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Paper No. 806. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

RETROFITTING OF R.C. BUILDINGS BY ENERGY DISSIPATING BRACING: NUMERICAL SIMULATIONS AND COMPARISON WITH EXPERIMENTAL TESTS

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ABSTRACT

Retrofitting r.c. framed existing buildings is one of the main problems to be tackled in order to reduce the seismic risk of recent settlements. Passive control techniques based on energy dissipation through special devices embodied in steel bracings is certainly a promising solution. Its efficiency was examined trough shaking table tests on 1/4-scale models of a 4-storey, two bays r.c. frame. The tests were carried out at the Department of Civil Engineering of the University of Bristol, for a joint research project of the ECOEST program, in cooperation with the Department of Structure of the University of Basilicata. To better evaluate and understand the test results, a detailed numerical model of the r.c. retrofitted frame was implemented. The results of the numerical simulations will be presented and compared with the experimental outcomes.

KEYWORDS

Seismic passive control; Retrofitting; Energy absorbing device; Braced frame; Numerical simulation

INTRODUCTION

Many design and practical problems have to be solved to attain a good engineered solution in retrofitting r.c. framed buildings. Besides the traditional techniques based on member jacketing and/or addition of stiff and resistant new elements, like r.c. walls or steel braces, the new passive control techniques, especially those based on energy dissipation through special devices, appear very effective (Dolce, 1994). Usually these devices are work in series with steel braces, to dissipate energy in the interstorey displacements. Great advantages come from the adoption of this technique with respect to the prevention of the existing frame from high ducility demand, the control of forces transmitted to the frame-brace connections, the dramatic reduction of the interstorey displacement and, consequently, of the damage to non structural elements. A major design problem regards the dimensioning of the bracings both for their stiffness and their strength. A design method was proposed by Braga, D'Anzi (1994) for hysteretic or friction dampers. It optimises the above two quantities to get a uniform ductility demand at all stories of the r.c. frame. In order to validate the design method, some shaking table tests on 1/4-scale models of a 4-storey, two bays r.c. frame were carried out. These tests were very promising for what concerns the performances of both the energy dissipating system and the retrofitted structure and are described in a companion paper (D'Anzi et al., 1996).
A detailed numerical simulation was subsequently carried out to better understand the overall and local behaviour of the tested models. The DRAIN-2DX finite-element code was selected, as it includes a fibre-

elements for accurate beam-column modelling. This is considered a fundamental feature for the retrofitted structure, where large variation of axial forces are expected due to the bracings. The consequent concrete cracking can produce important redistribution of forces between the r.c. structure and the steel bracings.
In this paper the results of the numerical simulations are presented and compared to the test records.

MODELLING ASPECTS

Experimental model

The experimental 1/4 model was referred to a r.c. framed building, which was designed only for self-weight and vertical live loads for residential purpose, without taking into account horizontal loads. The main characteristics of the model are shown in table 1.

Tab. 1 - Model characteristics

	BUILDING	1/4 MODEL
• No. of levels	4	4
• No. of bays	2	2
• Span length	4.5 m	1.125 m
• Column heigth	3.0 m	.75 m
• Floor length in perpendicular direction	5.0 m	
• I, II and III level - slab self-weight (floor finishing and partitions)	3.22 kN/m^2	
• IV level - slab self-weight	3.02 kN/m^2	
• I, II and III level - live load	2.0 kN/m^2	
• top storey load	.80 kN/m^2	
• Beam load for I, II and III level (p + 1/3 q)	19.41 kN/m	4.85 kN/m
• Load increase due to the dimensional analysis		.70 kN/m
• Beam load for I, II and III level	19.41 kN/m	5.55 kN/m
• **Global load for I, II and III level**	**174.69 kN**	**12.50 kN**
• Beam load on the top floor - IV level (p + 1/3 q)	16.42 kN/m	4.10 kN/m
• Load increase due to the dimensional analysis		.56 kN/m
• Beam load on the top floor	16.42 kN/m	4.66 kN/m
• **Global load for IV level**	**147.78 kN**	**10.50 kN**

The seismic retrofitting was achieved by inserting steel braces arranged in an inverted V fashion, equipped with steel energy absorbing devices. A "chessboard" bracing layout was used in order to minimise the seismically induced axial force variations in columns as shown in Fig. 1. Figs. 2 and 3 show the acceleration time history of the used shake and its acceleration response spectrum respectively. For the numerical simulations, only the motion with the maximum intensity, i.e. the one recorded during the last performed test, is used.

