

EXPERIMENTAL INVESTIGATION ON THE SEISMIC BEHAVIOUR OF NORTH EUROPEAN MASONRY HOUSES

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SUMMARY

This paper presents the results of shaking table tests performed on two full-scale masonry houses typical from North-European countries. The main objective of the experimental study is to assess the seismic behaviour of such houses for situations of low to moderate seismicity and to investigate the actual effect of some reinforcing details.

1. INTRODUCTION

Seismicity level in North-European countries is obviously lower than in other well identified seismic countries like Greece or Italy. Significant earthquakes can however occur, even if they are more spaced in time.

Layout of the walls and standard constructional details of simple non-engineered North-European constructions can be really unsafe and lead to dangerous partial collapse (fall of walls, opening of cracks) and even to total collapse for the maximum earthquake foreseen in such countries. Many of these damages have been observed for instance after the earthquake in Liège (Belgium) in 1983 (See Figure 1).

It comes then that architects have usually the choice between two main options: either to design without accounting for possible earthquakes, and therefore produce unsafe structures, or to realize reinforcements that are of common practice in high seismicity areas, but excessive and uneconomic in North-European countries with low to moderate seismicity level.

In this context, numerical and theoretical studies have been performed in University of Liège to estimate the forces transferred from floors to walls and from walls to walls for an earthquake with moderate intensity, assuming that the houses behave as rigid boxes. Technical detailing allowing an efficient transfer of these forces has then been developed. The outcomes of this study can be found in Ref. [1-3].

An experimental investigation has then been designed in order to assess the efficiency of these technical details in a real situation. It consisted in two shaking table tests on full scale specimens. The first one was realised according to the usual Belgian way of building, while the second one included some additional technical details aiming at improving the seismic behaviour of the house.

The main objective of the present paper is to present the results of this experimental investigation carried out on the 3D shaking table of the National Laboratory of Civil Engineering of Lisbon (LNEC). More than simply comparing the behaviour of both types of construction, the paper also provides interesting results about the seismic behaviour of masonry structures.

2. DESCRIPTION OF THE TEST SPECIMENS

The two tested models are perfectly similar for what regards the dimensions and loading. The first model is built according to traditional Belgian practice (although with Portuguese materials), while the second model includes the technical detailing referred to in the introduction. Figure 2 presents pictures taken during the construction of the reinforced model and shows some of these details: Murfor® (prefabricated wire reinforcement for masonry – Ref. [5]) are placed within the mortar layers (Fig. 2.a), bolted steel pieces are connecting the prefabricated concrete slab elements (Fig. 2.b), steel bars and angles are connecting the elements of the timber floor (Fig. 2.c) and steel plates and bars are anchoring the timber floor in the masonry wall (Fig. 2.d). Mechanical properties of the materials are described in section 3.



Figure 1 – Roof collapse during 1983 Liège earthquake

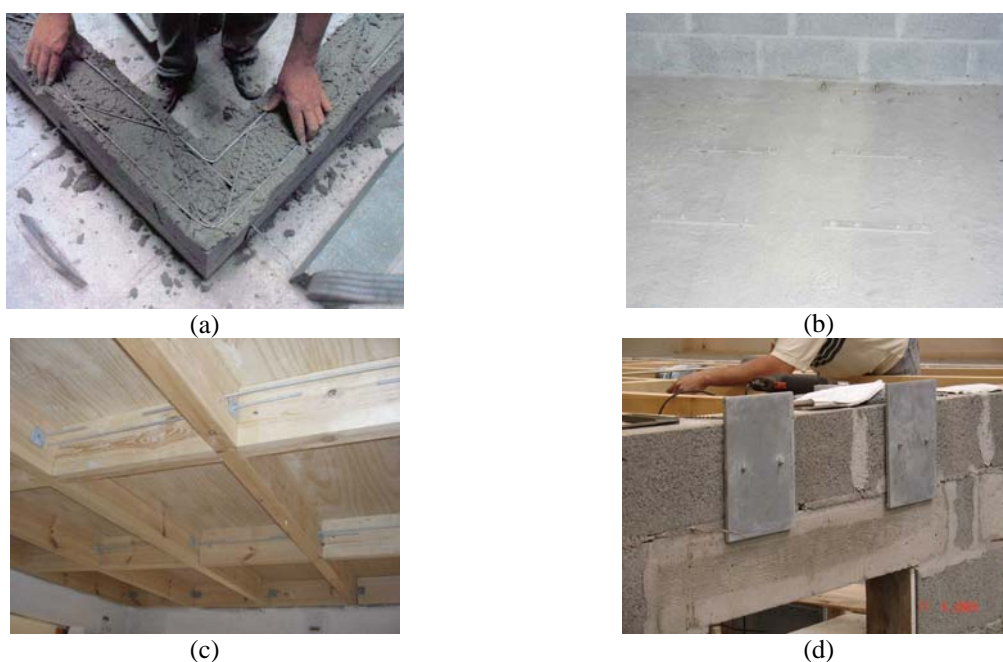


Figure 2 – Additional reinforcing details of the second model

The main limitation in the design of the experimental model was the capacity of the shaking table. The maximum pay load of the shaking table is 40t and the platform has a size of 4.6m × 5.6m. According to these limitations, the characteristics of the models can be summarized as follows:

- The models are full scale small houses comprising one room and two levels (see Figure 3);
- The overall dimensions are 4 × 3.6 × 5.2 m (see Figures 4 and 5) with a total mass of 24.7t (plus 10.7t for the foundation slab);

- The walls are classical Belgian double walls, i.e. an inner structural wall realised in concrete blocks and an external wall in clay bricks. Both walls are connected by steel hooks;
- The first floor is realised with five prefabricated concrete slabs;
- The second floor is realised with timber beams and crossings, with nailed plywood panels.

