Modal testing of an unreinforced masonry house

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ABSTRACT: There are a significant number of uncertainties in finite element models of unreinforced masonry structures related to the modelling assumptions and the properties of local materials. Therefore, it is necessary to implement calibration techniques for these models. Modal testing is a good option for assessing the dynamic properties of the structure. The experimental data is used to verify and improve the predicted response obtained by finite element model. The study presents the modal testing of a full-scale physical model of an unreinforced masonry house. The structure is tested under three different excitations: an impact by a calibrated hammer, a random excitation induced by a calibrated hammer, and a stepped-sine excitation induced by a shaker. In addition, an operational modal test has been performed using ambient and random excitations. Two different methods are used for system identification: peak picking and stochastic subspace identification. The results of this research will be used in future studies for updating the model.

1 INTRODUCTION

Masonry is basically a composite, anisotropic and non-homogeneous material. It is compounded of masonry units (bricks) and mortar joints. In general, masonry behaviour depends on the mechanical properties of its components, the interfaces between them, the arrangement of the bricks and the interaction with the others structural members and materials used in the building (concrete frames, steel or timber beams and columns and timber floors).

Numerical modelling of masonry structures is usually a very computationally demanding procedure. The high numerical cost is related to the intrinsic complexity of masonry (bricks connected by mortar joints) that requires a large number of degree of freedoms (Giordano et al., 2002) and excludes typical simplifications (e.g. rigid diaphragms and ideal connections) applied in modelling of other kind of structures. Another reason for this complexity is that the material constitutive models are not well defined, especially in the non-linear range.

So far, the numerical models have mainly been validated by studies based on structural component behaviour (e.g. a single wall or pier). However, a validation at system level (entire building or sections of a building) is not available. Adequate techniques to validate numerical models can be a significant contribution, because they provide a powerful tool to assess and predict the performance of URM structures. Two promising techniques for this purpose are modal testing and model updating. Modal testing is used for assessing the dynamic properties of the structure, such as, natural frequencies, mode shapes and damping factors. These properties are utilized to verify the degree of accuracy between the numerical model and the measured response of the structure. The measured response is employed as a target condition in the process of improving the numerical model (model updating). The updated model can then be used to predict the performance of the structure under different loading, for instance, earthquakes or other induced vibrations. Hitherto, only a few studies refer to the analysis of masonry structures (De Sortis et al., 2005; Ramos et al., 2005). We expect that this study will contribute to developments in this direction.

1.1 Motivation and objectives

Unreinforced masonry (URM) buildings is one of the oldest type of constructions, however the overall seismic performance of these structures are still not well understood. In general, URM buildings have performed poorly in past earthquakes, being responsible for high economical losses and death toll. However, their architectural and historical value is high. They represent an important part of the heritage building stock in many countries and, therefore, their preservation is desirable. Consequently, a number of initiatives (Bruneau, 1994; Griffith, 2008; Ingham, 2008; Lagomarsino, 2006; Lourenço, 2008) have been promoted in order to achieve a better understanding of URM structures behaviour and to develop proper retrofitting techniques.

In New Zealand, URM buildings were the most common type of commercial construction in the late 19th century and early 20th century. Their popularity began to decline after the Hawke's Bay earthquake (3 February 1931) because of their poor seismic performance (Ingham, 2008). Nowadays, it is estimated that around 3500 buildings of this type still remain throughout the country (Figure 1 and Figure 2). Recent studies (Russell and Ingham, 2008a; Russell and Ingham, 2008b) provide an extensive classification and description of URM building typologies, covering most URM construction in New Zealand.

Figure 1: Cuba Street, Wellington, New Zealand.

Figure 2: Jervois Road, Auckland, New Zealand.

The study presented here introduces the modal testing of a full-scale model of an URM house. An experimental modal analysis (EMA) is performed using three different excitations: an impact induced by a calibrated hammer, a random excitation induced by a series of impact with the calibrated hammer and a stepped-sine excitation induced by linear electro-dynamic shakers. Also, an operational modal analysis (OMA) is conducted using the structural response under ambient and random excitation. The aim of these studies is to contrast different system identification procedures applying different types of excitation. Another objective is to use the modal properties extracted from experiments for updating the numerical model. However, no results related to this second objective are presented, because this is a work still in progress.

