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# SEISMIC RESPONSE OF MASONRY BUILDINGS: EARTHQUAKE SIMULATOR STUDY OF THREE-STOREYED BUILDING MODELS

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## SUMMARY

Two three-storeyed masonry building models with peripheral reinforced masonry walls and respectively a central R/C column and a central cross-wall, constructed of prototype materials in 1:5 modelling scale, were subjected to a series of seismic excitations on simple earthquake simulator in both linear and nonlinear range of vibrations. In the linear range of testing synthetically generated accelerograms, derived from response spectra given in the european and italian codes, have been used; in the nonlinear phases, two records of the April 15, 1979 Montenegro earthquake have been employed. Although of specific structural layout, the buildings behave elastically when subjected to moderate earthquakes and posses sufficient ductility to withstand repeated severe shaking.

## INTRODUCTION

Lowrise, namely three- to four-storeyed masonry residential buildings, are very popular in both Italy and Yugoslavia. Reinforced concrete floor slabs, lightned by ceramic or polistyrene elements, are used in this type of buildings, and the mixed structural solutions obtained with peripheral masonry walls and central r.c. columns are mostly preferred because they allow for flexible architectural design in plan.

Since the post-earthquake observations in many cases indicated a rather poor seismic behaviour of some types of masonry buildings and since there is not sufficient knowledge of the actual seismic behaviour of the specific types of buildings previously described, they are sometimes subjected to severe seismic codes' requirements. In order to modify such requirements, a multi-year cooperative research project between Italy and Yugoslavia has been initiated to furnish experimental data on the seismic bahaviour of the cited types of buildings.

As the three-dimensional behaviour of the structures is crucial in this type of problem, and sufficient data are already available on the cyclic behaviour of single masonry walls and piers, shaking table tests have been decided on a number of 1:5 reduced scale models sufficient to study the seismic behaviour of both reinforced and plain masonry buildings, with both central r.c. columns and masonry walls. The data gathered during the tests are such to allow the comparison of the input and models' motions, the analysis of models' response at every floor level and in every testing phase, the evaluation of parameters defining the structural damages, and the calibration of reliable and as simple as possible structural models to analyze the seismic behaviour of the considered buildings.

DESCRIPTION OF TESTS

Modelling of prototype structures Within the first phase of the research project two three-storeyed reinforced masonry building models , one with peripheral walls and a central r.c. column (model 1), and the other with a cross-wall replacing the column (model 2), have been tested on a simple earthquake simulator. Plans and elevations of the two models are shown in Fig. 1; the results of the tests are reported in (Ref.1) and in (Ref.2).

