LOW INTENSITY EARTHQUAKES ON MULTI-STOREY R.C. BUILDINGS: RECORDS, IDENTIFICATION AND MODELLING

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SUMMARY

This paper presents experimental results obtained by recording several after-shocks on a real building, following the major earthquake at Irpinia, Italy, Nov. 1980.

The structural characteristics of the building can be listed with the following key words: reinforced concrete frames, 8-storeys, L-shaped plan, no structural shear walls, partitions in light hollow bricks, foundations on footings, ground of over-consolidated clay, no aseismic design, light damage due to the previous main shock.

Recordings were made at the foundation level and at the roof. Fundamental frequencies and modes are evaluated from the collected experimental data. A comparison with the responses of a numerical model (space frame) points out the importance of partitions for modal frequencies. An evaluation of the equivalent average stiffness of light brick walls in their partially cracked state is obtained, together with the damping ratio for the whole building.

Concluding suggestions for the design and strengthening of buildings of the same structural type are derived.

INTRODUCTION

Many modern buildings in mediterranean countries are built with a reinforced concrete frame structure, which is then completed by partitions and external walls made of light hollow bricks. In current design practice the latter elements are usually disregarded, although many field experiments (1) (3) have pointed out their importance for dynamic behaviour.

The justifications for this design method rest on the fact that non bearing walls are stressed and (partially) cracked in the very first cycles of a strong earthquake. After this has happened, the r.c. structure alone is thought to be the structural element, contributing to both stiffness and strength.

To verify this assumption a field investigation was undertaken by the authors after the main Irpinia shock, Nov. 1980. A partially damaged building was selected in an area abont 100 km from the epicenter, where the maximum ground acceleration of the main shock was estimated to have been 5% g (about 35% g at the epicenter).

So the actual damaged state of this building could approximate its hypothetical state after a few seconds of shock at the epicenter. Few responses of the selected building to after-shocks were recorded.

An analysis of these records is presented here to show the dynamic characteristics of the cracked state.

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FIG. 1

FIG. 2

FIG. 3

## THE CASE STUDY

Figs. 1 and 4 illustrate the elevation and plan of the building. It stands on a hill (slope of about 10%) of over-consolidated clay in Potenza.

The structural lay-out is described in fig. 5 (columns and main girders), showing that the main structure of the building has multi-bay frames in both orthogonal directions. Slabs are of r.c.joists and clay bricks. The stairs are supported by crooked r.c. girders (see fig. 7) that form a kind of bracing with excentric connexions around the stairs.

All the walls are in light bricks with horizontal holes (about 60% void ratio): two vertical layers of 12 cm thickness each for the external walls and one layer for the partitions.

The damage produced by the main shock consists of cracks in the walls, (see fig. 2), localized damage to the stair girders (see fig. 3) and small cracks in the columns at the first storey.

## EXPERIMENTAL RESULTS

The positions of seismometers is given in fig. 5. The characteristics of the recording net is given in ref. (4): it was active for about two months, recording a) several after shocks with a close epicenter, b) the earthquake of Greece, Feb. 24, 1981 and c) a few wind storms. Details on the records are given in ref. (4): in the present paper only a few samples are given in order to explain the processing and to justify the conclusions.

In fig. 6 the response spectra of one ground motion (close epicenter) are given. They show typical features for that region and soil conditions. In fig. 8 the Fourier analyses of the responses of the roof to the same excitations are given. Dominant frequencies are very sharp, lying in the range 1 to 2 seconds, which is rather high in comparison to similar uncracked building, see ref. (1) and (3) for istances. This discrepancy is in accordance with many other observations, see for istance (2).

On the basis of an examination of all the channels together and their phases at the peaks of fig. 8, an evaluation of the displacement of the roof in the first two modes is obtained, see fig. 9. Torsional effects, as expected in this kind of L-shaped building, are very evident. It is also clear that the building is less stiff in one direction (5 bay frames). This fact explains the damage in pictures 2 and 3 (see also plan 4) while the corresponding orthogonal elements (stair girders and walls) suffered no damage.

