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IDENTIFICATION OF DYNAMIC CHARACTERISTICS OF MASONRY BUILDINGS FROM FORCED VIBRATION TESTS

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ABSTRACT

The paper describes the key observations obtained from processing the experimental results of forced vibration tests carried out on an old masonry house using a vibrodyne and on a 1:5 scale model in the laboratory. Small amplitude oscillations are developed to determine the dynamic characteristics of the structure. Their evolution after large amplitude oscillations furnishes a correlation between modal quantities and structural damage. Identification techniques are used to process the experimental data. Interesting information is obtained on horizontal resistance and the manner of collapsing.

KEYWORDS

Masonry; experiments; damage detection; identification techniques; modal analysis; nonlinear behaviour.

INTRODUCTION

Most existing buildings in Italy are small old masonry houses. The characteristics of masonry depend on its components and on their organization. The uncertainties that affect the material also affect the assembly of structural elements and their connections, which strongly influence the mechanical behaviour of the structure, particularly its response to horizontal forces. These factors justify the expansion of experimental activities in this field. The present paper briefly illustrates the dynamic tests performed in-situ on an old masonry building and on a 1:5 scale model in the laboratory. The experimental data are processed using identification techniques so as to obtain as much information as possible from the experiments.

The in-situ experiments involved steady-state vibrations of a simple two storey masonry house excited by a vibrodyne (Capecchi *et al.*, 1990; Capecchi and Vestroni, 1991). Two series of tests are referred to here. In the first the whole house was involved and low amplitude vibration tests were performed. In the second series only part of the structure was involved, a simple square portion, excited with increasing force.

A similar investigation was carried out in the laboratory on the model of a typical building erected in the 1950s and consisting of external brick walls and an internal cross shaped columns. Several tests on a shaking table were conducted using simulated accelerograms with increasing intensity. In both investigations small amplitude vibration tests, developed before and after the damaging tests, were used to locate and quantify damage. A correlation between the modification of modal parameters and structural characteristics was sought using experimental modal analysis and parametric estimation techniques (Antonacci *et al.*, 1994).

