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Structural analysis and damage evaluation of existing masonry buildings by dynamic experimentation and numerical modeling

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ABSTRACT: The paper deals with the analysis of the dynamic behaviour and the evaluation of the seismic damage of masonry buildings. For such a purpose a special nonlinear time – domain identification approach is proposed, basing on the comparison between experimental time – histories and numerical results. The proposed method is applied to an existing four – stories stone masonry building in Florence, which has been tested dynamically by the Authors in the elastic and post – elastic range of response. The same building has been modeled through a finite – element nonlinear numerical approach; some of the parameters of the model have been estimated according with the proposed identification method based on the comparison between numerical and experimental results.

1. INTRODUCTION

The analysis of existing building through experimental tests provides important data that make the structural analyst richer of available knowledge and information. Nevertheless, in order to employ data provided by a direct experimentation, appropriate tools have to been employed to correctly interpretate the results acquired.

In analyzing existing buildings, in fact, damaging connected with environmental actions (traffic, wind etc.) and permanent loads, together with the changes of the structural material (maturation or degradation of mortar and stones etc.) produce continuous variations of the structure's dynamic behaviour during the years; new structural surveys and specific in – situ tests are thereby required in order to assess the new state.

Such considerations are to be faced especially when old constructions are treated, with a particular reference to monumental buildings, whose particular relevance makes necessary to reach an accurate evaluation of their behaviour.

In order of reaching a knowledge of the structural behaviour of existing buildings, identification procedures appears to be the most effective tools, as they combine results from experimental, numerical and analytical approaches, permitting in this way a complete understanding of the structure examined.

Fig. 1: Overall view of the building

2. EXPERIMENTAL RESEARCH PROGRAM

A stone – masonry typical Florentine building, Fig. 1, has been tested through forced excitations produced by a vibrodine; the amplitude of the excitation has been increased until structural damaging and partial collapse of some elements of the construction have been produced. Results from the dynamic testing of the building and the determination of the inelastic range of behaviour have been preliminarly reported in [1].

The building, now demolished, was constructed in the first decades of the century with stone – masonry principal walls of about 35 cm in thickness and brick – masonry secondary walls with thickness of about 12 cm. The floors and the roof were realized with timber structure. In order of transmitting the vibrodine excitation to the main part of the structure, the vibrodine has been placed on a reinforced concrete slab costructed on the upper floor, (see Fig. 9). The vibrodine was capable of producing a sinusoidal force with a frequency range of $0 \div 24$ Hz and a maximum value of 100 kN.

Before the dynamic testing of the whole building,

Fig. 2.2: Cracking in diagonal compression specimens

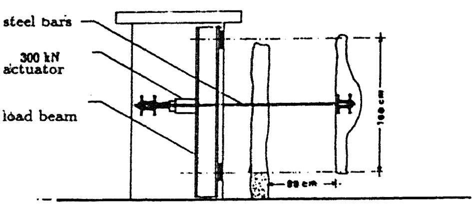

Fig. 3: In–situ shear test setup

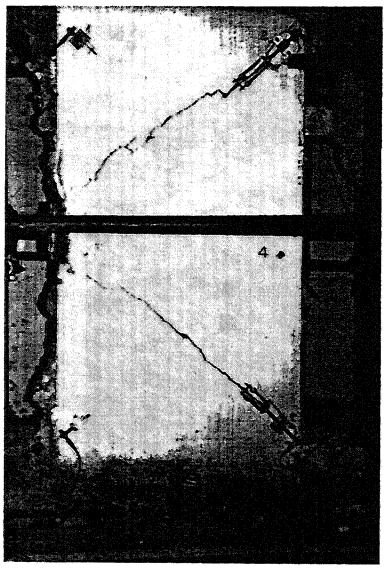

Fig. 4: Cracking at the collapse in the in – situ shear test

Fig. 5: Test for the assessment of the stress level

Fig. 6: Test for the assessment of the stiffness characteristics

an intensive experimentation has been performed on the same construction in order of evaluating the mechanical characteristics of the masonry and collecting all the necessary information to define its constitutive law [2].

The experimental program has comprehended tests on stone and mortar components, laboratory diagonal testing of panels re – constructed with material from demolished buildings of the same age of the one examined, in – situ shear testing of walls and in – situ testing with the flat – jack technique.

A real need of effective experimental data concerning masonry structures is in fact felt in order of correctly calibrating the numerical approach, also in view of the fact that several mechanical constants are required to express the very complex constitutive law of masonry. Despite the need, such data are generally not available for the more common masonry constructions. In order to redress this lack of data, the cited preliminary tests have been conducted by the Authors.