2 SPECIMEN

The physical specimen for this experiment corresponds to an almost full-scale unreinforced masonry house model (Figure 3). This house was constructed in the test hall of the Civil Engineering Department at the University of Auckland. The clay bricks used in this experiment were obtained from demolition sites of old masonry buildings. The mortar has a cement:lime:sand ratio equal to 1:2:9.

The house has 4 m × 4 m in plan. The north, east and west walls have 2.2 m height and 230 mm thick (two leafs of bricks), whereas the south wall has 1.9 m height and 110 mm thick (one leaf of bricks). The bricks follow a common bond pattern (header course at every fourth course). The east and west walls have one opening simulating windows and the north wall have two openings simulating a window and a door. There are no openings in the south wall (Figure 5).

At a height of 1.60 m a rigid timber diaphragm was built consistent in six equally spaced joists (45 mm × 140 mm) supported by the interior leaf of the east and west walls. These joists are connected by four equally spaced lines of blockings (45 mm × 140 mm). A floor was constructed over the joists. For this purpose, timber boards (32 mm × 140 mm) covered by plywood planks (12 mm) were used (Figure 4).

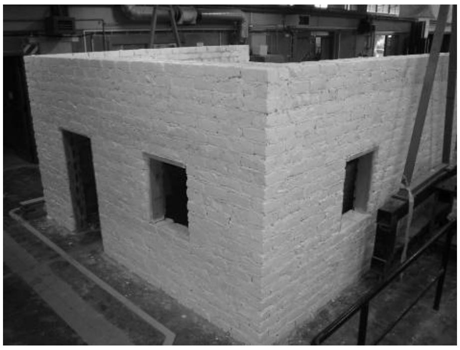


Figure 3: Laboratory specimen

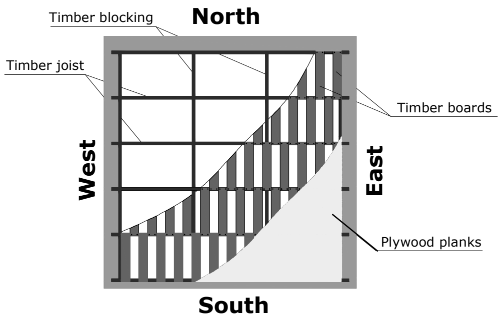
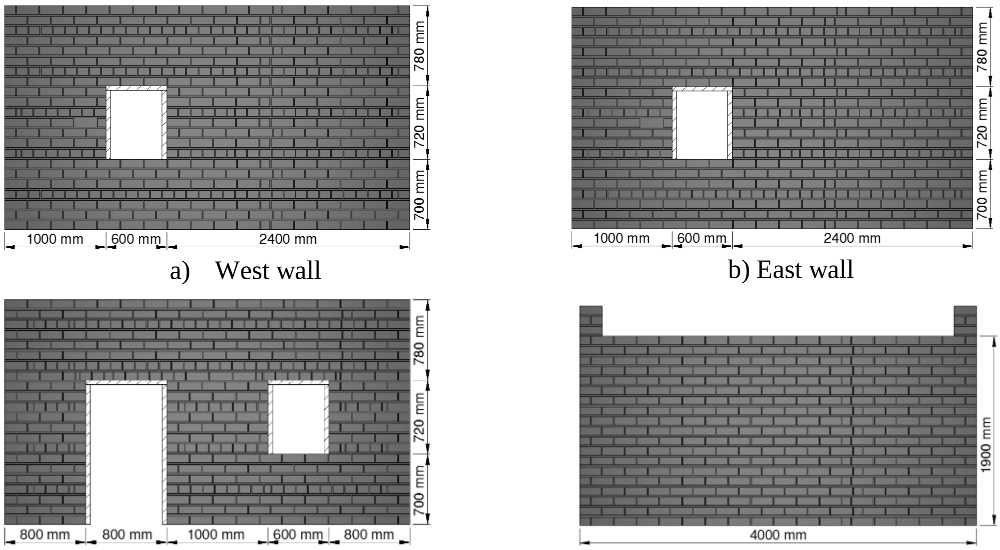


