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SEISMIC ANALYSIS OF A BUILDING BLOCK LOCATED IN AN URBAN AREA OF THE FAIAL ISLAND OF AZORES

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SUMMARY

This paper presents part of an ongoing research on the seismic behaviour of a whole building block at Horta (a town in the Faial Island - Azores), hit by an earthquake on July, 9th 1998. The outcome of a preliminary numerical analysis carried out locally on a single building and globally on the whole block is presented, in order to assess sensitivity both to the modelling type and material properties. After a preliminary identification of the most vulnerable areas of this block, an experimental campaign of *in situ* ambient vibration tests was performed. Through the results, both frequencies and vibration modes of some of the block buildings were obtained, which allowed calibrating the numerical modelling presented herein. Furthermore, numerical seismic analyses were performed for some houses of the block considering the actually recorded accelerograms during that earthquake. The first results obtained to date are briefly addressed and discussed in terms of stress and drift values due to seismic action.

INTRODUCTION

Any building within a block of a city shows different structural behaviour peculiarities from an isolated building. This is a key aspect to be taken into account while carrying out any repair and/or reinforcement actions. Any intervention in a building within a block might also affect the behaviour of either adjacent constructions or the structure of the whole block.

After having performed an experimental dynamic identification, the main purpose of this paper is to present a comprehensive analysis of the seismic response of a building block in the Horta town, hit by an earthquake on July, 9th 1998. This block (Fig. 1) located in the Matriz parish, near the Town Hall of Horta, is delimited by the Post Office building to the South, the Misericórdia lane to the North, the Serpa Pinto Street (Fig. 1-c) to the East and the Comendador Ernesto Rebelo Street (Fig. 1-a) to the West.

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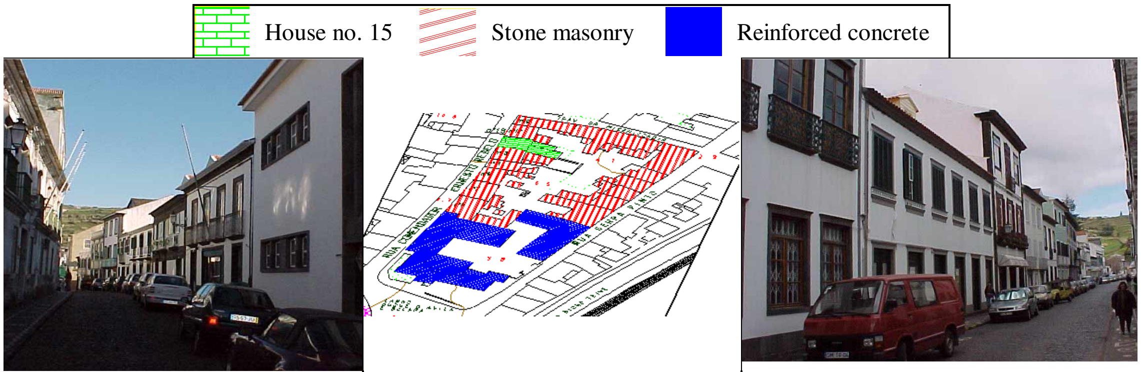
There are two different building types with distinctive dynamic behaviours: traditional masonry buildings over the major part of the block and the Post Office building with a frame structure of reinforced concrete.



a) Com. Ernesto Rebelo street   b) Block plan   c) Serpa Pinto street

Fig. 1- Block – Plan and surrounding streets: west (a) and east (c)

In this block there are several characteristics with potential structural behaviour problems. A careful and attentive analysis should, therefore, be carried out to decide the right reinforcement actions to reduce damages occurred after the earthquake. As a matter of fact, Fig. 1-b shows evidence of a more stiff and resistant structure in one of the block limits. During an earthquake the block dynamic behaviour might be conditioned by that significant stiffness concentration. There is also evidence of corners and narrower and longer structures at the rear of some buildings. They cause structural discontinuities in plan and are responsible for local and global eccentricities. These are extremely vulnerable areas, as can be seen in the outcome of several earthquakes: Azores (1980) and Northridge (1994). On the other hand, Fig. 1-a and Fig. 1-b show some of the existing irregularities in the block, due to land topography, to different heights of adjacent buildings and also to roof height. All these are responsible for relevant vertical discontinuities, that favour a stronger seismic vulnerability of the structures involved.
During this research, a structural modelling of the whole block was carried out with the help of CASTEM 2000 [1] (a general purpose structural analysis computer code based on the finite element method), in order to simulate and analyse the block response to a seismic action represented by accelerograms recorded "in-situ" during an earthquake. This modelling was performed after carrying out partial modellings of more deeply analysed houses, in order to establish adequate parameters for the modelling characteristics (structural discretization and components properties) to be adopted in the whole block.
The definition of parameters is a preliminary assessment and was supported by a first in-situ inspection, with a survey of several block buildings and measurements of vibration frequencies. After having enabled the characterisation of the global block behaviour and the identification of the most vulnerable zones during an earthquake, this preliminary study was followed by a large experimental campaign on the dynamic identification of several selected buildings to help calibrating the modelling strategies.

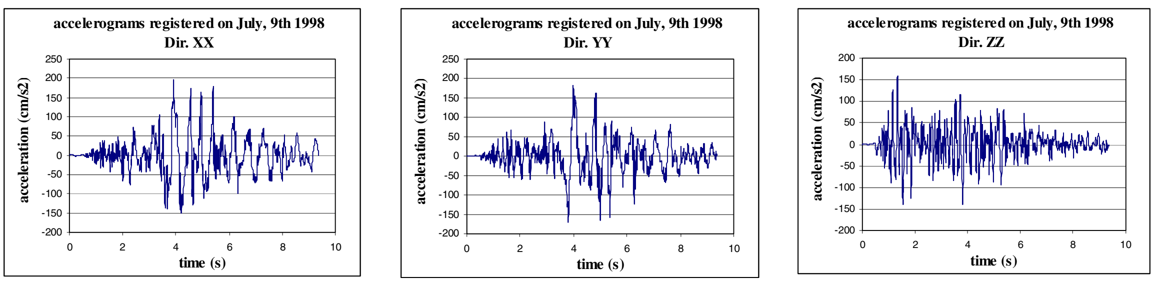
PRELIMINARY DYNAMIC ASSESSMENT

After the first survey in this block, a preliminary numerical analysis of its global and local (specific houses) dynamic behaviour was carried out. The following two modellings were the most deeply studied:

- a modelling of the house at no. 15 Comendador Ernesto Rebelo Street;
- a global block modelling.