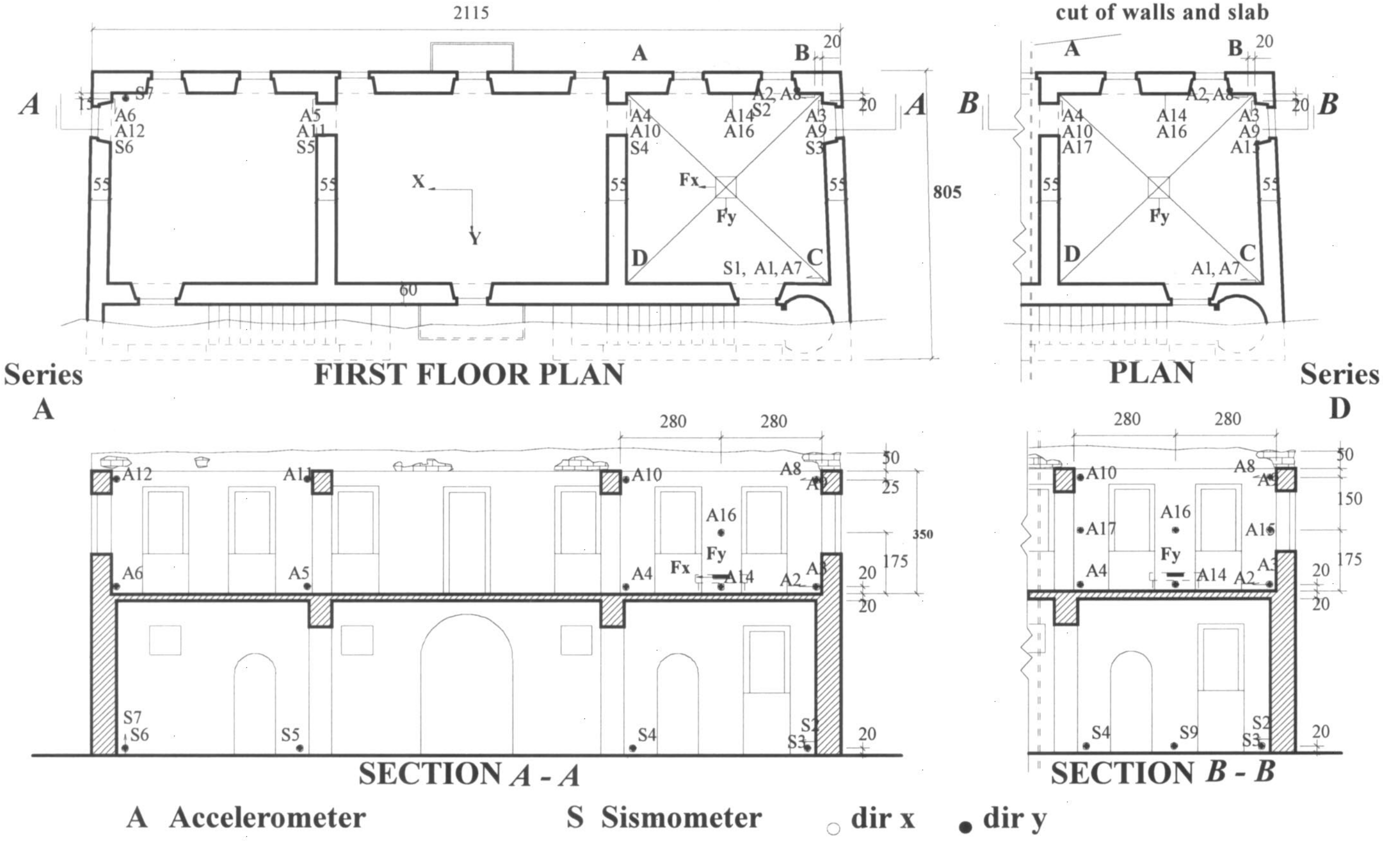


Fig. 1. Plan and sections of the house. Location of instruments in test series A and D

Useful information was obtained concerning the horizontal resistance of masonry elements and their way of collapsing.

THE IN-SITU TESTS OF AN OLD MASONRY HOUSE

The house was in L'Aquila, in Central Italy; it was built around the end of the 18th century. The structure has two storeys, including the ground floor, and is regular in plan (Fig. 1). Notwithstanding its two hundred years, the structure was not in bad conditions, since restoration had probably been carried out after the 1915 earthquake. In any case the first floor was strengthened by replacing the sand and mortar conglomerate with a reinforced concrete slab connected to vertical walls by steel bars. The roof structure consisted of wooden trusses; since its contribution to the resistance of the structure against horizontal forces was negligible, it was removed (Fig. 1).

Experimental Investigation of the Complete Structure

The structure was tested using a mechanical vibrodyne placed on the first floor in the center of the room ABCD (Fig. 1) acting in turn transversally and longitudinally. The experimental investigation was carried out in cooperation with ISMES (Institute of Experimental Analysis on Materials and Structures, Bergamo, Italy). Two series of tests are examined here. Series A with a medium intensity force shaking the whole

Table 1. Force and frequency range of steady state vibrations tests

Series	A					D							
Test (Dir)	1(x)	2(x)	3(x)	4(x)	5(y)	10(y)	11(y)	12(y)	13(y)	14(y)	15(y)	16(y)	17(y)
F_0	70	98	7	70	8	21	151	304	374	374	374	374	21
Ω_i	1	4	7	1	1	1	3	3	3	5	2.5	2.5	1
Ω_f	8	8	11	8	11	11	6	5	5	3	4	4	11