Figures 4 and 5 present respectively side and plan views of the models.

Sand-bags are placed on both floors to simulate loads associated with the occupancy of the houses in normal use. This overload is equal to 200 kg/m² for the concrete floor and to 50 kg/m² for the timber floor.



Figure 3 – The two models at the end of construction, Model 1 on the right and Model 2 (reinforced house) on the left

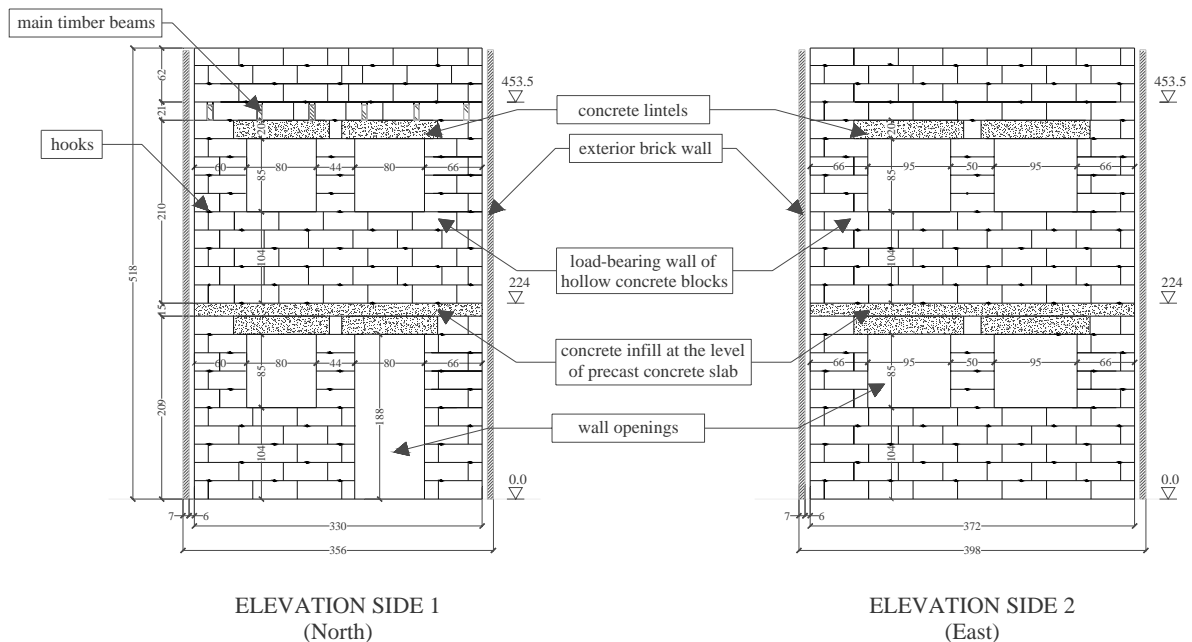


Figure 4 – Sides of the models

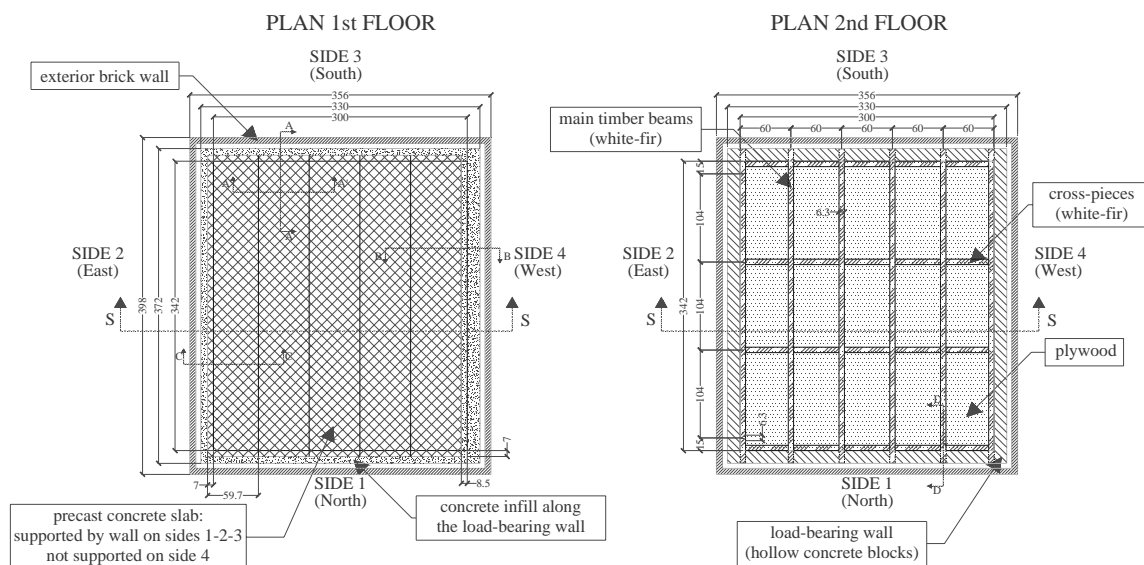


Figure 5 – Plans of the models

3. CHARACTERIZATION OF THE MATERIALS

Some of the materials previously described, considered to be the most relevant for the global seismic behaviour of the models, have been tested to determine their mechanical properties (see Ref. [4]) as will be described next. Starting with the hollow concrete blocks, they have overall dimensions of 200mm×150mm×500mm and 6 vertical holes as shown in Figure 6. A sample of ten specimens was taken to test for the dynamic elasticity modulus according to the EN 14146 norm. Since this norm is not directly applicable to hollow blocks, the dynamic elasticity modulus was estimated through a numerical simulation fine tuned to match the vibration frequency with the average of the measured ones. The main results obtained in this test, mass and dynamic elasticity modulus, are shown in Table 1; special attention is called upon the fact that the equivalent modulus of a block will be less than the one indicated due to the presence of the hollows and depending on the direction considered. The same sample of ten specimens was later tested for uniaxial compressive strength according to the NP EN 772-1:2002 norm and the results obtained are also summarised in Table 1.



Figure 6 – Hollow concrete blocks used in the inner wall

Table 1 – Summary statistics of the tests for longitudinal dynamic elasticity modulus and uniaxial compressive strength on hollow concrete blocks

Statistics	Mass	DEM	Strength
Mean	14.45 kg	27 GPa	4.9 MPa
Standard deviation	0.21 kg	-	0.9 MPa
Coefficient of variation	0.015	-	0.2

Next, the solid clay bricks are prismatic with dimensions of 23cm×11cm×7cm. A sample of ten specimens was taken to test first for the dynamic elasticity modulus according to the EN 14146 norm and later on for mechanical resistance to uniaxial compressive strength according to the NP EN 772-1:2002 norm. Both results are summarised in Table 2.

Table 2 – Summary statistics of the tests for longitudinal dynamic elasticity modulus (DEM) and uniaxial compressive strength on solid clay bricks

