OPERATIONAL MODAL ANALYSIS FOR DAMAGE DETECTION OF A MASONRY CONSTRUCTION

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Abstract

This paper presents a dynamic identification analysis of a masonry construction, built to be tested in "Laboratório Nacional de Engenharia Civil" (LNEC), in Lisbon, on the scope of the European Project ECOLEADER-LIS – Enhancing Seismic Resistance and Durability of Natural Stone Masonry. The masonry model was built with limestone units and lime mortar joints with polymeric grid reinforcement placed on the horizontal joins. The dynamic identification analysis was divided in several tasks. For the calculation of the expected dynamic parameters a preliminary FEM analysis was carried out. Two types of operational modal analysis were used, the EFDD and SSI methods. The main purpose of the analysis was to compare the classical modal analysis with the ambient based modal analysis and to verify if the ambient vibration methods are able to assess the damage in an earlier stage in the structure. Finally, concluding remarks of the work carried out are given.

1 Introduction

This paper presents the first results of the dynamic identification analysis of a rubble stone masonry structure (see Figure 1), built in "Laboratório Nacional de Engenharia Civil" (LNEC), in Lisbon. This structure was tested in the LNEC shake table, under the EU RP within the 5th EU program, ECOLEADER – Enhancing Seismic Resistance and Durability of Natural Stone Masonry.

Figure 1 Masonry model

The aim of those tests was to assess the potential benefits, in terms of collapse prevention, of using the reinforcement grids to absorb the seismic action effects arising from strong base motion inputs induced by earthquakes.

In parallel to that aim, several dynamic identification tests were carried out in the described structure with two main goals. The first one was to verify if the experimental analysis with output only methods, used for dynamic identification of structural systems, was able to predict dynamic characteristics assessed by a best signal to noise ratio experimental modal analysis based on input-output techniques. These comparisons were carried out before and after the seismic tests were preformed on the shake table using unidirectional horizontal random input motions for that purpose. The second goal was to verify if, under the hypothesis that operational modal identification gives good results, thus those methods are able to assess the damage in an earlier stage in the structure.

2 Description of the Masonry Mock-up

The experimental mock-up is a one storey limestone masonry building, with reinforced mortar joints. The reinforcement was composed by a polymer grid. The model geometry is representative of the traditional rural construction of South European countries and the reinforcement is a new developed product to increase the ductility capacity and strengthening of the composed material.

Figure 2 to Figure 4 shows the geometry of the mock-up which is asymmetric in plant. The walls are 3.6 m height without any slab on the top, i.e. the weakest possible structural behavior of the structure. The mock-up was built on a reinforced concrete slab with 0.21 m of thickness, which is used to fix the models to the shake table platform. Horizontal layers of a rigid polyurethane grid (RichterGard 20™) where used to reinforce the walls. Figure 5 shows the details of the reinforcement.

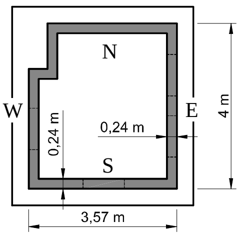


Figure 2 Top view of the model

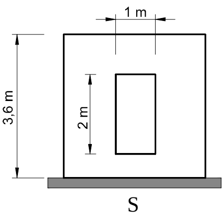


Figure 3 Principal facade

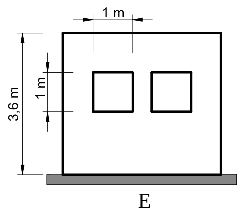


Figure 4 Right lateral view

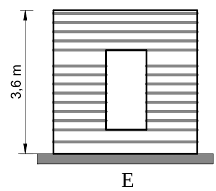


Figure 5 Reinforcement polymeric layers

3 Numerical Dynamic Identification

As a first approach to the behavior of the structure and to define strategies for the experimental identification, a modal analysis based on Finite Elements Model (FEM) was developed. A simpler model was constructed in DIANA [1] with 5166 plane shell elements of 8 nodes, isoparametrics and with quadratic integration. The polymeric reinforcement was neglected because it is believed that it has no influence in the dynamic response of the structure, has it only affects their strengthening and their ductility capacities. Taking into account those conditions the modulus of elasticity was taken is equal to 5.0 GPa and the mass equal to 2.3 ton/m^3.

