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Shaking table tests of three four-storey brick masonry models: Original and strengthened by RC core and by RC jackets

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ABSTRACT: The experimental results presented in this paper are part of the performed large joint project between the University of Bologna and the University of Skopje. For the selected hypothetical prototype, a true replica, 1/3-scale model was constructed and tested under various earthquake time histories. Applying different strengthening concepts to the same original model, two strengthened models were built and tested under the same earthquake as the original one.

1 INTRODUCTION

The results presented in this paper are part of the research project "Basic and Applied Study for Seismic Modelling of Mixed Reinforced Concrete Masonry Buildings", realized jointly between the Instituto di Tecnica delle costruzioni, and Instituto di Scienza delle Costruzioni, University of Bologna, and the Institute of Earthquake Engineering and Engineering Seismology (IZIIS), University "Kiril and Metodij", Skopje, Republic of Macedonia, in the period 1988-1991. Within the scope of this project, more than ten joint reports have been published, covering the entire preliminary experimental results. The primary objective of this project is to develop an appropriate strengthening technology for buildings in Rimini, constructed in mixed structural system of brick masonry and reinforced concrete.

One of the largest research phases of the project was dealing with testing of models on a large biaxial seismic table. Part of the obtained results are presented in this paper.

The considered building prototypes are characterized in most cases by basements constructed in mixed structural systems (RC columns and beams and brick masonry walls) while the upper floors are mostly constructed of brick masonry walls. The material properties of the prototype structures have been previously evaluated,

To meet the target of the project, it was decided to design a hypothetical prototype fragment with the first floor in a mixed structural system (RC beams and columns and brick masonry walls), and the remaining upper three stories as classical brick masonry walls. For this hypothetical prototype, a true-replica model was designed in geometrical scale 1/3, and it was fabricated for three times. The first model was used to study its dynamic behaviour under failure conditions. The remaining two models were strengthened in two different ways and were independently tested on a seismic shaking table as the first model. The results, presented comparatively for the three models in this paper, are aimed at pointing out the differences in the dynamic behaviour when subjected to identical seismic effects.

2 DESCRIPTION OF THE MODELS

Generally considered, the three tested models are designed and constructed as true-replica models. The similitude requirements for the 1/3 scale model are presented in the table below.

Similitude parameters	Scaling factors model/prototype	Model value
Length	$Lr = Lr$	1/3 Lp
Time	$Tr = (Lr)^{1/2}$	0.577 Tp
Frequency	$f_r = 1/Tr$	1.730 f_p
Velocity	$Vr = (Lr)^{1/2}$	0.577 Vp
Acceleration	$a_r = 1$	1.0
Density	$\rho_r = Er/Lr$	1.5 ρ_p
Strain	$\varepsilon_r = 1$	1.0
Stress	$\sigma_r = Er$	0.5 σ_p
Young's Modulus	$E_r = 1/2$	0.5 Ep
Displacement	$\delta_r = Lr$	1/3 Lp
Inertial forces	$Fr = Er.Lr^2$	1/18 Fp
Energy	$Wr = Er.Lr^3$	1/54Wp

