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THEORY ED EXPERIMENTATION ON PASSIVE CONTROL OF ADJACENT STRUCTURES

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SUMMARY

The introduction of passive dissipative devices between two adjacent structures has been shown to be a viable method to protect both structures from seismic excitation. In this paper a shaking table experimentation is presented, which refers to a coupled structures configuration, that might be typically used in civil and industrial applications. The test structure is a 1:5 scaled coupled frames model. The first structure is a 240cm high, 4-story model with a total mass of about 500 kg; the second one is a 120cm high, 2-story model with a total mass of about 250 kg. The adjacent structures are connected by using elasto-plastic dissipative devices, placed at the second floor of the two buildings. The model undergoes different earthquakes, (Northridge, Kobe, El Centro and Hachinohe), of different intensity. The control performances of the passive devices are compared with the uncoupled and rigidly connected structures cases. The experimental data have been also used to identify and validate a numerical model of both the structures and the connecting devices.

INTRODUCTION

In the field of civil and structural engineering applications, passive and semi-active vibration control techniques have emerged as simple and reliable, Housner [1]. In particular, passive control systems, based on seismic isolation and energy dissipation have already found several applications worldwide. Recently, also semi-active control systems, based on adjustable passive devices, have found some interesting real-scale application, Spencer [2], and have been used in large-scale experimental studies, e.g. Renzi [3].

A research activity is in progress at the Dept. of Structural and Geotechnical Engineering of the University of Rome "La Sapienza", Italy, on the application of passive and semi-active control techniques to both civil engineering structures and to critical components of industrial plants, Ciampi [4]. The research activity has been focalized both on theoretical aspects, such as optimal design and performances, Ciampi

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[5, 6, 7, 8, 9], De Angelis [10] and on experimental verifications by means of large-scale dynamic tests on shaking table, Ciampi [11, 12] Renzi [13].

The present paper refers to the case of two adjacent structures, connected by dissipative dampers, (instead of being, as usual, independent one from the other and separated by structural joints); this arrangement may lead to response reduction for both structures and has been also indicated as beneficial for preventing pounding damage. A similar connection may be used also between different parts of the same structure, as for the case of buildings which combine a moment resisting frame with a shear wall. The traditional way of providing the coupling would consist of rigid connections between shear wall and frame, at different floor levels; if dissipative connections are used, instead, again beneficial response reduction effects may be produced. In both cases it is essential that the adjacent structures have different dynamical properties, so that relative motions may develop during the response. While many theoretical studies are available in the literature on this topic, see e.g. De Angelis [10] Ciampi [7] Seto [14] Xu [15], only very few experimental studies are reported, see e.g. Soda [16], specially for multi-degrees-of-freedom systems.

The paper illustrates a shaking table experimentation on a coupled-structures configuration, which might be typically used in civil and industrial applications. The test structure is a 1:5 scaled coupled-structures model; the first structure, 240cm high, has 4 stories whereas the second structure, 120cm high, has 2 stories. These adjacent structures are connected by elasto-plastic dissipative devices, placed at the second floor of the two buildings. The tests have been performed at the Structural Dynamics and Vibration Control Laboratory of ENEA MAT-QUAL, at R.C. Casaccia, (Rome, Italy). In particular the paper reports the description of the experimental arrangement and a preliminary illustration of the test results; identification and numerical modeling of both the test structures and the control device are also discussed.

STRUCTURAL MODEL OF THE COUPLED STRUCTURES

The structural model has been designed on the basis of the characteristics of the shaking tables available at ENEA, see e.g. Renzi [17], in order to obtain a significant model, easy to transport and install, and adaptable for further future uses. In this first experimental phase, the control has been realized by means of passive dissipative connections. Further experimental developments, already in the design phase, will use also semi-active magnetorheological devices for the connection.

Physical model

The two steel structures represent a scaled model (length similitude ratio: $S_L = 5$) of two adjacent structures, having 4 and 2 floors respectively. Both structures have plan dimension of 60×60 cm and inter-story height of 60 cm, so that the first and the second structure are respectively 240 cm and 120 cm high. The vertical elements of both structures have been realized with commercial steel profiles L40×40×4 mm and all the connections are bolted. At every floor 4 steel blocks, of 19 kg mass each, have been installed in order to simulate the floor masses; the resulting total masses of the two structures are about 500 kg and 250 kg respectively. In order to avoid undesirable motions, both models have been properly braced in the direction orthogonal to the applied unidirectional seismic motion. Figure 1 gives a picture of the test structures installed on the ENEA shaking table and a design drawing of the structures in the plane of the motion.