Figure 4: Diaphragm layout



a) West wall

b) East wall

c) North Wall

d) South wall

Figure 5: Laboratory specimen elevations

3 NUMERICAL MODEL

3.1 Finite element model

A finite element (FE) model of the structure was constructed using the software Abaqus/CAE (DS Simulia, 2007). The URM walls were modelled using solid 8-nodes linear hexahedral elements (C3D8I), commonly known as “bricks elements”. The mechanical properties of masonry were obtained from compression tests applied to a set of six prisms elaborated with the same materials used to build the structure under study. The timber joists, blockings, floor boards and plywood planks were modelled using the same brick element (C3D8I), but in this case the mechanical properties were obtained from literature and were compatible with radiata pine timber and plywood. A summary of the mechanical properties used in the model is presented in Table 1. The structure-to-ground connection was considered as pinned. The connections between the structural members were modelled as tie connections, which is appropriate for the nailed connections (joist-floor and joist-blocking), but not necessarily for the joist-walls connections, that depend mainly on contact and friction conditions.

Table 1: Mechanical properties considered in the finite element model

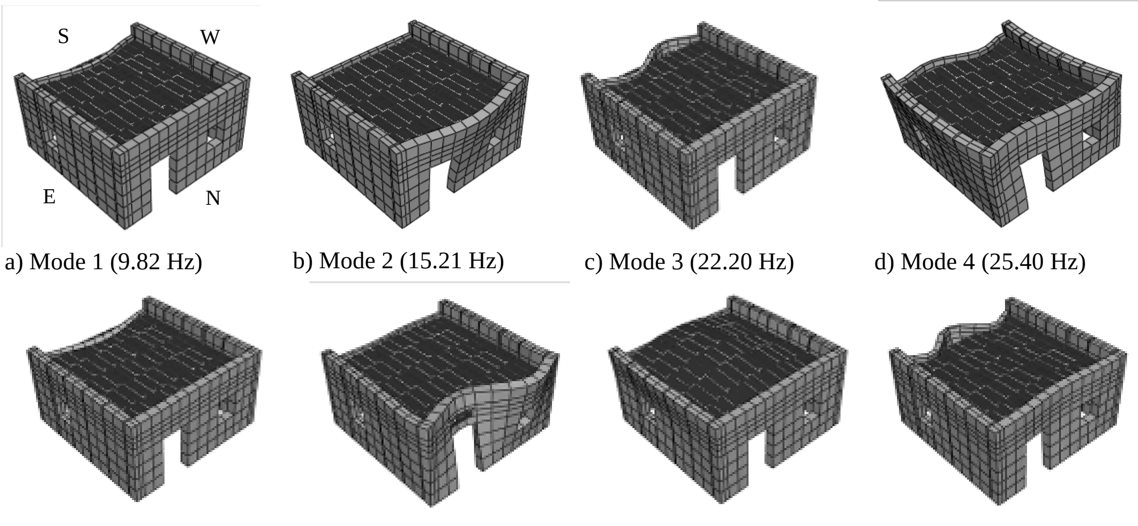
Material	**Density (kg/m^3)**	**E (GPa)**	**Poisson's ratio**
Masonry	1800	1.22	0.2
Timber	545	12.00	0.2
Plywood	545	12.00	0.2

3.2 Modal response

The results of the modal analysis are presented in Table 2 and Figure 6. These results show a predominant response of the south and north walls in the 1st and 2nd mode, respectively. The 4th mode corresponds to an overall EW translational vibration, while the 8th mode is principally an overall NS translational movement. The 7th mode is related to a vertical vibration. The 3rd, 5th and 6th modes have a significant lower mass participation. These three modes can be difficult to detect experimentally.