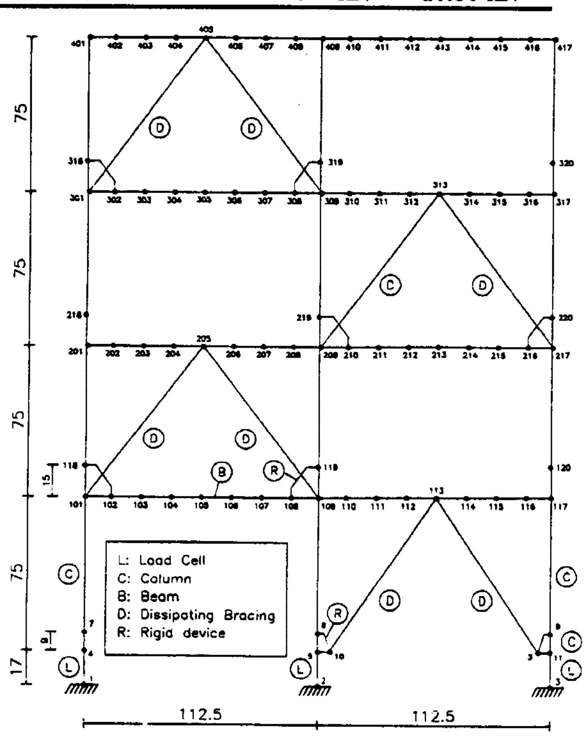

Fig. 1 Frame modelling

Numerical Model

The constitutive laws of the materials were drawn from the results of the experimental tests carried out on some specimens of the materials of the model. The constitutive law of concrete was assumed bilinear with no hardening (Fig. 4a), with 25 N/mm^2 compressive strength.

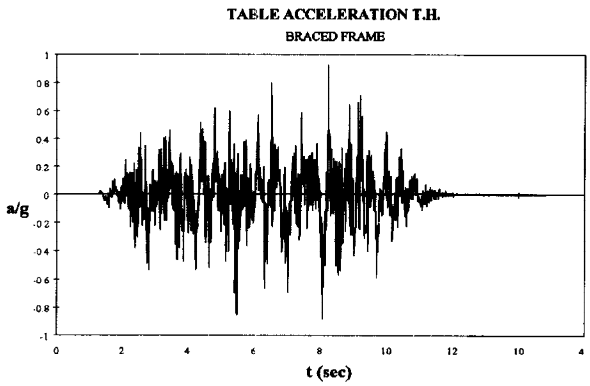

Fig. 2 Acceleration time history

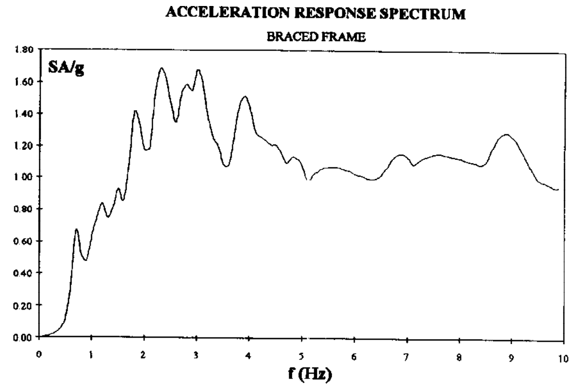

Fig. 3 Platform acc. response spec. (d=5%)

The Young modulus according to Eurocode 2 for a concrete C20/ 25 was assumed, with 5% reduction to take into account some scale effects. Its value resulted to be 27.55 kN/mm². The mechanical characteristics of the reinforcing steel were drawn from the results of the preliminary tests performed on steel bars whose diameter ranged from 2 to 5 mm. The bi-linear hardening stress-strain curve shown in Fig. 4b was assumed for both tensile and compressive strain.