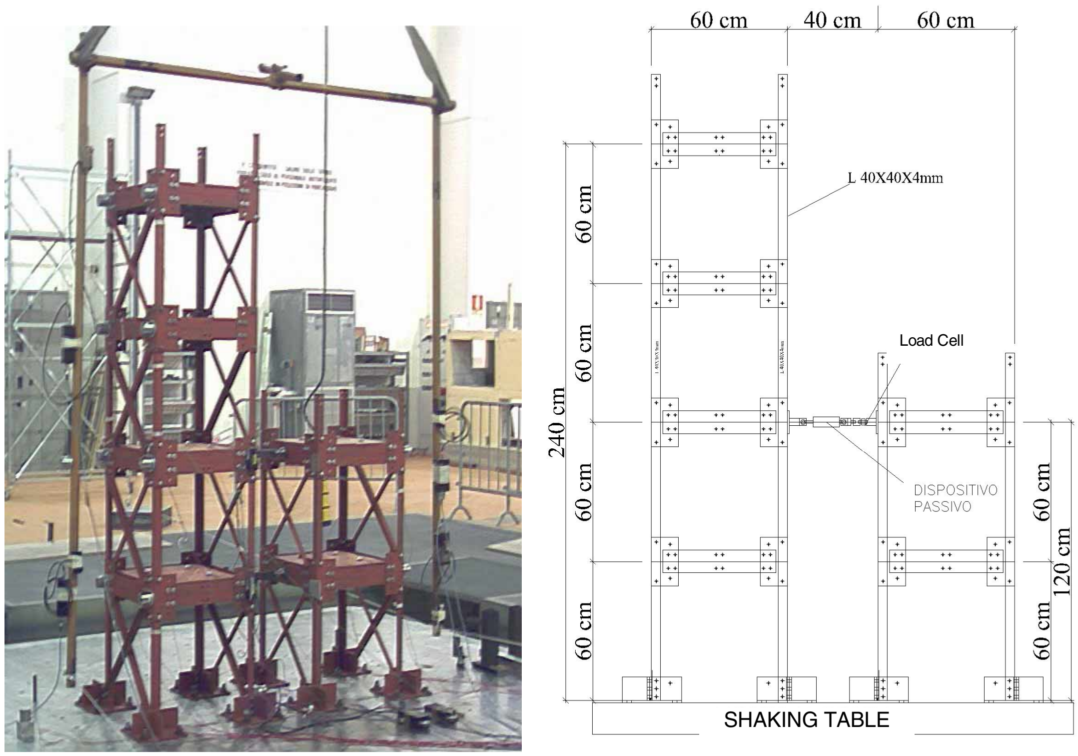


Figure 1. Structural Model

Preliminary characterization impact tests performed on the structural model have allowed the evaluation of the fundamental vibration periods of the uncoupled structures: $T_1 = 0.12$ s for the first structure and $T_2 = 0.06$ s for the second structure, in good accordance with the design assumptions. Note that, due to the scale reduction ratio, ($S_L = 5$), being unitary the similitude ratio for accelerations and stresses, the time similitude ratio results: $\tau = S_T = S_L^{-0.5} = 0.447$; this means that the structural model represents two real structures with fundamental periods $T_1^{(R)} = T_1 / \tau = 0.27$ s and $T_2^{(R)} = T_2 / \tau = 0.13$ s.

Design of dissipative connections

In the present experimentation, the two test structures have been connected by means of two steel elasto-plastic devices, placed between the second floors of the two structures (see Figure 1). In particular, E-shaped steel dissipators, (Fig.2), similar to devices already designed and tested for the seismic protection of framed structures, Ciampi [11, 12, 18], have been used.

Previous papers, De Angelis [10] Ciampi [7, 9], present design spectra for the optimal selection of the mechanical parameters of the connection control devices, obtained for simple two-degree-of-freedom systems. These spectra have been obtained by maximizing an energy-based index (*EDI*, Energy Dissipation Index), which represents a measure of the ratio between the energy dissipated in the connection and the total input energy. The design spectra have been used as a starting point for the optimization procedure which has led to the selection of the mechanical parameters of the elasto-plastic connection. In particular, for a fixed design input amplitude, (PGA = 1.0g), and for the considered structural model, the procedure has led to a design plastic force of the connection equal to 0.8 kN.