Statistics	Mass	DEM	Strength
Mean	3.52 kg	15409 MPa	113.1 MPa
Standard deviation	0.01 kg	2827 MPa	8.7 MPa
Coefficient of variation	0.003	0.18	0.1

The bonding mortar was tested for its bending and compression strength according to the NP EN 196-1 norm. A total of 62 prismatic mortar specimens with 4×4×16 cm were made during the construction of the models. The summary statistics of these tests for the specimens with 28 days or more of age is presented in Table 3 where the size of the samples was of 53 specimens in bending and 106 specimens in compression.

Table 3 – Summary statistics of the bending and compression test on prismatic mortar specimens

Statistics (age ≥ 28 days)	Bending strength	Compression strength
Mean	2.69 MPa	7.80 MPa
Standard deviation	0.49 MPa	1.59 MPa
Coefficient of variation	0.18	0.20

Regarding masonry, 9 wallets were made by the same craftsmen that built the walls of the models and using the same materials, 3 of each of the different types used in the models: (i) hollow concrete blocks and bonding mortar representative of the inner wall of Model 1, (ii) hollow concrete blocks and bonding mortar reinforced with Murfor® (Ref. [5]) in the bed joints representative of the inner wall of Model 2 and (iii) solid clay bricks and bonding mortar representative of the outer wall of both models. The dimensions of the hollow concrete blocks wallets were 101cm×105cm×15cm while the solid clay bricks wallets were 47cm×59cm×7cm. Because 3 of the wallets were damaged before the tests took place, only 6 were tested according to the EN1052-1:1998 norm. Table 4 summarizes the main results obtained in these tests. For each physical measure, two values are given, one according to EC6 and the other one according to EN1052-1. The first column of the table shows the compression strength of single blocks tests.

Table 4 – Compression strength (f_k) and elasticity modulus (E) of wallets

Type of wallet	f_b [Nmm ⁻²]	f_k [Nmm ⁻²]		E [kNmm ⁻²]	
		EC6	EN 1052-1	EC6	EN 1052-1
Block wallet	4.9	2.9	2.7	2.9	4.9
Block wallet with Murfor®	4.9	2.9	2.6	2.9	8.4
Brick wallet	113.1	31.7	11.4 ¹⁾	31.7	12.4 ¹⁾
	45.0 ²⁾	16.6		16.6	

1) Values obtained from a single test

2) Value given by the manufacturer

Last but not least, the concrete was tested for compression strength according to the LNEC E 226 specification. Since the concrete used was prepared and poured at the same time for both models a sample of only 3 cubic specimens with 15cm edge was made. The summary statistics of these tests are presented in Table 5.

