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Experimental evaluation of the dynamic behaviour of a stone masonry building causing progressive damage

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ABSTRACT: An experimental in situ dynamic investigation has been carried out to study the behaviour of a masonry house located in Florence. The study presents highly interesting features, because the structure considered is fairly representative one within the framework of the type of masonry building to be found in a large area in Florence; moreover the building was possessed of those features of symmetry and regularity that should make it easier to interpret the results and, finally, it stood alone, which made it possible to subject it to considerable stresses while avoiding transmitting detrimental actions to neighbouring buildings. The complete building has been shaken by sinusoidal loads developed using a two rotating masses vibration generator (vibrodine) capable of delivering forces of considerable amplitude. Twenty-two tests have been performed with different forces intensities and frequency range. At first, low-intensity forced vibration tests were carried out in order to identify the dynamic response of the building under operating conditions without damage; subsequently, dynamic tests with progressively growing intensity were effected, till vibrational levels comparable to those which were to be expected during an earthquake were obtained. The dynamic response of the structure has been measured by means of a suitable network of instruments consisting of accelerometers, seismometers and transducers of relative displacement.

1 INTRODUCTION

A series of dynamic tests of the same type of this given in (Capecchi, Vestroni, Panzeri, Pezzoli, 1990), has been performed on an isolated building in Florence. The building, of stones and bricks mixed masonry, which had been destined to demolition for purely functional reasons and was showing a perfectly solid structure.

This research has been planned with the purpose of obtaining as many information as possible on the structural behaviour of the building with particular reference to the inelastic field. The general criteria that have influenced this experimental research have been chosen with the aim of obtaining the dynamic response of the building in different situations. Particular emphasis is carried out to define the procedure for applaing the forced vibration and to the placing of the measurement devices.

2 DESCRIPTION OF THE BUILDING

The tested building is in Florence, just out of the historical centre. The construction made at the beginning of the century shows vertical walls of mixed stones and bricks masonry with timber floors and roof. For its structural organization the building can be considered a typically Florentine example dated between W.W.I and the first decade after W.W. II.

The building has a regular plan, three floors above ground and one in ground; the plan dimensions are approximately 18.3 m x 8.6. The walls are over 35 cm thick and are mainly constructed with by stones with some bricks and they include the external walls and some transversally interior ones. Inside partition walls some of which help sustaining the floors, are made in brick masonry.

The floors are in timber, with both a primary and a secondary structure or only just the primary one, with lime mortar base course and floor. The ground support floor instead is constructed with small volts in bricks and metal beams, filled with lean mortar. The roof covering has two pitches also in timber with a clay covering of pan tiles.

3 THE EXCITATION AND MEASUREMENT SYSTEM

To involve the structure as much as possible in the dynamic test, a reinforced concrete slab has been inserted on the floor, in that where the vibrodine has been placed. Such slab have been placed over the surrounding vertical walls by a boundary beam connected to the slab by reinforced bars. The slab layer has been banded on the inner walls with a welded net. The employed vibrodine, with two eccentric masses, can produce a sinusoidal force with a frequency ranging between the interval 0-24 Hz, and a maximum value of 100 kN (which can vary modifying the mass eccentricity).

The response has been measured with a net of accelerometers, seismometers and relative displacement transducers. The positioning of the instruments has been made to obtain bending and torsional modal shapes. Taking into account all 5 floors (basement, ground, first, second and roof), considering also the deformation of the floor layers. To investigate the localised damaging 8 displacement transducers have been placed on the junctions of the orthogonal walls of the second floor. Also consequently 4 additional transducers have been placed on the support beams of the second floor to detect the relative displacements between the beam and the wall.

4 PROGRAMME AND EXPERIMENTAL TESTS

Data obtained from the vibrations produced by the traffic have been considered as a first step for the dynamic experimentation. Such data have enabled a first dynamic characterisation of the building that afterwards has been used for selecting the frequencies to be attributed to the vibrodine in the induced vibration tests. Corresponding, the determination of the vibrations due to the traffic has allowed a first evaluation of the response threshold, connected to the dynamic load that has to be considered as a permanent load in the life of the building.