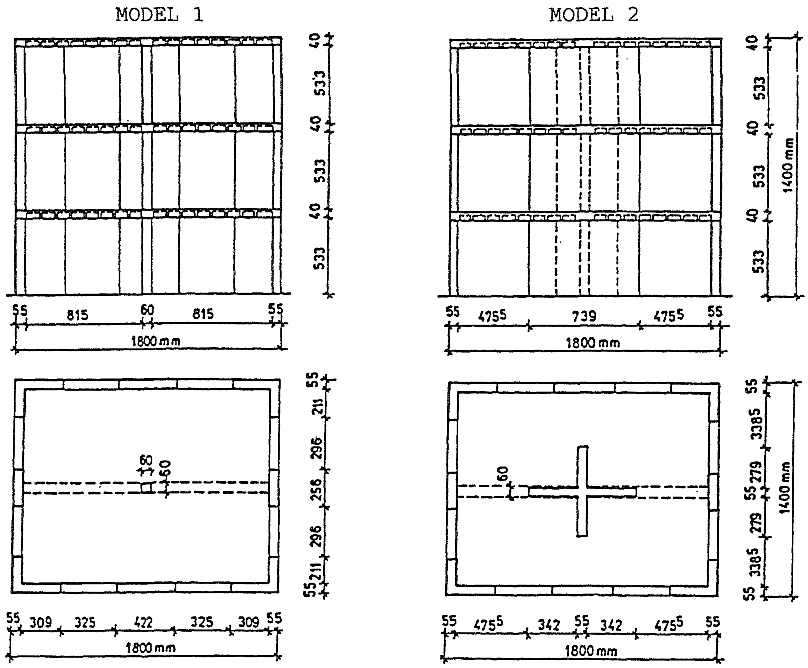



Fig. 1 Plan and elevation of model buildings

In the prototype structures the walls are constructed of lightweight ceramic perforated blocks, 280 mm thick, and are reinforced with vertical (minimum 10 mm diameter bars at 300÷400 mm intervals, and 16 mm diameter bars at the edge of window and door openings) and horizontal reinforcement (minimum two 6 mm diameter bars, placed in each second horizontal mortar joint).Deformed steel (yield stress 440 MPa) and smooth steel bars (yield stress 320 MPa) are used for for vertical and horizontal reinforcement respectively. The r.c. floor slabs are usually made lighter by using ceramic or polistyrene elements. The slabs are supported by peripheral walls and, when necessary, by r.c. beam at their midspan.

For the construction of models, the prototype blocks have been simply cut into the correspondingly sized model blocks. Concentrated rather than distributed vertical reinforcement has been placed at vertical edges of walls (two 2.3 mm diameter bars), at corners and at wall intersections (four 2.3 mm diameter bars). Horizonral reinforcement, however,was uniformly distributed in mortar joints (1.1 mm diameter wire in the form of closed stirrups in each second mortar joint). The so called "burned wire", commercially available on the market, has been used. Floor slabs have been modelled as one-way acting ribbed slabs, with r.c. beam at their mid-span, and were confined with r.c. tie beams, reinforced with four 3.1 mm diameter bars, atop of all peripheral walls. The beam was supported either by internal r.c. column, reinforced with four 4.2 mm diameter bars and 1.1 mm stirrups at 20 mm intervals (model 1) or by a cross wall, which replaced r.c.

column (model 2). Mortar used for the construction of walls and grouting of reinforcement consisted of Portland cement, lime and sand (aggregate size 0÷2 mm) in the proportion of 1:2:9. Micro-concrete consisted of Portland cement and sand (aggregate size 0÷4 mm) in the proportion of 1:3. The mechanical properties of the model materials are given in Table 1.

Table 1 Mechanical properties of the model materials (mean values)

| Masonry (MPa) | | | | Model 1 | Model 2 |
|---|---|---|---|---|---|
| Compressive strength of blocks | | | | 9.45 | 9.45 |
| Compressive strength of mortar | | | | 2.74 | 1.85 |
| Compressive strength of micro-concrete | | | | 27.95 | 23.43 |
| Compressive strength of masonry | | | | 6.33 | 6.33 |
| Tensile strength of masonry | | | | 0.40 | 0.40 |
| Modulus of elasticity of masonry | | | | 6450 | 6450 |
| Reinforcing steel | | | | | |
| Bar diameter (mm) | 6.00 | 4.20 | 3.10 | 2.30 | 1.10 |
| Tensile strength (MPa) | 382 | 448 | 443 | 113 | 43 |
| Yield limit (MPa) | 253 | 391 | 323 | 93 | - |
| Yield strain (%) | 1.21 | 1.86 | 1.54 | 0.44 | - |
| Ultimate strain (%) | 31.45 | 21.70 | 39.10 | - | - |

When studying the seismic behaviour of buildings by testing their models on earthquake simulators, the models must of course behave similarly to prototype structures subjected to similar excitations (Ref.3). If the behaviour of buildings is studied up to their collapse, not only the dynamic characteristics of models, but also the damage patterns and failure mechanisms obtained during model tests must be similar to those observed on buildings after earthquakes. In order to achieve this goal, the similarity of mass and stiffness distribution and the similarity of failure mechanisms between prototype and model sized walls as basic structural elements of masonry buildings play the most important role. In order to achieve the similarity of failure mechanisms, the similarity of the level of normal stresses in the walls due to vertical loading in prototype and model building must exist, i.e. the ratio between the working stresses in the walls and their compressive strength must be the same in both prototype and model building.