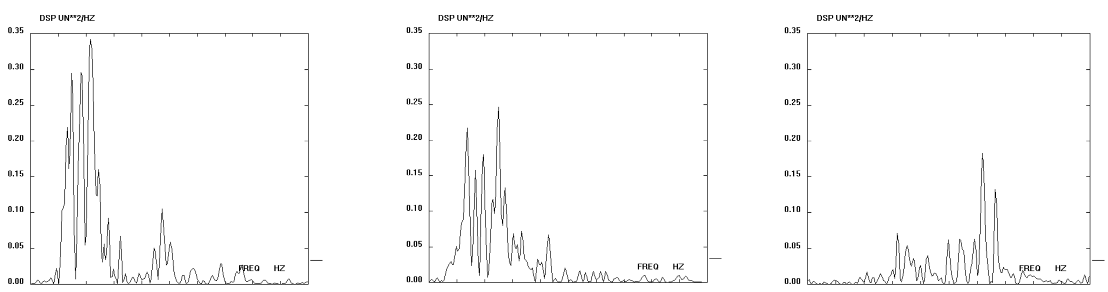
This research enabled an accurate identification and characterisation of the geometry and structural components of house no. 15 Comendador Ernesto Rebelo Street. This was decisive while choosing to

carry out a deeper analysis of this house. Its discretization process took into consideration every component of the house, namely: walls, floors and roofs. The model was then calibrated according to the materials and modelling criteria, in order to obtain results consistent with those recorded in-situ until this stage (mostly vibration frequencies).
Based on a careful analysis of house no. 15 results and on an adequate criteria selection for this kind of study, a global block modelling was then performed to assess the block dynamic behaviour and its global response during an earthquake.
Since these constructions were in fact hit by an earthquake, a seismic analysis based on this specific case is of utmost importance. It allows a comparison between the numerical results and the actual impact of the earthquake over the block buildings. This is an additional validation of the conditions adopted for the modelling.
In this case, the seismic information source comes from the accelerograms registered on July, 9th 1998 (Fig. 2), at the foundation level of the Prince Monaco Observatory, located in Horta - Faial Island, at an epicentre distance of approx. 10 to 15 km. The above referred seismic record showed a maximum value of approx. 400 cm/s^2 in one of the horizontal directions. The corresponding power spectra registered for its three components are illustrated on Fig. 3. According to the block orientation, the XX direction is considered as coinciding with the longitudinal axis.



a) XX Component a) YY Component a) ZZ Component

Fig. 2- Accelerograms registered on July, 9th 1998



a) XX Component a) YY Component a) ZZ Component

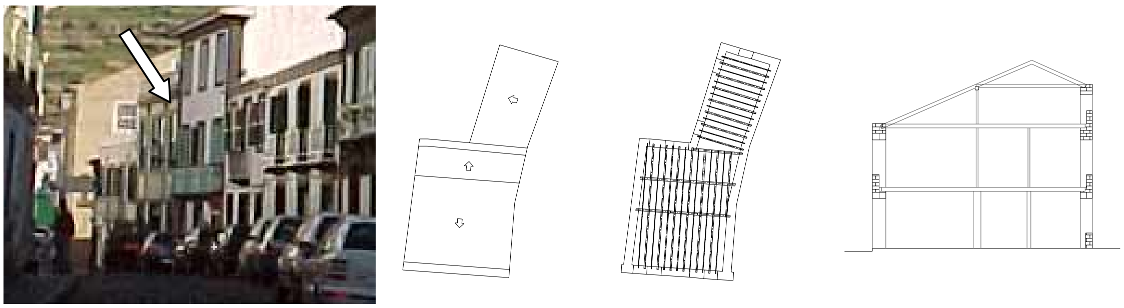
Fig. 3- Power spectra densities of each component during the earthquake on July, 9th 1998.

The power spectra analysis shows that the earthquake horizontal components (XX and YY) have greater content in the frequency range between circa 1 Hz and 2.5 Hz, whereas the vertical component (ZZ) is more intense between 6 Hz and 7 Hz. Considering the spectral contents for the relatively high frequencies (higher than 2 Hz), this earthquake can be associated with Portuguese code standard action type 1, whose characteristics are moderate magnitude and short focal distance.
Results from the structural seismic response were obtained from linear elastic analysis by time integration through the Newmark method, considering viscous structural damping according to Rayleigh formulation (proportional to mass and rigidity matrices) and calibrated to ensure that the damping coefficient does not exceed 5% in the frequencies range relevant for horizontal and vertical components.

Local assessment of house no. 15 at Comendador Ernesto Rebelo Street

Structural description

House no. 15, located near one of the block corners (Fig. 1-b), is a traditional dwelling (Fig. 4-a) with two façades, shared walls separating from the adjacent houses, two front storeys and a shorter upper storey (Fig. 4).



a) Location b) Roof c) Floor d) Sectional view

Fig. 4- Comendador Ernesto Rebelo Street – house no. 15

The structure consists of stone masonry walls and wooden floors laid on wooden joists and beams supported on the walls. Its stone masonry walls show an extensive homogeneity and good quality. The storey components are made of criptomeria (typical wood of this region), whose main characteristic is a low unit weight. The tests performed [2] and the vibration frequencies results from the early research allowed a first estimate of the main physical and mechanical properties of construction materials, shown in Table 1.

Table 1– House no. 15 – Material properties

Component	Material	ρ (ton/m^3)	E (GPa)
Walls	Stone masonry	1.8	1.1
Joists	Wood	0.27	3.8

Numerical modal analysis

The structure discretization of house no. 15 was carried out with the already mentioned computer code, using 3-node shell elements to model walls and 2-node bar elements for joists and wooden beams. All structural components that could condition the structural behaviour were modelled. Since one of the purposes of this research is the assessment of the group effect influence on the seismic behaviour of the existing structures, the following two modelling hypotheses were considered:

- A modelling – the house structure handled as a single component; the influence of the adjacent buildings was left aside (Fig. 5);
- B modelling – all structural components of the house were taken into account, including some walls of the adjacent buildings (Fig. 6).