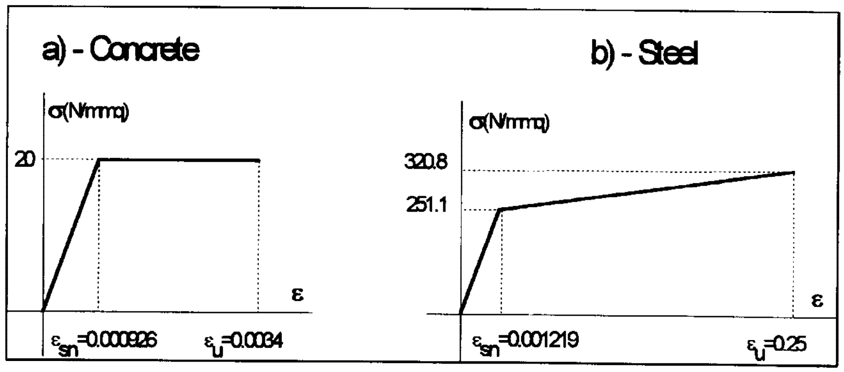

Fig. 4 - Concrete/steel Constitutive laws used in DRAIN modelling

The Young modulus was taken equal to 206 kN/mm².
The force-displacement relationships for the energy dissipating bracings was based on the preliminary cyclic tests performed at the Laboratory of DiSGG - University of Basilicata, on the same elements which were subsequently installed in the experimental model. Each brace was modelled by two parallel truss elements, with different bilinear hardening stress-strain relationships, so that a trilinear relationship resulted, as shown in Fig. 5. A very good fitting to the actual response of the devices was thus obtained (see Fig. 6). The characteristics of the truss couples modelling each energy dissipating brace are shown in table 2.

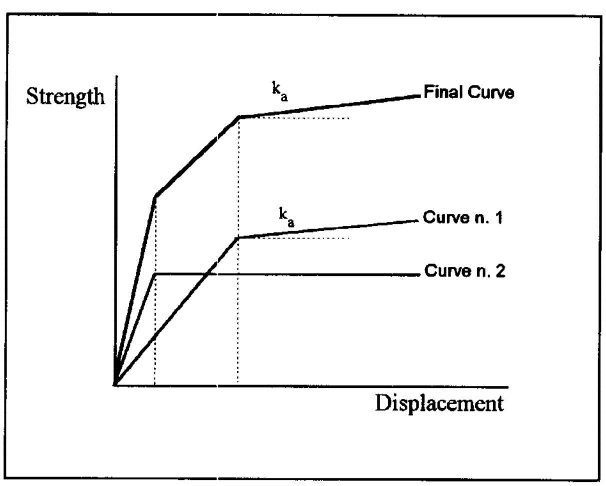

Fig. 5 - Three-linear curve Building

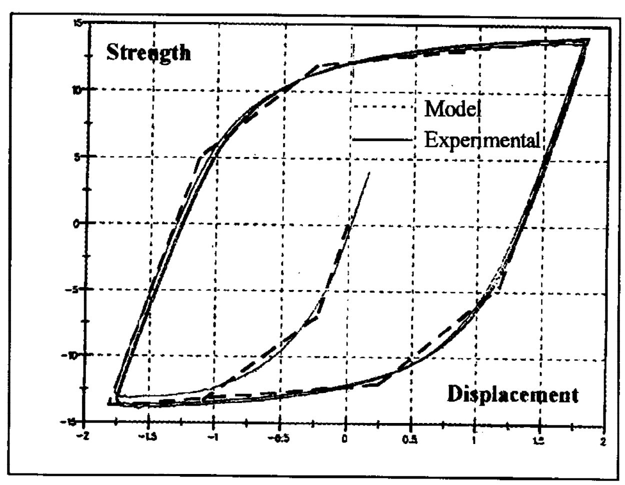

Fig. 6 - Comparison between complete cycles

Tab. 2 - Characteristics of the energy dissipating bracings (Truss elements)