The above requirements would be auotomatically fullfilled in the case of the so-called complete model similarity, which would, however, require the production of special model materials. Since the materials of prototype characteristics have been used for the construction of these models, two additional modifications were necessary. In order to achieve the similarity of mass distribution, additional mass has been added to the floor slabs of the model buildings; in order to achieve the similarity of working stresses in the walls of prototype and model struture, the models' walls were prestressed by means of steel knitted ropes (4 mm in diameter), fixed on the top slab through soft springs controlling the prestressing forces and anchored into the foundation slab.

Another important factor influencing the failure mechanisms is the strain rate, which is of course different in the prototype and in the model structure owing to the effect of the time scale factor: the similitude of the failure mechanisms can be then achieved only if the strain rates are such that similar cracks patterns are developed when the structures are similarly loaded.

<u>Modelling of seismic actions</u> In the linear range of vibrations, two sets of five earthquakes each, derived from the Eurocode n.8 response spectra (AG1E1÷AG5E1 and AG1E2÷AG5E2), and one set of five earthquakes derived from the draft Italian Seismic Code soil type II response spectrum (GNDT2.1÷GNDT2.5) have been used, in order to allow the evaluation of statistical parameters of the elastic

dynamic response. The shape of the response spectra and the time modulating function used for the generation of synthetic earthquakes are shown in Fig. 2; the characteristics of the response spectra are given in Table 2. In the non-linear range of vibration two records of April 15, 1979 Montenegro earthquake have been used ( ACC10 and ACC13).

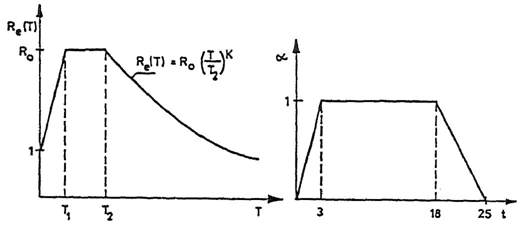



Fig. 2 Shape of the response spectra and time modulating function of the generated motions' intensity

Table 2 Characteristics of the used response spectra

| Group | T1 (s) | T2 (s) | K | Ro | Fractile (%) |
|---|---|---|---|---|---|
| AGE1 | 0.20 | 0.4 | 1 | 3.0 | 86 |
| AGE2 | 0.20 | 0.6 | 1 | 3.0 | 86 |
| GNDT2 | 0.15 | 0.8 | 1 | 2.2 | 50 |

Both models were subjected to seismic excitations in different testing phases ( in alltogether 74 and 67 test runs respectively in the case of model 1 and of model 2); the intensity level of earthquake simulator motion are controlled adjusting the magnitude of maximum ground displacement. However, owing to the limited capacity of the earthquake simulator ( which allows the programming of the displacements and has a reduced vibrating mass with respect to the models' mass) the actual table motion characteristics must be estimated by analyzing the records of shaking table accelerations. It is to point out moreover that, taking into account the equipment's limitations and in order to compare the effect of different frequency contents of shaking table motion to the models' response, not all the earthquakes have been modelled in time according to the laws of general model similarity, according to which the time modellimg factor should be equal to 5. The assumed values of the time scale factors (minimum 2.24 and maximum 6) were in any case such that the failure mechanisms were not influenced by the corrisponding different strain rates, as the observation of the cracks patterns during the tests demonstrated.

The characteristics of the imposed motions and of the corresponding motions recorded at the models' base have been compared in both the frequency and time domains (Ref.4). A sample of the results is shown in Figs.3 and 4 and in Table 3, where $\Omega c$ is the central frequency, bw the bandwidth, so the duration (time interval on which the total motion energy is uniformly distributed), and $\sigma o^2$ the intensity (mean square acceleration) of input motion (I), of the corresponding motion recorded at the base (B) and at the third storey of model 1.