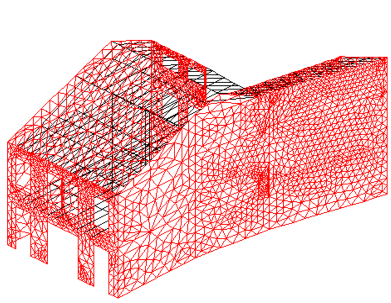


Fig. 5 - A Modelling

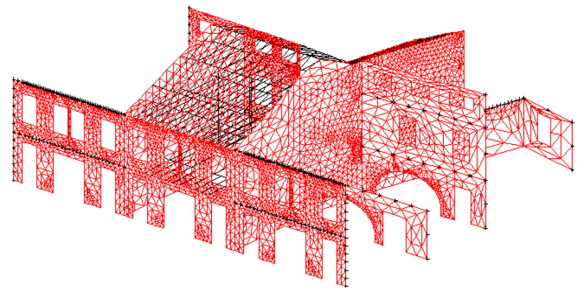
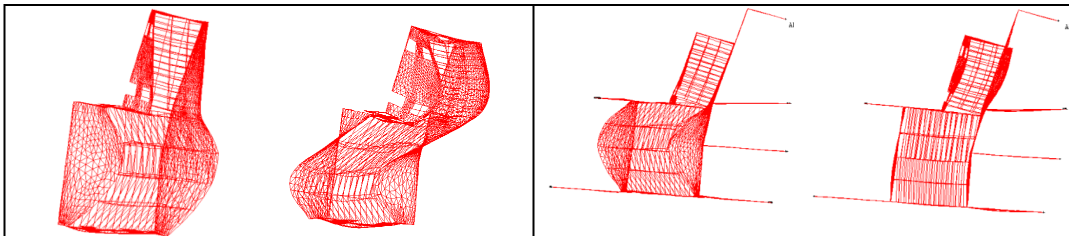


Fig. 6 - B Modelling

With regard to its components both models are similar. B modelling considers the existence of adjacent dwellings and models the relevant vertical components that might have an influence in the structure response. This is a first qualitative approach, since some simplifications are included concerning the adjacent structures supports and loads. Vibration modes were calculated for each modelling, some of them are illustrated on Fig. 7.



1st mode: 2.85 Hz 3rd mode: 4.43 Hz
a) A Modelling

1st mode: 3.89 Hz 2nd mode: 5.64 Hz
b) B Modelling

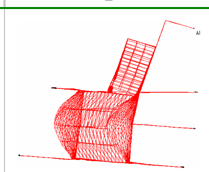
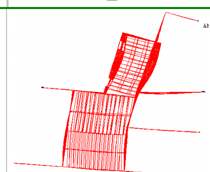
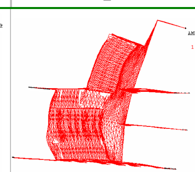
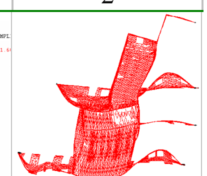
Fig. 7 - Vibration modes of house no. 15. Group effect impact assessment.

It should be stressed that in what the group effect is concerned conclusions can only be taken regarding the block longitudinal direction where adjacent structural components have a strong influence. In the transverse direction, the effect in this case cannot be adequately evaluated, since the group walls (due to the simplification process) were braced into the perpendicular direction of their development.
The modal result analysis helps to assess the relevance of the group effect in the building response to an earthquake. As expected, when comparing the first modes for A and B modellings (Fig. 7), the introduction of the adjacent houses walls represents an increase of the structural vibration frequency values in the longitudinal direction of the block. In both modellings, the vibration frequencies in the transverse direction correspond to higher modes than those shown in Fig. 7.
Besides the assessment of the group effect, an adequate calibration of the floors structural modelling was necessary, since their composition raised some difficulties in finding the most adequate strategy. In both cases (A and B), the floor structures were modelled with the main beams and joists. The wooden floor pavement was simply modelled with reduced inertia bar elements joining several storey joists, in order to achieve some homogenization of in-plan floor deformations. However this strategy does not guarantee that the storey acts in its plan as a rigid diaphragm. An accurate interpretation of the floor effect on the structure and the validation of its numerical model led to consider another modelling (C), based on the B modelling, but including very thin shell elements to simulate the wooden floors. These are settled on joists and main beams, but are not connected to the bearing stone walls, as can be seen on Fig. 8.
Table 2 contains the characteristics of the main global vibration modes of B and C modellings, namely its frequencies values, main direction and percentage of global effective mass mobilised in each mode. This last value helps to assess, for each direction, the relevance of each mode in the structure global seismic response, since it measures the inertia forces potentially mobilized during an earthquake.

Fig. 8 – Detail of the floor supporting structure

Between the vibration modes shown in Table 2, there are other local modes related to floor deformations that, because of their mass and rigidity, show similar frequencies between them and alternating with the values stated here. These modes may play a local key role at the element level, but do not represent a conditioning factor to the global response.
C modelling shows (Table 2) that the inclusion of shell elements adds stiffness to the floors, as can be understood from the difference in the frequency values of the first modes of B and C modellings.

Table 2 – Modal results of house no. 15. Assessment of the floor stiffness influence

Global modes	B Modelling		C Modelling	
	1	2	1	2
Configuração				
Frequency (Hz)	3.89	5.64	5.17	9.93
% effective mass - dir.	31 – xx	17 – xx	62 - xx	40 - yy

Whereas in the first case (B) the separating walls show a flexure around a vertical axis, in the second case (C) this situation is not so striking, since the shell elements in the floor acting as a rigid diaphragm, restrict its in plan deformation, increasing the structural stiffness and frequencies.
Vibration frequency values resulting from the preliminary campaign of in-situ experimental measurements are similar to those of B modelling. However, since there are two main variables conditioning the dynamic behaviour (wall elasticity modulus assumed with 1.1 GPa and floor stiffness), a conclusive calibration only based on the vibration frequency value could not be performed. The numerical model could only be calibrated after a more detailed dynamic identification that helped to achieve main modal configurations.
Nevertheless, the relevance of the wooden floor behaviour of the structure floor (Fig. 8) can be highlighted. Its rigid diaphragm effect is strongly dependent of existing effective connections both to the wooden floor supporting beams and to its own components. However, its quantification is still an open issue and requires more in-depth studies.
A further modelling without floors was also considered. The local modes of single walls are the result of an increased structural flexibility. This example shows the negative effect of the lack of walls bracing, a common situation in top walls. Without bracing against the horizontal components, these are the first to be affected during earthquakes.