Table 5 – Summary statistics of the compression test on cubic concrete specimens

Statistics (age ≥ 28 days)	Compression strength
Mean	25.17 MPa
Standard deviation	0.76 MPa
Coefficient of variation	0.03

4. INSTRUMENTATION AND TESTING PROCEDURE

4.1. Instrumentation

The models were instrumented with:

- 32 accelerometers (A1 to A32). The position of these accelerometers is plotted on Figure 7;
- 12 displacement transducers (D1 to D12), measuring the relative displacements of the corners of each wall (including the two floors). These diagonal transducers are plotted on Figures 7 and 8;
- Optical devices measuring the absolute horizontal displacement of the floors;
- Displacement transducers and accelerometers measuring displacement and acceleration of the shaking table in NS and EW directions.

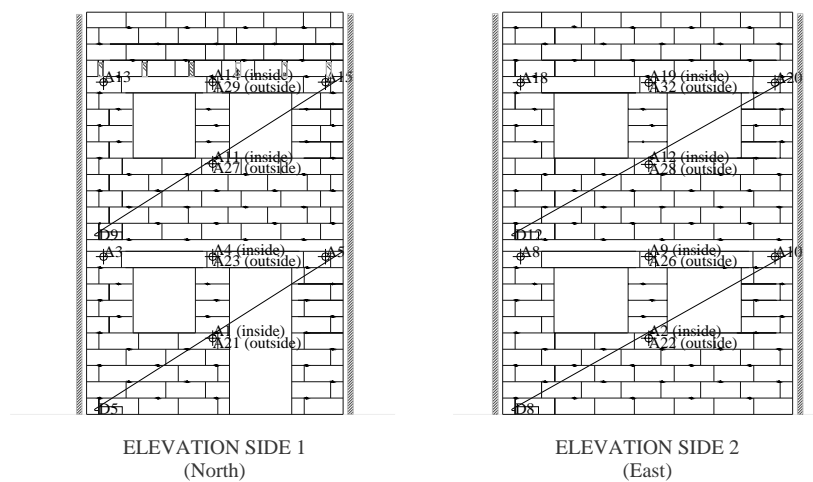


Figure 7 – Instrumentation of the North and East walls

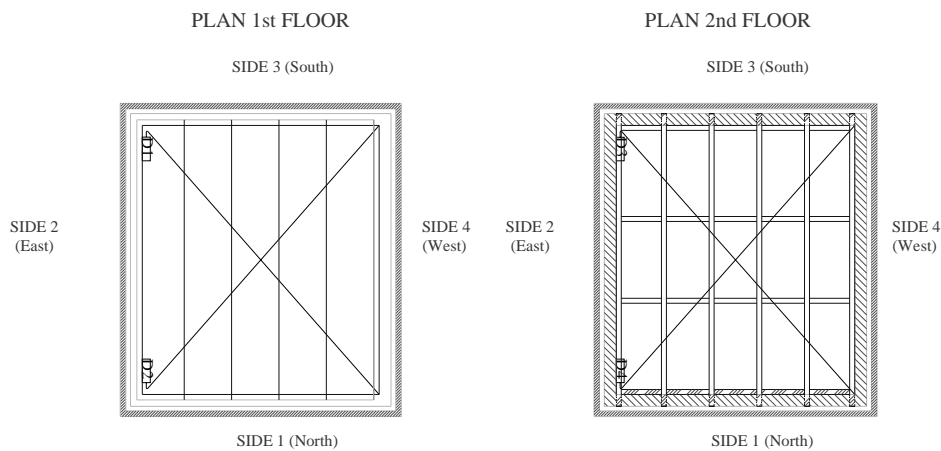


Figure 8 – Instrumentation of the floors

4.2. Testing procedure

In the shaking table tests were used fundamentally two kinds of input signals, one for the identification of the dynamic properties of the structure and the other one for the simulation of the earthquake itself. The 1st kind of input signal is a characterisation signal meant to be used in the identification of the vibration frequencies and modes. The 2nd kind of input signal is the earthquake signal that simulates the desired seismic demand. This signal, based on the Herceg Novi record of the April 15th 1979 Montenegro earthquake (M7.1), has 15s duration and two horizontal acceleration components scaled to 1g amplitude (PGA) as shown in Figure 9.a-b; the corresponding acceleration response spectra is also visible in Figure 9.c-d. The frequency content of the original signal was already fit approximately to the EC8 design spectra envelope.