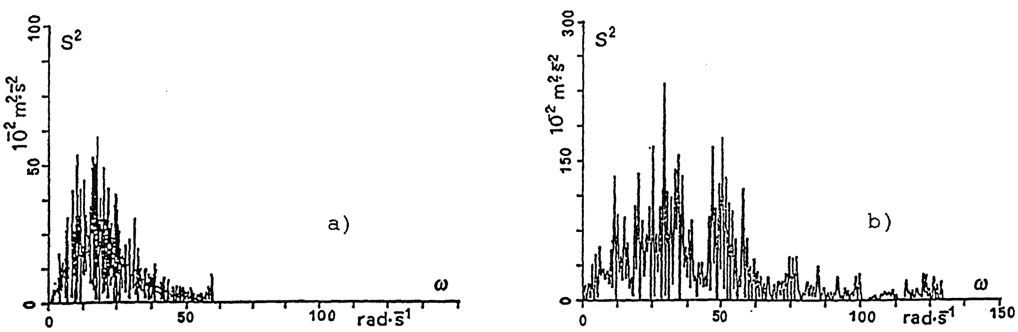



Fig. 3 Fourier spectra of a generated accelerogram (AG2E1)- a)- and of the corresponding motion recorded at the base of model 1 -b)-

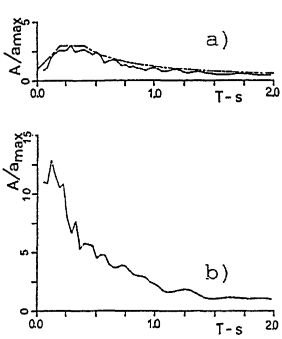



Table 3 Main properties in time and frequency domains of a sample of generated input and corresponding model 1 base motions

| Input motion | Test run | Ωc (rad/s) | | | bw | | so (s) | | $\sigma o^2$ ($m^2/s^4$) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | I | B | T | I | B | I | B | B | T |
| AG1E1 | 10 | 24.8 | 68.1 | 61.1 | 0.50 | 0.50 | 14.7 | 6.1 | 0.32 | 4.52 |
| AG1E2 | 20 | 23.7 | 64.2 | 59.3 | 0.55 | 0.55 | 14.8 | 6.2 | 0.41 | 5.02 |
| GNDT21 | 30 | 22.9 | 60.4 | 59.3 | 0.67 | 0.62 | 14.7 | 6.2 | 0.39 | 3.60 |
| AG2E1 | 44 | 24.7 | 55.4 | 55.4 | 0.50 | 0.54 | 14.9 | 6.3 | 1.41 | 13.2 |
| AG2E1 | 45 | 24.7 | 50.9 | 55.6 | 0.50 | 0.53 | 14.9 | 6.5 | 1.82 | 14.5 |

Fig.4 Response spectrum of a generated accelerogram, compared to the response spectrum from which the generation took place (dotted line) -a)- and response spectrum of the corresponding motion at the base of model 1 -b)-

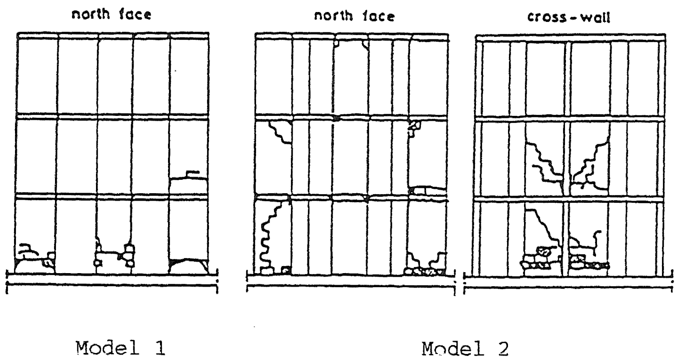
The analyses show an acceptable agreement between the input and the models' base motions. Considering in particular the response spectra of Fig.4, they correspond to maximum spectral displacement approximately four times greater in case a) with respect to case b), so that the model's scale factor is reasonably respected. An influence seems however to be observable of the model's dynamic properties on the frequency content of its base motion. Taking into account the time scale factor, it can be observed that the central frequency of the model's base motions are lower than the input motion's and closer to the third storey motion's central frequency.The differences between the central frequencies of the three groups of homogeneous accelerograms seem however to be reasonably retained.