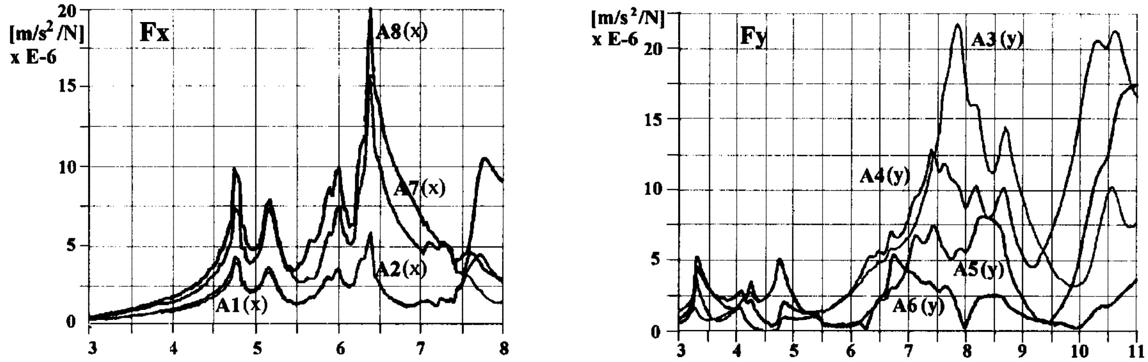


Fig. 2. Transfer functions of measured accelerations under F_x (T1) and F_y (T5) forces

structure to determine its dynamic characteristics. Series D with high intensity force shaking only a part of the house (ABCD in Fig. 1) to study the response beyond the elastic limit and to correlate dynamic characteristics and damage. Data on the force characteristics are reported in Table 1.

From the first processing of the results the transfer functions for each measured acceleration signal at points Ai are obtained. Samples of those of Series A are reported in Fig. 2. The most peculiar aspect is that, while the structure appears quite regular, the dynamic response is very complex. This results in a high number of resonance peaks, most of them simultaneously present in transfer functions related to the orthogonal forces in the *x* and *y* directions. Another worrying aspect is the slight but evident shift of frequencies passing from test T1 to test T5, which does not make the analysis easier. Moreover, the visible coupling among several modes requires the use of more refined tools to determine the modal parameters.

A six-dof model, three for each storey, is not able to describe the response accurately. The analytical transfer functions given by the identified model starts to diverge from the experimental ones from the fourth frequency. It is thus necessary to adopt a model with a higher number of dofs, close to the number of points measured. The modal parameters are determined by a procedure (Beolchini, 1995) which is based on the multi-mode method proposed by Goyder (1980); they are reported in Table 2. Only the identified mode shapes make it possible to relate resonances excited by F_x and those by F_y.; they appear in the same column, when the MAC index, used to recognize similarity between two modes, is close to unity. Some uncertainties remain, mainly due to difficulties in the evaluation of modal components, strongly influenced by the damping value.

The identified modal quantities are then used to draw the analytical transfer functions which satisfactorily agree with the experimental ones (Fig.3). Despite the ill-conditioning of the identification problem and the presence of experimental errors, the identified modal model is fairly accurate and can be used to predict a response reliably. The analysis of the dynamic characteristics shows that the structure is far from being regular and symmetric, as it appears. This is quite important in evaluating the seismic behaviour.

Experimental Investigation of a Part of the Structure

Series D involved part ABCD which was separated from the rest of the structure and excited in direction *y* (Fig. 1 and Table 1). From a general view of the response, the behaviour of this structure, which is more

Table 2. Identified frequencies and damping coefficients of the complete structure

Dir.	Test	Mode	1	2	3	4	5	6	7	8	9
F_x	T1	ω(Hz)	3.80	4.56		4.76	5.18		5.67	5.86	
		ζ(%)	1.84	1.25		1.20	1.98		1.20	1.03	
F_y	T5	ω(Hz)	3.33	4.18	4.30		4.79	5.39			6.75
		ζ(%)	1.75	1.98	1.63		1.96	1.86			1.60