To aid the reach of a true understanding of the effectiveness of the experimental research, particular importance has been given to the correlation between laboratory and in–situ tests; for this reason, a series of laboratory tests has been performed on big – size specimens of full–scale thickness (about 40 cm), constructed in order to actually reproduce the actual masonry texture, Fig. 2.

In order of detecting the actual properties of in – situ masonry, panels to be tested according to the diagonal compression method have been constructed using several types of mortar due to the difficulty of detecting the actual mechanical characteristics of mortars in existing buildings. The mechanical characteristics of in – situ mortar has thereby been determined by comparing overall strengths of laboratory specimens with the results from in–situ tests.

Several in – situ destructive shear tests have been performed on masonry walls, subjected to their actual vertical loads and to a testing horizontal force imposed by an hydraulic actuator, Fig. 3. The two panels have been isolated from the rest of the walls by two vertical openings 35 cm in

Table 1: Results from flat – jack tests

Test	Load cycle (MPa)	E (MPa)
1B	0.0 – 0.4	490
	0.4 – 0.8	420
	0.8 – 1.2	510
2B	0.0 – 0.4	560
	0.4 – 0.8	390
	0.8 – 1.2	350
3B	0.0 – 0.4	470
	0.4 – 0.8	590
	0.8 – 1.2	620
4B	0.0 – 0.4	1050
	0.4 – 0.8	680
	0.8 – 1.2	700

width and 160 cm in height. The resulting testing specimen had dimensions of 80 cm (width) and 160 cm (height) according to the optimal dimensions indicated in [3], Fig. 4.

Finally, a series of flat jack non – destructive tests has been performed [2]. Both tests to obtain the stress level in existing walls (employing a single flat – jack, Fig. 5) and tests to investigate the stiffness characteristics of masonry (employing two flat – jacks, Fig. 6) have been executed.

As example of the results obtained, the E modulus of masonry determined at various load cycles for four of the tests is shown in Table 1.

3. NUMERICAL MODELING

Numerical modeling of masonry structures represents a very complex problem due to the constitutive characteristics of the structural material and its highly nonlinear behaviour when subjected to strong actions. However, in view of practical applications, numerical methods seem to be at present the only suitable way to achieve effective results; this consideration explains the recent large interest in developing numerical models for masonry material [4, 5].

In the present work, a 3D elastic – plastic finite – element with continuum cracking capabilities (smeared approach) has been chosen for representing the masonry, in order of avoiding the strong influence of the mesh connected to the use of models with discrete cracking capabilities such as in [6]. The element employed (the STIF65, included in the ANSYS code [7]) is capable of cracking in tension and crushing in compression and is employed for modeling of geological materials, masonry or unreinforced concrete. The plastic behaviour has been modeled according to the the Drucker – Prager law, applicable to granular material such as soils, rock and concrete, which uses the outer cone approximation of the Mohor – Coulomb law.

According to this choice, only three constants are required in order to define the constitutive law of the material: the cohesion value C, the angle of internal friction ϕ and the amount of dilitancy (the increase in material volume due to yielding), reducing in this way the number of uncertainties about the mechanical characteristic employed in the model. In this case, dilitancy is permitted and the flow

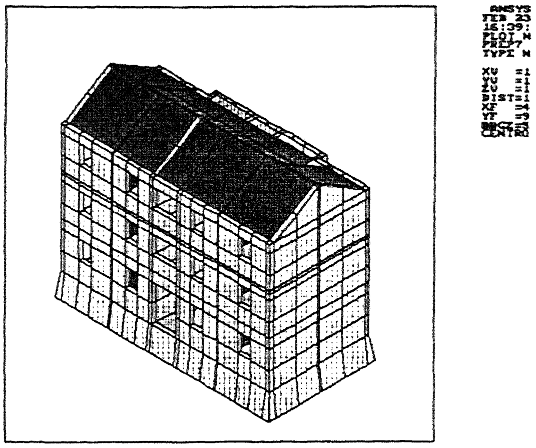

Fig. 7: overall view of the finite – element numerical model

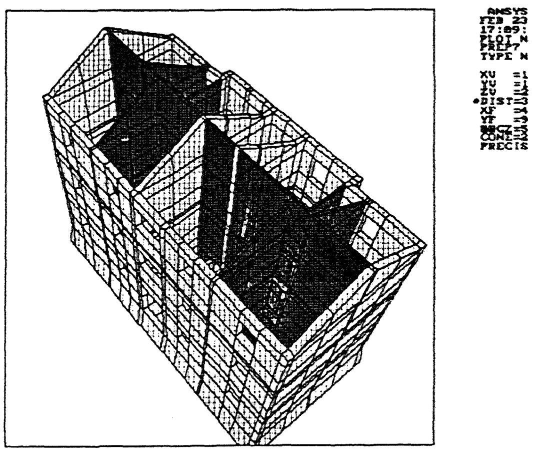

Fig. 8: view of the vertical structures inserted in the numerical model

rule is adopted in order to evaluate the plastic deformations.