Table 2 Modal response of the FE model

Mode	**Frequency (Hz)**	**Mass participation**		
		NS dir.	**EW dir.**	**Vertical dir.**
1	9.82	6%	--	--
2	15.21	14%	--	--
3	22.20	--	2%	--
4	25.40	--	60%	--
5	30.94	1%	--	--
6	34.39	--	1%	--
7	36.81	--	--	2%
8	39.82	18%	--	--



a) Mode 1 (9.82 Hz) b) Mode 2 (15.21 Hz) c) Mode 3 (22.20 Hz) d) Mode 4 (25.40 Hz)

e) Mode 5 (30.94 Hz) f) Mode 6 (34.39 Hz) g) Mode 7 (36.81 Hz) h) Mode 8 (39.82 Hz)

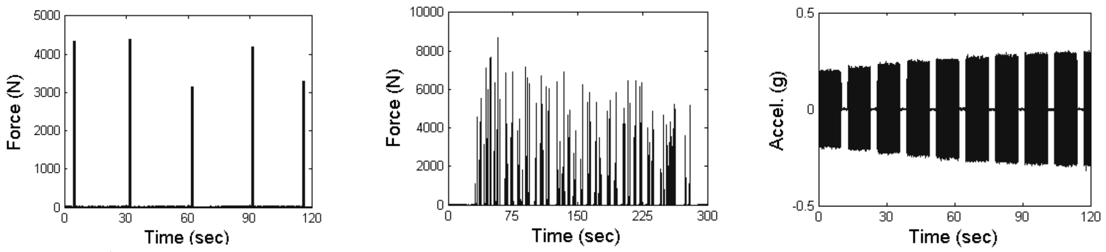
Figure 6: Modal response of the FE model

4 MODAL TEST

4.1 Experimental modal analysis

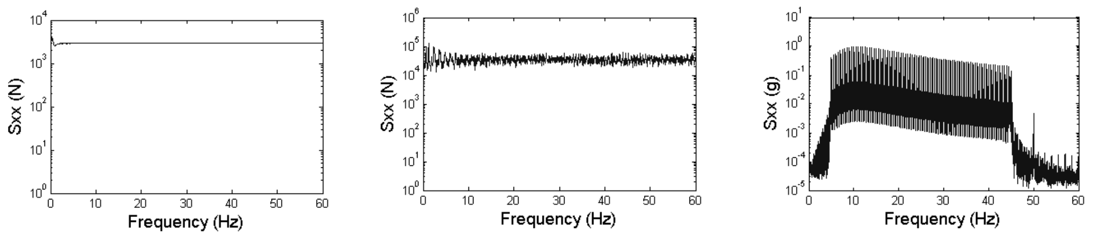
Experimental modal analysis (EMA) is a system identification technique based on the traditional input-output modal analysis. In this case, three different excitations were used to generate vibrations in the structure. The excitations were applied using a calibrated impact hammer (Dytran model 5803A) or an electro-dynamic linear mass shaker (APS Dynamics model 400).

In the first test, the walls were hit with the hammer and the response was measured during approximately 30 seconds. In the second test, the diaphragm of the specimen was randomly hit with the hammer during 300 seconds. In the third test, a harmonic horizontal excitation was applied by the shaker attached to the diaphragm. The excitation was a stepped-sine signal in the range of 5 to 60 Hz, with and frequency step of 0.5 Hz. The duration of each frequency step was 10 seconds with a pause of 3 seconds in between. Typical time-histories of the three excitations are presented in the Figure 7, and the typical power spectra of these excitations are presented in Figure 8.



a) Impact test b) Random test c) Stepped-sine test

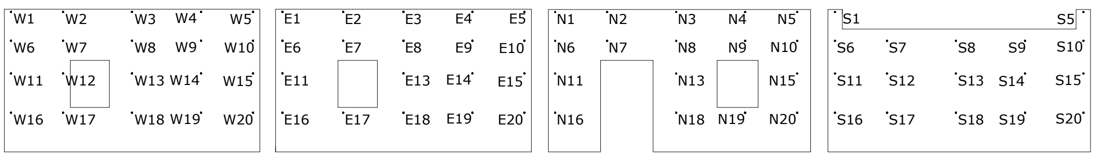
Figure 7: Time-history of excitations



a) Impact test b) Random test c) Stepped-sine test

Figure 8: Power spectrum of excitations

The response was captured using uniaxial accelerometers (Crossbow model CXL02LF1Z). The data acquisition was conducted using a Matlab code (MathWorks Inc., 2007) developed by the authors. The sample rate used in the impact and random test was 200 readings per second. The sample rate used in the stepped-sine test was 500 readings per second. The accelerations were measured in the direction normal to the face of the wall, over a grid of around 20 points per wall (Figure 9).