Block global study

Located in an area of approx. 8 400m^2, the block (Fig. 9) comprises two buildings types: a traditional one with two or three-storey houses and another with a single reinforced concrete building. The Serpa Pinto

Street is at a lower level than the Comendador Ernesto Rebelo Street, what was taken into account for modelling purposes.

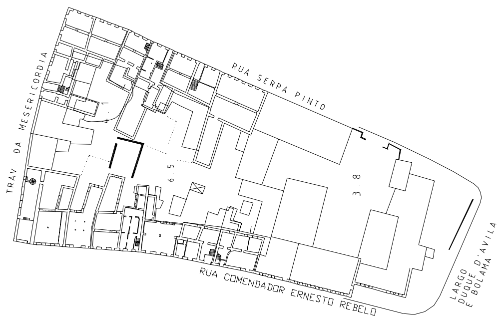


Fig. 9– Block plan

After calibrating the mechanical properties of materials and the modelling criteria according to the study of house no. 15, the block numerical analysis was carried out. This helped to assess the block global behaviour, considering the influence of the Post Office building located in one of its limits. Due to its location and strong stiffness, this building may have an influence in the global behaviour, setting the stiffness centre far from the mass centre, thus creating relevant eccentricities and aggravating the earthquake impact on masonry buildings. Another point to be considered is the interface between the houses and the reinforced concrete building. Contrary to the masonry houses with separating walls, in this kind of buildings there is a dilation joint between the two structures; this is a key point to explain the response.

A block global model (Fig. 10) was developed, including all buildings. Considering the model dimension, it was decided to refine the mesh only for the stone masonry structures, and to adopt a less discretized mesh for the reinforced concrete building. Using the already referred computer code, all components were simulated with shell elements, including wooden floors, since simulating main beams and joists was impossible, due to the block dimension. All mechanical properties of materials were considered as for the house no. 15 assessment.

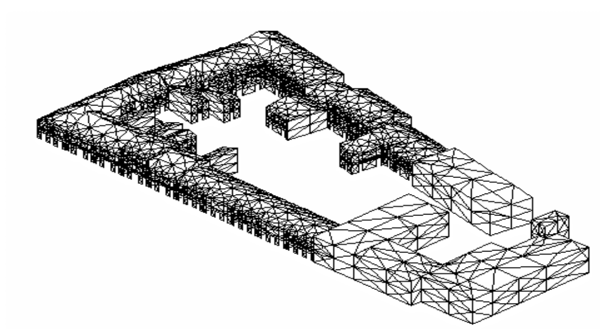


Fig. 10 – Block global modelling

Based on the calculated vibration modes calculation, several key points related to the block global dynamic behaviour could be understood.
From the results (see Fig. 11), it can be seen that corners (1st, 2nd and 3rd modes), plant irregularities (5th mode) and height irregularities (10th mode) represent the most vulnerable block zones in case of earthquake events.

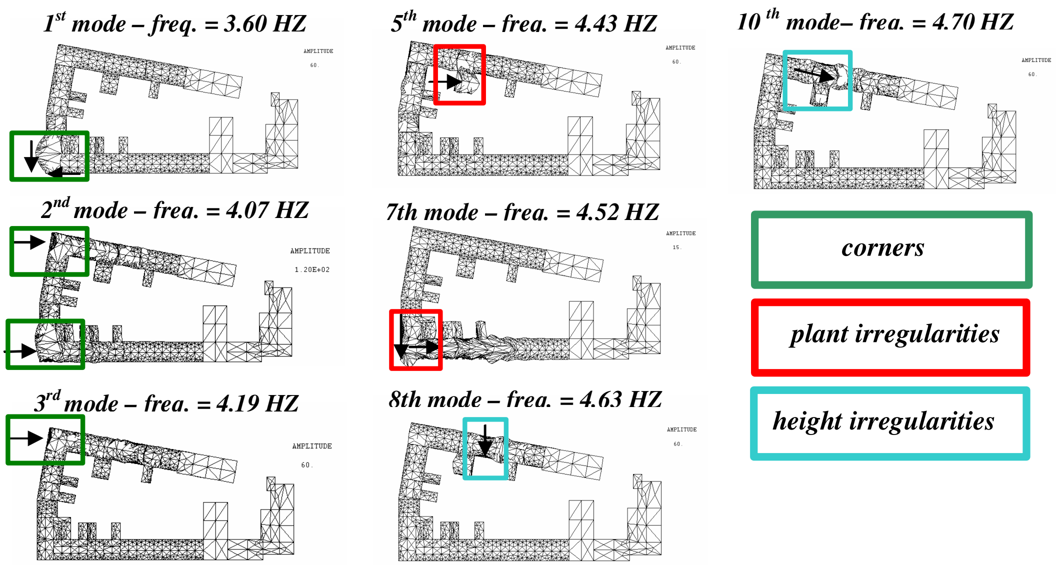


Fig. 11 – Block vibration modes

DYNAMIC IDENTIFICATION AND STRUCTURAL CALIBRATION

Further to the results presented and commented in the previous section, a campaign of dynamic tests was carried out in block zones found more vulnerable in the previous numerical analysis and with free access. Thus, the following structures were subject to ambient vibration measurements: Post Office building, house no. 15 (previously modelled and assessed) and two houses at Serpa Pinto Street, no. 16 and no. 24, all illustrated on Fig. 12. Nevertheless, for this paper purposes, only houses no. 15 and 16 will be referred to.

During this research, house no. 15 underwent some rehabilitation works, mainly on floors. Despite keeping the main structure (walls), its global behaviour changed, and some corrections had to be made to the numerical model, whose mesh is shown on Fig. 13.

a) House no. 15 b) Post Office building c) Houses no. 16 and 24 a) House no. 24

Fig. 12 – Buildings subject to ambient vibration measurements

Because the house no 16 stands for a typical case of height discrepancies, its numerical model had also to be developed, being the corresponding mesh discretization illustrated in Fig. 14. To get the group effect, the adjacent buildings were considered in both models; the mechanical properties of structural elements were assessed according to the dynamic tests results.

