Universidade do Minho
Escola de Engenharia

IMS
International Masonry Society

9th International Masonry Conference 2014 in Guimarães

Shaking table tests on unreinforced load-bearing masonry structures

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ABSTRACT: The recent interest of engineers for unreinforced masonry structures has led to improvements in the knowledge of their behaviour in normal conditions and to new applications, like multi-storey buildings. Nevertheless, additional investigations are still necessary regarding their seismic behaviour, considering the commonly admitted conservatism of the current seismic standards, limiting the applications even for low seismic areas. Moreover, new parameters have to be considered to fulfil the standards in terms of individual comfort. For the acoustic performance, rubber layers can be placed at walls extremities, but are likely to influence significantly the seismic response.

This paper describes shaking table tests on six specimens performed to contribute to these issues. The objectives are to improve the understanding of the seismic behaviour of masonry walls including rocking effects, to investigate the influence of soundproofing elements, to study the contribution of walls perpendicular to the seismic action and to characterize the frame behaviour.

Keywords: Unreinforced masonry, Shaking table, Acoustic insulation devices, frame behaviour

1 INTRODUCTION

Masonry is one of the oldest constructional materials and has been used for centuries for the construction of public buildings as town houses or churches, and of dwellings, like single family houses. Two main characteristics defined these structures: on the one hand, the huge diversity of the units (e.g. concrete, clay, natural or artificial stone, etc.) and their type of laying and bonding (veneer, dry, with mortar, glued, etc.) and, on the other hand, the design. Historically, the latter has been relying on good practice and empirical methods with a combined responsibility of the architects and builders, with no or limited engineering. Engineers have however been more and more interested in this field over the last fifteen years, leading to a better understanding of the structural behaviour in normal conditions and to a design with a more rational use of the materials, reducing thereby the cost and consumption of resources. All these structural, ecological and economical aspects are at the base of the “Eurocode 6 – Design of masonry structures” [1]. One of the main outcomes of the engineered approach is the extension of the range of applications, with the spreading of multi-storey apartment buildings (up to 5-6 levels) in pure unreinforced masonry or of lightweight concrete houses (see Figure 1).

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Figure 1. Multi-storey apartments (left) and lightweight concrete house (right)

Used all over the world and in particular in Europe as load-bearing system, masonry is thus also submitted to natural hazards, such as earthquakes for example. If not considered in the design, the seismic action can cause the collapse of the buildings and have tragic consequences, even in regions like Belgium (see Figure 2) where seismic event is unusual. Nevertheless, the earthquake impacts cannot be adequately and properly considered without a good understanding of the structural behaviour under these specific horizontal dynamic actions. One of the first main reference books dealing with this subject has been published in the late 90's by Miha Tomazevic [2] and gives the basic principles of this characterization. A similar approach is at the base of the "Eurocode 8 – Design of structures for earthquake resistance" [3] which describes the possible analyzes and design methodologies to be transposed to each particular type of masonry structures.

Figure 2. Earthquake in Liège (Belgium) - November 1983

In spite of the recent interest of engineers, a previous review [4, 5] of the technical literature shows up (i) the lack of researches focused on the most common types of masonry structural configuration used in North-Western Europe and (ii) the over-conservatism of the current seismic design standards and their mismatch with common construction habits. As explained in this review, the inconsistency comes in particular from the non-consideration in the current design models for the walls perpendicular to the seismic action and from the questionable modelling of the spanning structural elements.

In addition to these basic considerations, the design of new buildings, used for instance as apartments, implies new requirements in order to fulfil standards in terms of individual comfort. For the acoustic performance, a validated and convenient solution consists in placing rubber layers at the top and bottom of each wall to cut the propagation of acoustic vibrations (Figure 3). Such solution has been the purpose of recent researches performed at the University of Liège, but these aimed mainly at calibrating models in static conditions, without consideration of the consequences under dynamic action.

Figure 3. Rubber element at the bottom of a wall

With the view of contributing to these issues, dynamic tests on unreinforced masonry sub-structures have been performed in the Earthquake and Large Structures Laboratory (EQUALS) at the

University of Bristol, in the framework of the European project SERIES. The experimental campaign was carried out on six specimens, distributed in two sets and aiming at developing a better understanding of the behaviour of unreinforced masonry in dynamic conditions. More specifically, the first set investigates the influence of the soundproofing rubber devices on the seismic response, while the second set is focused on the contribution of walls perpendicular to the seismic action and of the horizontal spanning elements (frame behaviour).

This paper presents a global overview of the tests results and shows out the main outcomes in terms of dynamic properties (natural frequencies, modal shapes, damping ratio) and modelling of the seismic behaviour. A significant rocking behaviour is observed for both sets, strongly dependant on the presence of rubber, perpendicular walls and frame effect. The results are expected to be extended to full masonry structures in the future.