Further elements such as membranes or elastic – plastic plates have been employed to model the floors and the roof, see Figs. 7 and 9.

An important feature connected with the arrangement of the numerical model is given by the capability of representing the decrease of the stiffness properties of the structure when the yielding condition is reached in some points of it, so as to model the damaging of the structure. Especially in an earthquake – prone area, in fact, the possibility of evaluating the actual seismic behaviour of masonry buildings represents a very effective problem in order to assess the safety of the architectural heritage. In view of the above requirement, some preliminar studies have successfully been performed in [5].

Table 2 depicts the results of a first identification process performed in order of setting the modal outputs in the initial state (undamaged) so as to reproduce experimental results. The exact in – situ determination of the natural frequencies has been possible only up to 8 Hz; for

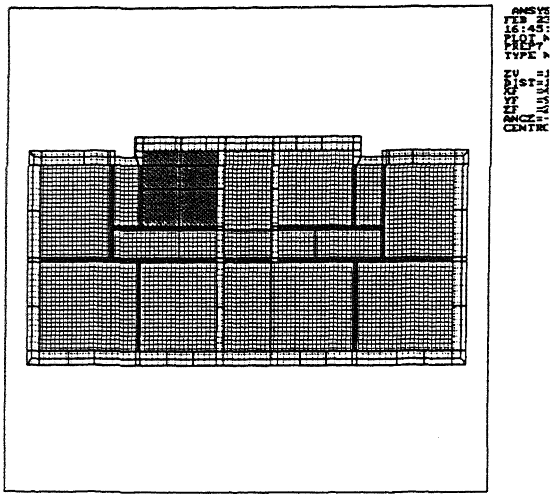

Fig. 9: view of the upper – story plan of the building with the reinforced concrete slab constructed in order of substaining the vibrodine (numerical model)

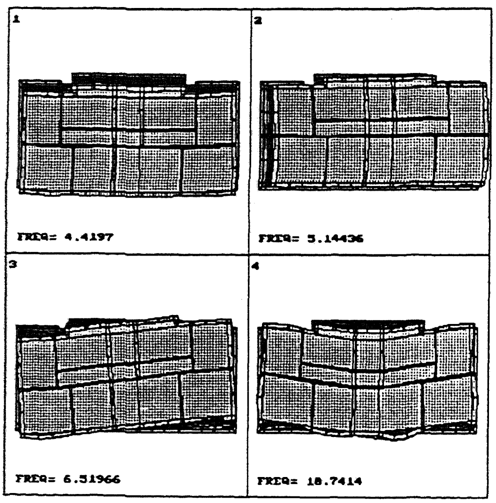

Fig. 10: modal shapes determined by the numerical model

higher values of frequency, a sure evaluation of the eigenfrequencies has not been possible.

4. STRUCTURAL IDENTIFICATION IN THE TIME DOMAIN

Structural identification is here referred to as the assessment of the model of a given construction by combining available a – priori knowledge and data from a direct testing of the building. In this sense, the differences between the monitored behaviour and the numerical results have to be carefully examined as they represent the starting information of an identification process; these differences are here named as *deviation*. According with the technique used to determine the *deviation*, different identification procedures are possible

Table 2: natural frequencies of the construction (Hz)

Mode no.	experimental	numerical
1	4.88	4.50
2	5.18	5.23
3	6.78	6.78
4	8.98	13.4

[8, 9, 10, 11, 12].

The process of developing the model, that may involve only some parameters or the whole mathematical formulation of the model itself, is here named as *estimation*.

When the identification process is performed with respect to a linear structure, the *deviation* and the *estimation* are performed referring to modal parameters (MP) [13] or, although less usual, transfer functions (TF) [14]. When the structure investigated is nonlinear, on the contrary, MP and TF cannot be defined uniquely, so that the use of these techniques requires more complex developments; such developments have to be based on the comparison between experimental and numerical time – histories [14]. Nevertheless, as time – histories are usually less workable in order to determine *deviation* and *estimation*, the time – history approach involves strong analytical difficulties; when a building exhibits nonlinear behaviour also under low – level actions (as in the case of masonry buildings), however, only a nonlinear identification procedure can correctly be developed.