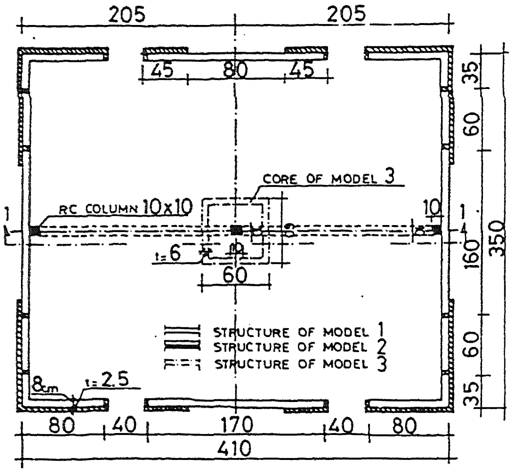

Figure 1. Plan of basement of all three models

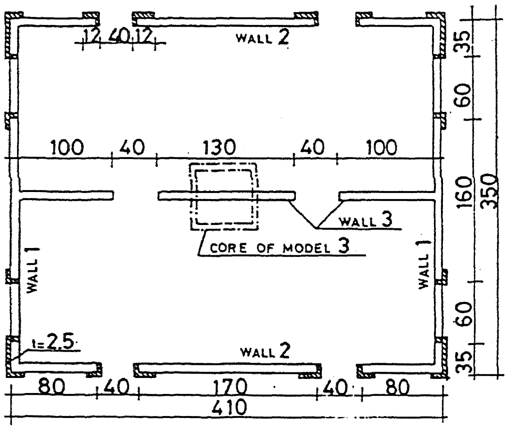

Figure 2. Plan of 2-nd and 3-rd floor of all three models

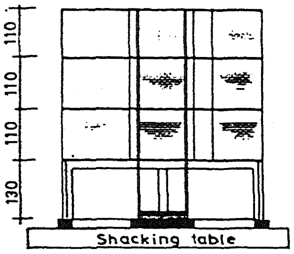

Figure 3. Cross-section 1-1 for model 3

Based on the above scaling model requirements, appropriate model materials have been developed. So, for the regular brick size of 25x12x6.5 cm, model brick of 8x8x4 cm in size was produced of the same material and in the same technology in the factory as the prototype brick. According to the similitude requirements, the Young's moduli of the mortar are approximately two times lower, compared to the prototype ones, and the same applies to the concrete material as well. For mass simulation, additional mass of 30% was necessary to be applied at each floor level.

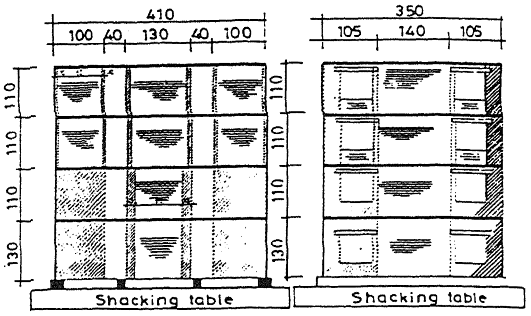

Figure 4. Strengthening of walls 1 and 2 for model 2

The model was designed using the geometrical scale factor of 1/3. In want of space, the geometries of the three models are shown in the same figures, whereby

- Model 1 applies to the structure of the hypothetical prototype model, consisting of brick masonry walls and RC columns and beams only at the first floor;
- Model 2 applies to the structure of model 1, strengthened by external RC walls and fragments of the RC walls;
- Model 3 applies to the structure of model 1, strengthened by central core, without the strengthening for model 2.

The geometries of the three tested models are shown in Fig. 1, for the first (basement) level, and in Fig. 2, for the other three floors, respectively. The cross-section of model 3 (which is the same for model 1 and model 2, but without the central core) is shown in Fig. 3. External RC wall distribution along the height of model 2 is shown in Fig. 4, for wall 1 and wall 2.

A detail of the central core of model 1 is shown in Fig. 5. In this figure, the geometrical elements of the core and its connections to the foundation can be seen. To reduce the considerable effect of the core contact to the foundation slab, which is rigidly fixed to the shaking table, a hard rubber layer is placed. The purpose of this layer is to simulate the interaction between the soil and the structure as in the case of the prototype structure.

Similarly, for model 2, in Fig. 6, a detail of the technical solution for the external walls around the corners are presented. This figure shows, also, that the reinforcement net is anchored to the wall, while the concrete, as torquete, is placed over the wall. The RC walls around the corners and the openings are strengthened.

3 EXPERIMENTAL PROGRAMME

The testing of the three models is performed on the biaxial shaking table at the dynamic Testing Laboratory of the Institute of Earthquake Engineering and Engineering Seismology in Skopje. The three models, with a total mass of 170 kN, for model 1, and up to around 200 kN, for models 2 and 3, and a height of 4.6 m were constructed on the shaking table, 5 m x 5 m in plan. Since the bearing capacity of the crane is 100 kN, the third and the fourth floor of the models had to be constructed on the shaking table itself. Consequently, the models were constructed and transported without initial cracks.

The experimental programme consists of two parts:

- definition of the dynamic characteristics of the models (natural frequencies, mode shapes and damping capacities) using the forced vibration method, of impulse excitation test and random motion test.
- seismic excitation, using five different earthquake time histories (El Centro 1940; Parkfield 1966; Montenegro 1979, records obtained at Bar and Petrovac and Friuli 1976 records at Breginj - Slovenia).