2 DESCRIPTION OF THE TESTS

2.1. Description of the specimens and objectives

The first set of specimens comprises four simple unreinforced load-bearing walls in thin bed-layered clay masonry with empty vertical joints. It includes two walls with an aspect ratio (Length x Height) close to 1 (long walls), while the other two have an aspect ratio of 0.4 (short walls), as shown in Figure 4. These two values are chosen with the view of targeting different failure modes, respectively in shear and in bending. The main dimensions of the walls are:

- 2.10 m x 1.8 m x 0.14 m (Length x Height x Width)
- 0.72 m x 1.8 m x 0.14 m (Length x Height x Width)

Figure 4. Walls with an aspect ratio close to 1 (left) and 0.4 (right)

Two walls (one for each aspect ratio) include soundproofing devices (rubber layers) placed at their bottom and top (see Figure 5, left). An additional mass of 5 tons lies on the top of the wall to emulate the structural floor load, with due consideration for the shaking table payload and for targeting a range of compression level comparable to what is commonly reached in regular masonry structures (see Figure 5, right). The resulting average compressive stress is about 0.15 MPa for long walls and about 0.5 MPa for the short walls.

Figure 5. Sound proofing devices and additional mass

This first set follows a double aim. Besides the development of a better understanding of the general behaviour of single walls in dynamics conditions, it is expected to investigate the consequences of the use of rubber elements on the seismic behaviour by comparing the structural response of walls having a same overall geometry, with and without soundproofing elements.

Regarding the second set, two single-storey frames with T- or L-shaped piers are built with the same construction method (Figure 6), namely thin bed-layered clay masonry with empty vertical joints, and connected by a RC lintel and a RC slab over the opening. Each pier is constituted by a "shear wall" and a "flange", which are respectively the part of the pier in the frame plan and the perpendicular part. The geometries are defined to fit with practical configurations. The dimensions are as follows:

- Shear walls : 0.74 m x 2.0 m x 0.14 m (Length x Height x Width)
- Flanges : 0.74 m x 2.0 m x 0.14 m (Length x Height x Width)
- Opening : 0.9 m x 1.8 m (Length x Height)
- Lintel : 1.8 m x 0.2 m x 0.14 m (Length x Height x Width)

Figure 6. Frames with T-shaped (left) and L-shaped (right) piers

The design of the frame with T-shaped piers has been done to trigger a global torsional behaviour and its piers are therefore oriented differently. The one with L-shaped piers has a geometrical axis of symmetry, but the shear wall and the flange are connected differently from one pier to the other. Indeed, one pier has its flange glued to the shear wall with a high performance mortar, while the other has its parts classically connected by an alternated mason work.

The structural floor load is here simulated thanks to a RC slab with additional steel blocks (Figure 7, left), still to reach common range of compression level in masonry structures whilst respecting the shaking table payload. The RC slab is supported by the piers through steel plates (Figure 7, right) used to be able to study different loading cases. The resulting average compressive stress in the loaded walls is about 0.135 MPa for fully loaded configurations and about 0.25 MPa when the load is only applied to the flanges. Steel frames are used beside the specimen, as it was for the first set, for the reasons of safety.

Figure 7. RC slab and steel connectors

The objectives of the second set are many. In addition to a better understanding of the frame behaviour and of the influence of the spanning elements in dynamic conditions, the contribution of the perpendicular walls is also to be assessed. Moreover, the consequences of global and local torsional effects are investigated through the specifically defined geometry of the frame with T-shaped piers, meanwhile the L-shaped frames is focused on the comparison between different connection methods between perpendicular walls and on the influence of the gravity loading case, especially the case of a frame with piers partially loaded shaken in the direction perpendicular to the plan of loaded walls.

2.2. Testing instrumentation and procedure

Both experimental sets use the same instrumentation devices. Those ones are divided in three different types, recording accelerations and relative or absolute displacements. The accelerometers are SETRA type 141A devices. The displacements are measured by LVDT sensors for the relative ones and an Imetrum Vision system with several targets is used for the global ones.

The instrumentation layout of the first set of specimens is the same for each wall, disregarding its length or the presence of rubber layers. A total of thirty-nine devices are used, distributed in seven accelerometers, fourteen LVDTs and ten internal table devices (Figure 8, left). The last eight devices are targets for the Imetrum Vision system, located on the top mass and on the fixed steel frame, assumed to behave as a rigid body (Figure 8, right).