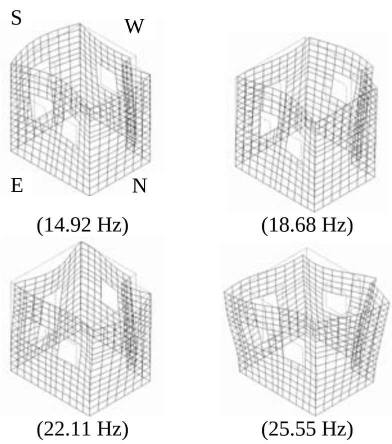


Figure 6 First four mode shapes

Furthermore, the degrees of freedom of the base nodes were clamped. This assumption is acceptable if the level of vibrations is lower and if is adopted a linear material behavior. For output only modal analysis vibrations are caused by ambient noise with very low amplitude, which makes this hypothesis realistic.

Figure 6 shows the results of the first four resonant frequencies. As can be seen they are all below the 25 Hz. The first frequency is closed to 15 Hz which demonstrates the high stiffness of the construction, even without any slab on the top of the structure. This value can be justified by the small lengths of the external walls (between 3.5 and 4.0 m) and by the geometry of the structure, similar to a box. All the frequencies are well spaced. Also it can be concluded that the higher points in the structure have larger amplitudes in every mode shape, as expected.

4 Operational Modal Analysis

For the ambient vibration based modal identification tests the software ARTeMIS Extractor [4] was used and for the classical modal identification analysis the software developed in LABVIEW by LNEC.

4.1 Test Conditions and Planning

With the aim of access the damage in an earlier stage in the structure, several modal identifications tests were carried out to check the application of operational modal analysis to the issue of damage detection. The identifications were developed according to a defined test planning for the seismic shake table test of ECOLEADER project.

In a first phase of that planning it was foreseen to apply seismic input intensity levels, so that a given level of damage was attained in the structure, without any partial or total collapse occurrences. This level of damage was conveniently selected by taking into account that the structure was to be strengthened afterwards. In deed, in the second phase the construction should be reinforced with the same polymeric grid applied as a reinforced plaster to the walls. The main purpose is to increase the strength and the structural ductility of the entire building. Finally, in the last phase of seismic testing, structural collapse may be reached and an ultimate load capacity is assessed for this type of strengthening technique.

At the present moment, according to the ECOLEADER project schedule, only the first phase of tests is completed. Table 1 presents the chronological tests series carried out within the operational modal analysis; test conditions and the observed damage are also described in that table. It is intended in future works to continue with these modal identifications tests, following again the schedule of the ECOLEADER project. Table 2 shows crack pattern observed in the mock-up during modal identifications tests referred in Table 1.

Table 1 Modal identification tests chronology

Modal Id.	**Date**	**Description**
A	08/07/2004	Modal identification after 28 days of the model construction. The ambient vibration data was acquired without any previous movement of the mock up in the laboratory, i.e. outside the shaking table. No cracks were observed during the identification
B	25/01/2005	Six months later, and after the first model transportation, for logistics reasons in the laboratory, and apparently without cracks (see Table 2)
C	28/01/2005	Modal identification after model transportation and fixing to the shaking table (2nd transportation); localized cracks appear in the structure due to a shock occurrence during that operation (see Table 2)
D	02/02/2005	Modal identification after the forced dynamic characterization of the structure with impact, sine-sweep and random input excitation. The amplitudes responses in sine-sweep tests were large enough to produced additional cracks in the structure (see Table 2)
E	03/02/2005	Modal identification after applying a sequential of ground motions intensities, up to 0.25 g maximum, to the shake table; more cracks could be observed in the structure (see Table 2)

To identify the modal parameters 49 points were selected for ambient response measurements (see Figure 7) in out-of-plane wall directions. Two points at the top of the model, in orthogonal directions, were chosen as reference points for the several test setups, namely the 35 and 45 points presented with a circle in Figure 7. For signal measurements, 8 accelerometers were used through 11 test setups. The total sampling duration of every setup was 10 min.