Tests of ambient vibration measurements were carried out with five GEOSYG strong motion recorders (seismographs), GSR-12 and GSR-16 models of 12 and 16 bits resolution, respectively. Equipped with three direction accelerometers (two horizontal and one vertical) and a certain storage capacity of acquired data, these equipments can record accelerations in those directions. These records are transferred into a computer where a first analysis of the collected material can be carried out. In each house, records were made in series of measuring stations, each with three movable seismographs and two of reference. Some of them are shown on Fig. 15 and Fig. 16 for houses no. 15 and no. 16 respectively, with the location of

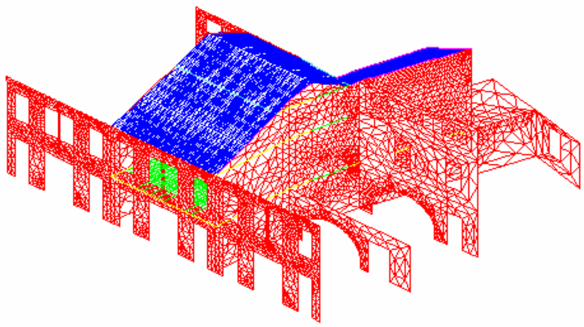


Fig. 13 – House no. 15. New numerical model.

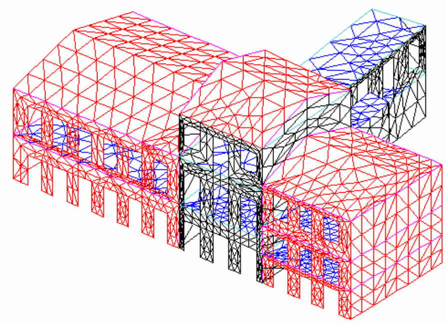
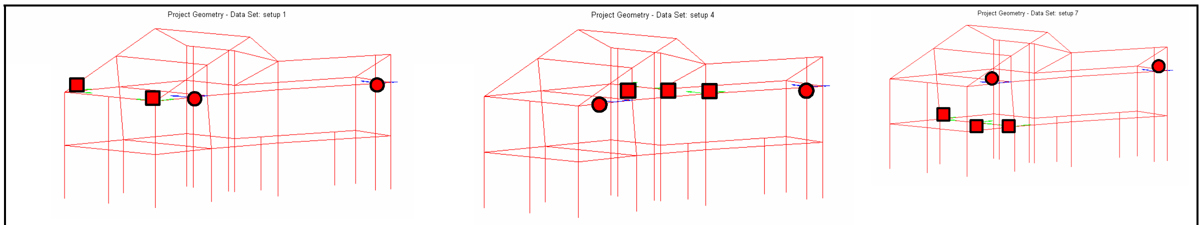


Fig. 14 – House no. 16. Numerical model.

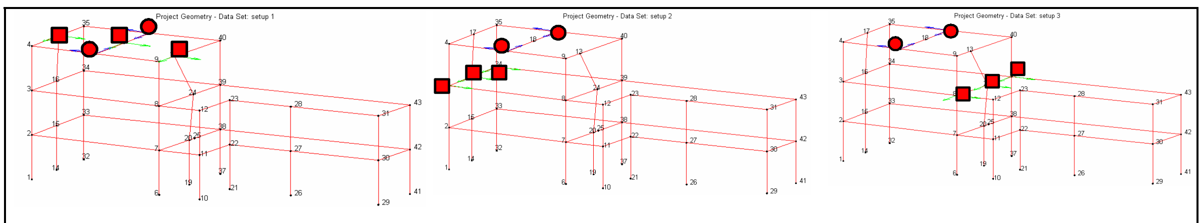
From the acceleration records of each station the corresponding power spectra densities were calculated by procedures based on the Fast Fourier Transform (FFT), capable of estimating the transfer functions for the monitored degrees of freedom (two horizontal and one vertical per seismograph).



a) Station no. 1 b) Station no. 4 c) Station no. 7

Reference Seismograph Movable Seismograph

Fig. 15 – House no. 15. Some ambient vibration measuring stations



a) Station no. 1 b) Station no. 2 c) Station no. 3

Reference Seismograph Movable Seismograph

Fig. 16 – House no. 16. Some ambient vibration measuring stations

These procedures are available in the CLOSEVIEW analysis computer code (associated with GEOSYG seismographs) and were used in records processing, according to the following operations sequence, [3],

- Hanning filters application to record time windows, in order to reduce the Leakage effect [4];
- Digital filtering of records with a low-pass filter regulated to 1/8 frequency of the sampling, in order to reduce the Aliasing effect and eliminate high frequency components;
- Digital filtering of records with a high-pass filter regulated to 1 Hz;
- Baseline correction to cancel the residual offset of records;

In a later stage, the modal identification based on the Frequency Domain Decomposition method (FDD) implemented on the ARTeMIS [5] program was performed. In a first stage, this technique, [6] and [7], provides the calculation of the eigenvalues of the response power spectral density matrices obtained through the auto- and cross-spectrum of each test. In a later stage, the spectra peaks of those eigenvalues allow to obtain the system natural frequencies, as illustrated on Fig. 17 and Fig. 18 (concerning the application of this method in two measuring stations).

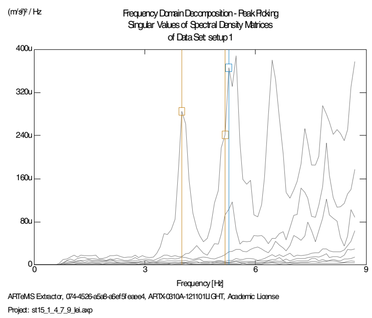


Fig. 17 – House no.15; FDD - station 1

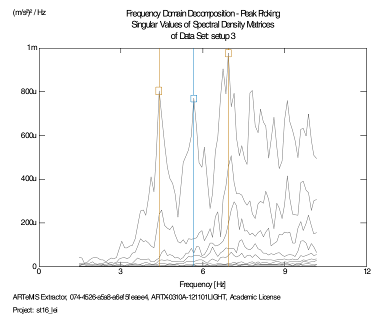
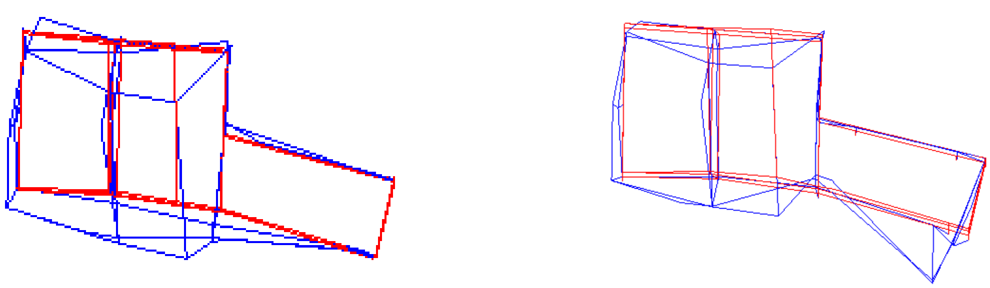


