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Shaking table study of a reinforced masonry building model

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ABSTRACT. A shaking table investigation on a reduced scale model of a mixed reinforced masonry-R.C. building struture is presented and briefly discussed. The testing procedure proved to be capable to reproduce the most relevant aspects of the actual seismic behavior of the considered structure. Model's dynamic parameters identification allows quantitave evaluations of the structural damages induced by actions of given intensity.

1 INTRODUCTION

Shaking table investigations on reduced scale models of buildings made with plain and reinforced masonry are being used in Italy with the main scope of calibrating theoretical models of the overall behaviour under seismic actions of these structures. The models are intended to be as simple as possible, capable to reproduce the essential aspects of the seismic response of the structures under consideration.

The basic mechanical properties of the materials have been already experimentally studied and analytically modelled, as for example reported by Bernardini-Modena-Vescovi (1983)and by Giuffre'-Macchi-Modena (1984). The simplified analytical-numerical approach to the dynamic analysis, which is intended to be used for calibrations, has been presented by Modena (1982) and Modena-Barel(1986).

Simplified experimental techniques have been first extensively used as described by Modena-Tomazevic (1990). The more sophisticated testing facilities available at the laboratory of ENEA-Casaccia, in Rome, are being being used to control and validate the simplified procedure.

2 MAIN ASPECTS OF THE INVESTIGATION

The building model which has been tested has the same characteristics of one of the four models described by Modena-Tomasevic (1990) and used for the simplified investigations. The 1:5 scale model is shown in Figure 1. It reproduces the most relevant architectural and structural characteristics of typical italian residential buildings. Namely, it is three storey high, made with peripheral reinforced masonry walls and internal r.c. column and with r.c. floor structures (slabs and tie-beams).

Additional masses are placed at each floor level to reproduce the prototype mass distribution. Moreover, as prototype materials are used for the construction of the model, vertical prestressing ensures adequate distribution of stressess for reproducing the mode of failure of the masonry walls.

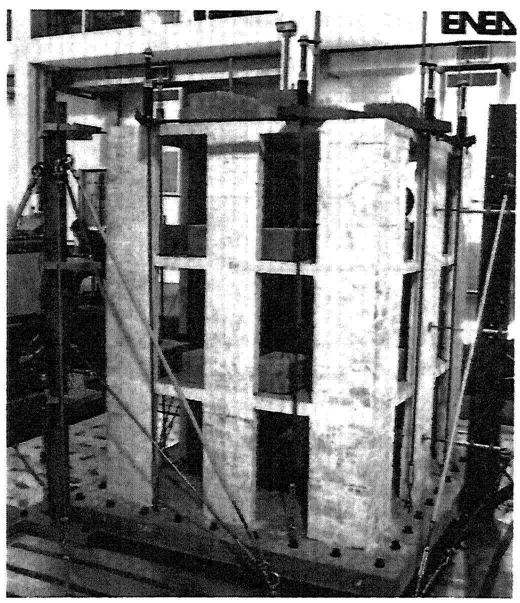

Figure 1. General view of the model as it was instrumented and installed on the shaking table.

The shaking table is a MTS system, capable of controlling six degrees of freedom and operating in the frequency range between 0.5 and 50 Hz. The table size is 4 meter x 4 meter, and the maximum weight of the specimen can be 10 metric tons.

Several series uni-directional and bi-directional excitations have been applied in the linear elastic range of vibration of the model. The input acceleration time histories used in this phase has been numerically generated from response spectra given in the Italian an European seismic codes. The uni-directional excitation has been have been used to simulate the test conditions of the previously cited simplified dynamic test procedure. The excitations applied in two orthogonal directions were assumed to have a fixed ratio between their maximum intensties equal to 0.7. Only bi-directional excitations of progressively incresing intensity have been applied in the non-linear range of vibration of the model. A real accelerogram, recorded during the 1978 Montenegro Earthquake, has been in this case used.

The model has been instrumented as shown in Figure 2 in order to control both X-Y translations and Θz rotations.