For recording of the dynamic response of the model, a data acquisition system was used consisting of 30 channels, out of which 6 accelerometers, 13 displacement transducers (LVDT's), 4 linear potentiometers, 2 clip gages and 5 strain gages.

The definition of the dynamic characteristics was carried out at the beginning, then after a number of earthquake simulations, and at the end of the testing. From these tests, the relationship of the natural frequencies reduction upon the level of cracking was obtained.

In the beginning, the seismic testing of the models was carried out for all five earthquake types in linear range, in order to determine the sensitivity of the models to the considered earthquake time histories. For this purpose, 15 runs were performed, and then, the testing was continued applying El Centro, Friuli (Breginj) and Montenegro (Petrovac) earthquakes. Applying these three earthquakes, totally 28 runs were performed, until the appearance of visible nonlinearity, and then simulation with only the Petrovac earthquake was carried out until the occurrence of considerable damage to the model.

The above testing scenario was performed for all the three models, simulating always earthquakes of the same acceleration on the shaking table (SPAN). Having in mind that models 2 and 3, compared to model 1, are of considerably higher strength, destructive tests are performed only with the Petrovac earthquake and for significantly higher peak acceleration level of the shaking table (SPAN). Having in mind that models 2 and 3, compared to model 1, are of considerably higher strength, destructive tests are performed only with the Petrovac earthquake and for significantly higher peak acceleration level of the shaking table (SPAN). The maximum simulated peak acceleration was 0.51 g, for model 1 and 1.07 g, for models 2 and 3.

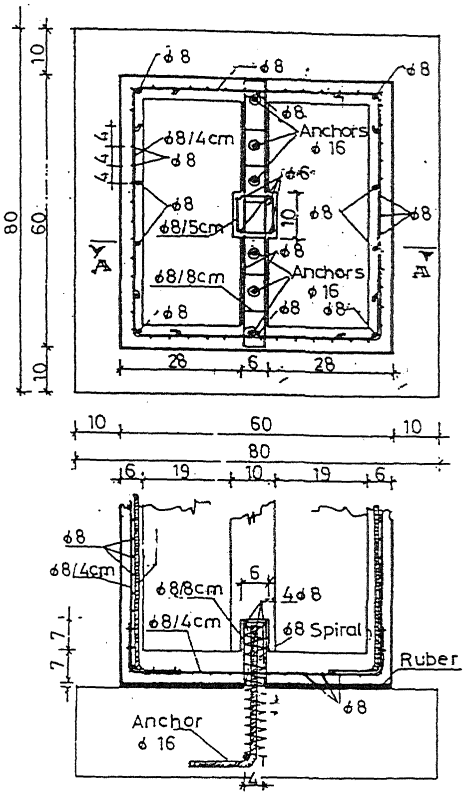

Figure 5. Connection between central core and foundation

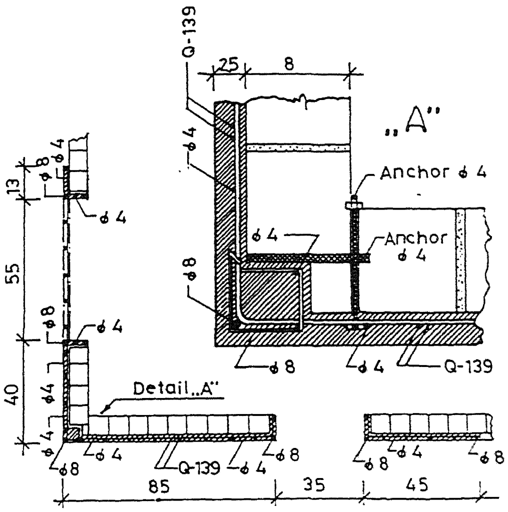

Figure 6. Detail of corner strengthening of model 2

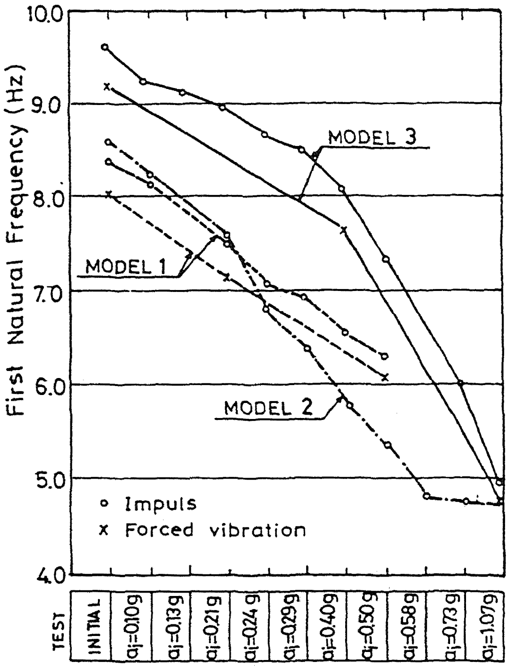

Figure 7. First natural frequency decreasing as a function of peak shaking table motion for all tested models