Figure 8. Instrumentation layout of the first experimental set

The layout of the second set is slightly different from one specimen to the other to target the specific objective of each frame. For example, the frame with L-shaped piers has LVDTs measuring the relative displacement between the shear wall and the flange of its piers. A total of fifty-seven devices and fourteen targets are used in this case. Unlike the first set, the targets for the Imetrum Vision system are all located on the specimen, namely on the frame top or on the RC slab. This is illustrated in Figure 9 in the case of the specimen with L-shaped piers. Figure 9 emphasizes on the positioning of the measurement targets. Details of the instrumentation layouts for both sets are given in [6].

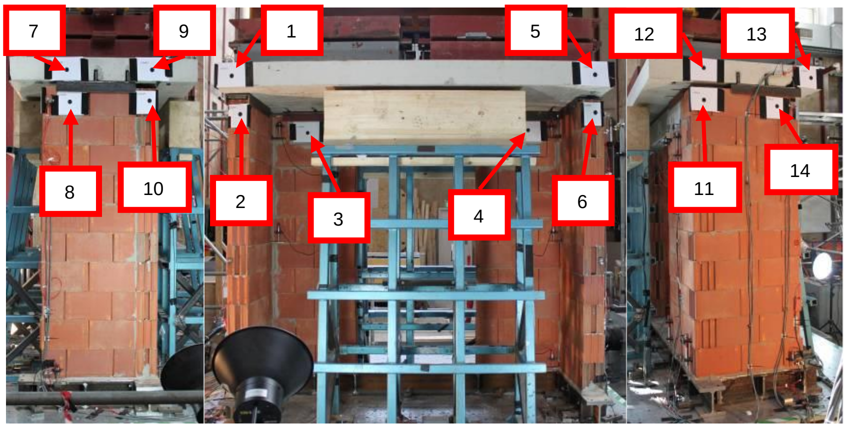

Figure 9. Targets for the Imetrum Vision system (L-shaped piers)

The testing procedure is extensively described in [6]. It consists in an alternation of two types of tests. The first type is done under “white noise" table excitation and is performed to characterize the dynamic properties of the specimens, while the second one is the seismic test strictly speaking with increasing acceleration input and is carried out by sending out to the table a chosen waveform compatible with a Eurocode 8 – type 2 spectrum. The procedure differs between the two experimental sets on a few points:

- The white noise tests are only performed along the wall length for the first set, but in two directions for the second one, with the view of measuring the specimen properties in the frame plan and in the perpendicular plan;
- For the second set, the seismic input is alternatively imposed along the frame plan, and perpendicular to it;
- The acceleration increment in the first set is not constant. Some levels are indeed repeated in the perspective of studying the effects of repeated earthquakes at a same level.

For latter exploitation, it is important to insist on the fact that the frame with L-shaped piers is tested under two different gravity loading cases, namely a fully loaded case and a partially loaded case with the slab resting on flanges only. This second case was applied after the tests of the first configuration and on the same specimen that had been previously slightly damaged.

A preliminary assessment had been carried out in order to assess the maximum acceleration that the specimens could withstand, in order to prevent premature collapse. This assessment is developed in [6] and is based on the normative procedures of the dedicated chapters of Eurocodes 6 and 8.

3 EXPERIMENTAL TESTS RESULTS

This section describes the results of the experimental tests. The data of the first experimental set has been extensively processed and provides results in terms of dynamic properties of the specimens, useful for the seismic design of masonry structures. Further developments, based on a classification of the tests with regard to the acceleration level, allow the modelling of the tests thanks to two different models. The second experimental set still requires additional investigations, but preliminary results about the natural frequencies and modal shapes are available and presented hereby.

Some first conclusions can already be made already from the direct visual observations and direct analysis of the seismic input [4, 5]. The main one leads to an obvious need to improve the seismic design rules used in the standards. Indeed, these rules are based on a theoretical static equivalent models assuming given behaviour and failure modes, while the experimental failure modes were actually completely different. The first set of specimens reached higher acceleration than expected, due to a general rocking behaviour strongly dependent on the aspect ratio and on the presence of rubber devices. In this case, the standards have to be improved in order to take into account the dynamic character of the seismic input. On the contrary, the collapse of the specimens of the second set happened prematurely, although a global rocking behaviour has been also observed. The associated failure modes were due either to torsional effects or to local collapse of the walls connection, which are not explicitly covered in the current code design procedures.

The maximum acceleration input (PGA) being the main parameter used to characterize the seismic action for practical engineering applications, the measured experimental values are provided in Table 1 for the first set and in Table 2 for the second one.

