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Retrofitting of structures built and designed by different materials

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ABSTRACT: The problem of strengthening of buildings by using other materials than those that have been used for their construction is discussed in this paper through a presentation of a case study. Investigations of the dynamic behaviour of strengthened structural systems is performed on large scale models tested on a seismic shaking table. Six models were tested: the original model, two models strengthened by using different methods and three repaired models. All tested models were subjected to an identical experimental programme in both elastic and elasto-plastic range. The comparative differences between the models were discussed via the experimentally obtained shear force - relative storey displacement relationship and the failure mechanism.

INTRODUCTION

In everyday practice, structural engineers are most frequently facing the need to elaborate a project for repair of structures damaged by earthquakes or to design strengthening of structures having an inadequate earthquake resistance capacity. Multiple problems arise and have to be solved when designing strengthening of structures. First of all, due consideration has to be given to the codes for repair and strengthening of structures and the corresponding specifications. In many of the national codes for earthquake resistant design of structures, the requirements for repair and strengthening are simplified to an extent that it is recommended that damaged or old buildings are to be strengthened, during their reconstruction, to the earthquake resistance level required for the design of new buildings. In some way, this simplified requirement, leaves out of consideration that old buildings are most frequently composed of archaic materials that are not longer in use in modern construction and have mechanical characteristics that have not been sufficiently explored. If, in addition, the aging factor is also considered, it is clear that diagnosis of the seismic resistance of such structures prior to their strengthening is a very complex task. In such conditions, the engineers are designing the strengthening of structures by using modern materials. In such cases, engineering judgment is required and lateral strengthening of the structure is more or less successfully performed depending upon the experience and the knowledge of the engineer /20/.

Starting from the complexity of this problem, a large number of scientists have dealt with investigations in this field and a large number of papers on different methods and techniques of repair and strengthening of structures have been presented at many conferences and symposiums. The recommendations for strengthening of structures can be conceived only as an endeavour of the investigators to systematize the practical solutions /6, 17/. In addition, engineers have a lot of liberty regarding the selection of the strengthening methodology that should provide a seismic resistant structure. So, steel structures are usually strengthened by additional steel elements, concrete structures are repaired by using different expoxides and strengthened by adding RC structural elements or steel structural elements. Many such examples have been presented at WCEE and ECCE-s held for the last years. The problem becomes even more complex in case of structures constructed of local materials or mixed materials for which strengthening has to be performed due to insufficient seismic resistance. In these cases, little or nothing is known about whether the structure experienced earthquakes in the past, whether it was previously strengthened, what are the aging effects upon modification of the mechanical properties of its materials, etc. The application of modern materials for strengthening of such structures imposes the need of solving many problems as to the interaction between the old and the new materials, i.e., isotropy, ductility, deformability, resistance capacity and chemical reaction between the materials as a function of time. How the strengthened structures generally behave during an actual earthquake that is stronger or equal to the design one is not known according to the above stated references, or more exactly, the behaviour of a structure strengthened by using "modern" techniques that result from most recent scientific achievements is still not proved by a "natural experiment". It therefore seems that laboratory or in-situ tests may provide valuable results as to this problem /1, 5, 7, 9, 10, 11, 12, 13, 14/.

THE NEEDS OF STRENGTHENING

On the basis of a large number of professional and scientific papers, the need of strengthening arises from the following reasons:

1. Strengthening of old historical buildings;
2. Strengthening of buildings when they are redesigned to serve another purpose;
3. Strengthening of buildings damaged due to earthquakes;
4. Strengthening of buildings having insufficient earthquake resistance capacity.

Strengthening of historical buildings is performed in almost all countries, even in those with a relatively low seismic hazard. The historic value of a building, especially its architecture and interior dictate to a great extent the strengthening techniques to be used. If the selection of the strengthening technique can be presented in a simplified form, then it may be said that strengthening of the structure should be performed without disturbing the authenticity of the structure. The proposed strengthening techniques are therefore usually evaluated and approved by art historians, conservators and architects. Strengthening of such structures is generally a very delicate task for structural engineers.

In case of functional redesign of the buildings, especially when it means a considerable increase in the mass of the building, it is important that apart from addressing the problem of earthquake resistance, stability of the structure against gravity loads has to be provided previously. During functional redesign of structures there may be cases when the redesigned structure is reclassified into another category of structures that are designed for a higher level of earthquake effects according to seismic regulations. In such cases, special attention has to be paid to the strengthening of the foundation old and the new part of the foundation structure.