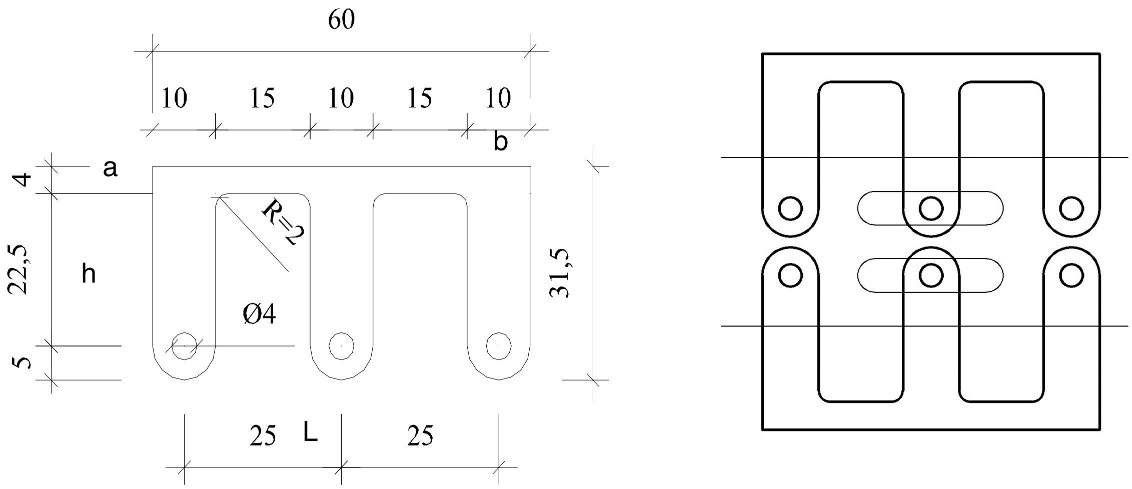


Figure 2. The elasto-plastic steel device, (measures in mm)

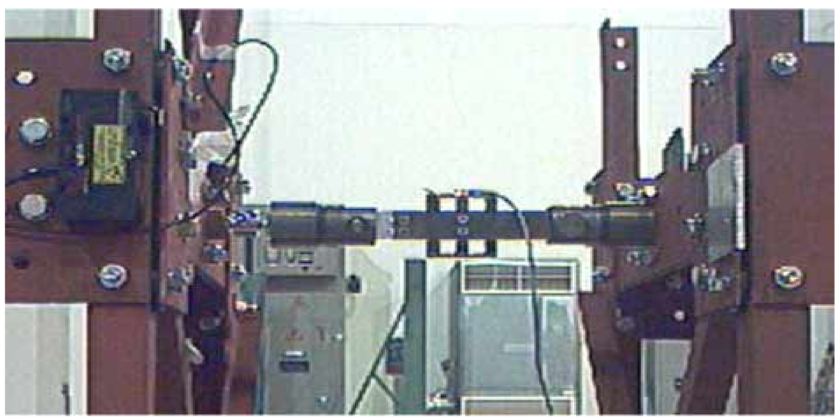


Figure 3. Detail of the dissipative connection installed between the structural models

The actual device used in the experimentation is made of two E-shaped dissipators, arranged in parallel. In order to avoid unstable lateral behaviour, they have been inserted in a special "package", with functions of guide. The adopted dimensions of the dissipators are reported in Figure 2, the thickness s of the dissipators is 3.5 mm, the yielding stress of the used steel is σ_a = 340 N/mm^2 and the Young modulus E_a = 206000 N/mm^2. Under the assumption of a perfectly elasto-plastic steel behaviour, the following values are found for the initial yielding force F_y, the full plasticization force, (corresponding to complete plasticization of the section), F_u, the first yielding displacement d_y and the corresponding elastic stiffness k_d , of the device:

$$F_y = 2\sigma_a \frac{sa^2}{3H} = 0.52 \text{ kN}, \qquad F_u = 1.5\, F_y = 0.78 \text{ kN} \qquad (1, 2)$$

$$d_y = \sigma_a \frac{2a^2 H}{E_a}\left(\frac{H}{b^3} + \frac{L}{a^3}\right) = 0.54 \text{ mm}, \qquad k_d = F_y / d_y = 0.965 \text{ kN/mm}, \qquad (3, 4)$$

where $H = (h + a/2)$. Finally Figure 3 shows a detail of the installation of the dissipative connection.