Table 1. First set – Measured PGA [g]

Test	S1	S2	S3	S4	S5	S6	S7	S8	S9
Long wall without rubber	0.039	0.078	0.078	0.158	0.238	0.323	0.450	0.572	0.688
Long wall with rubber	0.043	0.090	0.088	0.187	0.278	0.356	0.457	0.569	0.639
Short wall without rubber	0.041	0.065	0.064	0.087	0.136	0.133	0.178	0.187	0.234
Short wall with rubber	0.042	0.060	0.061	0.080	0.124	0.128	0.171	0.042	0.060

In Table 1, the seismic tests S2 and S3, as well as the S5 and S6, are repeated at a same acceleration level. The values of the measured PGA show the well-known difficulty to control the table response, seen that the measurements are different for similar theoretical input. This comment is also valid for the Table 2. Indeed, the seismic input should be unidirectional, but the measured PGA is bidirectional. The residual component transverse varies from 8 to 29% of the main one. This range of values, although reasonably low, can however influence the seismic response and is likely to be the source of unexpected damages to the specimen.

Table 2. Second set – Measured PGA [g]

No Test		S1	S2	S3	S4	S5	S6	S7	S8	S9
T-shaped fully loaded	x y	0.007 **0.074**	0.018 **0.148**	0.046 **0.285**	**0.038** 0.005	**0.083** 0.008	**0.176** 0.025	**0.276** 0.057	0.107 **0.477**	/ /
L-shaped fully loaded	x y	0.006 **0.066**	**0.038** 0.006	0.014 **0.149**	**0.087** 0.013	0.036 **0.221**	**0.135** 0.023	0.077 **0.269**	**0.180** 0.042	/ /
L-shaped flanges loaded	x y	0.011 **0.063**	**0.039** 0.007	0.014 **0.083**	**0.080** 0.018	0.026 **0.133**	**0.125** 0.035	0.039 **0.195**	**0.170** 0.037	0.058 **0.197**

3.1. First experimental set

The direct outcomes of the first experimental set are processed in terms of damping ratio, natural frequencies and corresponding modal shapes. The natural frequencies are derived from the white noise tests and the modal characterisation procedure is described in [6]. This allows identifying the possible deterioration of the walls. The first and second natural frequencies are given in Figure 10, respectively for the long and short walls.

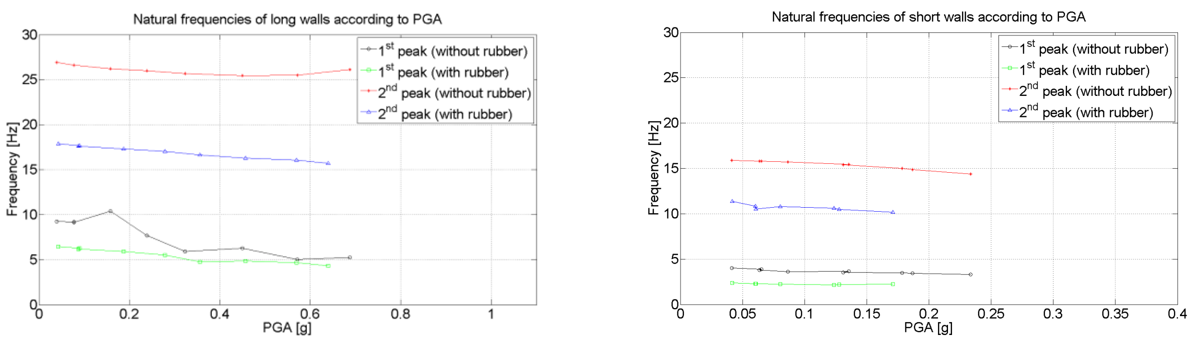

Figure 10. First set – Natural frequencies of long (left) and short (right) walls

The analysis of Figures 10 highlights the influence of the rubber layers. Indeed, the specimens without soundproofing devices have clearly higher natural frequencies. The relative difference in the undamaged situation is around 30% to 40%. The natural frequencies decrease after all seismic tests with increasing PGA, but the drop is more pronounced for the walls without rubber. Such decrease translates a progressive damaging of the specimens. Therefore, it can be stated that the presence of rubber devices has a positive effect since it results in reduced damages for similar ground acceleration level. This can be explained by the change from a classical rocking phenomenon to the situation of a wall resting on an elastic foundation.

The corresponding modal shapes are plotted in Figure 11. The first identified mode is the classical triangular modal shape. The major difference between the plots comes from the presence of the rubber and appears as a strong discontinuity, translating a more deformable zone located at the extremities (Figure 11, right) and similar to the common modal shape of base-isolated structures. The second mode (not plotted here) is associated to a vibration of the additional mass in phase opposition with the respect to the wall.