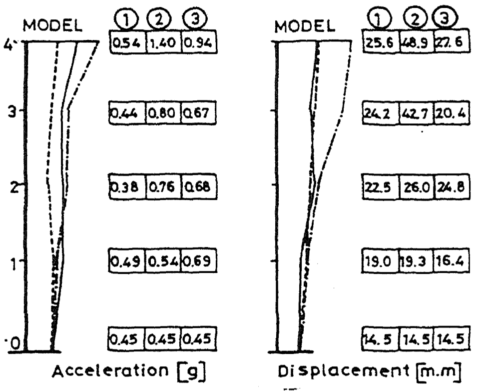

Figure 8. Acceleration and displacement responses of all three models for Petrovac earthquake simulated on the shaking table with peak acceleration of 0.5 g at time t = 4.40 s

4 EXPERIMENTAL RESULTS

Out of the significant volume of experimental results, obtained by the performed tests on the three models, consideration in this paper will be given to the dynamic characteristics of the models, the displacement and acceleration time histories and the failure mechanism. The experimental values can be easily compared, since in all the three cases the same monitoring system and the same data processing and data presentation pattern as well as the same order of earthquake simulation on the shaking table was applied.

The dynamic characteristics were defined applying the known methods in experimental mechanics. It is interesting to be pointed out how the resonance frequency was modified depending on the previously generated acceleration level on the seismic shaking table, i.e., the intensity of the micro and macro cracks of the model (Fig. 7). It is obvious from this figure that model 3 has the highest initial stiffness, then model 2 and, finally, model 1. However, on the other hand, the stiffness deterioration is the lowest for model 1, while model 2 and model 3 show identical behaviour. The model shapes are characterized by prevailing shear type (model 1 and 3), while model 2 has prevailing bending type of vibration. The viscous damping coefficients, defined from the frequency response curves, in all three cases, range between 2 and 3. The torsional frequency is twice the value of the first frequencies of the translational mode shapes.

Fig. 8 shows the acceleration and displacement responses for all floor levels, for the three models, for the effect of the Petrovac earthquake simulation, with SPAN 260, i.e., peak acceleration of 0.5 g, recorded at time t = 4.40 s. It is obvious from the figure that model 2 has the highest values for both accelerations and displacements. Naturally, this shape changes depending on the time and the intensity of the excitation, i.e., the level of cracks.

The acceleration time histories recorded at fourth floor for simulation of Petrovac, SPAN 260 earthquake, for the three models is shown in Fig. 9. For this excitation level, model 1 has reached the ultimate failure mechanism, which proves that in such a case, the acceleration level at the upper floor (0.55 g) is approximately the same as the level of excitation of the shaking table (0.51 g), i.e., without amplification. For the other two models, the amplification is 2 for model 3, and almost 3 for model 2. The damage mechanisms are quite different for the three models.

If one considers the displacement time histories for the same case of excitation (Fig. 10), it can be concluded that model 1 has larger deformation than model 3, and almost twice as small as model 2. On the basis of this, as well as the previous figure, it can be concluded that model 1 has reached a higher level of permanent damage, compared to models 2 and 3, which also points to the different mechanism of damage development.

The damage mechanism, observed in wall 2, for the three models, is shown in Fig. 11 for the stated levels of generated accelerations. It is obvious from the figure that model 1 is characterized by intensive damage to the first floor, when parts of the wall failed totally, slight damage to the second floor and almost none to the upper two floors.