## COMPARISON WITH A NUMERICAL MODEL

A three dimensional frame model was numerically simulated for the case study and subjected to the same (recorded) ground motions. Partitions and external walls are simulated by plain shear panels, and therefore their "equivalent" average stiffness takes into account holes in the bricks, openings (like doors and window, abont 15% of the total panel surface on average) and cracks.

Several numerical cases were run, proportionally varying the equivalent stiffness of the walls. The following table summarizes the results in terms of modal frequencies.

The value of the equivalent tangent modulus $G = 5000$ kg/cm$^2$ best matches the first modal frequency. Howewer, the effect of "clustering" of the frequencies is more relevant in the numerical model than in the real case. The value $G = 12000$ kg/cm$^2$ already existed in the literature as a best fitting for similar buildings and similar, but uncracked, brick walls (see ref. 1). The shift in the dynamic behaviour from uncracked to craked brick walls is then made evident by the two columns of the above table, while the importance of partitions is clear from the column for r.c. frames alone.

| Period (sec.) | Numerical model: Only r.c. frames | Numerical model: Tangent modulus of walls $G = 5000\ kg/cm^2$ | Numerical model: Tangent modulus of walls $G = 12000\ kg/cm^2$ | Experimental results |
|---|---|---|---|---|
| $T_1$ | 1.88 | 0.93 | 0.67 | 0.93 |
| $T_2$ | 1.54 | 0.86 | 0.59 | 0.84 |
| $T_3$ | 1.34 | 0.77 | 0.53 | 0.67 |

Using the best fitting for G (5000 $kg/cm^2$) a comparison between the responses in the time domain was attemped, varying the damping ratio. A close agreement between thoretical and experimental data was difficult to obtain because of the already mentioned "clustering" of the frequencies both in the numerical and in the experimental model. However, reasonable agreement for the displacement peaks was obtained for a damping ratio of 7%, for all the modes. Note that ref. (1) indicates a damping ratio of about 5% for similar buildings with uncracked walls.

## CONCLUSIONS

1. In the dynamic behaviour of buildings with a r.c. structure, partitions and non bearing light hollow brick walls are very important, even when the latter are partially cracked.

2. For the case study a figure of G = 5000 $kg/cm^2$ was obtained for the equivalent tangent modulus of the walls, i.e. approximately one half of the stiffness in the uncracked state. This figure takes into account on average openings in the walls (about 15%) and cracks.

3. For the same case 7% was obtained for the damping ratio (5% for the estimated uncraked case).

4. These results are related to a moderatly damaged building and low intensity after-shocks. Care must be taken in extrapolating these results to extreme cases (high intensity, large damage).

The present conclusions can be used more directly however, in the design and/or the strengthening of similar buildings by using r.c. shear walls. In fact, in these cases the storey drift is supposed to be limited (as it was in the present experiments), even for strong motions.

## ACKNOWLEDGEMENTS

Ing. Calogero Alongi ran the numerical cases. The field investigations were performed also by the other authors of ref. (1).

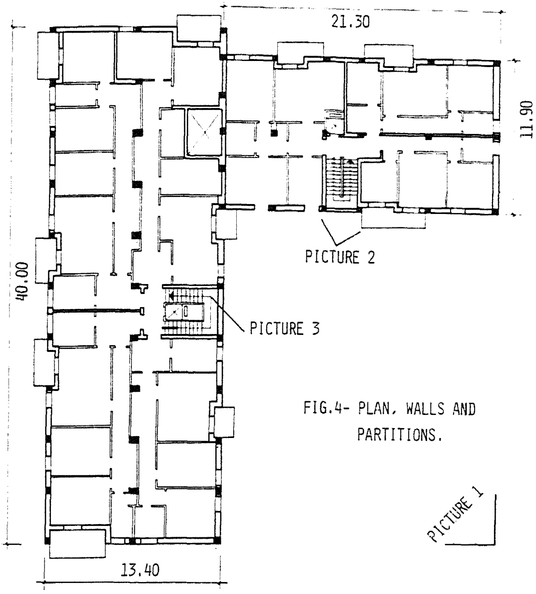



FIG.4- PLAN, WALLS AND PARTITIONS.