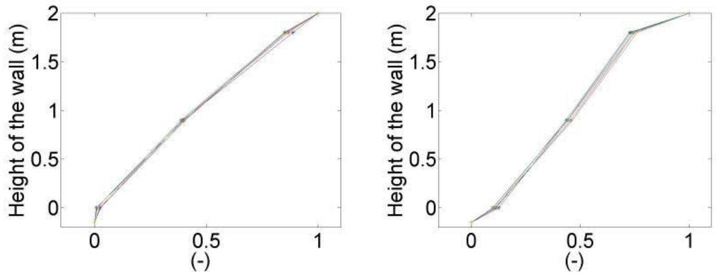

Figure 11. First set – First modal shapes

The progressive damaging is generally also related to an increase of the damping ratio. Such an increase is observed for all walls (Table 3), but is especially important for the long wall without rubber. The accuracy of the procedure is however questionable with regard to the numerical values obtained for highly damaged states (more than 100%).

Table 3. First set –Damping ratio [%]

After No Test		Before	S1	S2	S3	S4	S5	S6	S7	S8	S9
Long wall without rubber	1st peak 2nd peak	96.49 2.76	8.94 1.56	23.71 1.74	28.46 1.82	93.77 2.21	82.94 2.42	126.3 2.50	132.2 2.50	160.9 2.65	95.8 2.41
Long wall with rubber	1st peak 2nd peak	44.88 8.60	8.33 5.88	14.30 5.78	13.90 5.93	28.16 6.23	40.43 6.59	26.29 7.18	42.54 6.81	36.83 7.80	31.93 8.37
Short wall without rubber	1st peak 2nd peak	17.05 2.02	3.86 1.30	7.27 1.45	14.97 1.45	10.87 1.56	15.06 1.91	15.54 1.74	17.44 2.05	19.60 2.00	17.84 2.32
Short wall with rubber	1st peak 2nd peak	3.51 2.80	9.14 4.01	6.53 3.76	6.70 3.87	6.52 3.99	9.41 2.74	8.45 4.29	9.19 5.22	/ /	/ /

A last result from the direct exploitation, useful for the assessing the practical design procedure is the measurement of the contact length between the wall and its foundation. This parameter is derived from the direct measurements during the seismic tests, assuming that the base section remains plane. The post-processed results are presented in Figure 12 and illustrate a major difference between the preliminary assessment theoretical model and the experimental reality. According to the static equivalent model, a zero contact length is assumed as failure by overturning of the specimen. This conclusion is not valid when considering dynamic action. A comparison with design rules is performed in [4] and shows that the static procedure is actually relevant for low acceleration level but gives underestimated results (in terms of maximal acceleration) for the highest inputs. The presence of rubber layers leads to higher compressive lengths, but larger horizontal displacements.

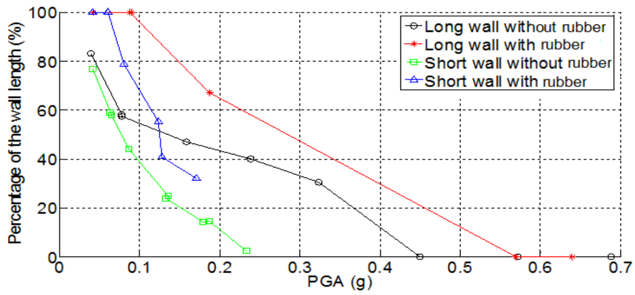

Figure 12. First set – Compressive length

The following developments classify the tests based on a comparison of the top and bottom rotations (Figure 13). This figure shows that the rotations are different for seismic tests at a low acceleration level, but are close one to the other when the acceleration level increases. Based on this classification, it is has been demonstrated that the specimen behaviour can be accurately modelled as (i) a cantilever-beam system for the low levels and (ii) as a simple rocking rigid block for the highest ones.

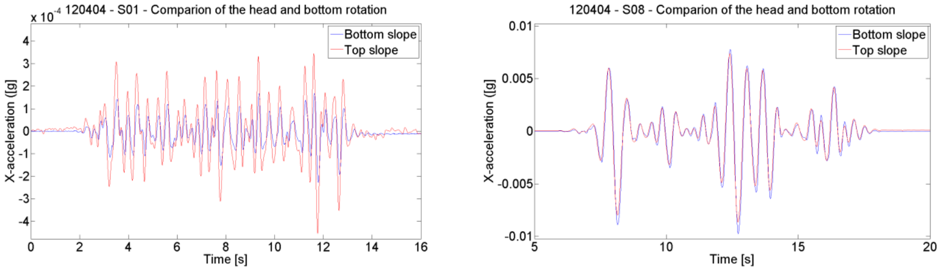

Figure 13. First set – comparison of the top and bottom rotations

(i) The developed cantilever-beam model is based on the Timoshenko beam theory [7] and used to derive a frequency equation with modified boundary conditions, so as to fit with the testing configurations. This equation depends on the specimen geometry and on its mechanical properties, namely the elastic and shear moduli. These two parameters need either to be characterized by tests or to be assessed according to standards recommendations. Establishing the frequency equation is thus interesting since it allows comparing the frequencies measured during the tests with the one theoretically obtained from standard properties of the materials. Details of such developments are given in [8] for the first natural frequencies walls without soundproofing devices. These are expected to be extended to walls with rubber devices in an upcoming contribution.