When strengthening of structures damaged by strong earthquakes has to be done, there are usually two cases. The first case involves buildings that are seismically designed and constructed, but may suffer allowable damages to the structural system and some structural damages which depends upon the code requirements. The main task of the engineer in this case is to determine the way of repair and design additional lateral strengthening, if necessary. The second case involves older damaged structures that are not designed to sustain seismic loads. In this case, the strengthening design should provide the required earthquake resistance. It may be stated that the long world experience has proved the existence of a lot of such cases after occurred earthquakes.

Finally, there are known cases in practice when governments initiate programmes for organized and planned strengthening of buildings within considered urban areas that, according to experts, are characterized by an unacceptable vulnerability level, i.e., insufficient earthquake resistance capacity. The case of seismic strengthening of the existing reinforced-concrete school building in Shiruoka City in Japan might be taken as an example /16, 19/. Such examples are also found in other parts of the world, but it seems that in many towns throughout seismically active regions of the world, there are still structures of insufficient seismic safety and even such that require urgent strengthening.

STRENGTHENING DEMANDS

The strengthening demands determine the strengthening strategy that has to provide the required seismic safety of the considered case.

The selection of the strengthening strategy depends upon many factors among which is the age of the building and its historic value, the quality of the materials and the structural system, the expected seismic hazard level, the soil deposits, the geometrical configuration, the purpose and the category of the building, the urban disposition, etc. All these factors, along with the economic cost of the building, have an essential influence upon the decisions of the engineer regarding the selection of the best possible strengthening technology. According to existing concepts /6, 17/, the aims of strengthening are classified into the following three categories:

- increase in strength;
- increase in ductility, and,
- increase in both strength and ductility.

However, it is most likely that the

concept of controlling structural behaviour will be most frequently used in practice. Hence, strengthening demands can be classified into the following categories:

- to increase the strength;
- to increase the ductility;
- to control structural behaviour, and,
- combination of the above three aims.

In the case of buildings constructed of brittle materials (brick and stone masonry), the incorporation of new structural elements composed of the same materials increases structural strength. Such an approach is most characteristic for cultural-historic monuments and generally, old buildings.

The buildings that are characterized by a relatively high strength sufficient enough to sustain the expected seismic forces, but a relatively low ductility, are strengthened by structural elements that primarily increase the ductility of the structure. A much more realistic and frequently applied strengthening technology for buildings constructed of local materials or reinforced concrete structures characterized by insufficient seismic resistance and brittle failure is incorporation of structural elements that lead to an increase in both strength and ductility. In such cases, it is disputable whether and for how long the mutual interaction between the different materials will be preserved, especially in the cases when the structure is forced to vibrate in the state of considerable nonlinearity. The separation of the ductile and brittle structural elements in such cases may reduce the total seismic resistance of these structures.

There are, however, some practical solutions by which the capacity of controlling seismic energy and deformations is considered. Such technological solutions may successfully be applied when strengthening of important structures has to be provided. So, for example, the vibration base isolation system might provide sufficient seismic resistance /3/ to buildings of cultural-historic values characterized by brittle failure mechanism, i.e., masonry buildings of sufficient strength capacity. The elements for controlling of deformations or elements that increase the energy absorption capacity may successfully be applied if supported by sufficient experimental evidence.

STRENGTHENING OF BUILDINGS BUILT AND STRENGTHENED BY DIFFERENT MATERIALS

The buildings prevailing in old urban units are mainly constructed of local materials like bricks, stone, timber or combination of these materials. Buildings dating from the fifties and the sixties of this century are often constructed of reinforced-concrete in combination with stone and brick masonry. Being built in a period of non-existence of seismic design codes, most of these buildings are characterized by an extraordinarily high level of vulnerability for the expected seismic hazard. The decisions of the regional and federal governments regarding strengthening of certain structures in these towns is therefore completely justified. Such a decision was also made by the regional government of Emilia Romagna - Italy regarding strengthening of some buildings in the town of Rimini during their reconstruction. The investigations were realized jointly by the Istituto di Tecnica delle Construzioni, Istituto di Scienza delle Construzioni from the University of Bologna, Italy and the Institute of Earthquake Engineering and Engineering Seismology, University "Kiril and Metodij", Skopje, Republic of Macedonia. Some of the results obtained from this project are presented in this paper.