From this preliminary investigation the first three frequencies of this building have been obtained. The first three natural frequency values are obtained from the dynamic response of the structure (the first two bending and the first torsional) as those frequency values corresponding to a peak in the FFT and where the coherence functions increase considerably (Bendat, Piersol 1980), though not near the unit.

The induced forced vibrations with the vibrodine have been performed to obtain the following results:

- evaluation of the dynamic behaviour of the building;
- individualisation of the structural damage starting;
- evaluation of the forcing vibration level producing a non-linear behaviour, and evolution of the response non-linearity with the damaging;
- study of the structural damaging.

The conceptual points characterising such experimental test are the following:

a) F is the amplitude of the forcing such as to produce a vibration of the possible lower amplitude, but such as to allow measurements sufficiently precise (maximum value = 7 kN); a first test with the amplitude of the forcing equal to F and then subsequent tests with values of the forcing amplitude multiple of F.

b) The structural response is considered as transfer functions(h): these have been obtained both with increasing scanning frequencies h^s and decreasing, h^d.

c) As the forcing amplitude increases, possible differences between the obtained response and the one produced by F, can be interpreted both as the starting point of the structural damaging or as the beginning of a non-linear behaviour. The damaging is reviled by a further initial test with initial amplitude F. If the h^s functions appear during such test equal to those obtained during the initial test as described in a), then the difference in the response has to be attributed only to a non-linear behaviour; otherwise the threshold beyond which begins a damaging phenomenon is. In those cases where the beginning of the damaging is localised, the comparison between the responses h^s and h^d reveals if with the damaging also non linear behaviours have occurred.

d) In case a damaging state is found, to establish if this may have produced a stabilised degrade, or if the started degrade increases progressively, a further test with F equal to the level of the forcing amplitude has been prefixed: if h^s equal to the value obtained in c), that is different from the initial one, but equal to the one obtained right after the damaging, then the degrade is stabilised; otherwise the degrade has started and progresses even at the lowest level of excitation.

e) To establish if the new dynamic behaviour produced by the starting of the structural degrade involves thresholds of non-linearity different from the initial ones, a further test with F excitation level is repeated and compared with that obtained as in d): if the two responses appear equal a linearity in the case is found, otherwise a different threshold of non-linearity is revealed.

Many tests (22) have been performed, varying the configurations of the analysed structure, in particular the excitation direction, the intensity of the used forcing and the considered frequency field. The so obtained data have been elaborated and organised to give directly the diagrams of the transfer functions. This diagrams regarding acceleration, velocity or displacements and the modal parameters of the first modal shapes of the building.

The transfer functions h_{kj} have been computed as ratio between the response q_j in position j and the exciting force F_k in position k in the frequency domain:

$$h_{kj} = \frac{q_j}{F_k}\, e^{\,i(\phi_j - \phi_k)} \tag{1}$$

and ϕ represents the phase.

Such magnitudes have been obtained directly during the acquisition phase obtaining the frequency, the amplitude and the phase of the response and of the forcing, to obtain automatically the transfer functions. From the transfer function peaks the modal parameters have then been obtained (Bendat, Piersol, 1980).

5 ANALYSIS AND INTERPRETATION OF THE EXPERIMENTAL RESULTS

From the data obtained with the test performed in the experimental programme above described, the analysis and the interpretations bellow described have been developed.

5.1 Floor behaviour

The number and positioning of the transducers have been studied to allow also an accurate evaluation of the floor response. Particularly, from the difference between the experimental response and that theoretically obtained in the hypothesis of floors infinitely rigid, the limits of this hypothesis have been evaluated about the structure here examined.