SHAKING TABLE EXPERIMENTATION

The tests have been performed on the largest shaking table, (4×4 m), available at the ENEA Laboratory. This has the following principal nominal characteristics: frequency range from 0 to 50 Hz, peak acceleration 3g, maximum displacement ±125 mm; maximum velocity ±0.5 m/s and maximum overturning moment about 300 kNm (corresponding to 3g PGA for 10 tons of rigid mass at 1m height).

The structural configurations tested have been the following:

- NC: no connection between the structures;
- RC: rigid connection at the second floor;
- PC: dissipative connection at the second floor.

For each structural configuration, dynamic characterization tests and seismic tests have been performed. In the characterization tests a mono-directional random signal corresponding to a white noise (0 ÷ 100 Hz frequency range) at various intensities, has been used. The seismic actions are the natural records of El Centro, Northridge, Hachinohe and Kobe. These well known records are widely used in the literature on structural control; in particular they have been proposed as inputs for the benchmark problem on structural control of seismically excited buildings, Spencer [19]. With reference to the scale reduction of the model, also the accelerograms have been scaled, using the appropriate time scale factor $\tau = S_L^{-0.5} = 0.447$. For each structural configuration and for each input, the mono-directional seismic tests have been repeated for increasing intensity, starting from very low nominal PGA (e.g. 0.1g) up to the attainment of a defined limit state for the structures or the device.

In order to measure the structural response, the test structures have been instrumented with the following transducers (see also Figure 4):

- n. 14 seismic accelerometers, located at the table (A0, AV) and on both sides of the different floors of the first (A1S_A, A1D_A, A2S_A, A2D_A, A3S_A, A3D_A, A4S_A, A4D_A) and second (A1S_B, A1D_B, A2S_B, A2D_B) structure;
- n. 4 laser displacement transducers, to measure the absolute displacements of the table (SL0), of the 4th and 1st floors of the first structure (SL4_A, SL1_A) and of the 2nd floor of the second structure (SL2_B);
- n. 2 laser displacement transducers, to measure the relative displacements between the two first floors of the two structures (SL1_AB and SL2_AB);
- n.2 load cells, to measure the forces in the connections (LCD1 and LCD2);
- n.1 strain gauges, to measure the deformation of the dissipative devices (SG2).

The data acquisition has been made by using a *MTS 469D* system, with sampling rate of 200 Hz. The absolute displacement measures have been obtained by using a suspended reference frame, isolated from the vibrations induced by the shaking table motion (see Figure 1).

In the various configurations, over 200 characterization and seismic tests have been performed; in particular, for the Passively Controlled configuration, the seismic tests have reached very large PGA levels, up to 2g. Thus a large number of results has been obtained and recorded. A preliminary, but significant, selection of these experimental results is here reported and commented, in order to show the performances of the proposed control approach.