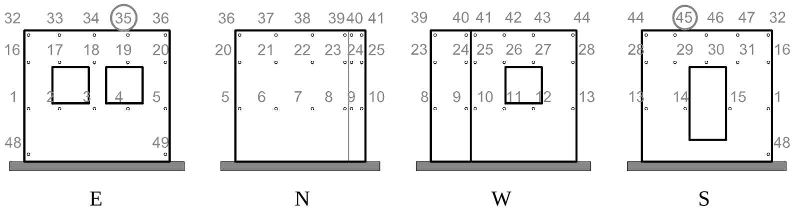
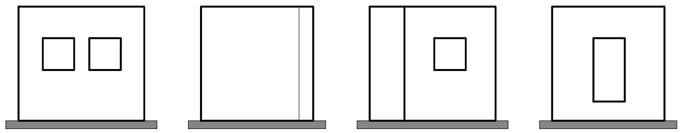
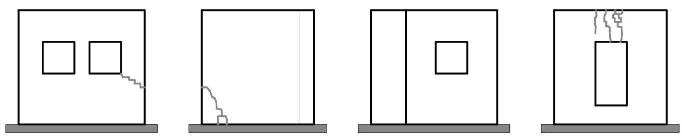
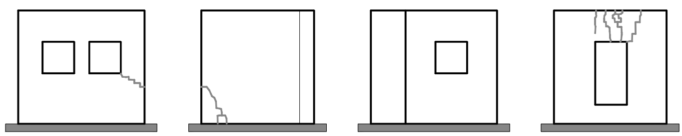
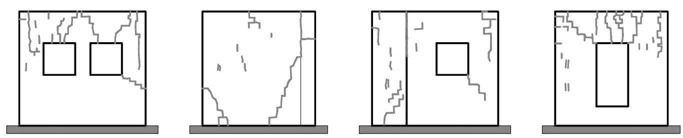


Figure 7 Location of the measuring points

Table 2 Crack pattern observed for each modal identification width

Modal Identification	**Crack patterns of each test. All cracks varies from 0 to 1 mm opening**
A an B	
C	
D	
E	
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4.2 Operational Modal Results

Table 3 and Table 4 summarize the results for all modal identifications tests carried out in this study in terms of resonant frequencies values and mode shapes, respectively. The first conclusion is the decreasing of resonant frequencies along the several identifications tests (in particular a reduction of 50%, on average, for all mode shapes between the A and E tests) associated with significant differences in the modes shapes. Second remark is the divergence between experimental and the numerical prediction values; although the value of the first computed natural frequency can be considered closed to the experimental value of 15.05 Hz. The correlation between measured and the predicted natural frequencies can be analyzed in Figure 8, where both results are graphically compared. The figure shows that model updating is necessary for every step, even for the first case where the mock-up was in is “ideal” undamaged conditions.

Table 3 Resonant frequencies (Hz)

Mode Shape	Modal Identification Test				
	A	B	C	D	E
1st	15.05	12.28	10.60	7.55	4.62
2nd	19.79	13.97	12.29	9.60	6.13
3rd	20.50	18.30	16.63	12.96	8.71
4th	26.57	21.27	17.60	13.83	12.80
5th	28.91	25.94	19.56	17.58	13.61
6th	36.85	32.87	28.06	23.82	15.40
7th	39.73	33.69	32.06	28.99	21.64

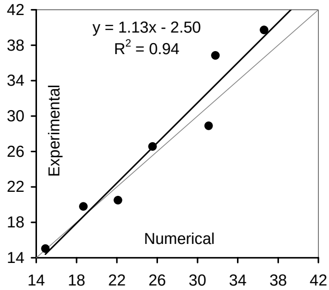
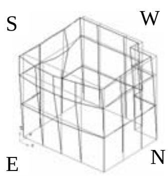
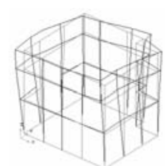
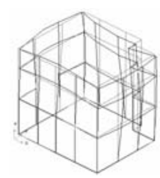
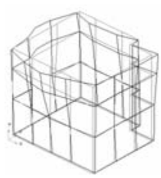
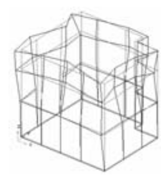
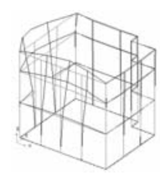


Figure 8 Modal identification test A and predicted natural frequencies

Table 4 First four mode shapes configuration for the modal identifications

Modal Id.	Mode shape			
B	 (12.28 Hz)	 (13.97 Hz)	 (18.30 Hz)	 (21.27 Hz)
C	 (10.60 Hz)	 (12.29 Hz)	 (16.63 Hz)	 (17.60 Hz)
D	 (7.55 Hz)	 (9.60 Hz)	 (12.96 Hz)	 (13.83 Hz)
E	 (4.62 Hz)	 (6.13 Hz)	 (8.71 Hz)	 (12.80 Hz)

Regarding the variation of each frequencies values of every modal identification test, from A to E (see Table 3), is possible to conclude that the difference in percentage between two consecutive modal identification varies from 1 to a 40%, with 20% on average. Those variations are very high changes in frequencies.