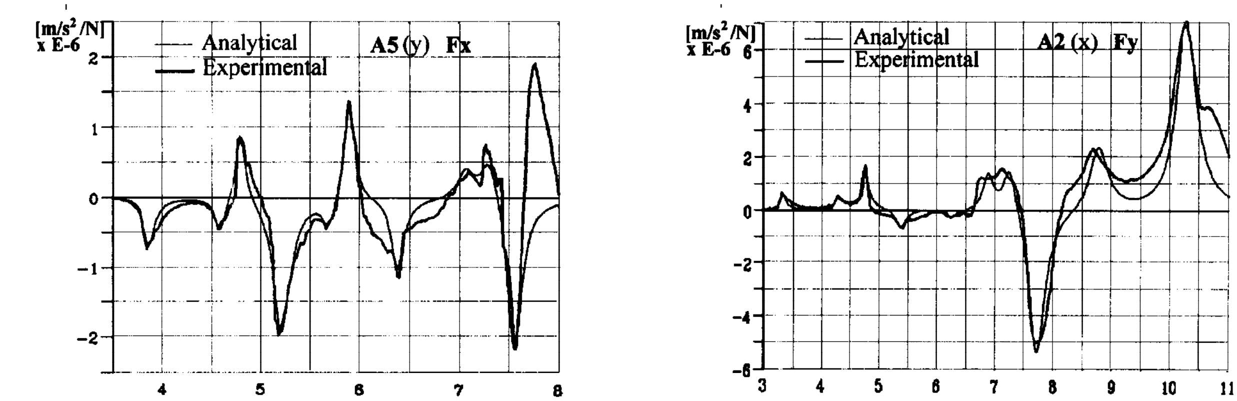


Fig. 3. Experimental and identified transfer functions of the accelerations of two points

regular and compact than the complete structure, is more complex, characterized by translation, rotation and distorsion, with the last two increasing percentally with the increasing oscillation amplitude. The displacement transfer functions of one measurement point in the range of low modes is in Fig. 4. A notable decrease of resonances in the tests with higher force intensities is observed, which is primarily related to the nonlinear behaviour. However, a decrease is also observed when tests 10 and 17 (with the same intensity) are compared, which is certainly due to damage caused by previous tests. Furthermore, the modification of the shape of the curves must be related to a non uniform distribution of damage among the structural elements.

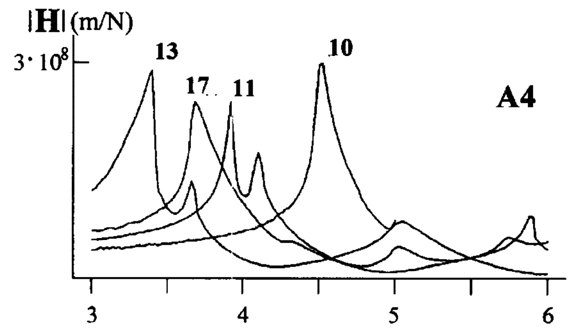


Fig. 4. Displacement transfer functions of a point for different tests

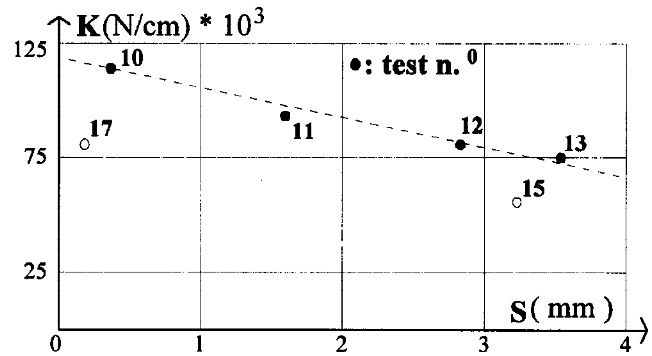


Fig. 5. Identified first modal stiffness for each test versus oscillation amplitude

An interesting picture of the overall nonlinear behaviour experienced by the structure is given by the first modal stiffness, shown in Fig. 5. It decreases with the amplitude of oscillation, but a notable decrease is also observed between tests 10 and 17, with similar oscillation amplitude, clearly due to a certain degree of loss of structural integrity. A more accurate analysis of the damage suffered by the structure between tests 10 and 17 is developed by evaluating modal quantities from these two tests and identifying the stiffness parameters of a finite element model of the structure in the initial and final conditions.