Most of the considered buildings represent mixed structural systems. The first storey of these buildings is mainly constructed of reinforced-concrete elements (beams and columns) and walls made of bricks; the upper storeys are constructed of brick masonry. An economically justified strengthening method that would provide the necessary seismic resistance had to be defined for such type of structures. It is evident that for such structural systems, it was necessary to increase equally both strength and ductility by applying strengthening materials other than those of the principal structural system. The arguments in favour of space functioning were decisive for not strengthening the existing RC elements at the first storey.

Considering the importance of the problem, the following analytical-experimental-analytical programme was proposed and accepted:

- Structural analysis of the considered buildings;
- Forced and ambient vibration measurements on a real structure /12/ and mathematical model formulation using experimental data;
- Material quality test of a tested building /5/;
- Shaking table test of a hypothetical fragment in large scale (1/3) simulating the most relevant characteristics of the forced vibration tested building considered as an original model /11/;
- Shaking table test of a model strengthened with RC jackets /9/;
- Shaking table test of a strengthened model with a central core /10/;
- Strengthening of the considered (tested) building by using the strengthening methods developed by this research programme;
- Forced vibration test of a strengthened

building and mathematical model formulation for the needs of correlation /12/.

In addition to this programme, two additional shaking table tests were performed after repairing of the damaged models /13, 14/.

Discussed in this paper will be only part of the seismic shaking tests and other relevant analytical and experimental data.

Material quality test

The quality of materials and structural elements of the building to be strengthened are defined for the purpose of determining the mechanical properties of the basic material and the structural elements. So in this case, performed were investigations for definition of the strength characteristics of mortar (Fig. 1), bricks and masonry itself. For this purpose, wall fragments were taken in order to define the shear strength of mortar, bending and shear strength of bricks and shear strength capacity of the masonry specimens.

Similar tests were carried out for the model materials in order to define the shear strength capacity of the brick masonry wall (Fig. 2).

Shaking table tests

Three basic shaking table tests were performed within the frameworks of this programme after which the same models were repaired and the shaking table tests repeated. The following three plus three shaking table tests are performed.

- Model M1 represents a hypothetical model of the original structure in scale of 1/3;
- Model M2 represents model M1 strengthened by one side RC jackets;
- Model M3 represents model M1 strengthened by RC central core;
- Model M1-R represents model M1 repaired by injection after completion of testing of model M1;
- Model M2-R represents the repaired model M2, and,
- Model M3-R represents model M3 repaired by injection after completion of testing of model M3.

All shaking table tests were performed under similar (almost the same) testing programme. According to this programme, tests for identification of the dynamic properties of the models and shaking table tests simulating different earthquake time histories were performed. For the linear tests, five different earthquake time histories were used, while for the nonlinear tests, one earthquake time history was used to which responses of the models were most sensitive. For all models, a sufficient number of experimental data were recorded in respect to acceleration and displacement time histories, stress-strain distribution, uplifting of the model, sliding, diagonal deformations, etc.

The main model for all these investigations was model M1. Its geometrical characteristics are given in Fig. 3, 4 and 5. From Figs. 3 and 4 it is clear that RC structure exists only at the first storey and consists of RC columns and a beam connected by a floor slab. The model is symmetrical and has four storeys and a first storey that is relatively more flexible in respect to the remaining three storeys.

Model M1 was most sensitive during the simulation of the Petrovac earthquake record (Montenegro earthquake, 1979, Petrovac record, N-S component). By simulation of this earthquake on the shaking table by 0.295 g (PET.SPAN 150), obtained were the first cracks in walls W1 and W2 (Figs. 3 and 4), i.e., point A (Fig. 6). After this test, another simulation of the same earthquake (PET.SPAN 260) was carried out whereby an acceleration of 0.51 g was generated. At this excitation level, walls W1 and W2 suffered heavy damages (diagonal cracks and partial failure) mainly at the first floor level. At the second floor, there were small cracks, while no visible damage was observed at the third and the fourth storey. It should be pointed out that all four W1 walls and the two W2 walls at the first storey did not suffer damages of the same type although a crack development mechanism exists. It should also be pointed out that the mechanism of damages is such that it primarily takes place along the joints.