For such purpose the structure has been represented like MDOF system with fifteen degrees of freedom (three kinds of motion, translation x and y, rotation t, for the 5 floors). We have then computed the stiff motion vector of "best fitting" $\{q_r\}$ starting from the six-dimensional vector of the experimental results $\{q_s\}$ for each floor:

$$\left\{\frac{\partial^2 q_r}{\partial t^2}\right\} = [A]^{-1}\left\{\frac{\partial^2 q_s}{\partial t^2}\right\} \tag{2}$$

where $[A]^{-1}$ is the 3 x 6 dimensional matrix, pseudo-inverted of matrix $[A]$ of "best fitting" according to the minimum square error, having the direction cosine of the measured displacements and the distances between the instrument point and the center of gravity. We have then evaluated the

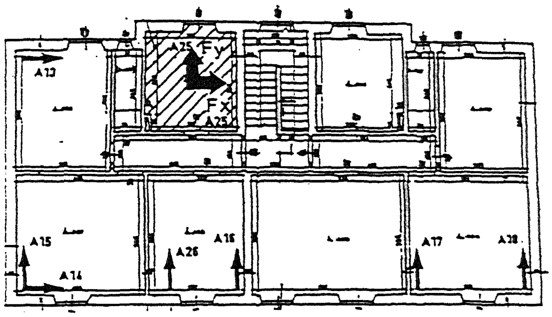

Figure 1. Plan of second floor with reinforced concrete slab where the vibrodine has been placed and position of the accelerometers.

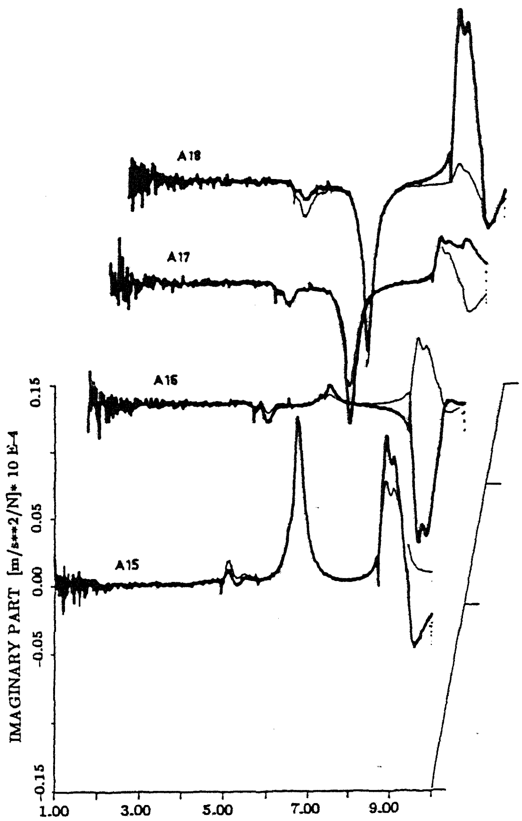

Figure 2. Imaginary part of the responses, test n. 3, DC= F, excitation x direction (— experimental, — analytical).

vector of the theoretical rigid movement $\{q_t\}$ in the instrumented points, starting from those evaluated with the hypothesis of rigid floor $\{q_r\}$:

$$\left\{\frac{\partial^2 q_t}{\partial t^2}\right\} = [A]^{-1}\left\{\frac{\partial^2 q_r}{\partial t^2}\right\} \tag{3}$$

To evaluate how much the hypothesis of rigid floor has been disagreement, index χ associated to the discrepancies between theoretical and experimental behaviour has been introduced (Bendat, Piersol, 1980, Richardson, Potter, 1974):

$$\chi(\omega) = \sqrt{\frac{\sum_{1}^{N} \epsilon_i^2(\omega)}{N}} \tag{4}$$

where:

$$\epsilon_i = \mathrm{I}\left[\left\{\frac{\partial^2 q_{si}(\omega)}{\partial t^2}\right\} - \left\{\frac{\partial^2 q_{ti}(\omega)}{\partial t^2}\right\}\right] \tag{5}$$

The analysis of the results shows how, in the field of frequencies between 1-8 Hz the three floors can be considered as rigid. For higher frequencies, instead, the behaviour changes and considerable discrepancies can be found between the theoretical and the experimental results. Besides we have noticed how such discrepancies become higher proportionally with the vibration amplitude. (See Figs. 1, 2, 3, 4 and 5). Through the prepared instrumentation net we have besides observed that the junction floor-wall behaves like a hinge, preventing relative creeps, or after the creeps are disconnected, as the frequency varies. Particularly, the junction can be considered like a hinge in the same frequency interval where the floors can be considered rigid in own plane.