Although time – histories are more difficult to be handed than MP or TF, their use can sometimes considerably reduce errors due to numerical processes, as the passage from time – domain to frequency – domain, FFT technique etc.

In order to combine time – histories analysis with the available a – priori knowledge, a special identification procedure is here proposed, with a particular reference to the building examined.

4.1 Time – domain identification for a nonlinear system

In order to compare the numerical model results with the experimental ones, a comparison between time – histories corresponding with the same inputs is performed. Fig. 11 depicts the comparison process in a simple single – input single – output scheme.

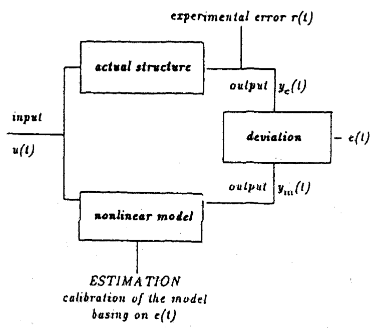

Fig. 11: identification process scheme

As represented in Fig. 11, the function $e(t)$ allows to compare the experimental and numerical outputs in the time domain; $e(t)$ is defined as:

$$e(t) = y_e(t) - y_m(t) \qquad (1)$$

still referring to Fig. 11, in order to point out the experimental error, the function $y_e(t)$ can be written as:

$$y_e(t) = x_e(t) + r(t) \qquad (2)$$

where $r(t)$ is the error that adds to the ideal errorless measure $x_e(t)$.

Such a scheme sets the experimental error only in the output, while it exists also in the input; however, the input error can be transferred in the output error only in the case of linear system [8, 9]; thus, with eq. (2) an approximation is thereby introduced. Moreover, in the proposed scheme numerical errors are assumed to be negligible, assuming their magnitude much inferior than the experimental ones.

Due to the sampling at a fixed pitch of the experimental results, difference equations have to be introduced to hand the numerical results:

$$y_m(k) = - \sum_{v=1}^{n} a_v \, y_m(k-v) + \sum_{v=0}^{n} b_v \, u(k-v) \qquad (3)$$

where a_v and b_v are the $n+(n+1)$ parameters to be estimated in the identification process and k a discrete variable referring to the time sampling. Thus, the response at time k depends on the previous output values from time $k-n$ and on the input values from $k-n$ up to the time of observation. The parameter n is thereby an additional important characteristic of the system, to be considered as the order of a differential equation.

Once the a_v and b_v parameters involved in eq. (3) are evaluated through the comparison between numerical time – histories and results from the same equation, the use of this expression can substitute the numerical model in order of obtaining solutions for different values of the input parameters. For such a purpose, eq. (3) should be transformed in the following:

$$y_m(k) = - \sum_{v=1}^{n} a_v \, y_e(k-v) + \sum_{v=0}^{n} b_v \, u(k-v) \qquad (4.4)$$

in which y_e replace y in the first term at the second member.

The application of the above general formulation does not usually suit with the use of a parametric numerical model with a large number of degrees of freedom. In the case of analyzing a complex structural system, however, a complex numerical model appears as the only way to take into account the available a – priori information. Due to this consideration, an identification approach with the following requirements has been develop:

- the model to be identified is a nonlinear finite element model;
- the model is characterized by a large number of degree of freedom, together with a nonlinear constitutive law for the structural material; due to these unavoidable conditions, the time spent to obtain a step – by – step solution for a significant load – history is very relevant;
- some of the less evaluable characteristics of the model are assumed as unknowns, i.e. these characteristics are referred to as the parameters to be identified;
- due to the nonlinear formulation, the comparison between numerical and experimental results have to be conducted in the time domain.

Basing on the above assumptions, an identification procedure has been arranged involving an appropriate parameters estimation technique to apply in the case of masonry buildings.

Estimation problem has been stated in the following general form:

$$m \in \{ M(x) \mid x \in H \} \qquad (5)$$

where m represents the final (identified) model, $M(x)$ the class of models in which m is searched, depending on a set of parameters x, and H the space of the parameters x.

Eq. (1) is then rewritten as:

$$e(k) = y_e(k) - y_m(k, x) \qquad (6)$$

and the parameters estimation problem is solved by introducing an appropriate functional $\mathcal{F}$, depending on $e(k)$:

$$\mathcal{F} = \mathcal{F}\,[k, x, e(k, x)] \qquad (7)$$

to be set carefully with regard to the specific problem treated, since the identification results depend on the $\mathcal{F}$ definition.