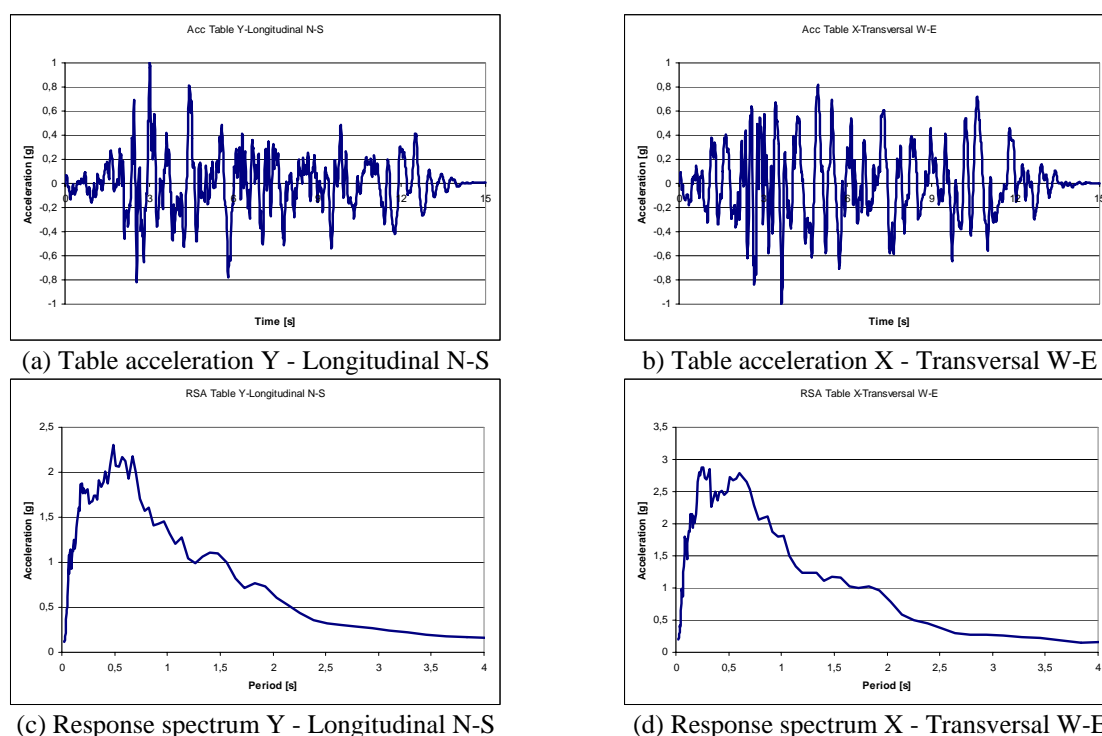


Figure 9 – Acceleration time series scaled to 1g PGA and corresponding response spectra

For both models, the testing procedure was a succession of stages comprising the three following steps:

- Identification step in EW direction;
- Seismic shake in NS direction;
- Seismic shake in EW direction;

In each stage, the amplitude of the seismic shakes is increased until collapse of the model. Table 6 summarizes the whole loading sequence. The values of PGA in Table 6 were obtained from the time-history of accelerations recorded at the level of the shaking table.

Table 6 – Load sequence – PGA

Ident.	Model 1		Ident.	Model 2	
	NS	EW		NS	EW
Id00	S00 - 0.08g	S01 - 0.04g	Id00	S00 - 0.07g	S01 - 0.04g
Id02	S02 - 0.14g	S03 - 0.08g	Id02	S02 - 0.12g	S03 - 0.09g
Id04	S04 - 0.23g	S05 - 0.21g	Id04	S04 - 0.22g	S05 - 0.20g
Id06	S06 - 0.46g	S07 - 0.39g	Id06	S06 - 0.46g	S07 - 0.39g
Id08	S08 - 0.65g	S09 - 0.45g	Id08	S08 - 0.54g	S09 - 0.42g

In order to fully understand the following results, it is also important to mention that for the second model (reinforced house), an unexpected re-initialization of one of the jacks occurred during the initializing process of the hydraulic system. This resulted in a significant shock on the model and produced an important longitudinal crack over the whole width of the model at the base of the first floor. Therefore, all the results obtained on the second model must be analyzed keeping in mind this pre-damaged situation.

5. TEST RESULTS

5.1. Frequencies and damping coefficient

The identification is performed by estimating the transfer function between the shaking table acceleration and the accelerometers placed on the structure. The transfer function is evaluated as the ratio between PSD of the shaking table acceleration and PSD of the model accelerations. In order to obtain a precise value of the natural frequencies, a cubic spline function is fitted on the discrete transfer function. The frequency corresponding to the maximum of the fitted curve is considered as the natural frequency of the system. This method also allows the additional identification of the damping ratio through a half-power band-width approach. The identification is made on the basis of the 'identification' stages (low acceleration level, frequency content more or less corresponding to a white noise, duration of 100s), which were performed only in the EW direction.

In order to get complementary information, the same procedure is also applied on the seismic stages. However for these cases, the results have to be considered carefully since (i) the signal is not stationary, (ii) the duration of the signal is equal to 32s and the consequent resolution of the transfer function is thus less refined and (iii) for the last stages, the behaviour of the system is non linear. Further, in the seismic stages, it is not possible to identify properly the damping ratio, due the rather high value of the frequency step of the transfer function and to the consequently bad conditions for being able to apply the half-power bandwidth method.

Figure 10 presents examples of the calculated and fitted transfer functions for the first model, respectively corresponding to the first EW seismic shake (M1 – S01) and to the identification stage immediately following this seismic shake (M1 – Id02) in the vicinity of the peak of the transfer function. Table 7 gives the estimated natural frequencies and, for the identification stages, the level of damping.