5.2 Structural damaging

The main approach for the evaluation of the structural damaging is that such phenomenon can be produced forcing the structure under large displacements, where the post-elastic behaviours become predominant. This is underlined by the production of an irreversible variation of the structural behaviour of the building also during small displacements.

The threshold of the starting damage has been evaluated during test No. 13, with an amplitude of the forcing equal to 16 F. The corresponding variations of the frequencies belonging to the building consist in a reduction of themselves of 2%. We have then determined the reduction of the frequency produced by the maximum damaging consequent to a force of amplitude 54 F, corresponding to 10%.

Coinciding with the frequencies variations, the modal forms have remained almost unaffected by the structure stiffness reduction; at last the damping ratio has shown variations of discordant sign from one vibration to another. Further interpretative results concerning the propagation of the damage: it has been noted that the damaging appears stable for small amplitudes of the forcing vibration while it tends to increase as the excitation increases (starting from an amplitude of 4 F).

The relative displacement transducers in the orthogonal walls have shown to be very significant for evidencing the damaging. The transducers have shown a progressive decrease in the junction stiffness. This result has inadequate to find in the junction between the orthogonal walls some points of great vulnerability of the building during horizontal actions. It can also be noted that, been such damaging localised, it can be better found with local measurements, rather than with global investigations, that show significant variations only in presence of relevant and diffuse damaging.

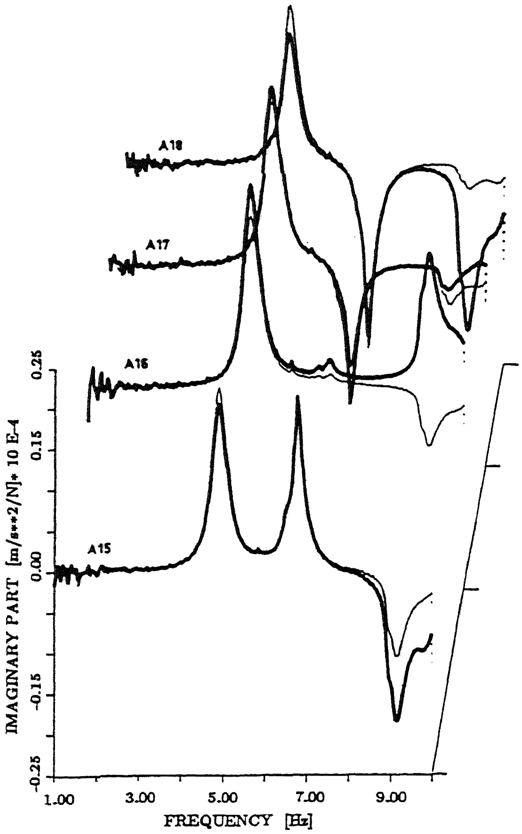

Figure 3. Imaginary part of the responses, test n. 5, A=F, excitation y direction (— experimental, — analytical).

From this first examination of the results it appears evident that considering the high damaging produced by a force of 54 F (which has produced vibrations comparable to those established by the actual Italian code for medium seismicity zones assuming a behaviour coefficient $\beta = 2$), the lowering in the frequencies, although considerable, is not such as to modify mostly the dynamic behaviour and the level of the seismic design action of the project.

5.3 Non-linearity

The experimental tests, as previously reported, have been conducted to obtain information on the non-linearity dynamic behaviour of the building. The knowledge of the level in which a linear analysis is possible and the knowledge of the errors that can be brought in when using such hypothesis, is essential for the modelling of masonry structures. For such structures, considering the small tensile and shear strength of the masonry, depending besides on the compression value, it is very important to define the acceptable limits of the linear behaviour hypothesis.