Focus on the first two identification tests, A and B, one possible explication for the significant decreasing values in frequencies without any apparent cracking (see Table 2) can be explained through changes in the boundary conditions at the base of the model, as a result of the model transportation inside the laboratory (see Table 1). In fact the concrete slab that supports the model is rather flexible and relative deformations along that boundary surfaces can occur during model suspension; those deformations associated with low tensile strength of foundation joint can justify changes in boundary conditions. It is worth mentioning that temperature inside the lab is almost constant, during the day and the year, and also that mortar drying along the time would probably increase the stiffness, instead of reducing it. Changes in modal characteristics due to changes in boundary conditions are not new and were also mentioned by other authors [4].

In the case of the difference between the identification B and C a small accident occurred during the mock-up transportation to the shaking table. Localized cracks could be seen in the model with a maximum width of 1 mm (see Table 2). Again, also some changes in the boundary conditions could contribute for the significant frequencies changes. But the most interesting aspect was the differences founded in the spectrums before and after the mock-up was placed in the shaking table. Figure 9 shows the EFDD pick peaking method of identification test B, where eight peaks can be clearly identified. In the case of identification test C (see Figure 10) more peaks and consequently more resonant frequencies can be observed. Those frequencies are not from any excitation source but from the new observed structural system, which includes the structure of the shaking table with six degrees of freedom. For every identification test in the shaking table further analyses were performed to separate the mock-up frequencies from the shaking table contribution frequencies.

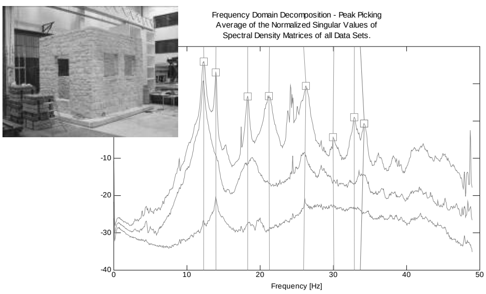


Figure 9 Average of the normalized singular values of modal identification B

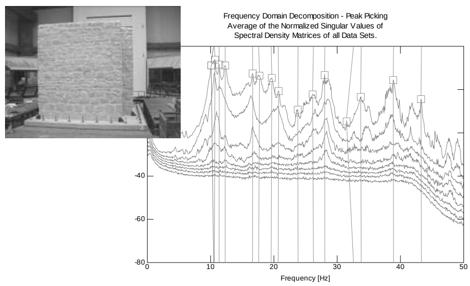


Figure 10 Average of the normalized singular values of modal identification C

The differences founded between tests C and D were due, as already mentioned, to the forced input identification tests carried out before the seismic tests. In particular, the sine-sweep tests were performed in open loop (without any adaptive control) and vibration were quite large during the passage through the first resonance frequency. The damage induced by this test was reflected by a new observed crack in the south facade (see Table 2).

Modal identification test E was made after the seismic characterization test. The frequencies values for all modes decreased (see Table 3) and the mock-up was significantly damaged, but no partial collapse was occurred, as can be seen in Table 2, and the crack did not reach 1 mm width.

Table 5 MACs between tests D and E

(Hz)	**4.62**	**6.12**	**8.71**	**12.80**	**13.61**	**15.40**	**21.64**
7.55	0.23	**0.90**	0.02	0.18	0.02	0.08	0.10
9.60	0.05	0.09	**0.89**	0.32	0.08	0.07	0.07
12.96	0.03	0.11	0.14	**0.77**	0.23	0.09	0.17
13.83	0.03	0.18	0.26	0.45	**0.50**	0.06	0.12
17.58	0.13	0.05	0.06	0.18	0.41	**0.62**	0.28
23.82	0.18	0.20	0.06	0.12	0.40	0.02	0.24
28.99	0.16	0.11	0.10	0.31	0.41	0.16	0.29

An important conclusion can be observed in the comparison of results of test identifications D and E: although the structural damage was induced by the seismic tests, those tests induced the smallest differences (11% on average) between frequencies values. However, the changes in frequencies and mode shapes are significant, as can be seen in Table 5 by the MAC values between the last consecutive experimental analyses. In the MAC table a good correlation can be observed for modes 2, 3 and 4, with the MAC values upper 0.77. But for the first (new localized mode) and higher modes the value decrease. With this fact it seems that higher frequencies and mode shapes are more sensitive to damage than the lower cases.