The variation of the shear force at the first floor in function of the relative storey displacement is presented by a full line in Fig. 6. From the Q-Δ relationship, it is clear that the initial stiffness of the model at the first storey is about 18.4 kN/mm (part OA), while for part AB it drops to 2.8 and further deteriorates. The ductility ratio is higher than 4 (for Δc/Δa) which basically depends on the presence of the RC frame only at the first storey.

After testing of model M1, it was decided that the model be repaired by injection of special cement mortar. The repaired model M1-R was subjected to the same testing procedure as that for model M1. It is interesting to note that the initial stiffness of model M1-R was not affected by the repair process, i.e., the initial stiffness of models M1-R and M1 is identical. An ideal value for the first yielding point (A) and an ideal post-yielding behaviour were achieved.

As to the mechanism of damages, there have been some essential differences. Namely, cracks in model M1 were not opened but some new developed especially at the second storey, some indications of their occurrence at the third storey being evident. The Q-Δ relationship for the first storey is presented by a dashed line in Fig. 6. This relationship points out that the repair of model M1 by injection of special cement mortar does not affect the overall seismic behaviour which is completely maintained. After the tests, investigations were performed for the purpose of finding out whether there were any disturbances of the connection between the injected mortar and the bricks during the dynamic tests. As such a mechanism of damages was not observed, it was concluded that there is a sufficient adhesive connection between the injected mortar and the brick masonry walls.

Two concepts for strengthening of model M1 were adopted. According to the first concept, strengthening was performed by means of external RC jackets (Model 2). Strengthening of model M3 was performed by a central RC core according to the second concept.

The disposition of the RC jackets for the first and the second storey is presented in Fig. 7, while that for the third and the fourth floor is given in Fig. 8. From these figures (Figs 7 and 8) and Fig. 9 as well, it is concluded that the width of the RC jackets is variable along the height of the building. Reinforcing of the RC walls was performed by net Q139 ($\phi 4$) reinforcement that was anchored into the brick masonry walls. The corners of model M2 were additionally strengthened (see Fig. 7). While selecting the disposition of the RC jackets and their thickness, care was taken to maintain the balance between the existing strength capacity and that of the RC jackets. This was performed for the purpose of achieving the best possible interaction between the brick masonry structure and the RC jackets, special emphasis being put on detailing, too.

The tests on the seismic shaking table were performed by the same order of earthquake simulation as that in the case of model M1 (Petrovac earthquake, a = 0.51 g - PET.SPAN 260). At this excitation level, small cracks occurred at the upper storeys, while the first resonance frequency dropped from 6.20 Hz in the beginning to 5.80 Hz after the test. Testing was further performed for three additional acceleration levels (0.53g; 0.64 g and 0.75 g) which resulted in occurrence of cracks at the corners of the jackets, diagonal shearcracks at the two upper brick masonry walls and bending cracks along the connection between the RC jackets and the foundation beam. Analyzing this mechanism of occurrence of damages and considering the fact that most of the jackets remained intact, it was decided to repair model M2. Repair was mainly done at the contact between the RC jackets and the foundation beam by incorporation of a side angle profile L40 anchored previously into the foundation, the reinforcement of the RC jackets being welded to it. The repaired model M2-R was of a somewhat lower initial stiffness (the first natural frequency for M2 was 6.20 Hz, while for M2-R it was 6.08 Hz). The shaking table tests for the M2-R model were repeated in the same order as that for model M2. During the last test, essentially different failure mechanisms were obtained. Namely, diagonal cracks occurred in the W1 walls, in both the brick masonry part and the RC part, as well as heavy damages and partial failure of walls W4 took place at the third and the fourth storey. For this model, it is characteristic that the failure mechanism is primarily concentrated at the third and the fourth storey. After the last test, the first natural frequency dropped to 4.20 Hz.

The seismic shear force at the first storey versus relative storey displacement relationship for both Model M2 and model M2-R is presented in Fig. 10. The figure shows that compared to model M1, model M2 is not characterized by an essentially higher initial stiffness, its strength capacity is higher for almost 80% and its ductility ratio is somewhat lower since, as it was already stated, the bending capacity between the RC jackets and the foundation is lower than the expected. Therefore, the behaviour mechanism is essentially changed after strengthening, although this has to be proved by an experimental model test.