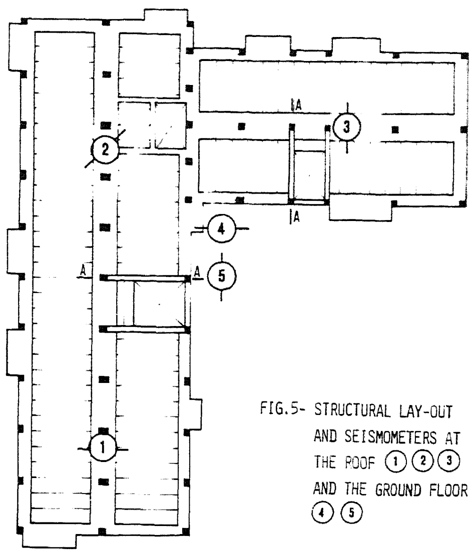



FIG.5- STRUCTURAL LAY-OUT AND SEISMOMETERS AT THE ROOF ① ② ③ AND THE GROUND FLOOR ④ ⑤

FIG.6- RESPONSE SPECTRA

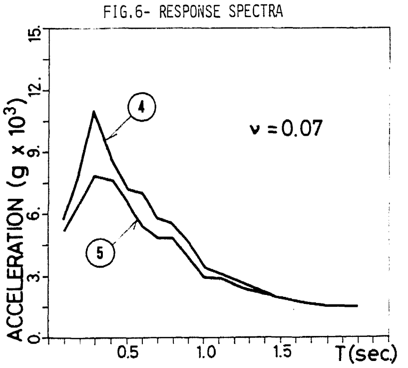



FIG.7- SECTION AA (STAIRS)

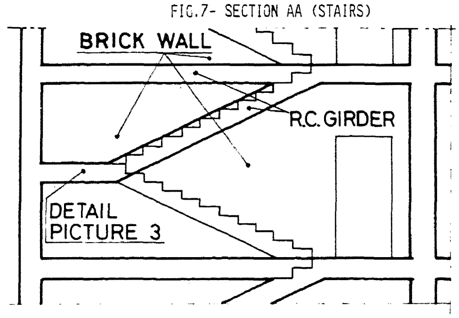



FIG.8- FOURIER SPECTRA OF THE RESPONSE (VELOCITY) OF THE ROOF.

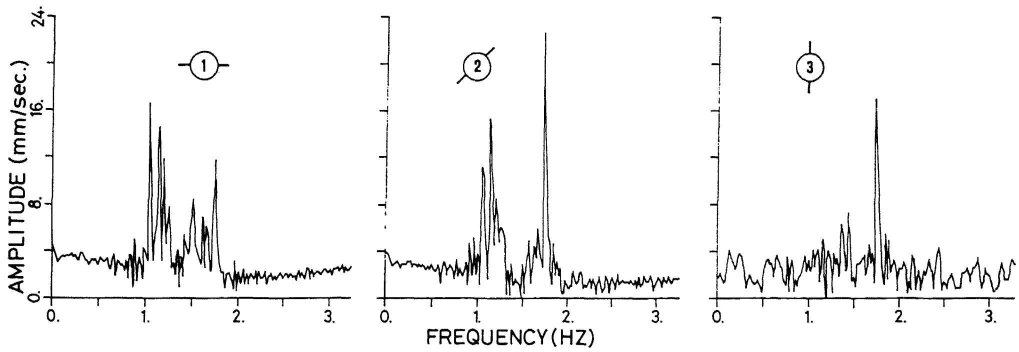
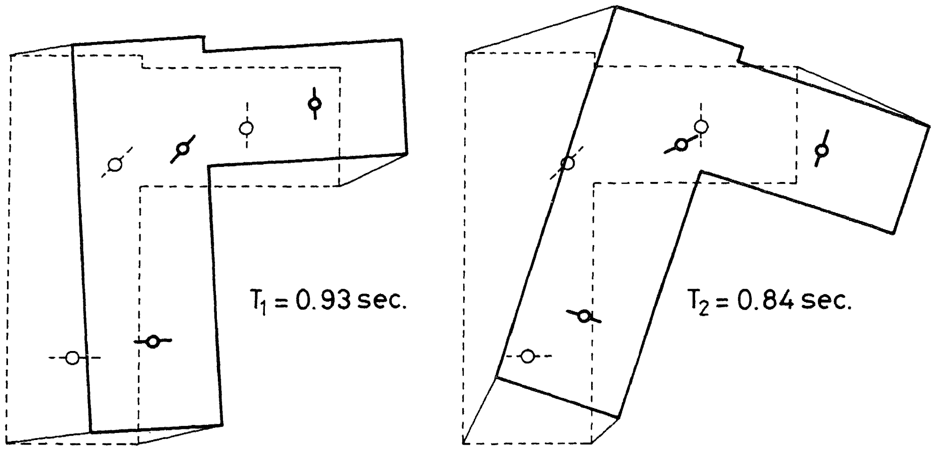