The analytical transfer functions are expressed by a model with viscous damping complex modes and modal quantities are obtained using rational fractional polynomials. The curves determined by the identified

Table 3. Identified frequencies and damping coefficients of the structure ABCD

Test	Eigenv.	1	2	3	4	5	6	7	8	9
10	ω(Hz)	4.50	5.00	6.22	7.11	7.54	7.92	8.31	9.18	10.36
	ζ(%)	1.67	2.98	1.60	1.21	2.10	1.38	1.41	2.11	2.15
17	ω(Hz)	3.66	4.31	5.02	5.74	6.00	6.95	7.19	7.69	7.99
	ζ(%)	2.39	3.27	2.57	1.93	3.23	1.33	1.65	3.31	2.47

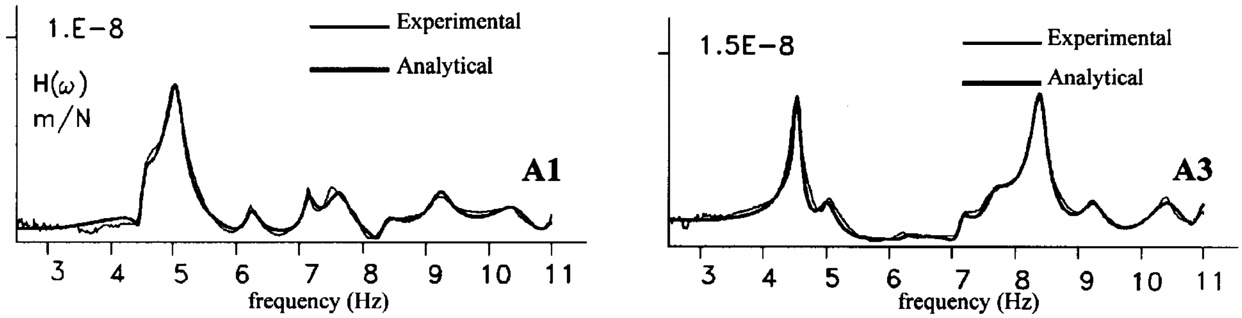


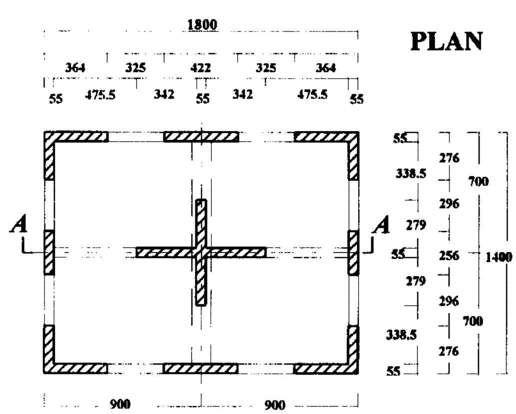
Fig. 6. Identified and experimental displacement transfer functions of two points

parameters of the first nine modes, reported in Table 3, are in good agreement with the experimental ones (Fig. 6).

The experimentally evaluated modal quantities are then used to determine the stiffness characteristics of a finite element model. The comparison between the models identified from the T10 and T17 results makes it possible to locate and quantify the damage suffered between the two tests. The stiffnesses of the four walls are assumed as parameters to be identified. A nonlinear parametric estimation procedure, based on an output error equation, is adopted (Antonacci *et al.*, 1994). The specific problem is very ill-conditioned; indeed, a preliminary analysis on the f.e. model has revealed that this structure has two very close frequencies (the 2nd and the 3rd) and very similar relevant modes. This would suggest that these two modes should be disregarded; unfortunately this was not possible, since higher modes are not reliable enough. A preliminary analysis should have been developed before the experimental investigation, in order to select the optimal quantities to measure for identifying the interesting parameters. In any case, numerical difficulties, due to the irregularities of the objective function to be minimized and the presence of some local mimina, were overcome and an optimal estimate of parameters was obtained. The comparison of the identified stiffness parameters leads to the following conclusions. All four walls suffered damage, but in different degrees; the average reduction of stiffness was 0.79, 0.69, 0.78, 0.58 for walls AB, BC, CD and AD respectively. The most affected are the walls in the excitation (y) direction and in particular the wall AD, caused by considerable rotational components. These results are in very good agreement with the survey of damage.