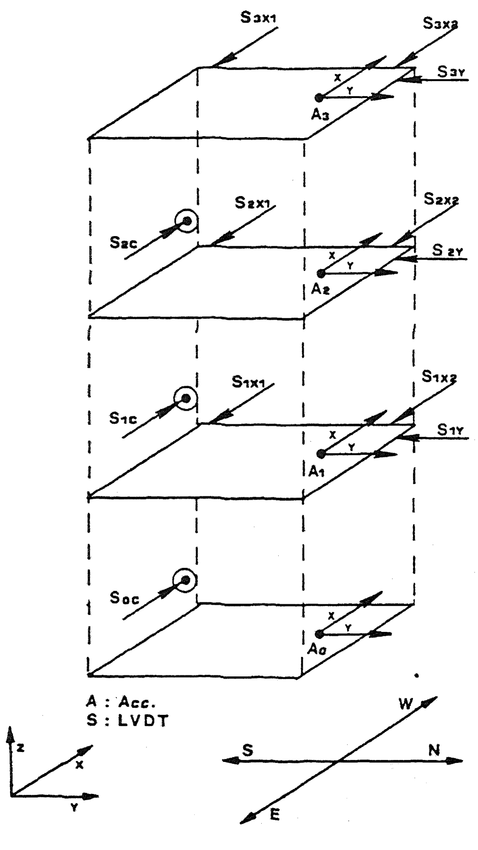

Figure 2. Model instrumentation.

After each series of test runs in the linear range of vibration and after each run in the non linear range, the model's dynamic properties has been determined applying sinusoidal X-Y translational and Θz rotational excitations.

3 DISCUSSION OF THE MAIN RESULTS

Relevant results of the investigation are detailed observations of the formation and progressive development of the crack patterns of the masonry walls and the numerical records of displacements and accelerations time histories.

The first type of records (pictures and diagrams) are indispensable first of all for validating the experimental procedure. The comparison with the damages observed in masonry buildings striken by earthquakes and during full scale testing of masonry panels should in fact demonstrate that similar failure mechanisms are reproduced, at least of the major structural components (the masonry walls) during the dynamic test on the reduced scale model.

Very significant appears, from this point of view, the results obtained during this research. A typical example of the evolution of the crack pattern in one of the base walls of the model (the south wall of the S-E corner) is shown in Figure 3.

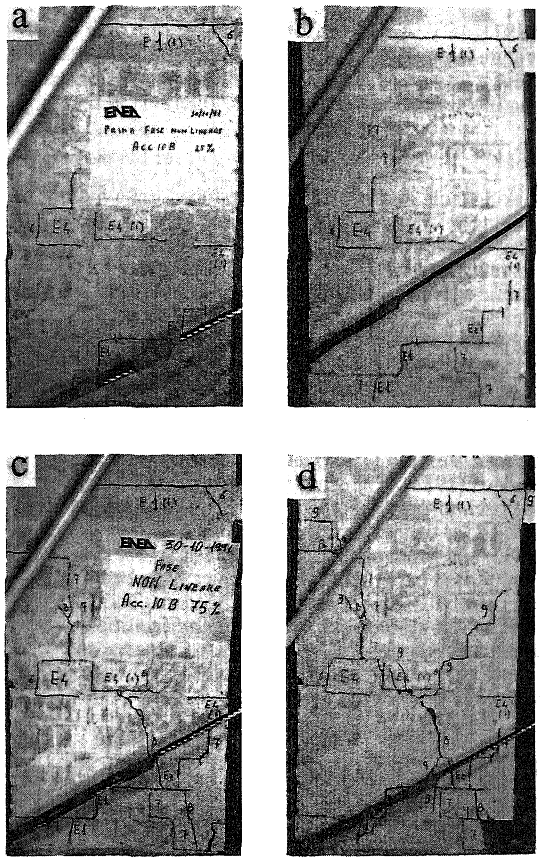

Figure 3. Typical crack pattern evolution on one of the base walls of the model.

The four pictures show the cracks which have formed after base excitations were applied whose maximum intensities were respectively (having taken into accoun the scale effect) 0.25, 0.5, 0.75 and 1.0 times the maximum intensity of the real Montenegro Earthquake accelerogram (approximately 0.4 g).

The first, and most interesting observations regard the final collapse of the model. It is in fact clearly due