The results based on reference models are given in Figure 14. They provide the coupled values of elastic and shear moduli required to reach a given natural frequency (in this case the first frequency of the undamaged walls). Comparison with the standards recommendations leads to the conclusions than the suggested values are too stiff for the considered type of masonry, as explained in [8]. Indeed, the recommended couple is (E = 1950 MPa and G/E = 0.4), which is above the drawn lines.

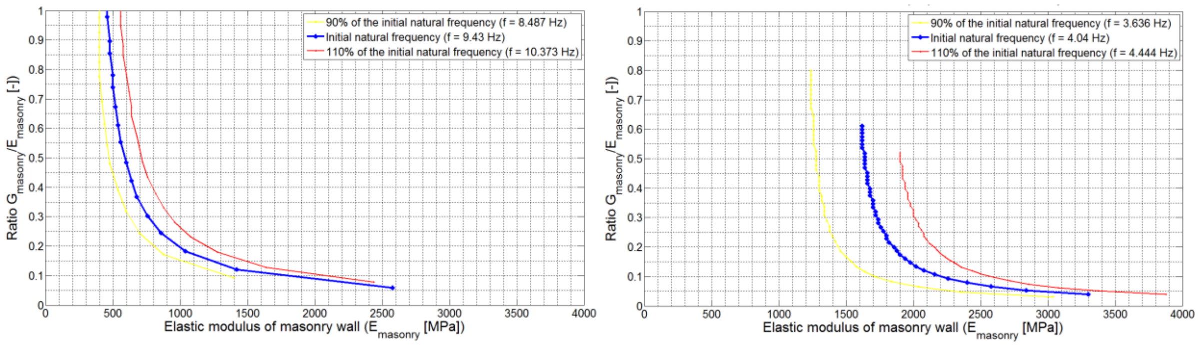

Figure 14. First set – Results of the frequency equation (left : long wall ; right : short wall)

(ii) The simple rocking model is derived from the historical developments proposed by Housner [9], extended here to consider the additional mass lying on the top of the wall and the intrinsic deformability of the specimen. These considerations result in a modification of the criterion for the initiation of the rocking motion and of the restitution coefficient. Details and explanations of the developments are given in [10]. These results have to be extended to the walls with rubber devices.

Figure 15 compares the rotation of the wall measured during the experimental campaign to the one predicted by the adapted rocking model, in the case of the short wall without rubber. The fitting of the curves is reasonably correct, except at the end of the signal. This difference can be explained by the influence of the table motion, which modifies artificially the damping through its breaking system. Having a perfect correspondence between the curves is no easy task because of the formulation of the restitution coefficient in the theoretical rocking model. This coefficient is indeed a useful numerical trick but has a limited physical background, which makes its calibration rather difficult.

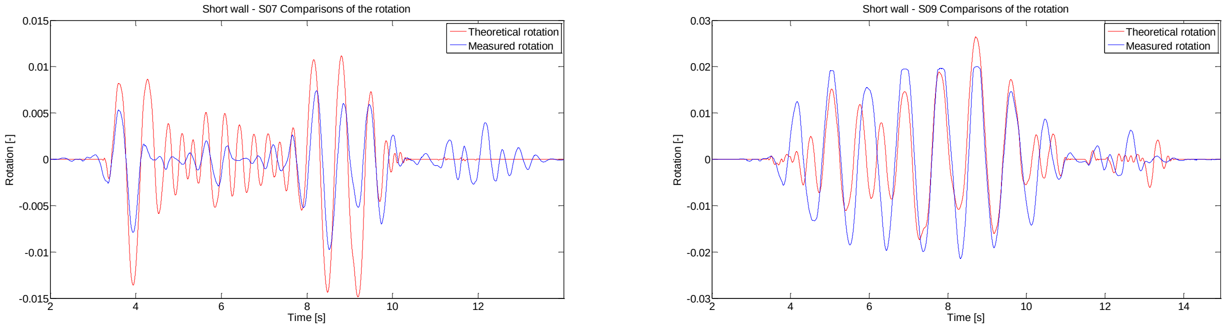

Figure 15. First set – Results of the rocking model (top : S07 ; bottom : S09)

3.2. Second experimental set

The exploitation of the results for the second experimental set provides again information about the natural frequencies and corresponding modal shapes [5]. The procedure used to get this information is the same as the one of the first set. The results are presented in Figure 16 and Figure 17.