In the present approach, $\mathcal{F}$ have to be defined in order to reduce the computational charge, as:

$$\mathcal{F}_i = \frac{\sum_{k=1}^{L} |e(k, x)| \, \Delta t_k}{T_L - T_1} \qquad (8)$$

where the index i refer to the i – th degree of freedom of the numerical model; considering the whole model, hence, a second functional Υ is defined:

$$\Upsilon = \sum_{i=1}^{N} \frac{\sum_{k=1}^{L} |e(k, x)| \, \Delta t_k}{T_L - T_1} \qquad (9)$$

being N the total number of degrees of freedom of the model. The general form of the *estimation* is finally represented as:

$$m = \{ M(x) \mid x \in H, \; \Upsilon(x) = min \} \qquad (10)$$

4.2 Parameter estimation technique

The finite – element numerical model has been revealed capable of reproducing the behaviour of the actual construction with a closer agreement due to the great accuracy of the geometric representation and the appropriate setting of the constitutive relations for the masonry material. For the same characteristics, the time required to obtain a solution in a step – by – step dynamic integration is quite relevant. Nevertheless, in order of calibrating the model in an identification process, a large number of numerical results would be required.

To overcame such a problem, the functional Υ, whose minimum represents the solution of the identification process, has been expressed in an analytical form. The expression of Υ has been obtained through an appropriate interpolation of results from a little number of solutions provided by the numerical model. Hence, the functional Υ has been replaced by the paraboloid Υ_p:

$$\Upsilon_p = \alpha + \sum_{i=1}^{N} (\beta_i \, x_i + \gamma_i \, x_i^2) \qquad (11)$$

being α, β_i and γ_i some coefficient to be evaluated and x_i the set of free parameters to be identified in the process.

As appears in Eq. (11), the expression of Υ_p lacks of the cross terms x_i x_j; such an approximation is performed in order to simplify the formulation and it has been found well acceptable in the case examined. It is easy to note that in evaluating the $2N+1$ values of α, β_i and γ_i in the Υ_p expression, at least the same number of solution of the model is required for different values of the x_i parameters.

The minumum of Υ_p, obtained via simple analytical operators in a pre – fixed range of the parameters, provides the estimated x_i values. A much more expensive computational charge to obtain a large number of numerical solution is thereby avoided and, moreover, usual estimation techiques can be employed [8].

Obviously, the minimum of Υ_p represents only an approximation of the minimum of Υ, to be accepted with regard to the problem that is faced. In some pathological cases, a sensitivity analysis may be opportune to evaluate the level of accuracy of the solution.

5. THE ASSESSMENT OF DAMAGE

Structural damage will result in degradation of the resistance properties of structural elements, for example when cracking or buckling occur. Very often, especially in the case of reinforced concrete or masonry rather than in the case of steel structures, the stiffness characteristics of an element or of a joint will degrade when damage occurs. From this local or global stiffness degradation, a general shift of the natural frequencies towards lower values will result. The assessment of a correlation between the severity of damage and the stiffness degradation as a function of the variation in the fundamental period of a given structure has been often attempted in the past.

For the studies here described, both the shift of the natural frequencies and the modification of the transfer function during the dynamic excitations of the structure have been considered to assess the damaging of the structure [1].

By alternating small and large – amplitude tests, the damaging threshould have been detected when the shift of the natural frequencies recorded in a large – amplitude test was maintained also in a successive small – amplitude test, distinguishing in such a way damaging from the nonlinearity of the response.

During the dynamic testing, a maximum permanent shift of the eigenfrequencies of about 10% has been reached at the highest level of excitation (compatible with the necessity of avoiding damages at the nearest buildings).

The dynamic testing has been developed also in view of collecting information about the nonlinearity of the dynamic behaviour of the building, so as to detect the limits of admittibility of a linear elastic model. At the maximum level of excitation, an overall shift of about $35 \div 40\%$ of the natural frequencies has appared, of which about $25 \div 30\%$ due to the nonlinearities of the construction and only 10% due to damage.

6. CONCLUSIONS

A stone – masonry building in Florence, destined to demolition, has been subjected to an experimental research program comprehending a series of dynamic tests, performed so as to produce the damaging of the structure. On the basis of the intensive experimentation, a nonlinear finite – element model of the whole building has been set, capable of reproducing the actual behaviour of the structure. A particular attention has been focused on the modeling of cracking phenomena and on the possibility of reproducing the decrease of the natural frequencies of the structure due to damaging.

A time – domain identification procedure has then been defined in order of evaluating some of the parameters of the numerical model. Such a procedure, related to an highly nonlinear structure, has been based on time – histories analysis, and it has been appropriately arranged in view of the peculiar characteristics of the problem so as to require only a little number of numerical solution in order of performing the identification of the structure.

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