Fig. 18 – House no.16; FDD - station 4

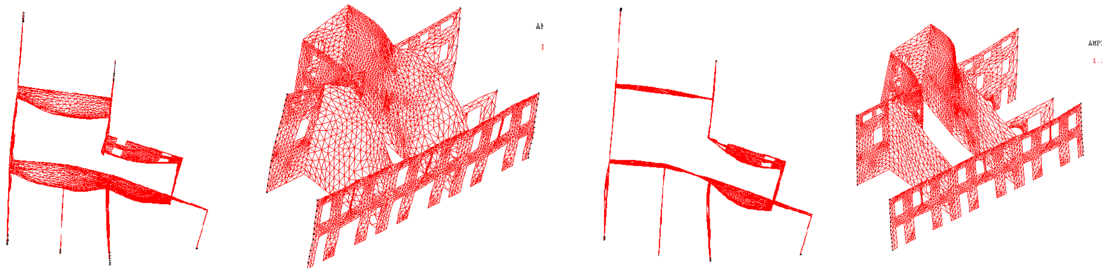
One of the relevant findings to be analyzed is the coherence function between different seismograph signals in each measuring station. For each station and according to the frequency, the use of this dimensionless parameter makes it is possible to observe the trend of the linearity degree and the noise influence on the relation between the seismograph signals. This allows the detection of vibration modes for frequencies whose coherence shows high values (larger than approx. 0.7).

After a careful analysis of the obtained results, it was possible to assess for both houses the two first vibration modes used for calibrating the numerical models. This calibration was done by adjusting the elasticity moduli of the stone masonry walls. A good agreement between numerical and experimental results
for those vibration modes was achieved, as can be seen on Fig. 19 and Fig. 20 for house no.15 and Fig. 21 and Fig. 22 for house no. 16. The values considered for those elasticity moduli are presented on
Table 3.



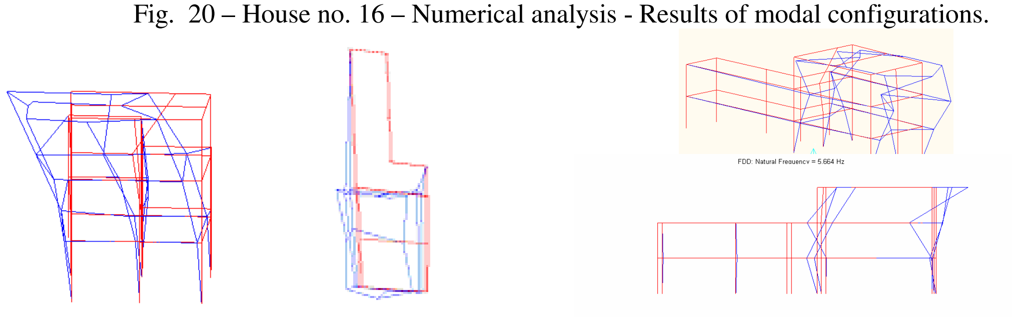
a) 1st Mode: f = 4.00 Hz b) 2nd Mode: f = 5.18 Hz

Fig. 19 – House no. 15. Experimental modal analysis - Results of modal configurations.



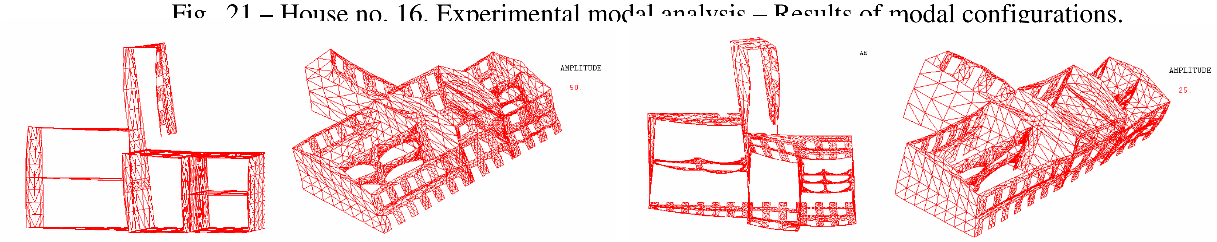
a) 1st Mode: f = 4.06 Hz b) 2nd Mode: f = 5.20 Hz

Fig. 20 – House no. 16 – Numerical analysis - Results of modal configurations.



a) 1st Mode: f = 4.39 Hz b) 2nd Mode: f = 5.66 Hz

Fig. 21 – House no. 16, Experimental modal analysis – Results of modal configurations.



a) 1st Mode: f = 4.34 Hz b) 2nd Mode: f = 5.57 Hz

Fig. 22 – House no. 16 – Numerical analysis - Results of modal configurations.

Table 3– Houses no. 15 and 16. Stone masonry properties

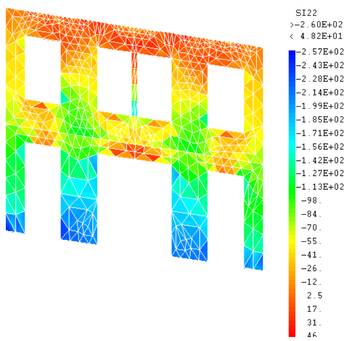
Walls	Material	ρ (ton/m^3)	E (GPa)
House 15	Stone masonry	1.8	0.65
House 16	Stone masonry	1.8	0.8

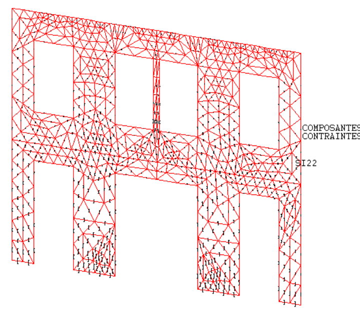
According to the frequencies and mainly to the corresponding modal configurations, it was proved that the wooden floor has diaphragm behaviour with a slight in plan stiffness, restricting the walls flexure along the vertical axis. Therefore, there is evidence that in what the storey stiffness degree is concerned, these buildings behaviour is framed between B and C modellings (referred to on a previous section). The wooden floor maintenance, the interfaces with beams and supporting walls are, therefore, key conditioning factors of dynamic behaviour for this kind of structures, since they play a very important role in the bracing of bearing walls.