a) West wall b) East wall c) North wall d) South wall

Figure 9: Measurements grid

The system identification was conducted using two methods: peak picking (PP) (Ewins, 2000) and stochastic subspace identification (SSI) (van Overschee and de Moor, 1996). The range of frequencies considered as valid for the system identification is 5 Hz to 45 Hz, according to the range of periods defined by power spectrum of the stepped-sine excitation. The results are presented in Table 3.

Table 3: Natural frequencies identified using EMA

<table>
<tr><th rowspan="2">Mode</th><th colspan="3">PP</th><th colspan="3">SSI</th></tr>
<tr><th>Impact</th><th>Random</th><th>Stepped-sine</th><th>Impact</th><th>Random</th><th>Stepped-sine</th></tr>
<tr><td>1</td><td>--</td><td>--</td><td>--</td><td>6.18</td><td>5.92</td><td>6.06</td></tr>
<tr><td>2</td><td>--</td><td>--</td><td>--</td><td>12.59</td><td>13.48</td><td>13.61</td></tr>
<tr><td>3</td><td>17.97</td><td>17.97</td><td>16.60</td><td>16.28</td><td>17.17</td><td>16.30</td></tr>
<tr><td>4</td><td>21.88</td><td>21.49</td><td>--</td><td>20.17</td><td rowspan="2">22.55</td><td>20.92</td></tr>
<tr><td>5</td><td>--</td><td>25.40</td><td>25.39</td><td>26.21</td><td>24.83</td></tr>
<tr><td>6</td><td>31.25</td><td>29.96</td><td>29.30</td><td>30.81</td><td>28.18</td><td>28.91</td></tr>
<tr><td>7</td><td>34.82</td><td>33.65</td><td>34.18</td><td rowspan="2">35.68</td><td>33.04</td><td>32.88</td></tr>
<tr><td>8</td><td rowspan="2">39.94</td><td>--</td><td rowspan="2">39.06</td><td>38.62</td><td>37.07</td></tr>
<tr><td>9</td><td>41.08</td><td>40.19</td><td>--</td><td>41.44</td></tr>
</table>

4.2 Operational modal analysis

Operational modal analysis (OMA) utilizes an unknown input force to excite the structure, assuming that the input force is a random, stationary and ergodic signal (Silva and Maia, 1999). In this study, two tests were conducted. In the first test, the response of the structure due to unknown ambient excitation was measured. These measurements were performed during 900 seconds with a sample rate of 200 readings per second. The second test corresponds to random test same to the random test performed in EMA, but the data of the excitation was neglected in the analysis. The system identification was performed using the SSI method only. The range of frequencies considered for the system identification is 5 Hz to 45 Hz. The frequencies identified through OMA are presented in Table 4.

Table 4: Natural frequencies identified using OMA

Mode	SSI	
	Ambient	**Random**
1	--	--
2	12.05	12.33
3	16.66	16.46
4	20.28	19.43
5	24.39	24.45
6	27.50	28.08
7	32.59	33.89
8	36.93	36.60
9	40.95	40.50

5 RESULTS DISCUSSION

In EMA results (Table 3), the frequencies detected by PP methods are slightly higher than those identified by SSI. When the PP method is applied, the stepped-sine test shows the clearest peaks related to the natural frequencies in the frequency response functions (FRFs), followed by the impact test. In general, the random test present several peak in FRF not related to natural frequencies (Figure 10). The PP method is not able to detect the 5th mode when the structure is excited by an impact and neither the 4th mode when it is excited by a sine signal. Something similar happens to the 9th mode under random excitation. The 1st and 2nd mode are not detectable with the PP method, no matter what kind of excitation is applied.