The non-linear behaviour is recognised from the damaging phenomena reducing the amplitude of the forcing vibration: the non-linear behaviour mobility can be cancelled reducing the induced oscillation levels. The non-linear behaviour begins to show during test No. 9 (See Figs. 6 and 7), with a

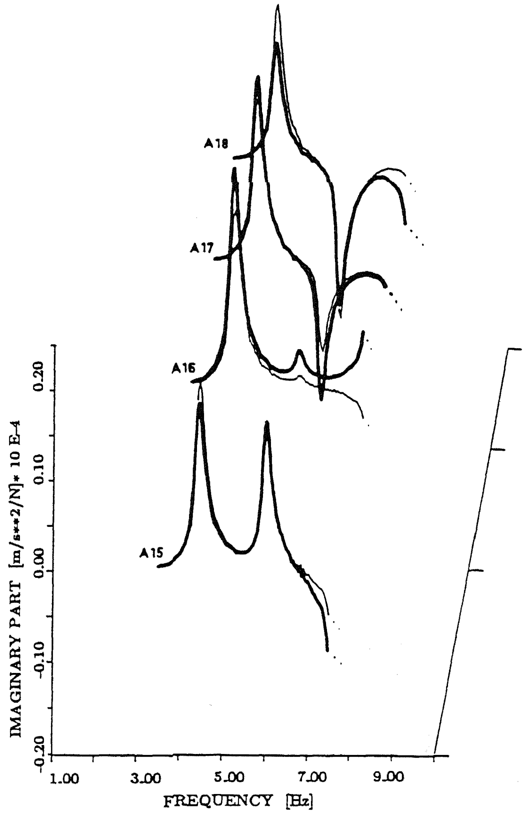

Figure 4. Imaginary part of the responses, test n. 13, O=16F, excitation y direction (— experimental, — analytical).

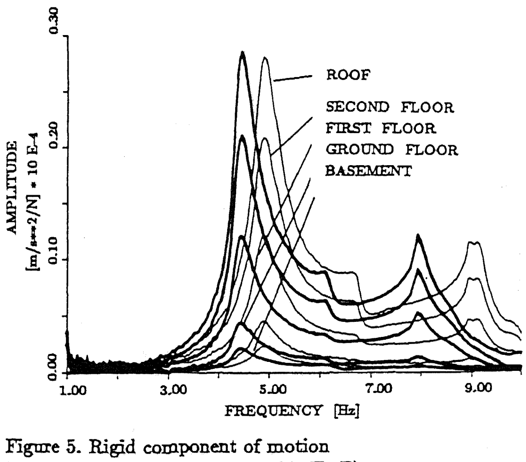

Figure 5. Rigid component of motion
(— test n. 5, A=F; — test n. 20, U=F)

value of the force of 4 F (1/4 of the value that produces the first damaging).

The following considerations can be made on the researches performed:

a) With a forcing vibration of 4 F a reduction of the

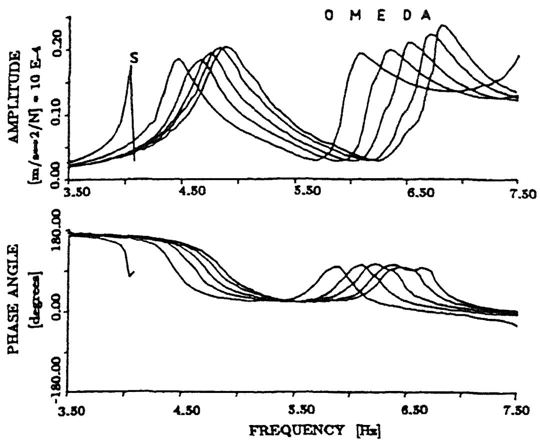

Figure 6. Transfer function (accelerometric pos. A15), tests: A=F, D=2F, E=4F, M=8F, O=16F, S=46F.