SEISMIC ANALYSIS

According to the results of the dynamic identification and the estimated material properties, a seismic analysis was made for both houses (no. 15 and no. 16) based on the accelerograms actually recorded on July, 9th 1998 (Fig. 3). The bases for the structural analyses of seismic response were mentioned above, namely concerning the linear elastic behaviour assumption. Some of the results obtained are shown in this

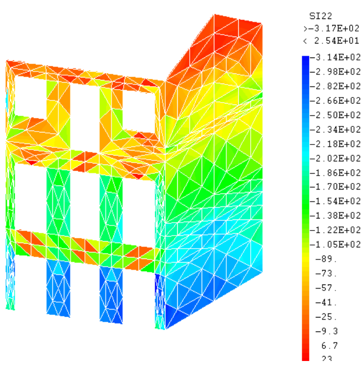
section, mainly focusing on the structural effect amplifications due to the seismic action and on the lateral drifts of bearing walls.
In this context, Fig. 23 and Fig. 24 show a group of principal stress results obtained from static analyses of one structural element of each house, due to the action of vertical static (dead plus live) loads. The main purpose is to compare these results with those arising from the inclusion of the recorded earthquake effect, thus allowing estimating the amplification factor.
The results of vertical static loads show maximum stress values of 257 kPa and 314 kPa, respectively in houses no. 15 and no. 16, and are in good agreement with commonly installed stresses on this type of walls and are lower than their estimated resistance.
The corresponding results of the seismic action are shown in Fig. 25 and Fig. 26, and consist of the maximum envelope of principal stress distributions rather than the stress pattern for a given time instant.

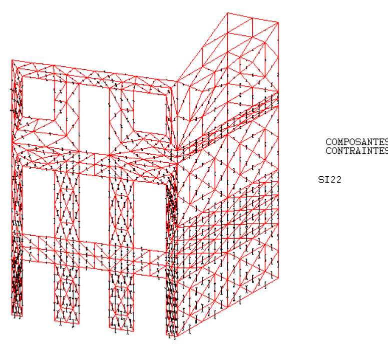




a) Principal stresses (kPa) b) Principal stresses - Directions

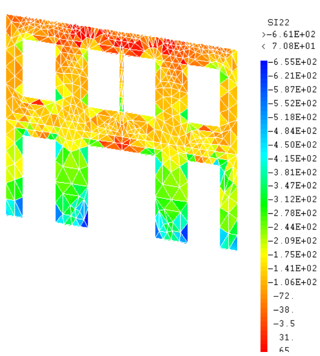
Fig. 23 – House 15 - Principal Stresses (Compression) due to static loads – Frontal Wall

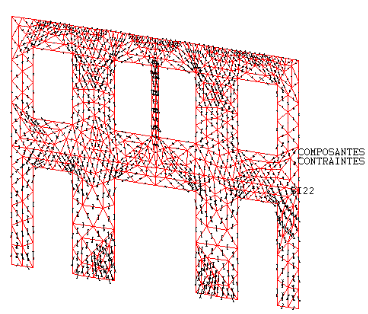




a) Principal stresses (kPa) b) Principal stresses - Directions

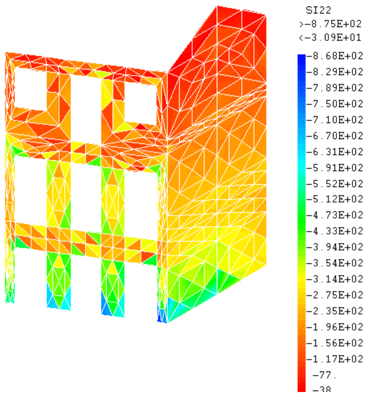
Fig. 24 – House 16 - Principal Stresses (Compression) due to static loads – Frontal Wall



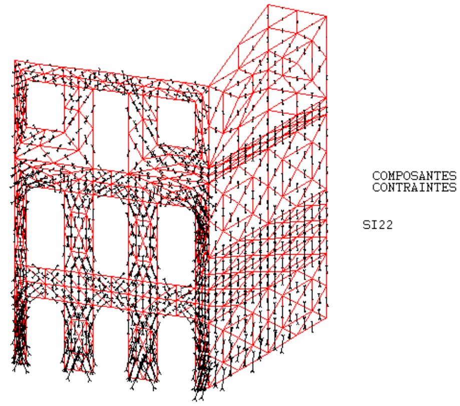


a) Principal stresses (kPa) b) Principal stresses - Directions

Fig. 25 – House 15 - Principal Stresses (Compression) due to seismic loads – Frontal Wall



a) Principal stresses (kPa)

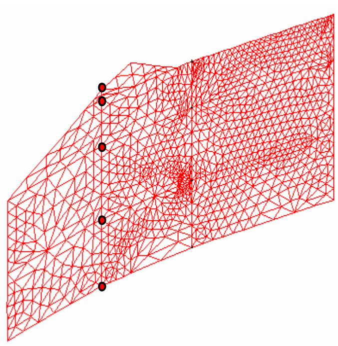


b) Principal stresses - Directions

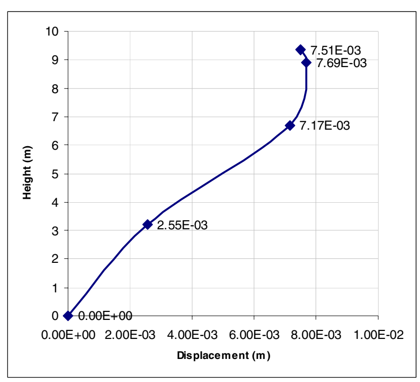
Fig. 26 – House 16 - Principal Stresses (Compression) due to seismic loads – Frontal Wall

The comparison of static and seismic loading effects, show that principal compressive stresses become increased by a factor of about 2.4 in house no. 15 whereas for house no. 16 this factor is slightly higher, reaching 2.76. For the reasonably quality of the stone masonry of the houses under analysis, these increased compressive stresses are still compatible with their strength. However, for tensile stresses such amplification level becomes critical in comparison with the very low tensile strength of these masonry walls, and leads to visible and extensive cracking throughout the walls as observed after the earthquake occurrence.