Level	*Element*	*Area* (cm^2)	*Young Mod. E* GPa	*Hardening* Eh/E	*Yield. T.* kN	*Yield. C.* kN
1	1	1.0	157.89264	0.0	6.7607	6.7607
1	2	1.0	36.88979	0.0236	7.3714	7.3714
2	1	1.0	170.09661	0.0	6.3654	6.3654
2	2	1.0	28.01084	0.0089	5.6816	5.6816
3	1	1.0	111.20431	0.0	3.8479	3.8479
3	2	1.0	27.16840	0.014	4.1063	4.1063
4	1	1.0	47.18211	0.0	1.6489	1.6489
4	2	1.0	11.15301	0.026	2.4286	2.4286

R.c. beams and columns were modelled with the fibre element available in DRAIN2DX, which allow the behaviour of concrete and steel fibres to be considered separately. The beam sections were divided in 10 concrete fibres and 2 steel fibres, while column sections were divided in 8 concrete fibres and 2 steel fibres as shown in Fig. 7. Each beam element was divided in sub-elements of different length, in order to follow closely the zones of different reinforcement without increasing the number of elements.

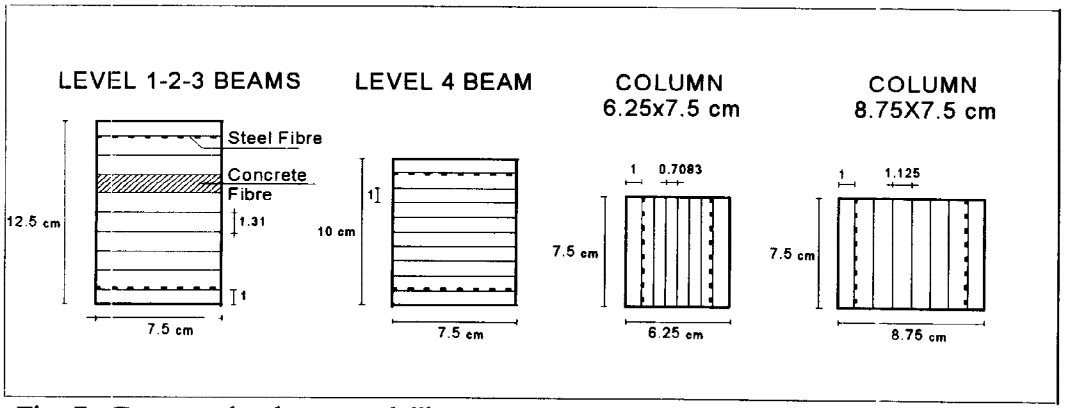

Fig. 7 - Beam and column modelling

Beam and column reinforcements were arranged according to the model design. Moreover the beam elements had a rigid end whose length was taken equal to half the column thickness.

Tab. 3 - Chain pretensioning

Level	n. bars	ϕ (mm)	Tension stress (N/mm²)	Prestress forces (kN)
I	4	6	203.21	13.73
II	4	6	159.67	10.79
III	4	6	106.83	7.22
IV	4	6	50.51	3.41

In the experimental model the beams of each level were equipped with four ϕ 6 pretensioned plane steel bars (Tab. 3) to counteract the tensile forces induced in beams by the bracing action. In the numerical simulation this effect was simulated by inserting an additional reinforcement fibre in the beam sections and by applying compression forces to the beam ends.

The braces are connected to the column-beam corners by steel brackets (Fe430), clamped to the beams and columns by tensioned bars. The bracings are connected to the brackets by friction bolts. The contribution of these brackets to the global stiffness of the structure is very important because of their own stiffness and their not negligible size. Therefore their effect was considered in the model by stiffening the joint with rigid elements connected to the beam and to the column, as shown in Fig. 1. The real contribution of these elements on the global behaviour of the r.c. frame was verified via modal analyses. The comparison of the fundamental frequencies confirmed the need to consider this stiffening effect. The numerical model considers also the influence of the three load cells attached at each column base and bolted to the shaking table on the global structural behaviour. These cells were modelled via beam-column elements (Fig. 1).

A damping matrix proportional to the stiffness matrix was considered. The damping ratio was taken equal to 5% and the value of the fundamental frame frequency f used in the evaluation of β resulted to be equal to 6.1 Hz from the modal analysis.