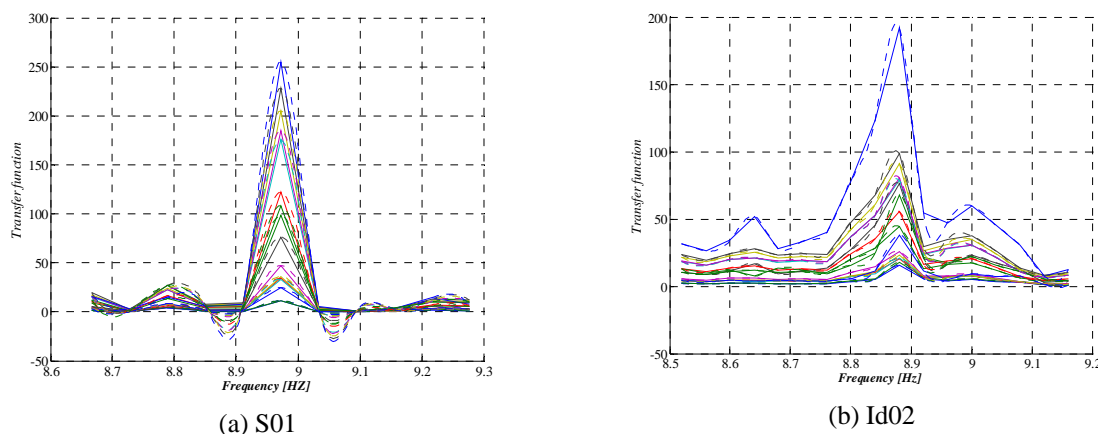


Figure 10 – Examples of transfer functions

Table 7 – Frequencies and damping

Model 1				Model 2			
Id – Freq [Hz]	Id – damping [%]	NS – Freq [Hz]	EW – Freq [Hz]	Id – Freq [Hz]	Id – damping [%]	NS – Freq [Hz]	EW – Freq [Hz]
8.9	3.6	8.6	9.0	6.0	4.2	6.0	6.3
8.9	2.8	7.8	8.9	6.3	7.3	5.7	6.4
8.7	3.0	7.9	7.3	5.9	4.0	5.4	5.5
7.8	5.0	7.4	5.0	5.9	3.7	5.4	6.1
6.6	2.9	5.3	-	6.3	4.1	4.6	6.2

The analysis of the results in Table 7 shows that:

- For the non reinforced model (model 1), the frequency decreases progressively during the loading process, while it remains more or less stable for the reinforced one (model 2). However, the initial frequency of this latter model is much smaller than the frequency of the first one, which can be related to the initial shock reported above;
- The damping ratio is rather stable for both cases, between 3 and 4%. The difference between both models can not be considered as significant, as the variability of the results for one model is greater than the variability between both models;
- For small earthquake intensity, the identification performed on the basis of the EW seismic signal provides results that are comparable with those obtained with the identification signal. When the PGA increases, the identification on the earthquake signal tends to give frequencies smaller than those obtained with the identification signal;
- For low PGA level, the frequencies obtained in both directions are similar, even if slightly smaller in the NS direction. No clear difference can be identified between the elastic behaviour in both directions.

5.2. Accelerations

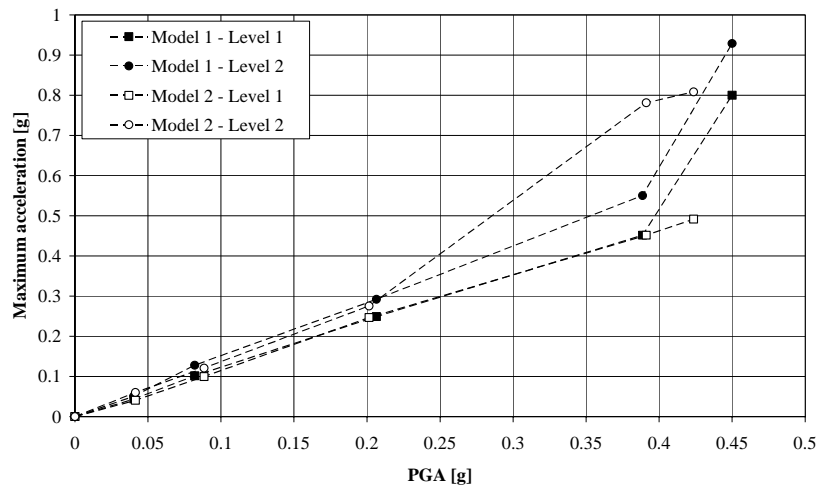
Figure 11.a-b presents the evolution of the maximum acceleration of both floors when the PGA increases, respectively for seismic input in NS and EW directions. The acceleration of the floor is taken as the average of the maximum acceleration recorded on all the accelerometers of the considered floor.

It can be observed that the difference between models 1 and 2 is rather small. The reinforcement devices are indeed designed to provide additional strength at the connection between the walls but not to modify the stiffness of the structure.