Model M2-R is characterized by a somewhat lower initial stiffness in respect to model M2, but its rigidity considerably increases after the first yielding point A. The maximum strength capacity of model M2-R is larger than that of model M2. Although it is not possible to determine precisely the ductility ratio for M2-R, it is evident that it is higher than that of M2.

Fig. 11 shows the disposition of the RC central core in model M3. The RC central core stretches from the foundation to the fourth storey. In order to decrease the effect of the RC core in respect to brick masonry, i.e., balance the participation of both the RC core and the brick masonry walls on one hand, and decrease the overturning moment of the RC core at the foundation level (which would create additional problems as to the soil-structure interaction of the actual structure), it was decided that the RC core be elastically supported at the foundation level. This was achieved by incorporation of hard rubber under the base of the RC core. In this way, the mechanism of distribution of seismic forces along the height of the structure was essentially made different. A detail of this connection is presented in Figs. 11 and 12.

The experimental testing procedure applied for this model was the same as that applied for the previous four models. The initial resonant frequency was 9.60 Hz (Model M1, f_1 = 8.0 Hz; model M2, f_1 = 8.60 Hz). During simulation of the Petrovac earthquake with peak acceleration of 0.50 g, small cracks occurred at the level of the first storey. The same earthquake was further generated with a peak acceleration of 0.76 g inducing severe damages to the floor slabs, the external and the internal brick masonry walls at all the four floors. The first natural frequency dropped to 4.80 Hz.

The relationship between the seismic shear force at the first floor and the corresponding relative floor displacement is displayed in Fig. 13. The figure shows that the strength capacity of this model is almost twice higher than that of model M1. This means that the RC core and the brick masonry walls have approximately the same strength capacity. The ductility ratio for this model and for the first floor is approximately 4.0.

After these tests, it was decided to repair the model by cement mortar injection, i.e., injection of the same material as that used for model M1-R. The repair was done by the Ediformaciai company from Bologna, while the material was provided by Emacoresto MAC.SPA, Treviso, Italy. After repairing the brick masonry walls by injection, the complete testing programme was repeated. The results of these tests are presented in Fig. 13 where the relationship between the shear force and the relative storey displacement is presented by a dashed line. A significant reduction of initial stiffness and ultimate strength capacity can be observed for model M3-R. This reduction is primarily due to the fact that the floor slabs were not repaired and the fact that the same level of interaction between the RC core and floor slabs as that obtained for the damaged model M3 was accepted. For the M3-R model, the initial resonant frequency of the first mode was 6.4 Hz, which dropped to 4.10 Hz after the last test. During the last shaking table test performed by simulation of the Petrovac earthquake record with peak acceleration of 0.76 g, severe damages to all the external and internal brick masonry walls was observed at all the four floors.

The inspection made after this test was aimed at finding out whether there were any disturbances of the connection between the bricks and the injection mortar. Such a disturbed connection was not observed, but it was evident that cracks in model M3-R were developed at different places than those in model M3. There were cases when cracks were displaced for two rows of bricks only, but they never occurred at the same place as those in model M3.

The comparative presentation of the shear force - relative storey displacement relationships for the first storey of all the six tested models is given in Fig. 14. This figure shows the variation of initial stiffness of the models with the application of different repair and strengthening methods and the variation of the ultimate strength capacity, as well. A global insight into the ductility of the structure, i.e., ductility ratio is provided, too.

PROCEDURE FOR REPAIR AND STRENGTHENING OF STRUCTURES

While choosing an economically and technically justified strategy of repair and strengthening of structures, a certain procedure has to be followed. This procedure should be based on one's own knowledge and the world experience and should be developed in compliance with the requirements of the corresponding codes, if any. The following procedure was adopted in this case study:

- Synthesis of the aims and the constraints of strengthening in respect to the natural, urban, cultural, architectural, historical, age, purpose, structural system, structural materials and other factors that are relevant for decision making;
- Synthesis of the national code requirements for seismic design and construction of buildings as well as repair and strengthening of structures, if any;
- Analysis of the existing seismic resistance of the considered structure and definition of the required seismic resistance;
- Field and laboratory tests for definition of the existing dynamic properties of the structures, mechanical properties of the construction material and structural fragments by forced and ambient vibration tests and quasi-static field and laboratory tests.
- Definition of a strengthening strategy, strengthening construction technology and design calculations, detailing, specification and cost estimation;
- Experimental verification of the repaired or strengthened structures and verification of whether the expected seismic resistance has been achieved.