The numerical model was finally realised by using the following DRAIN-2DX finite elements:
- Load cells (steel) - No. 3 beam-column elements (Elem. 2 DRAIN)
- Columns (r.c.) - No. 24 fibres elements (Elem. 15 DRAIN)
- Beams (r.c.) - No. 64 fibres elements (Elem. 15 DRAIN)
- Dissipating bracings (steel) - No. 16 truss elements (Elem. 1 DRAIN)
- Rigid joint elements (steel) - No. 10 beam-column elements (Elem. 2 DRAIN).

A total number of 117 elements, 88 joints and 255 degrees of freedom were considered in the model.

STATIC AND MODAL ANALYSIS

Before running the non-linear dynamic analysis a static analysis and a modal dynamic analysis were carried out. The static analysis was aimed at initialising the element stresses due to vertical loads. The modal analysis was aimed at making a first comparison of some parameters with the experimental outcomes. It was performed on the vertically loaded structure, so that the actual cracking situation was taken into account. The main dynamic characteristics of the structure are shown in Tab. 4. The fundamental period drawn from the experimental results is equal to 0.163 and therefore is practically identical to the numerical value.

Tab. 4 - Dynamic Characteristics

Mode	Period (sec.)	Modal Mass
1	1.62E-01	*.84924042*
2	*6.90E-02*	*.10893476*
3	*4.38E-02*	*.02675009*
4	*2.95E-02*	*.00980731*
Total		*.99476918*

COMPARISON OF RESULTS

A general satisfactory agreement between the test outcomes and the numerical results was obtained, at least in terms of the keinematic quantities (displacements and accelerations) which are examined in this paper.