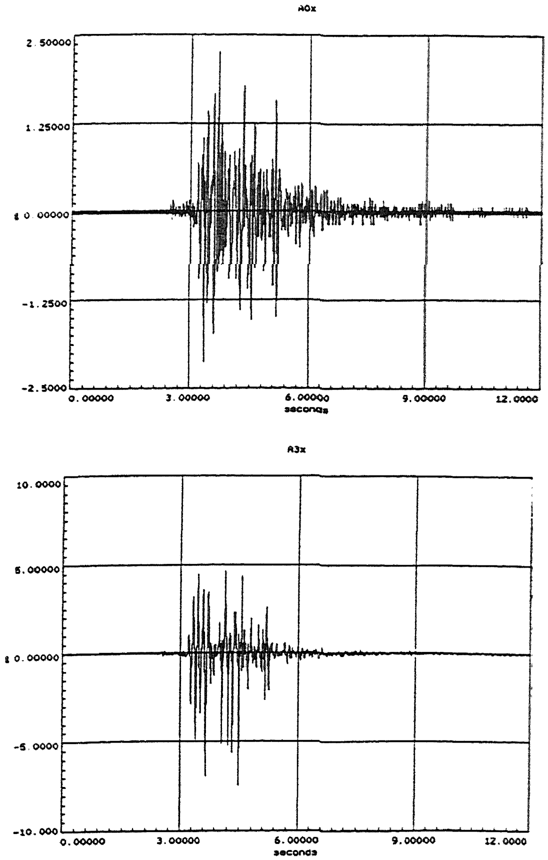

Figure 4. Typical acceleration time history recorded in the elastic range of vibration at the base (A_{ox}) and at the top floors (A_{3x}) of the model.

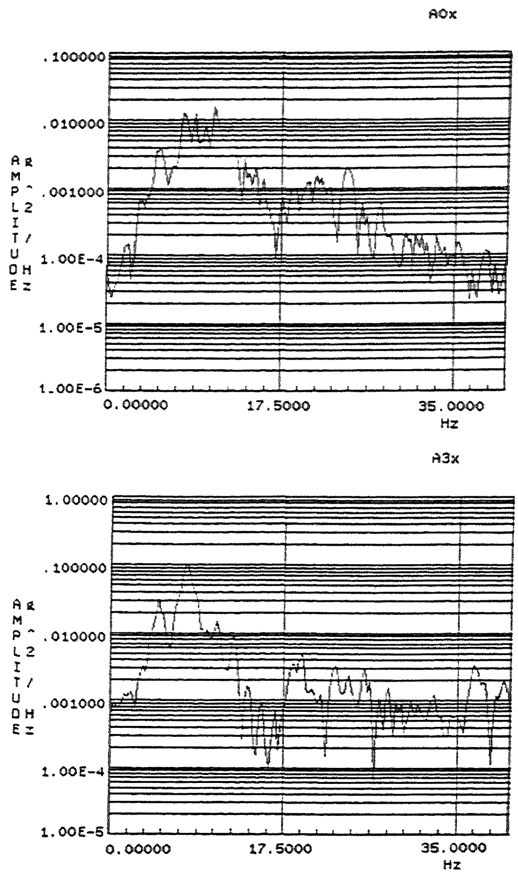

Figure 5. Fourier transform amplitute of the signals shown in Figure 4.

to in-planė, prevailing shear mode of failure, as expected taking into account its structural and material characteristics. Moreover, while the first, slighter cracks occur along the mortar joints, at failure the cracks pass through the units (Figure 3 d) as observed during the previous tests conducted on panels made with the same mortar and units, as reported in Giuffre'-Macchi Modena (1984). This is a very crucial pojnt, and seems to be a sound proof of the test significance.

The second type of considerations which can be derived from Figure 3 regard the seismic behaviour of the considered structure. In spite of having a small quantity of seismic resistant walls (there are no walls inside the building) and being the walls very slightly reinforced (approximately 0.4 % in the horizontal direction) the seimic behaviour seems to be very good.

Damages are in fact very slight, and easily reparable, under ground excitation intensities up to 50% of a very strong motion. The heavy damages observed after the same motion has been applied at its maximum intensity are practically concentrated at the ground floor level of the model and do not definitely reduce the load bearing capacity od the walls.

As regards the numerical records, they are typically as shown in Figure 4.

Appropriate numerical treatment af these data allows the identification of the modal parameters of the structure in the different phases of the investigation. The amplitude of the Fourier transforms of the above signals are for example shown in Figure 5.

The precise evaluation of the first natural frequency of the model can ba obtained according to well estabilished procedures as shown in Figure 6, where amplitude and phase are represented of the complex-frequency-response function derived from the above cited Fourier transforms.

This simple dynamic parameter identification can furnish precious informations on the material and structure behaviour.

It confirmed first of all the importance of the accurate simulation of the prototype state of stress in order to reproduce the prototype behaviour not only at failure bat also in the linear elastic phase. The dynamic characterization revealed in fact that an increase of about 15 % of the first natural frequency is connected to prestressing. This means that the material exhibit not negligible (at least as far as dynamic actions are considered) non linearity before attaining its maximum strength.