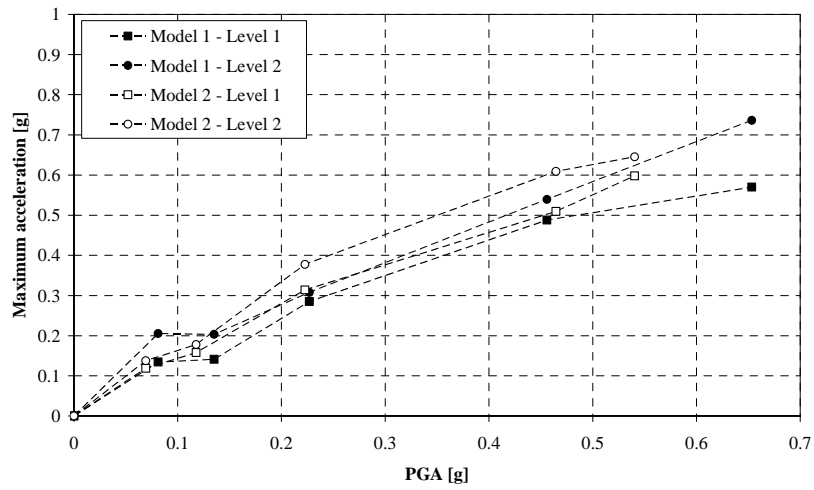
Consequently, Figures 12.a-b proposes curves obtained by averaging the 'PGA-Maximal acceleration' curves obtained for both models respectively for the behaviour in NS and EW direction. On these figures, additional straight lines defined by the origin of axes and by the first experimental dot are also plotted. These additional curves can reasonably be considered as representative of an elastic 'PGA-Maximal acceleration' relation, assuming that the behaviour of the model for a PGA equal to 0.1g is elastic.

From Figure 12.a, it can be concluded that the behaviour in the EW direction is elastic up to the collapse of the model (behaviour factor q equal to 1), this collapse being absolutely non ductile. Figure 13.a illustrates this collapse mode. It is initiated by a rocking of the central panel (1). The load is then transferred to the two external panels (2), which are too weak to sustain this overload, leading thus to the total collapse of the model. It must be noted that the central panel does not fulfil the requirement of Eurocode 8 regarding the ratio l/h between the length of the wall and the maximum height of the adjacent openings ($l/h = 0.25 < 0.4$). This wall should thus normally not be considered as primary seismic element. However it can also be noted that the behaviour of both models was still admissible for the PGA level corresponding to the design earthquake action defined for Belgium (i.e. 0.2g).

On the contrary, Figure 12.b shows that the behaviour in the NS direction exhibits a certain level of ductility. The maximum acceleration recorded on the model is indeed not a linear function of the PGA. The ratio between the elastic extrapolation and the actual measured maximum acceleration for the last point of the curves (PGA = 0.6g) gives an estimation of the behaviour factor that could be considered for the analysis of the structure. On the basis of the results related to the first level, q is equal to 1.4, while it is equal to 1.55 on the base of the results related to the second level. For this NS direction, the minimum l/h ratio is equal to 0.53 (> 0.4) and the entire wall can then be considered as a primary seismic element. Figure 13.b shows the collapse mode in this direction: it consists in an excessive shear of the panels situated at the base of the walls and at the level of the first floor.