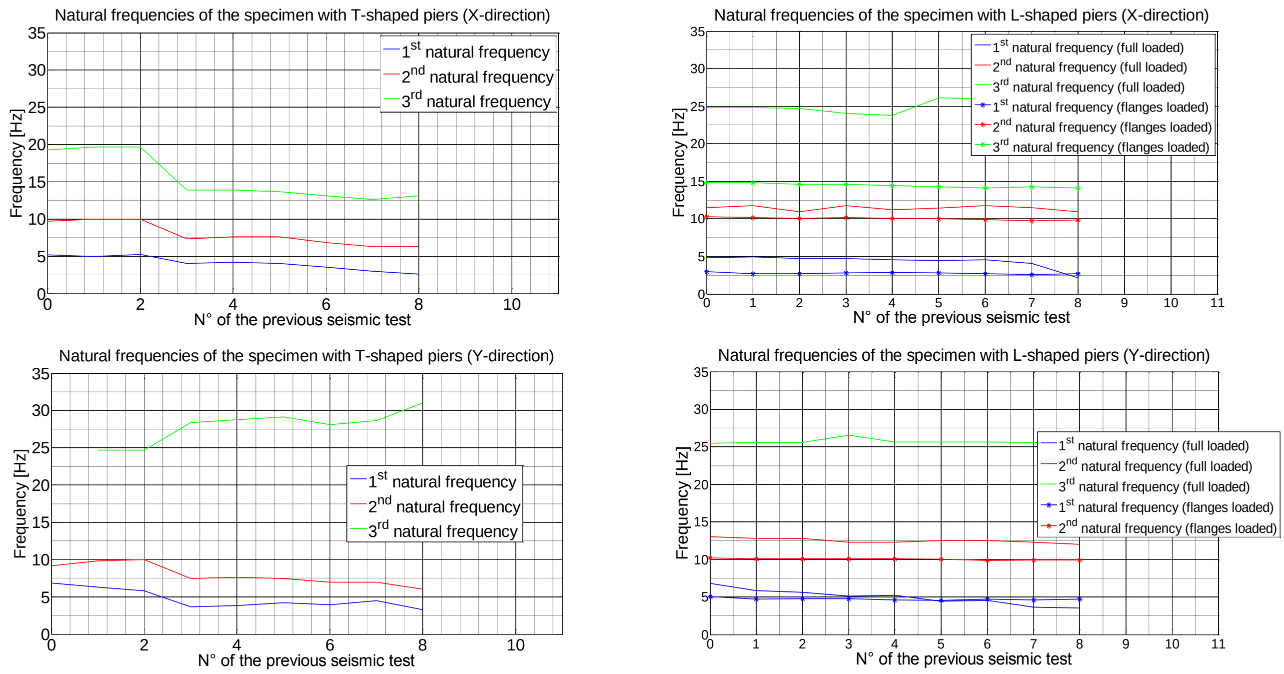

Figure 16. Second set – Natural frequencies

As for the specimens of the first set, a frequency drop is observed in Figure 16 when the acceleration level increases, translating the progressive damage of the masonry frames. Another observation is that the first frequency is higher for the frames with T-shaped piers. This result is unexpected with regard to the theory and should be further investigated.

The modal shapes of the frame with T-shaped piers (Figure 17) are obtained from the four accelerometers located on the RC slab. They describe the overall response of the specimen in the horizontal plan. The identified modes are a combination of translations and rotation. The translation component is more important for the first and third modes, while the second one is essentially rotational.