It is clear that verification and proving of the quality of the performed strengthening is the most difficult task to be performed within the described procedure. Some other procedures based on the experience of others should always be considered.

CONCLUSIONS AND RECOMMENDATIONS

Strengthening of structures is, in general, a very responsible and complex task for structural engineers, especially when it has to be done by using structural materials other than those the structure is composed of. Each particular case requires a research for itself before decisions are to be made as to choosing the appropriate strengthening strategy. One of the most responsible tasks in making decisions regarding the strengthening strategy is diagnosis of the existing bearing capacity of the structure and the mechanical properties of the used materials and structural elements.

The existing codes for seismic design of structures primarily refer to modern materials. Recommendations have to be given in the national codes in order that the engineers might come to the right decision regarding the strengthening concept.

As far as it is known, a large number of buildings constructed of different materials, characterized by different structural systems, age, geometry, etc. are strengthened throughout the world each year. All these strengthened structures are functioning and have most probably been exposed to moderate or strong earthquake effect. A record on the behaviour of such buildings during an earthquake will be quite valuable. To that effect, the experience of engineers in making an appropriate strengthening concept is very important. It is therefore a time that our association proposes a mechanism for creation of a data bank on repaired and strengthened structures in seismically prone areas in the world and especially records on the behaviour of these structures during strong and moderate earthquakes.

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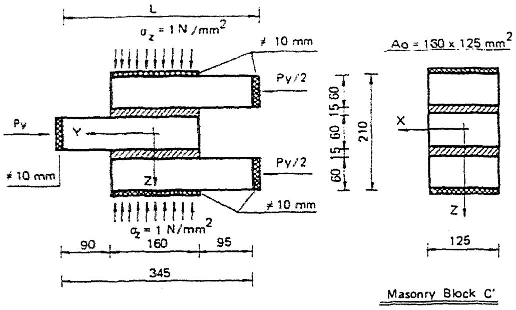

Figure 1. Masonry specimens subjected to shear test.

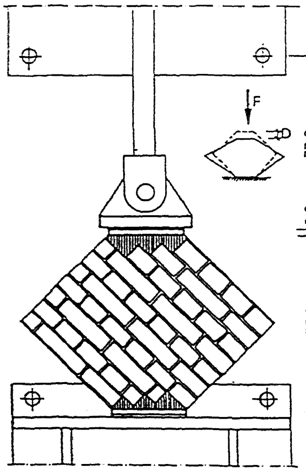

Figure 2. Laboratory test of 4 wall specimen.

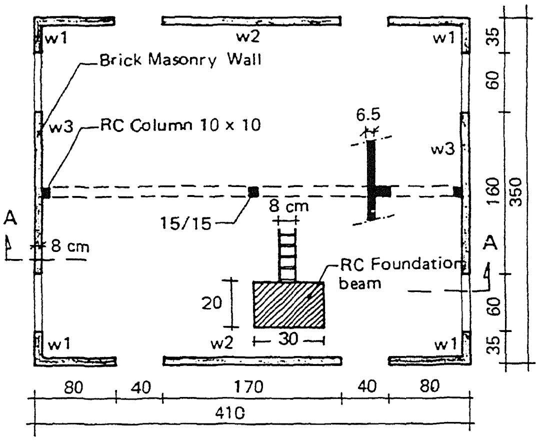

Figure 3. Plan of the M1 first floor.

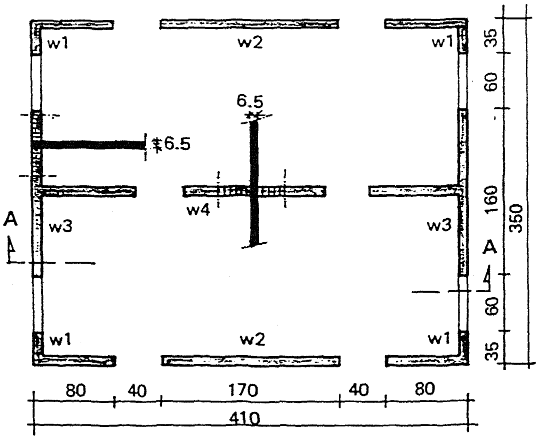

Figure 4. Plan of the 2nd, 3rd and 4th M1 floor.