5 Classical Experimental Modal Analyses

Experimental dynamic identification tests, based on traditional input-output experimental modal analysis, were performed parallel to the operational modal tests already referred. Two tests were performed using the shake table as the input motion apparatus: a first one, prior to the seismic test and after the sine sweep tests and, a second one, after the last stage ground intensity seismic tests. Results of those two tests are directly comparable with the results obtained in the operational modal analyses performed in stages D and E, respectively (see Table 1).

Output absolute accelerations, perpendicular to the walls, were measured in 31 points of the structure, in which some of them corresponds to the degrees of freedom also measured in the operational modal analysis. It is important to stress that transducers used during these tests are not the same as the ones used previously and that the sensitivity of this transducers are ten times lower then the accelerometers used in operational modal analysis. This feature, combined with the fact that amplitude of vibrations in this tests are about two orders of magnitude greater then amplitudes of ambient vibration, thus it turns out that signal to noise ratio for the classical modal analysis is only one order of magnitude greater then the corresponding value of the operational modal analysis.

Input-output experimental modal analysis was performed using the vibrations data acquired during the random input characterization test. Traditional methods for experimental modal analysis [3] were used to obtain the frequency response functions (FRFs), amplitudes and phases, and coherence functions between input table motion and 31 output acceleration measurements. Some of the results of classical experimental modal analysis are depicted in Figure 11 to Figure 13.

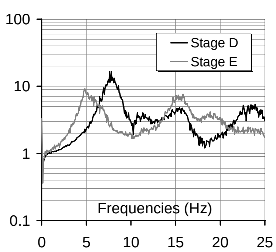


Figure 11 FRFs

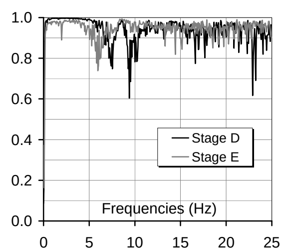


Figure 12 Coherence

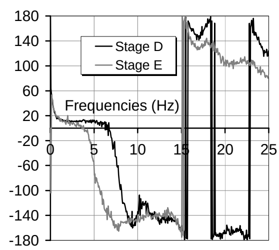


Figure 13 Phase angle

From the analysis of the information gathered in the classical modal analysis the principal points that deserve mentioning are: (a) prior to the seismic tests (stage D) and after major cracking produced by sine-sweep tests, FRFs (magnitude and phase) show a frequency peak around 7.8 Hz corresponding to a global mode shape with larger amplitudes in the out-of-plane direction of the walls; (b) after seismic testing, stage E, further cracking in the structure induced a localized substructure deformability, namely in the spandrel beams, above the windows of the East facade and above the door of the North facade. This increase in deformability is responsible for a low frequency localized mode shape (4.8 Hz) with larger amplitudes in the out-of-plane direction of those walls, precisely in the zones where higher cracking was observed; and (c) natural frequencies and mode shapes corresponding to in-plane distortion deformations of the walls are much higher (above 15 Hz) then the natural frequencies corresponding to the out-of-plane flexural deformations of the walls. Furthermore those in-plane natural frequencies didn't seem to decrease in substantial way from test E to D.

6 Conclusions and Further Developments

The present paper presents the first results of an operational modal analysis performed on a rubble stone masonry structure built in "Laboratório Nacional de Engenharia Civil" (LNEC), in Lisbon. In framework of present study the man issue was to verify if the operational modal analysis, an output only based method, is able to assess the damage in an earlier stage in the structure. For that purpose comparison with classical input-output experimental modal analysis results were used as reference terms.

All the identification tests are described, in terns of test conditions, test planning and observed crack pattern. During the several tests it was possible to observe the decreasing of all resonant frequencies. The corresponding mode shapes also suffers significant changes, especially for the higher modes. In particular, a new mode shape associated with localized damage was detected, either by ambient and forced vibration identification tests.

At the moment, experimental tests series are not finished and the structure is in the phase of suffering structural strengthening improvements for further seismic testing. Thus more identification test should be performed in the future, to evaluate both the structural strengthening improvements and the damage progress in the subsequent seismic testing.

Moreover further analyses of the entire data gather during the tests described in this paper should be accomplished. For instance, damage detection methods [4] should be carried out to evaluate their relative accuracy in terms of damage detection (Level 1) and damage localization (Level 2) for masonry structures.

7 Acknowledgments

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8 References

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