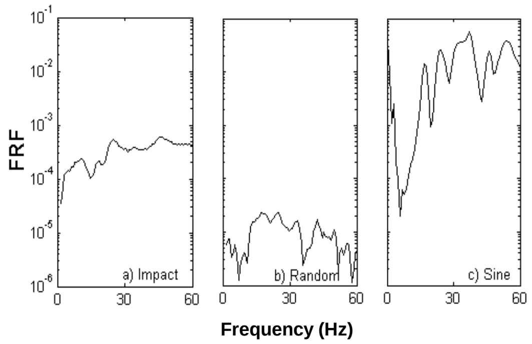


Figure 10: Typical FRFs from PP method

The SSI method is able to identify nine frequencies in the range. In general, the frequencies identified by SSI method coincide with those identified by the PP method. The frequencies are clearly distinguishable when the structure is excited by stepped-sine signal. Nevertheless, this is the longest test (900 seconds). The response in this test is better, because the load has only one specific frequency in each excitation interval. The impact test is the shortest (30 seconds). This test identified satisfactorily the first four frequencies, but the upper modes are coupled. The random test (300 seconds) has the worst performance in this study. It detects adequately three natural frequencies only. The higher modes are coupled (4th and 5th modes) or their natural frequencies are misidentified.

In OMA results (Table 4), the frequencies coincide reasonably well with those obtained using EMA. An exception is the frequency of the 1st mode that is not detectable. In the 2nd and 4th modes, both frequencies identified using OMA are slightly lower than those identified using EMA. The frequency identified for the 2nd mode coincides with the value obtained with EMA. The upper modes (5th to 9th modes) are more difficult to identify.

The difference between the natural frequencies predicted by numerical model and those detected experimentally are considerable (Table 5). The 4th mode has the highest mass participation (60%) in the model, therefore it is expected that this mode is easier to detect experimentally. Based on this criterion and for our study, it is possible to pair the 4th mode of the numerical model with the experimental frequency of 16.6 Hz (reference frequency). This is the frequency most easy to identify using the results of both, EMA and OMA, and the SSI method. An exception is the random test (Table 3) that instead of the reference frequency of 16.6 Hz, a value of 17.17 Hz is detected.

The first two frequencies detected experimentally correspond to the first two modes obtained by numerical modelling. These modes have a 6% and 14% mass participation, respectively. The 3rd mode obtained by the model is not detectable, because its low mass participation (1%).

In further steps, the properties of the first four modes will be used for model updating, especially the frequency of the 4th mode. The higher modes will be determined using the updated model.

Table 5: Results Comparison

Mode	**Natural Frequency (Hz)**		
	EMA-SSI	**OMA-SSI**	**FEM**
1	6.05	--	9.82
2	13.23	12.19	15.21
3	--	--	22.20
4	16.58*	16.56*	25.40

* Reference frequency 16.6 Hz

6 CONCLUSIONS

The results of this study demonstrate that EMA identifies the dynamic properties of an URM structure more clearly than OMA, because the system identification is improved when excitation history is considered.

The SSI method is able to detect more accurately the natural frequencies than the PP method. SSI is especially effective in identifying modes with low mass participation.

With EMA the stepped-sine signal produces the best response; nevertheless, this is the longest test. The impact test is able to identify satisfactorily the first four natural frequencies, even though, it is the shortest. The random test has the worst performance. It is able to detect satisfactorily three natural frequencies only, and the reference frequency is misidentified.

With OMA the results obtained from the ambient and random perform similarly. They are able to reveal accurately the highest mass participation modes only (2nd and 4th modes).

FE model prediction is not completely satisfactory. There are differences of around 50% between the experimental and numerical natural frequencies. However, it was possible to pair the modes based on a mass participation criterion. According to this criterion, the experimental mode whose natural frequency is 16.6 Hz should correspond to the 4th mode obtained from the FE model. If a better prediction of the structural performance under other kind of excitation (e.g. earthquake) is desired, then model updating is necessary. For this purpose, we recommend to match the frequency of the 4th mode in the FE model to the reference value 16.6 Hz.

A verification of the mode pairing based on a mode shape criterion (e.g. MAC factor) and the results of the model updating procedure are still pending. We expect to introduce these results in our presentation at the conference.

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