FIG.9- MODAL SHAPES (EXPERIMENTAL)



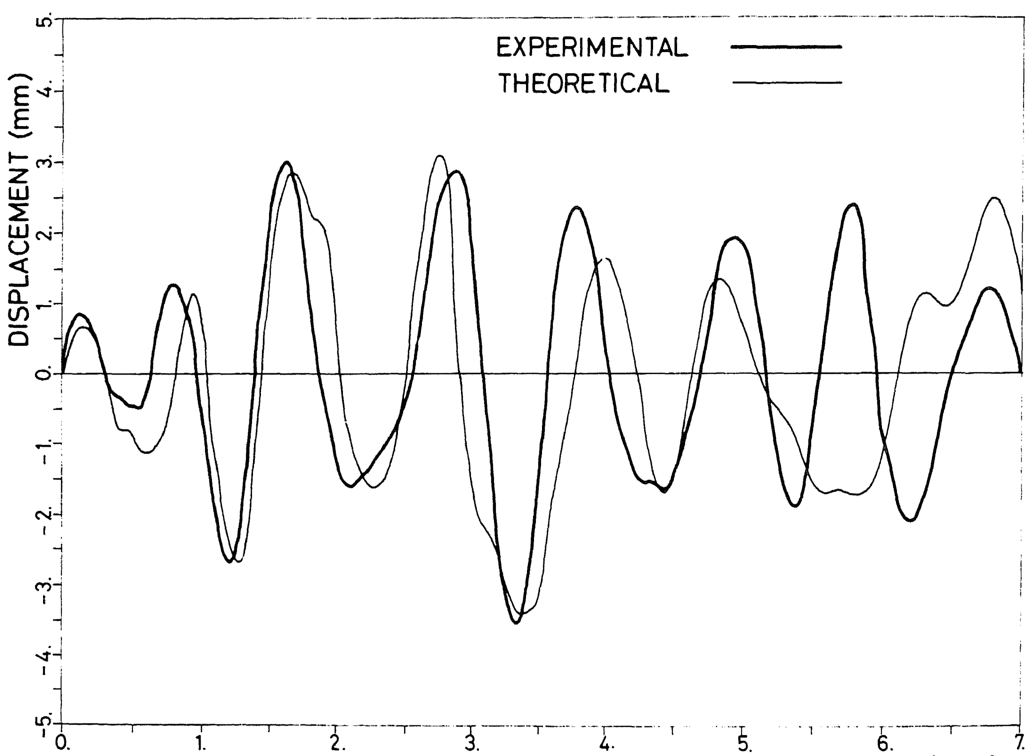




FIG. 10 RESPONSE OF SEISMOMETER (1)

THEORETICAL : G 5000 KG/M$^2$, ν 0.07

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