EXPERIMENTAL TESTS ON A SCALED-DOWN BUILDING

The model reproduces, on a scale of 1:5, a mixed building with an internal cross shaped column and external brick walls. A detailed description of the prototype is given in Modena *et al.* (1992); only the basic information is given here. The model has three floors and is 1917 mm high; its plan and section are shown in Fig. 7. All elements, such as bricks, floors, etc. were designed to reproduce correctly the behaviour of the actual structure, according to the similitude law: thus, masses were added to adapt the ratio between the masses of walls and floors, prestressed cables at the four corners of the model adequately increased the compression stress in the walls, etc. The model was mounted on a shaking table.



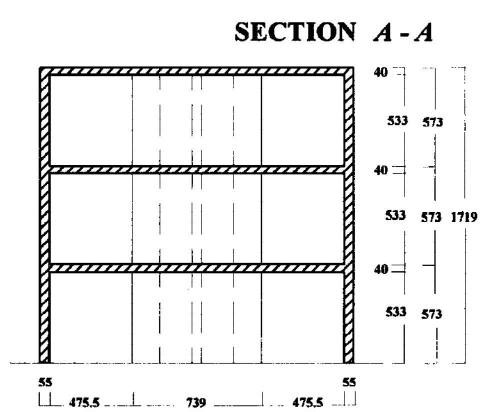


Fig. 7. Plan and section of the model

Table 4. Identified parameters of 1st mode (frequencies from hammer tests (1), and free oscillations of forced tests (2))

Acceler. no.	Intensity	First frequency (1)	First frequency (2)	Damping	1st eigenvector
20	0.26	12.79	12.60	2.7	.28, .67, 1
30	0.32	12.79	12.70	2.0	.23, .64, 1
41	0.25	12.69	12.69	2.5	.31, .63, 1
42	0.41	12.45	11.80	3.3	.28, .64, 1
43	0.52	11.87	11.00	2.9	.28, .63, 1
44	0.90	11.00	10.40	3.2	.31, .67, 1
45	0.89	9.83	8.10	4.6	.37, .70, 1
47	1.10	6.59	5.70	3.8	.38, .73, 1
48	1.55	6.20	4.60	4.2	.38, .73, 1
46	1.72	7.35	5.00	3.0	.41, .76, 1
50	2.45	5.63	4.30	3.7	.40, .75, 1
49	2.74	5.57	4.40	3.5	.37, .75, 1

The tests on the shaking table were performed using simulated accelerograms with assigned spectra and increasing intensities. The design accelerograms belong basically to two groups. The first group was generated using the Eurocode n.8 and GNDT-CNR spectra. They correspond to low intensity earthquakes and consequently the responses of the model in these cases are supposed to be linear. Accelerograms of the second group are still synthetic, generated using the Eurocode n. 8, or recorded (component N-S of the Petrovac earthquake, 1979). They have higher intensities and in these tests the structure experienced nonlinear vibrations. Actual intensities was measured on the shaking table: they are summarized in Tab. 4

This paper does not discuss the identification of the linear dynamic characteristics of the structure, which was not easy on account of the type of excitation: an imposed base motion with a particular frequency content. Since the response is dominated by the first mode, attention is focused on the estimation of its modal parameters. The recorded signals were analyzed taking two windows into consideration. The first corresponds to the last part of the time history, where free oscillations occur. The second window corresponds to a middle interval, where the response is more intense. The transform of the free vibrations permits to determine the natural frequencies of the model in the elastic range and their possible variation due to damage; the transform at a more intense level makes it possible to recognize the nonlinear behaviour and the modification of frequency due to large amplitude oscillations. The effectiveness of the two windows is illustrated using the numerical responses of a sdof oscillator.