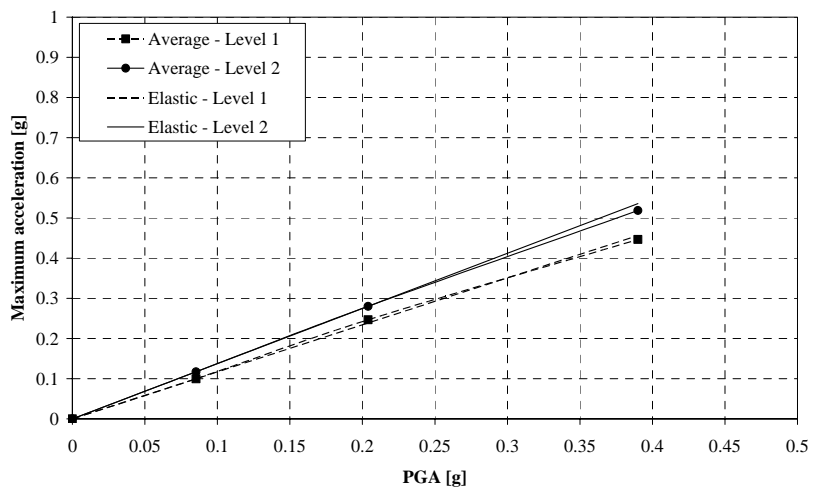


(a) NS behaviour



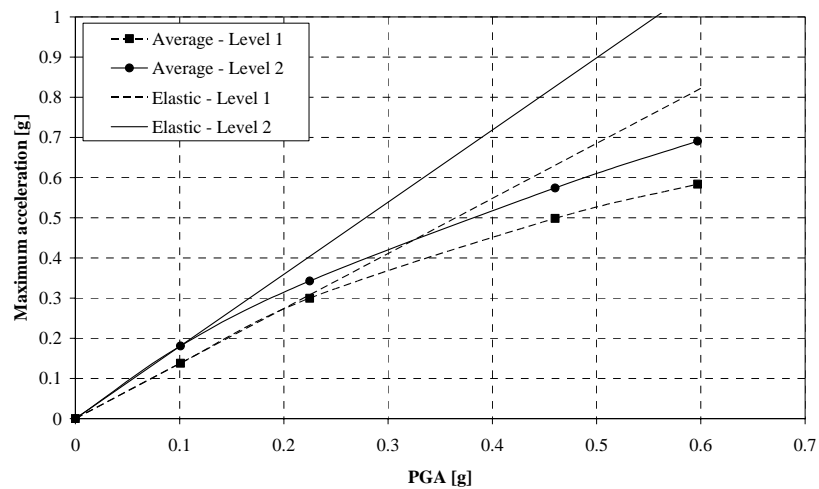
(b) EW behaviour

Figure 11 – Maximum acceleration of the model



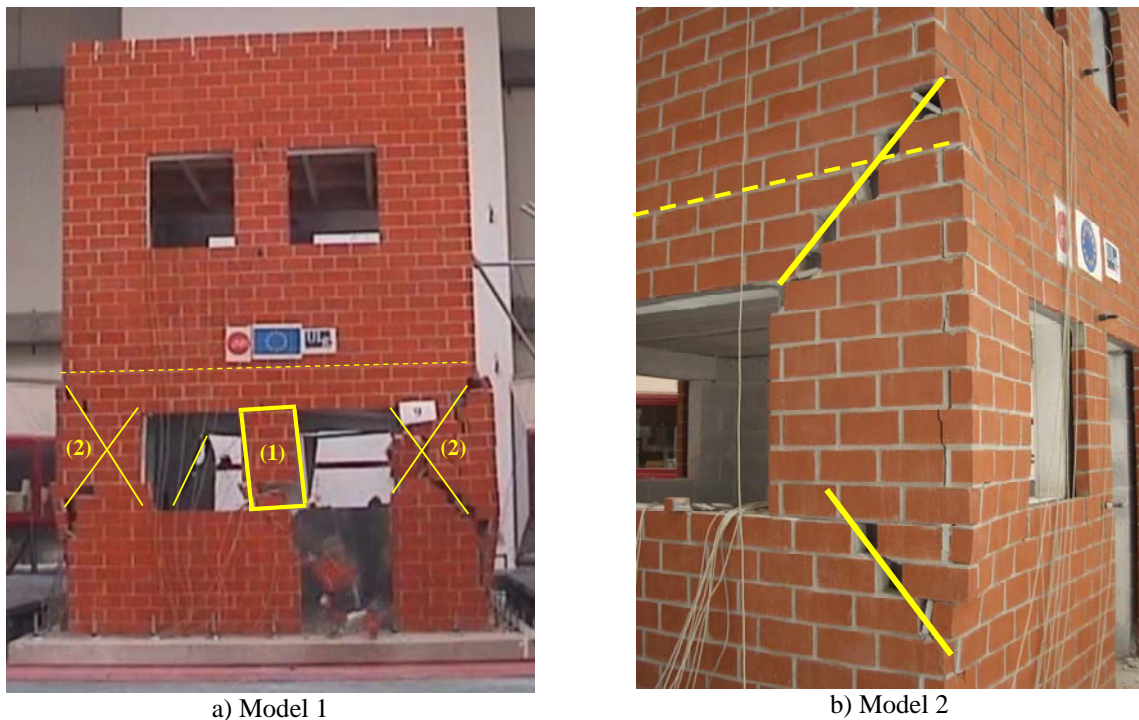
(a) EW behaviour

Figure 12 – Curves for the estimation of the behaviour factor



(b) NS behaviour

Figure 12 (continued) – Curves for the estimation of the behaviour factor



a) Model 1

b) Model 2

Figure 13 – Collapse modes of the model

5.3. Displacements

Detailed information about the interpretation of the displacements measurements can be found in Ref. [3]. Figure 14 presents the most significant result obtained from these measurements. It compares the standard deviation of the displacements measured by transducers D1 to D4 during the successive seismic shakes (S00 to S07, defined in Table 6), respectively for non reinforced (D_NR) and reinforced (D_R) models. D1 and D2 correspond to elongation of the diagonals of the concrete floor while D3 and D4 correspond to elongation of the timber floor (See also Figure 8).

It can essentially be observed that (i) the reinforcement devices included in the second model have a real beneficial effect on the behaviour of floors, (ii) the behaviour of the reinforced model starts to deteriorate from the third shaking level S04, which corresponds actually to the maximum earthquake level for Belgium and (iii) the constructional improvements seem more efficient against earthquakes acting in the EW direction.

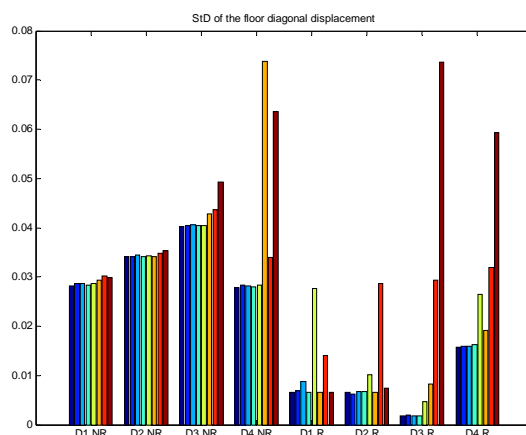


Figure 14 - Comparisons of the standard deviation of the diagonal displacements of the floors

6. CONCLUSIONS

In conclusion, it can be stated that the values proposed by EC8 for the behaviour factor of unreinforced masonry ($q=1.5$) and for the limit on the ratio l/h between the width of the panels and the height of the adjacent openings are confirmed by the experimental results. Furthermore, the devices proposed to improve the seismic behaviour are found efficient for the range of seismicity level for which they are intended in the sense that they limit the local displacements of the floors and thus reduce the local damages. However their impact on the global structural behaviour of the models remains negligible.

7. ACKNOWLEDGEMENTS

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