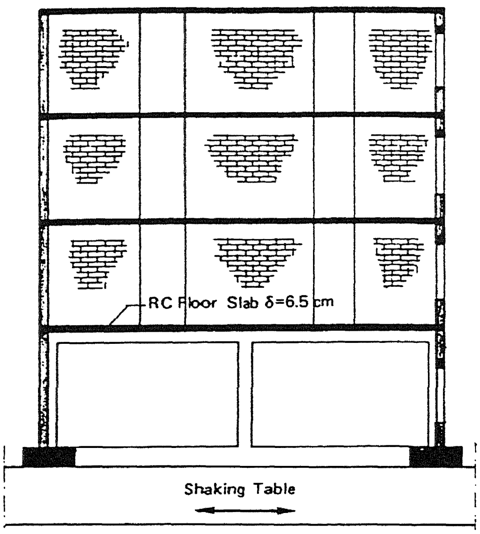

Figure 5. Cross-section A--A of M1.

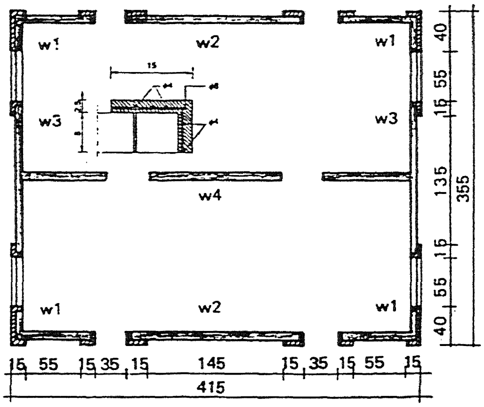

Figure 8. Plan of the 3rd and 4th M2 floor.

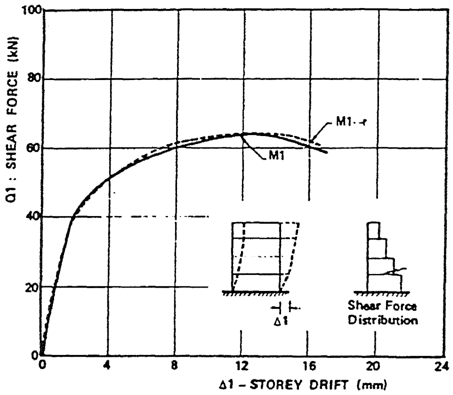

Figure 6. Shear force - storey drift relationships for the first floor of M1.

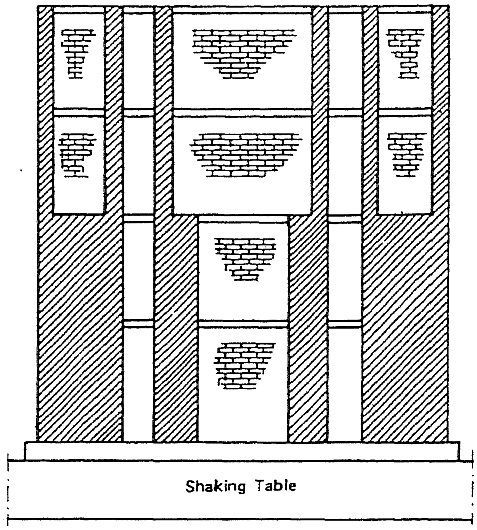

Figure 9. Front view of M2

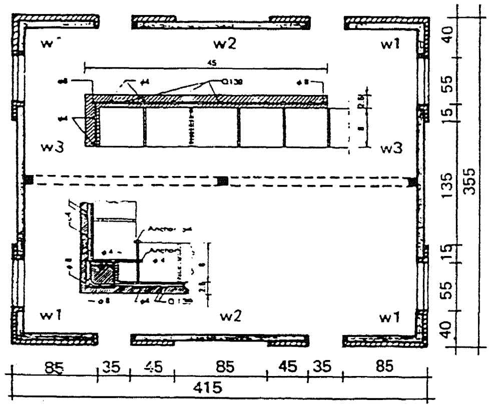

Figure 7. Plan of the M2 first floor.