In Fig. 8-a the comparison between the transforms of the excitation and the response of an elastic oscillator with a frequency of 12.5 Hz allows to define the natural frequency by separating the fundamental component, associated with the natural frequency, from those forced by excitation. A similar analysis (Fig. 8-b) is developed for an elastic-plastic oscillator. Due to nonlinearities, the response transform exhibits several peaks at low frequencies; however, even in this case the natural frequency component is well defined and its modification, from f_0 to f'_0, can be obtained by comparison with the free oscillations transform.

The mean values of the most significant frequencies and corresponding eigenvectors identified using the free oscillations after each test are shown in Tab. 4. The eigenvectors refer to floor displacements; they are normalized to the displacement of the third floor. The frequencies and damping factors (Tomazevic *et al.*, 1989) evaluated from hammer tests carried out after each testing phase are also reported. The data are arranged with increasing intensity; the numbers of the original sequence of tests are also given.

During the first 40 tests the model did not suffer damage and only a slight stiffness degradation occurred: in these cases the agreement between frequencies evaluated by the two above methods is very satisfying. In contrast, the frequencies obtained by hammer tests are much higher than those evaluated by the free oscillations following the last ten forced tests: during these tests the model suffered damage varying from

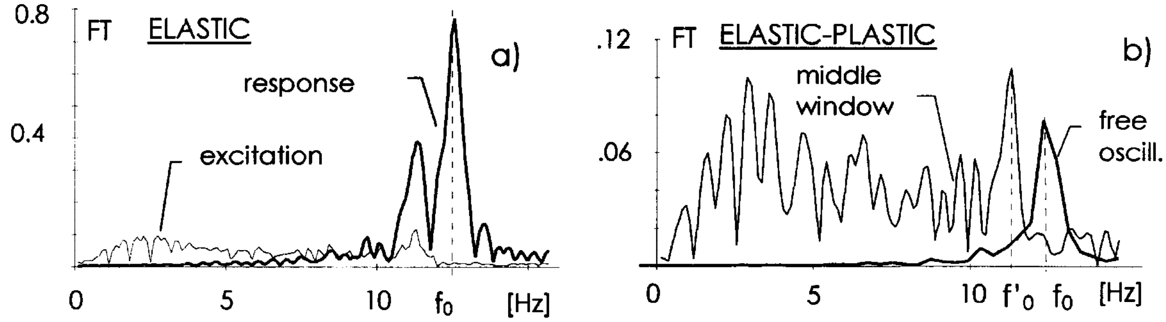


Fig. 8. Sdof oscillator: a) the transforms of the response and the excitation; b) the transforms of the same response by using two windows

small horizontal cracks in corner walls to shear failure of the cross shaped column when the intensity reached maximum value. The gap between frequencies, which becomes more evident for accelerograms with higher intensities, can be attributed to the difference between the amplitudes of the vibrations, which is much smaller in the case of oscillations produced by hammer, and to the rapid variation of the stiffness near the origin.

Widely different damage effects are observed after the 44th and 45th accelerograms, which are basically of the same intensity. The first produces a smaller reduction in the frequency than the second. If the frequencies measured by hammer hit are considered, test 44 produces a reduction in the frequency equal to 2.3%, whereas test 45 causes a further reduction of 15.3% (respectively 5.5% and 22.1% if the other frequencies are considered). This apparently odd behaviour can be explained by examining the response spectra of both accelerograms computed using a damping factor equal to 2% (Fig. 9). The value of the pseudo-acceleration corresponding to the second input around 0.09 s, which is the first period of the model, is about 2.5 times that of the accelerogram 44: the stresses basically depend on the first mode, so the observed reduction in frequency and the damage effects are justified.

Figure 10 shows the maximum measured displacement versus the acceleration intensities: it can be seen as a damage measure. The first part confirms that displacements are basically the same until test 40, due to the elastic behaviour of the model and to the equivalence of excitations. The first increase in the maximum displacement can be observed in tests 41-42: the behaviour is no longer elastic, although no evident cracks appear in the walls. This is confirmed by a reduction, of about 1 Hz, in the natural frequencies related to the decrease in the generalized stiffness, due to internal damage and not revealed by a visual inspection. When the model suffers manifest damage (accelerograms 45-50), a sharp variation in the tangent to the curve is observed. In substance the diagram still reveals a basically linear relationship in this range, corresponding to a gradual spreading of cracks over the structure. It results in a satisfying global ductility of the model, notwithstanding material fragility.