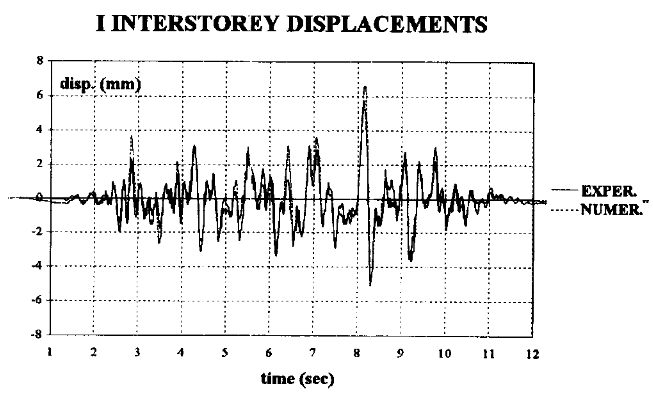

Fig. 8 I interstorey experimental and numerical displacement time history

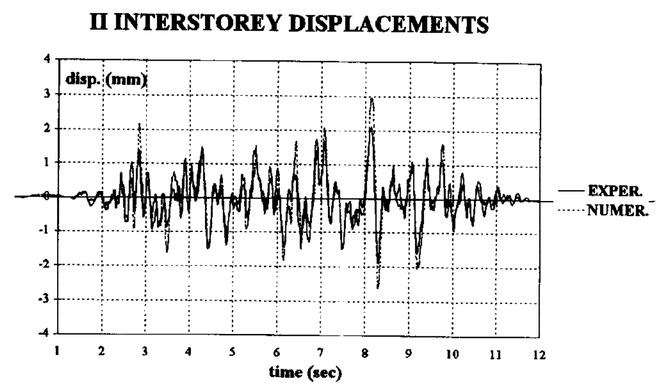

Fig. 9 II interstorey experimental and numerical displacement time history

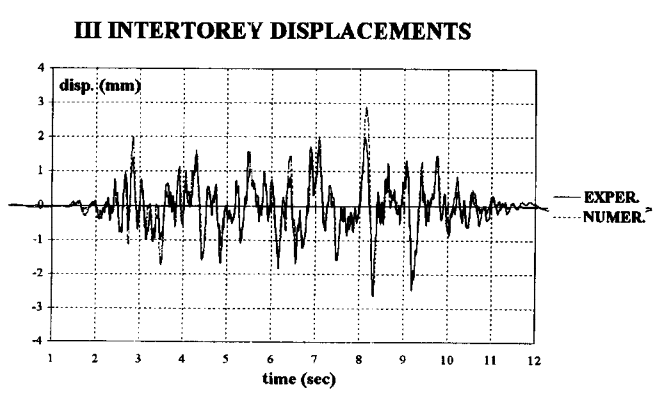

Fig. 10 III interstorey experimental and numerical displacement time history

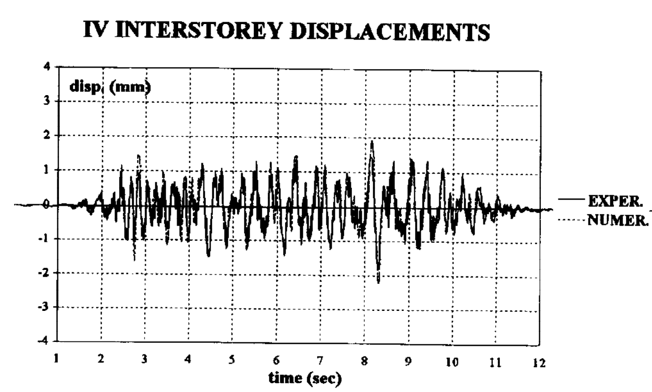

Fig. 11 IV interstorey experimental and numerical displacement time history

As can be seen in Figs. 8÷11, the interstorey displacement time histories of the numerical simulation fit fairly well the corresponding experimental time histories. In particular the fitting is very good for the shape of the curves relevant to all stories, while some differences can be noted in the peak values. A somewhat better agreement can be noticed at the upper levels with respect to the first one, although some differences on the peak values can be still observed at the 2nd and 3rd levels. The time history scattergram relevant to the first storey is presented in Fig. 12. It confirms the consistency of the numerical and experimental results in terms of shape (distance from the regression line) and of magnitude (deviation from the 45° line). In Figs 13-16 there are represented the power spectral density of the time histories shown in Figs. 8-11. In general a good agreement can be noticed for the components above 2 Hz in all the diagrams. The lack of some low frequency waves in the numerical analyses, especially at the first storey, is probably due to some local effects at the base column-load cell interface. Some difference, which increases with the order of storey, can also be noted for the components at 6 Hz, corresponding to the initial fundamental period of the structure, which is however of minor importance for their small amplitude.

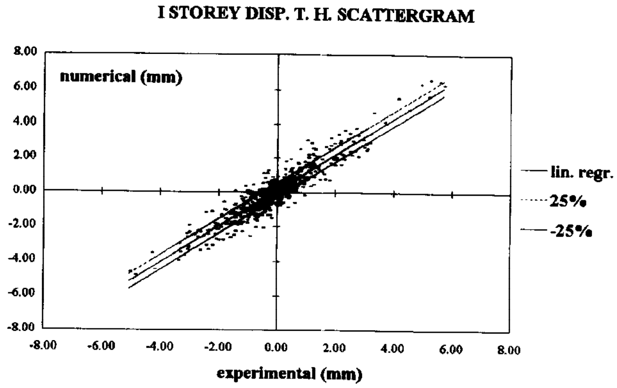

Fig. 12 Scattergram of the I storey experimental and numerical displacements.

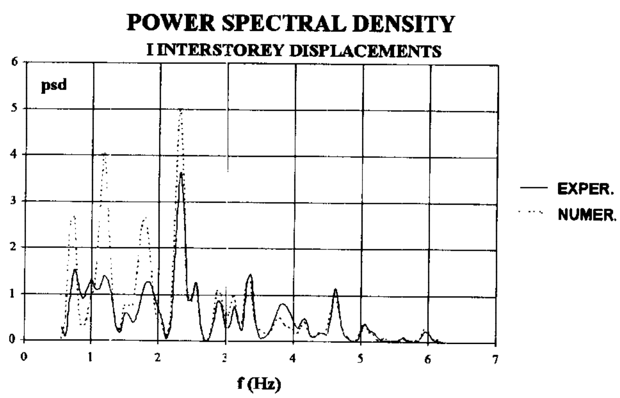

Fig. 13 Power spectral density. Experimental and numerical I interstorey displacements