Model 2 is characterized by intensive damage to the masonry of the first, second and third floor, bending

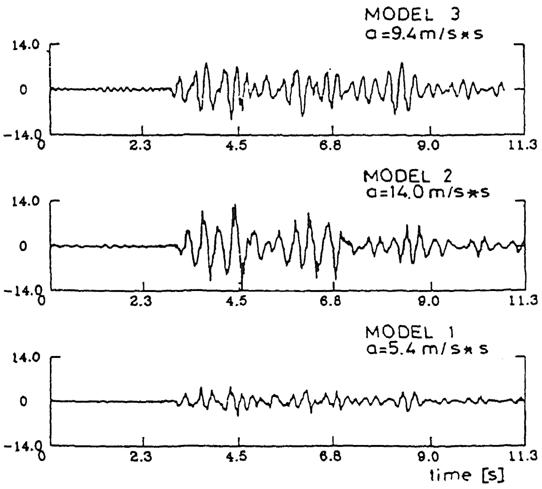

Figure 9. Acceleration time histories of 4-th floor for all three models recorded for Petrovac earthquake simulated on shaking table with peak acceleration of 0.51 g

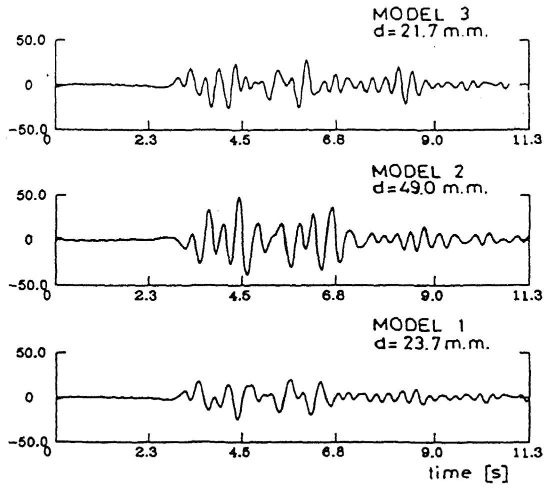

Figure 10. Displacement time histories of 4-th floor for all three models recorded for Petrovac earthquake simulated on shaking table with peak acceleration of 0.51 g

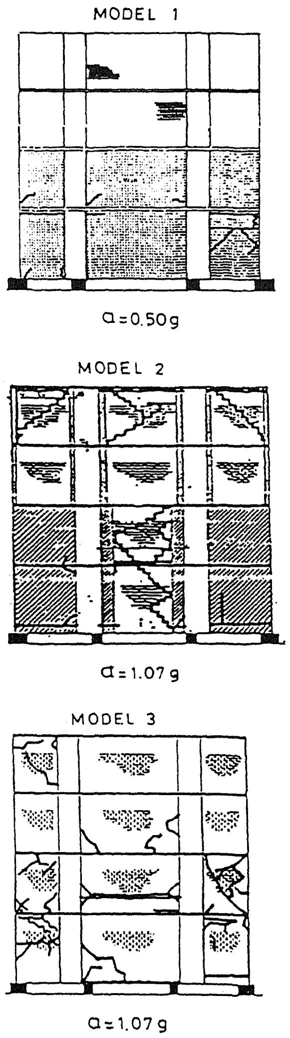

Figure 11. Failure mechanism of wall 2 for all three tested models

failure mechanisms to the RC walls of the first floor and almost no damage to the third floor.

Model 3 has failure mechanism developed at the masonry of all the floors, while small cracks are observed to the central core of the first and second floor. Due to interaction between the central core and the floor slabs, damage to the first and second floor slabs is also observed.

As a general comment, drawn based on the targeted objectives of the investigations and the obtained experimental results, it can be pointed out that the two suggested strengthening strategies, obviously, increase the seismic safety of the model, i.e., the prototype. Each of the two approaches has both advantages and disadvantages, therefore the decision-making on the selection of the most adequate approach for strengthening of the prototype will be affected by the economic, in addition to the technical aspects. The possible combination of the two approaches for strengthening of structures in practice should certainly be considered, parallel to the two individual solutions.

5 CONCLUSIONS

The experimental results, obtained by testing of three models, provide a sufficient volume of data for a selection of a technical solution for strengthening of masonry structures, that will ensure the required seismic safety of a structure at an economically acceptable cost. The studied strengthening technologies enable application of combined strengthening solutions of the two studied solutions or fragments of them, depending on the geometry of the structures and their purpose. The selected strengthening technology has such a failure mechanism which is distributing over the height of the structure and is providing a high energy absorption capacity.

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