## TEST RESULTS

In model 1, a combination of shear and flexural cracks has first occurred in both middle walls and web parts of corner walls of the first storey of model building, and the damages remained concentrated in this storey when the intensity of shaking table motion was increased. Plastic hingening of r.c. column's ends were also observed at the first storey. Middle walls finally failed in shear and the horizontal reinforcement prevented the complete disintegration of such walls. As indicated by crack patterns, walls behaved as vertical cantilevers, coupled by flexible floors and bond-beams.

In model 2, shear cracks have first developed in the cross-wall as expected, and in one of the corner wall of the first storey of the model. With the increased intensity of shaking, cracks in the walls have propagated into the upper storeys, and precisely in the cross wall on the second storey and in the middle peripheral walls on the third storey. Whereas typical shear behaviour has been observed in the case of the cross-wall, a combination of shear and bending governed the in-plane behaviour of peripheral walls.

Typical crack patterns in structural walls before collapse of both models are shown in Fig. 5. Based on the observed models' response, three limit states have been defined in models' behaviour: elastic limit, corresponding to the first visible cracks in the models' structural elements; maximum resistance, corresponding to the maximum base shear acting on the models, calculated on the basis of mass distribution, recorded mode shapes and maximum recorded accelerations (in model 1 cracks appear in the walls and bond beams, in model 2 in the walls of 1-st and 2-nd storey, between walls and slabs, in bond beams); ultimate state, corresponding to the situation of model before collapse (crushing of blocks, plastic hinges in bond beams, and in columns in case of model 1, tearing and buckling of reinforcement are observed in both models).



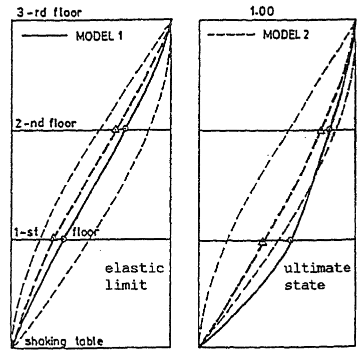




Fig. 5 Typical crack patterns before collapse

Fig. 6 First natural mode shapes

Correspondingly to the above limit states, the shaking intensities given in Table 4 have been recorded. In the linear range of vibrations, greater amplification of the base motion is observed in the stiffer model 2 then in model 1 which however corresponds to the greater value of its walls resisting area. The maximum resistance of the two models are practically the same, while model 2 seems to retain such resistance also up to failure ( it losed it faster however after the testing phase corresponding to the conventional ultimate state ). In Fig. 6 the first natural mode shapes at the extreme limit states are plotted, compared with pure bending and shear mode shapes of systems with uniformly distributed masses and stiffnesses. The differences in damage distribution over the models' height are clearly evidenced by the mode shapes at the ultimate state , deformed in the case of model 1, where the damage is concentrated in the first storey, more regular and close to the pure shear mode shape, correspondingly to a more uniform distribution of damage and predominant shear behaviour in the case of model 2. The first natural frequencies in the elastic range of vibrations varied slightly during the tests, from 11.9 to 9.5 s and from 13.3 to 11.1 s respectively in model 1 and model 2; the mean values fairly agree however with the results of the modal analysis of the models performed by means of ETABS computer code.

Table 4 Shaking intensities at the conventional limit states

| | Elastic limit | | Max. resistance | | Ultimate state | |
|---|---|---|---|---|---|---|
| | Model 1 | Model 2 | Model 1 | Model 2 | Model 1 | Model 2 |
| Max. shaking table acceleration ($m/s^2$) | 16.22 | 13.92 | 23.02 | 31.20 | 21.16 | 36.60 |
| Max. top floor acceleration ($m/s^2$) | 19.94 | 25.28 | 39.88 | 40.16 | 22.74 | 38.19 |
| Base shear coefficient | 1.29 | 1.52 | 2.62 | 2.57 | 1.73 | 2.50 |

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