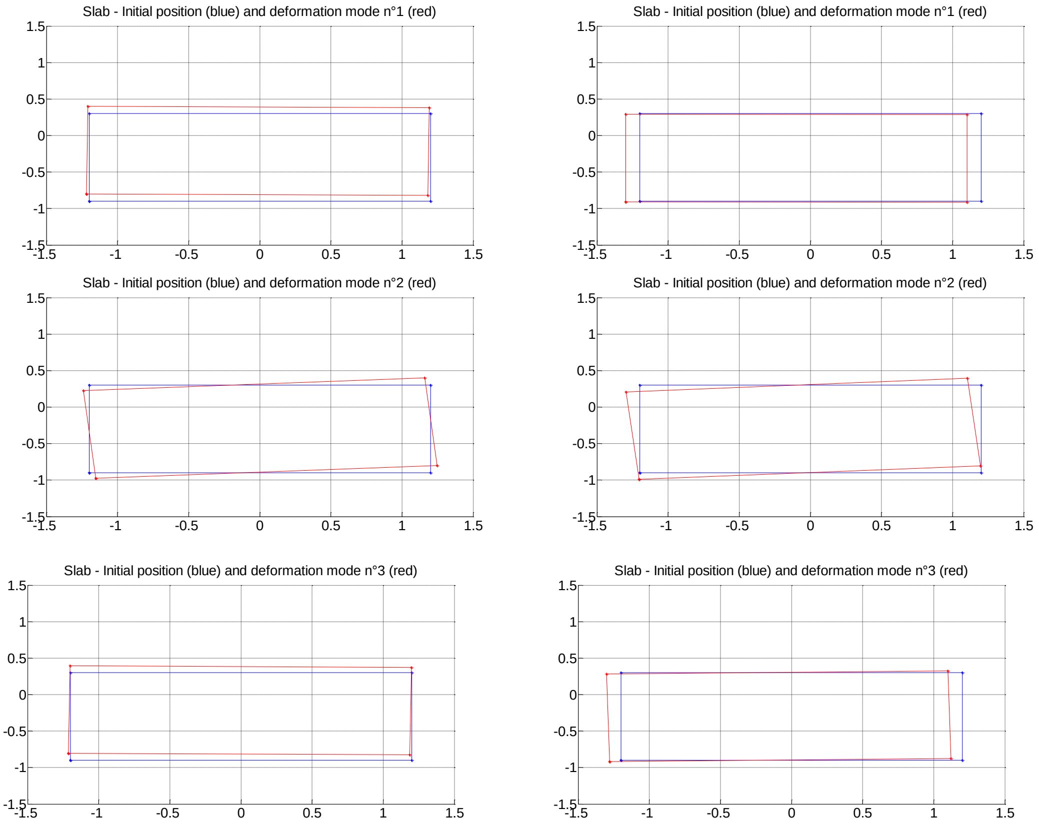

Figure 17. Second set – Modal shapes (left : x-direction ; right : y-direction)

4 CONCLUSIONS

This paper gives an overview of experimental shaking table test results on unreinforced load-bearing masonry sub-structures with glued horizontal joints and empty vertical ones. The tested specimens are divided in two sets. The first set is composed of four simple unreinforced masonry walls including or not soundproofing devices. The main objectives of this set are to develop a better understanding of the seismic response of simple walls and to study the influence of rubber layers used for acoustic reasons. In addition to the direct exploitation of the tests results, further developments lead appropriate suggestions for modelling. The following conclusions are drawn:

- When submitted to successive earthquakes, the natural frequencies of masonry walls decrease. Such frequency drop translates a progressive damage of the walls with a concentration of damages in the base mortar joint.
- The presence of soundproofing devices decreases the natural frequencies of the specimens but has a beneficial influence by reducing the frequency drop and by increasing the contact length. Nevertheless, this conclusion is mitigated by larger horizontal displacements which can be problematic at the scale of an entire building.
- The compressive length can be easily assessed and the procedure given by the standards is relevant for low acceleration level, but underestimates the experimental value for higher acceleration.

The specimens of the first set have been further modelled by a cantilever shear-deformable beam. This type of model is found relevant for the lower levels of seismic input where no uplift of the base is observed. It has been used to derive a frequency equation and to assess the mechanical properties of the studied type of unreinforced masonry walls. A comparison with the standards recommendations leads to the conclusions that the couple elastic/shear moduli proposed by the norms is too stiff for the considered type of masonry. Concerning the highest seismic input, with a significant rocking behaviour induced, a simple rocking model is actually shown as able to provide relevant results and to reproduce correctly the experimental observations.

The second set consists in two unreinforced masonry frames with T- or L-shaped piers. The main objectives of this set are to develop a better understanding of the seismic response of masonry frames, to investigate the contribution of the spandrels elements and of the walls perpendicular to the seismic action and to study the influence of the torsional effects, of the type of connection between perpendicular walls and of the gravity loading case. This set still requires a deeper post-processing although preliminary outcomes have been presented and discussed hereby.

Further perspectives cover the investigation of the modelling and behaviour of walls with rubber, the extensive exploitation of the results on the second set of tests and the globalization of the theoretical model to study entire buildings.

ACKNOWLEDGEMENTS

The research leading to these results has received funding from the European Union Seventh Framework Programme (FP7/2007-2013) under grant agreement n° 227887, SERIES. H. Degée also acknowledges the direct support received from F.R.S.-FNRS (Belgian Fund for Research). C. Mordant acknowledges the F.R.I.A. for the support of his PhD.

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