Another important structural feature to be analyzed refers to the out-of-plane wall displacements, along the wall height. In particular, estimates of drifts between floors are of special importance since they are strongly related to the potential pathologies that can occur in the walls. Therefore, drift values were calculated for both lateral and front walls of houses no. 15 and no. 16, and the corresponding results are listed and plotted in Fig. 27 to Fig. 30.



a) Considered points

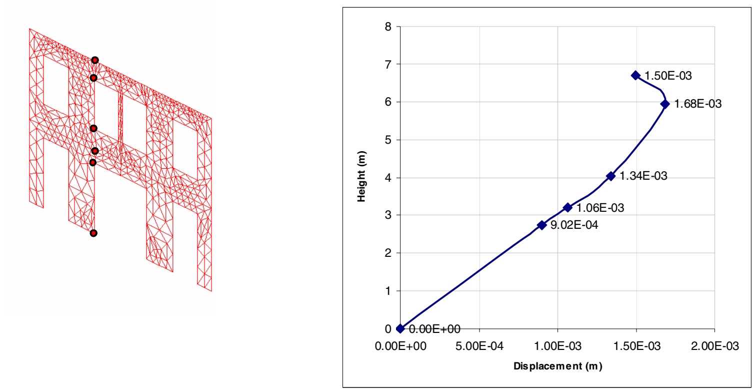


b) Perpendicular displacements

DRIFT = $(d_a-d_b)/(h_a-h_b)$	
Floor 2 - Floor 3	0.02%
Floor 1 - Floor 2	0.13%
Floor 0 - Floor 1	0.08%

c) Drift

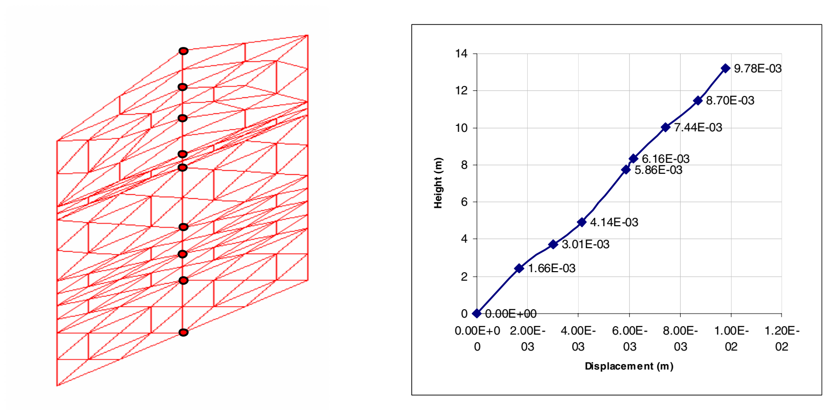
Fig. 27 - House no 15 - lateral wall – displacements



DRIFT = $(d_a-d_b)/(h_a-h_b)$	
Floor 1 - Floor 2	0.03%
Floor 0 - Floor 1	0.03%

a) Considered points b) Perpendicular displacements c) Drift

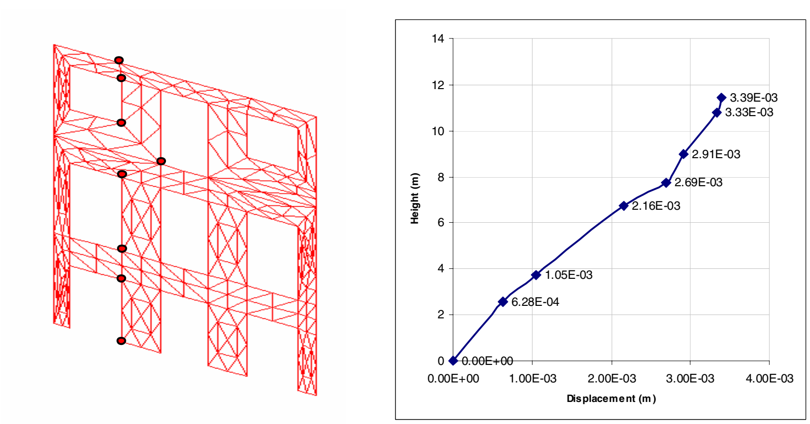
Fig. 28 - House no 15 - front wall - displacements



DRIFT = $(d_a-d_b)/(h_a-h_b)$	
Floor 2 - Floor 3	0.08%
Floor 1 - Floor 2	0.07%
Floor 0 - Floor 1	0.08%

a) Considered points b) Perpendicular displacements c) Drift

Fig. 29 - House no 16 - lateral wall – displacements



DRIFT = $(d_a-d_b)/(h_a-h_b)$	
Floor 2 - Floor 3	0.02%
Floor 1 - Floor 2	0.04%
Floor 0 - Floor 1	0.03%

a) Considered points b) Perpendicular displacements c) Drift

Fig. 30 - House no 16 - front wall - displacements

Results show that, in both houses, the drift values are higher in the lateral walls than in the front ones. This is in agreement with the fact that, for the considered seismic action, the XX direction (the longitudinal one, along the larger block dimension) is mainly mobilized; this can be confirmed by observing that the main vibration modes of either the single houses or the global block refer to the longitudinal direction.

It should be noted that, for the lateral walls, the obtained drift values are close to (and even exceed) the 0.1% value above which the masonry is currently expected to exhibit some important damage under lateral loads such as the seismic action. Finally, it is worth recalling that such deformations are very likely to be increased if a more adequate analysis is performed, where the non-linear behaviour of masonry is taken into account.

CONCLUSIONS

This paper presented some results of one of the stages of a comprehensive research on the seismic behaviour of a specific block in the Horta town. Its purpose is to check and characterize the most conditioning factors to perform an adequate modelling capable of simulating the in-situ material and structural reality.

The structural elements within the block show mechanical characteristics with significant differences, either locally or as a global structure, since the block comprises several buildings that, despite the same construction type, have different qualities. These heterogeneities can only be accurately quantifiable through tests; this is the reason why this paper was based on an experimental campaign of ambient vibration testing.

Despite some sensitivity limitations of the testing equipment when used in very stiff structures, the obtained measurements gave satisfactory results for numerical model calibration. These allowed realistic estimates of the mechanical characteristics of materials and an adequate assessment of modelling strategies for some of its elements (such as wooden floors). These results were of utmost importance to continue with this research, since they were implemented in the numerical modellings of all block buildings, in order to achieve a trustworthy evaluation of the seismic response. Some results of this seismic response were also reported herein, with particular emphasis on the stress amplification and on the maximum out-of-plane wall drift values due to the seismic action.

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