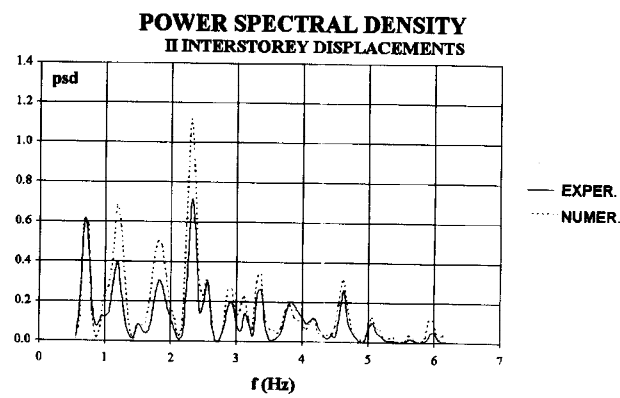

Fig. 14 Power spectral density. Experimental and numerical II interstorey displacements

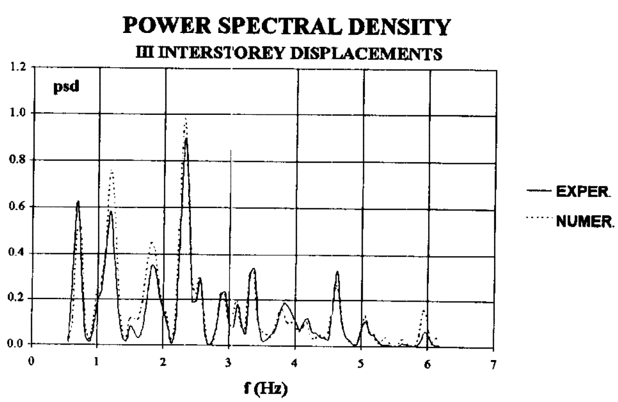

Fig. 15 Power spectral density. Experimental and numerical III interstorey displacements

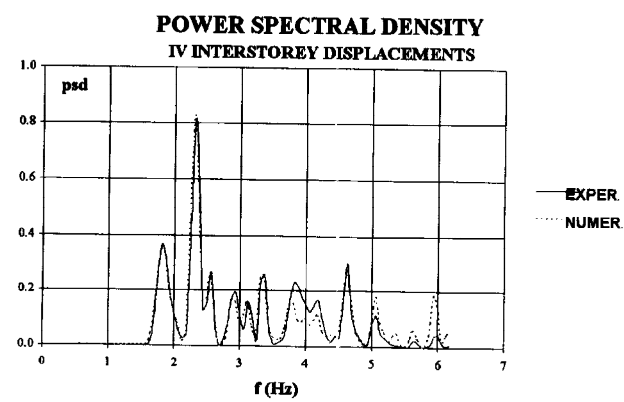

Fig. 16 Power spectral density. Experimental and numerical IV interstorey displacements

Finally Figs. 17÷20 present the complex transfer functions between the numerical interstorey displacements and the corresponding experimental values. At all the storeys the transfer function remains reasonably close to one up to 5 Hz, while some increase can be noticed near 6 Hz, especially for the upper storeys.

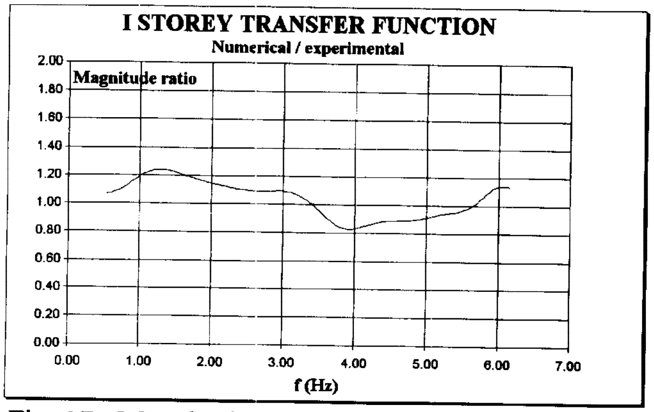

Fig. 17 Magnitude ratio of the complex transfer function. I storey displacements.