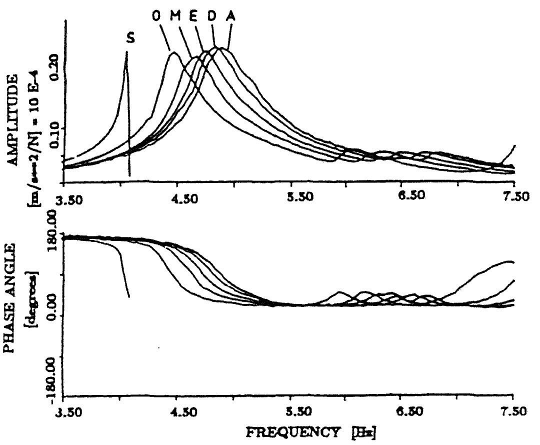

Figure 7. Transfer function (accelerometric pos. A16), tests: A=F, D=2F, E=4F, M=8F, O=16F, S=46F.

frequencies of 2-3 % can be obtained.

b) With a forcing vibration of 16 F (beginning of the damaging) the variation of the frequencies, which appears of 8%, belongs 6% to the mobilisation of nonlinear components, while 2% to the degrade.

c) With a forcing vibration of 54 F variations of the frequencies of 35-40% are produced, 25-30% of which belongs to the non-linear mobilisation, while about 10% to the degrade.

d) After the maximum damaging, a test with amplitude F does not produce non-linearity; nevertheless such phenomena began to appear at a smaller level of that previously recorded (forcing vibration equal to 2 F).

We can therefore observe that the degrade phenomenon, while progressing constantly during the tests, tends to reduce the resonance frequency value. While the excitation frequency, during the test, increases: because we have a sudden outgoing from the resonance condition, which appears on the amplification peak shape, with a tendency to a hardening spring jump.

6 SEISMIC TESTING

The experimental tests have been extended to very heavy excitations with the only limit of not produce damages to the surrounding building. This has unable to produce inside the building a vibration comparable to that of an earthquake of considerable level.

Since the experimentation has made its own frequencies, the modal shapes and damping, after having defined the mass matrix it is possible to know the response of the building compared to any dynamic excitation, except the non-linear behaviour. Particularly it has been possible to derive the seismic participation coefficients C_k. Starting from the relation that gives the response q_{ki} of point i referring to the k mode, we have considered the acceleration response spectrum $S_a(\omega_k, \nu_k)$ according to the present Italian code:

$$\left\{ \frac{\partial^2 q}{\partial t^2} \right\}_{max} = C_k \, \Phi_k \, S_a(\omega_k, \nu_k) \tag{6}$$

being Φ the k-th shape form. We have considered the medium seismicity zone; the adopted behaviour coefficient, β = 2, corresponding to an exercise limit design. About reference to the evaluated modal parameter maximum damaging and the coefficients of participation of the first mode deriving from simplify MDOF system adopted to modelling the structure. The maximum accelerations due to the earthquake have been evaluated and they appear almost equal to 1.5 m/s^2. It has to be noted that, during the tests with maximum forcing vibration, acceleration of 1.6 m/s^2 has been reached. It is therefore possible to say that the tests performed have produced, from a seismic simulation point of view, vibrations comparable to those attributed by our code to second category zones. In this sense however such seismic event, having $\beta = 2$, should be considered absorbed in exercise, i.e. in an elastic field. The structure of the building instead had to overcome amply in the inelastic field to absorb such vibrations. Therefore the structure does not have the security requirements according to the standards for new buildings or a seismic upgrading of an existing building.

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REFERENCES

Vestroni, F., Capecchi D., Panzeri P., Pezzoli P., 1990. *Analisi sperimentale del comportamento dinamico di una casa in muratura del '700*. Ingegneria Sismica, Anno VII, 2. (In Italian).

Bendat, J. S., Piersol, A.G., 1980. *Engineering application of correlation and spectral analysis*. John Wiley & Sons, New York.

Richardson, M., Potter, R., 1974. *Identification of the modal properties of an elastic structure from measurement transfer data*. New Mexico, XX International Instrumentation Symposium.