Another description of the damage evolution is given in Fig. 11 where the decrease in the fundamental frequency versus excitation intensity is reported. According to Fig. 10, there is a rapid reduction in the first range, while for the higher intensities the curve tends to become horizontal. This behaviour explains the potential high resistance of masonry structures to horizontal seismic forces, due to the modification of its dynamic characteristics. Unfortunately, some conditions must be present for collapse to be avoided; the damage can have even a strong effect on the walls in the excitation direction, but the structure must maintain

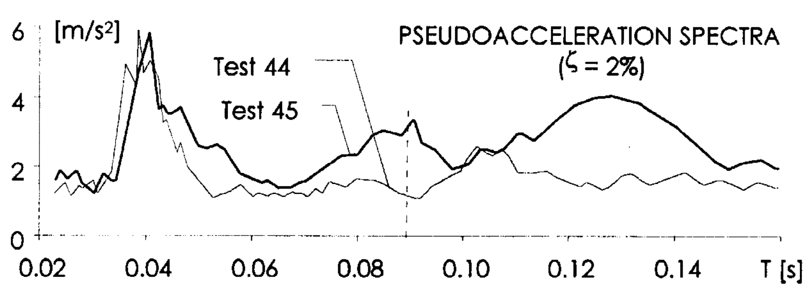


Fig. 9. The response spectra of accelerograms 44, 45

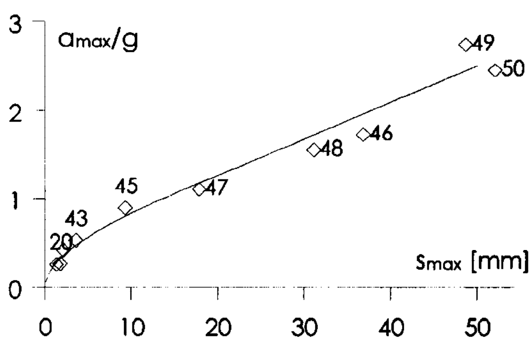


Fig. 10. Maximum measured displacement and accelerogram intensities

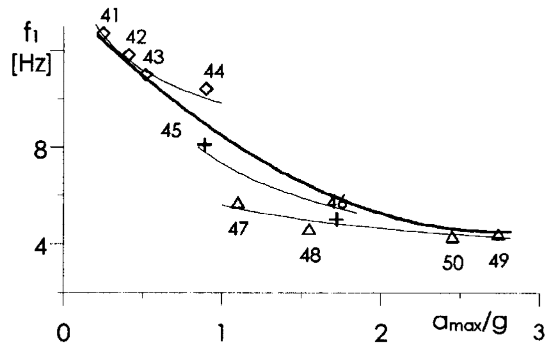


Fig. 11. The decay of the first frequency

its regularity and effective connections between floors and vertical walls. This reduces the probability of rotational motions which are quite frequently the origin of a partial and the subsequent total collapse. These conditions are difficult to find in the real world: the old masonry house in L'Aquila shows a continuous decay of the generalized stiffness with an excitation intensity that would lead to collapse, mainly related to the rotational components of the response and to the out-of-plane fall of the walls, as frequently observed during past earthquakes.

CONCLUSIONS

Forced vibration tests carried out in-situ or in a laboratory are a very effective tool for a more thorough understanding of the dynamic behaviour of masonry buildings, for which a priori estimates of the characteristics of materials, structural integrity and the effectiveness of connections between vertical walls and horizontal floors are difficult. From the two presented investigations some important aspects of the dynamic response beyond the elastic behaviour are shown, together with basic observations on resistance and the manner of collapsing. The role of identification techniques in processing experimental data is shown to be fundamental to a correct interpretation of test results and to extending their meaning beyond the cases studied.

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