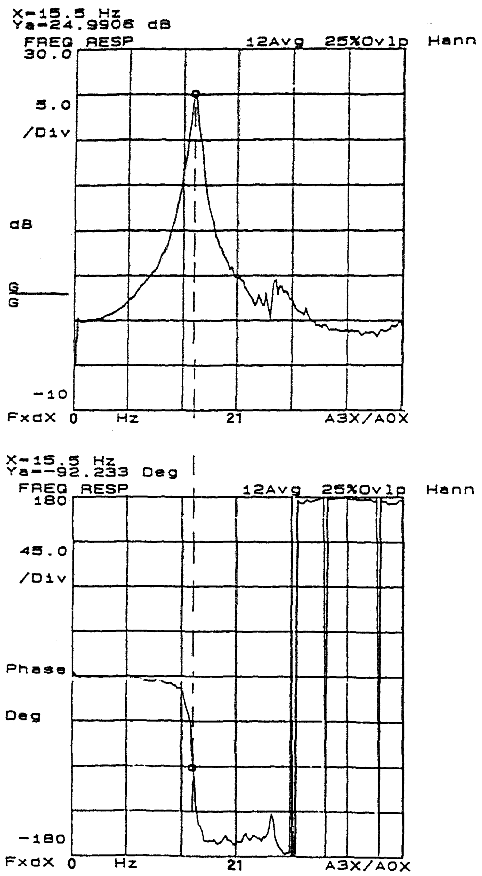

Figure 6. Amplitude and phase of the complex-frequency-response function corresponding to the Fourier Transform shown in Figure 5.

Very significant are the remarks which can be drawn from the variation of the modal frequency during the test. A synthetic representation of this phenomenon is given in Figure 7, where the values of the frequency correponding to the model characterization (N Car.) perfomed in different testing phases are shown.

The ferquency remains in fact constant during the test phases we have above assumed to correspond to a linear elastic behaviour of the structure.

The points marked A, B, C and D in the figure correspond to the different levels of damages induced in the model by motions of increasing intensity (non-linear range of vibration of the structure) represented in Figure 3. The qualitative definition of "slight" and "reparable" damage which has been associated to the damage levels represented in Figure 3 a and b receives here a more precise and quantitative significance.

The values of the first natural frequency corresponding to the points A and B of Figure 7 are in fact quite similar to the values obtanied before the model's prestressing. The craks formed along the mortar joints should then not correspond to dramatic changes in the material properties. The frequency is on the contrary reduced to a half of the original value

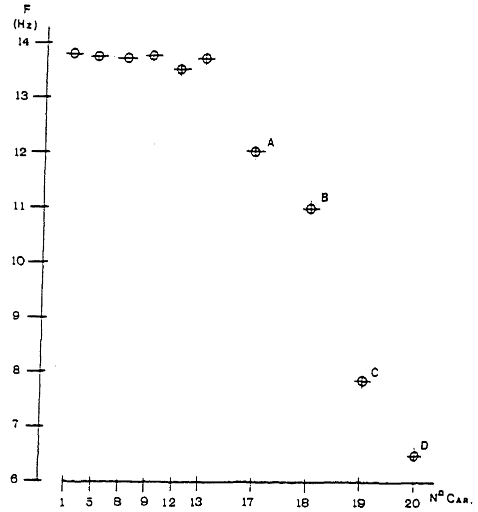

Figure 7. Variation of the first natural frequency of the model during the tests.

(ponit C) after cracks form passing through the units, as shown in Figure 3 c. These cracks represent in fact, as observed by Bernardini-Modena-Vescovi (1983), the actual "shear" failure of this kind of masonry, as confirmed by the fact that the subsequent frequency value reductions are much slighter (Point D).

REFERENCES

Bernardini, A., Modena C. & Vescovi U. Deformation and failure model for hollow clay brick masonry shea walls. Proc. 4th Canadian Conference on Earthquake Engineering, Vancouver, Canada. 1983.

Giuffre' A., Macchi G. & Modena C. Research on reinforced masonry walls for seismic regions. Proc. 3rd Symposium on Wall Structures -CIB, Warsaw, 1984.

Modena C. Sulla valutazione delle azioni sismiche di progetto per gli edifici a muratura portante. Proc. VI International Brick Masonry Conference, Rome, 1982.

Modena C. & Barel S. Studies on the seismic response of masonry wall constructions. Proc. Pacific Conference on Earthquake Engineering, New Zealand, 1987.

Modena C. & Tomazevic M. A research program on the seismic behaviour of modern masonry buildings. Proc. 9th European Conference on Earthquake Engineering, Moscow, 1990.

La Mendola P., Terrusi A. & Modena C. Shaking table test on a reduced scale model of a mixed reinforced masonry- R.C. building structure. Technical Report, ENEA, Rome, 1992.