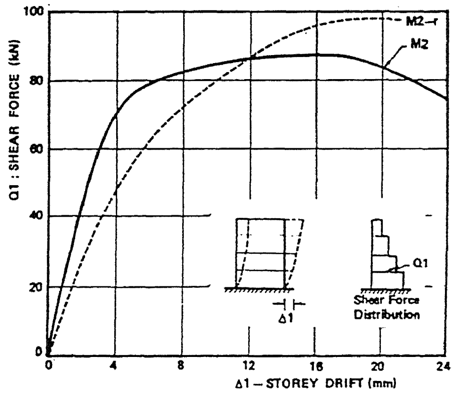

Figure 10. Shear force - storey drift relationships for the first floor of M2.

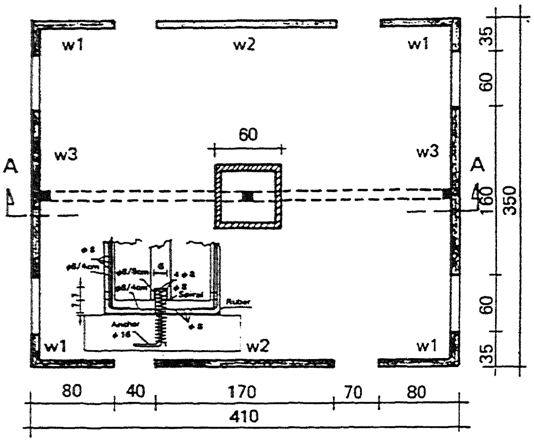

Figure 11. Plan of the M3 first floor.

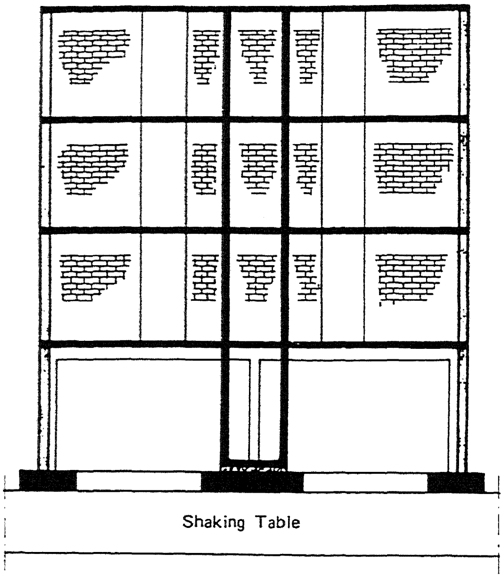

Figure 12. Cross-section of M3.

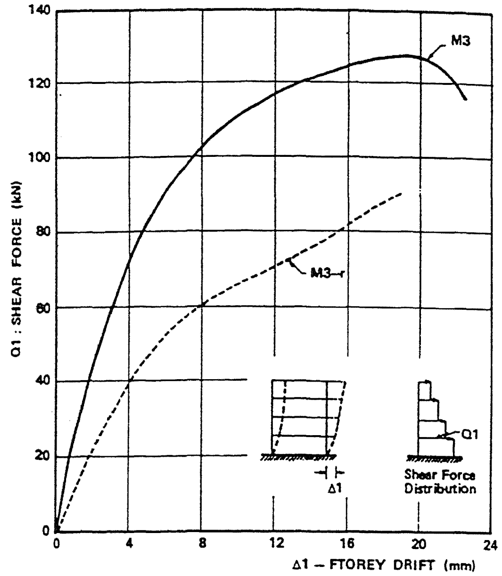

Figure 13. Shear force - storey drift relationships for the first floor of M3.

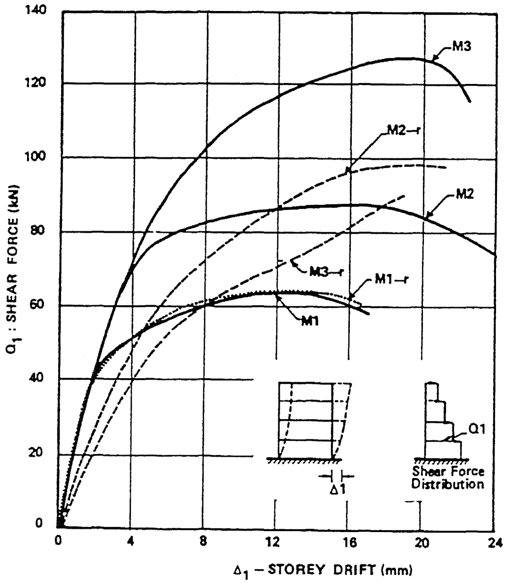

Figure 14. Shear force - storey drift relationships - first floor.