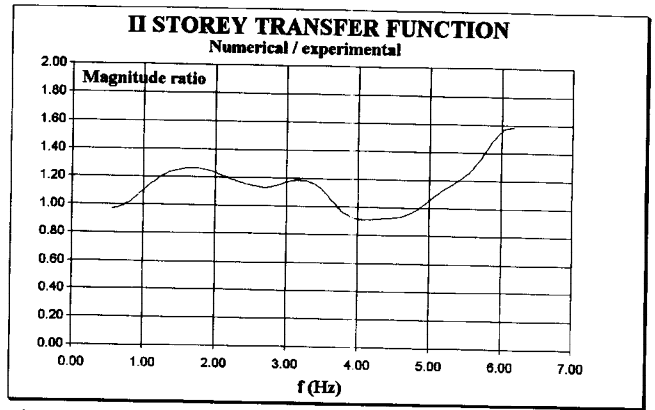

Fig. 18 Magnitude ratio of the complex transfer function. II storey displacements.

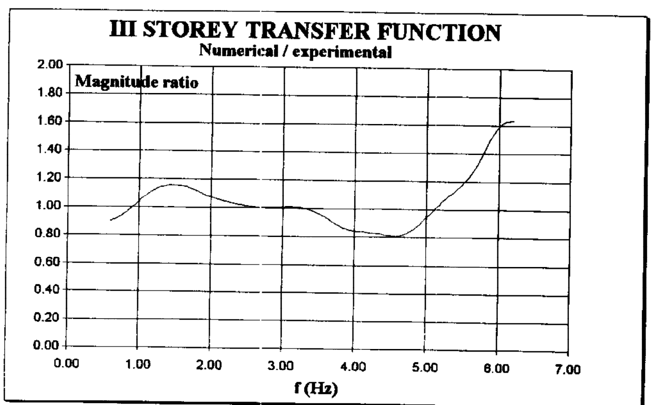

Fig. 19 Magnitude ratio of the complex transfer function. III storey displacements.

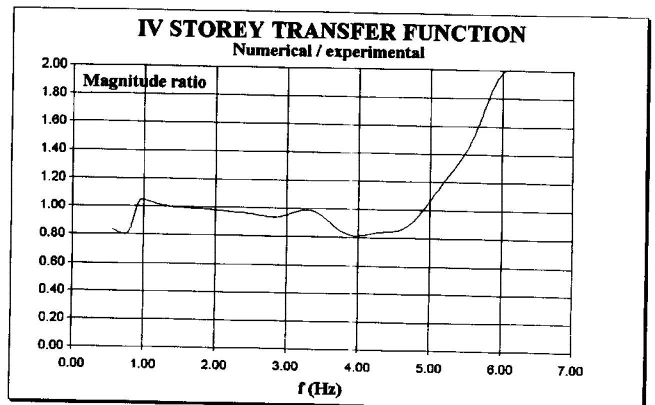

Fig. 20 Magnitude ratio of the complex transfer function. IV storey displacements.

CONCLUSIONS

The comparative analyses of the experimental and numerical results indicate that an accurate mathematical modelling of the r.c. structure and of the energy dissipating bracing elements permit to simulate very closely the experimental behaviour of a real structure. The mathematical model must have the capability of taking into account the effects of cracking in concrete and of reproducing faithfully the actual non linear behaviour of the bracing elements.

The interstorey displacements have been compared and a good fitting has been found in terms of shape of both their time histories and of their power spectral density functions. Some differences have been detected for what concerns their peak values. Although a direct comparison of stresses has not been made, a good agreement is expected since the relative interstorey displacements are strictly related to the main stresses induced in the structural elements.

The use of numerical models like the one herein shown presents a great potential for the understanding of the non linear behaviour of complicated structures, like that examined in this paper, in which structural members made of different materials are combined together. Their use can overpass some limits of the experimental tests, such as the control of local behaviour (stresses and strain), the specimen interaction with the earthquake simulator, the scale effects and, which is most important, the great amount of money and time needed to carry out shaking table tests. They are therefore a precious tool for the development and validation of new technologies in structural retrofitting, such as the one based on energy dissipation presented here.

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