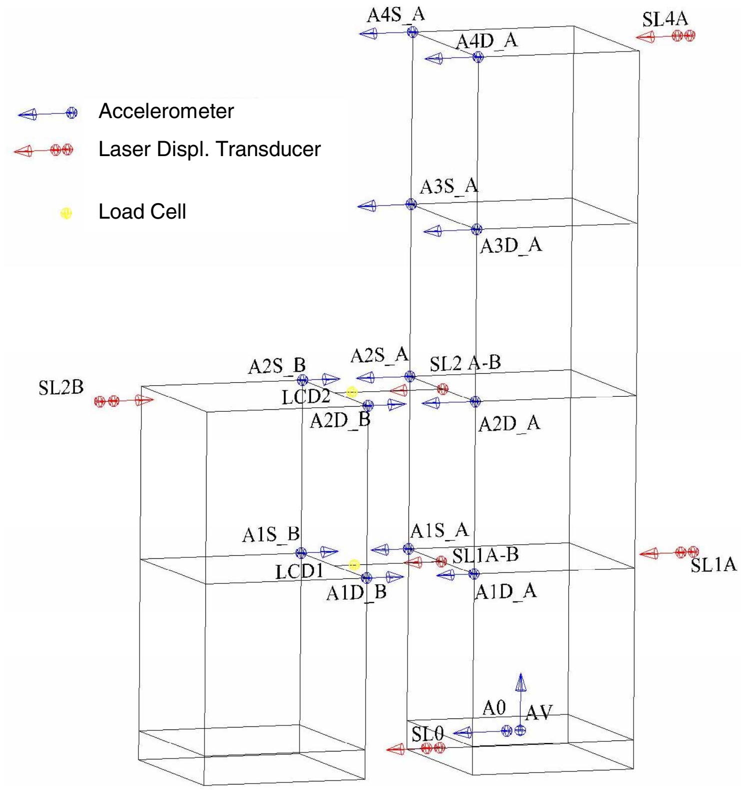


Figure 4. Transducers location

EXPERIMENTAL RESULTS

For the non-connected structures (NC configuration), Figures 5 report the maximum values of the total base shear of both structures, versus the PGA values actually measured at the table, for two representative inputs, (El Centro and Hachinohe). The figures clearly show the linear behaviour of the structures for increasing PGA, in a properly limited range. Similar linear dependence is also obtained by observing other structural responses, in particular floor displacements. This permits use of linear extrapolation for a correct comparison between results referring to different configurations, even when the actually measured PGA, for the different cases, are not exactly the same.

Figures 6 present a comparison, in terms of maximum total base shear for both structures, of the results obtained for the cases of no connection (NC configuration), dissipative connection (PC configuration) and rigid connection (RC configuration), for different seismic inputs and PGA intensities. The figures show the substantial response reductions which may be obtained by using the dissipative connection with respect to both the non-connected and the rigidly connected cases. These reductions, which depend on the PGA intensity and on the used accelerogram, are, e.g with respect to the NC case, of about 10 ÷ 20% for the first structure and of about 20 ÷ 40 % for the second one, which has the major benefits from the control.

Similar results have been obtained also in terms of displacements, Figures 7, where the maximum top displacements of both structures are shown. The effectiveness of the dissipative connection depends even more significantly on the seismic input. In fact, for the Hachinohe seismic record, characterized by a long period pulse component, the top displacement of the first structure shows no reduction with respect to the non-controlled case.

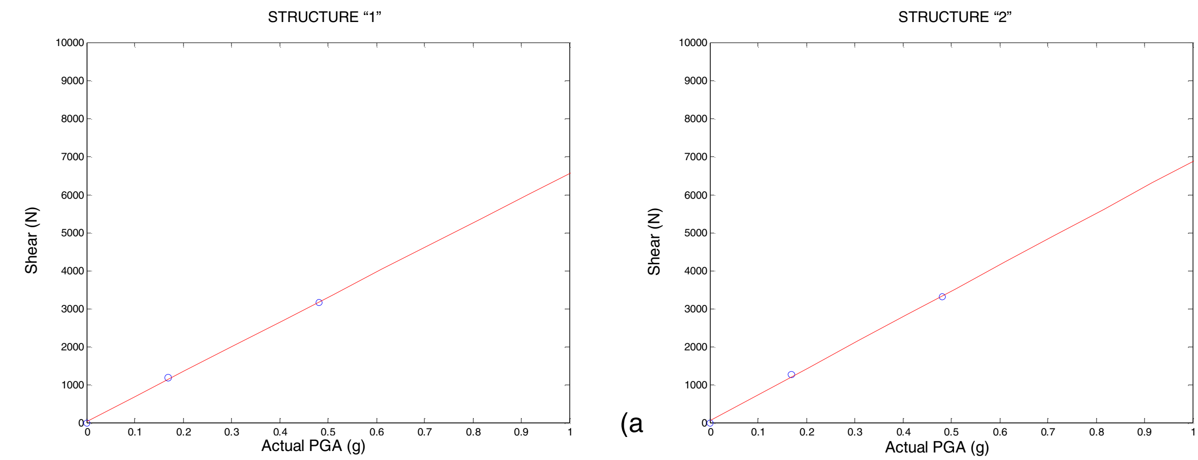


Figure 5a. Maximum base shear *vs* actual PGA (NC configuration). (El Centro)

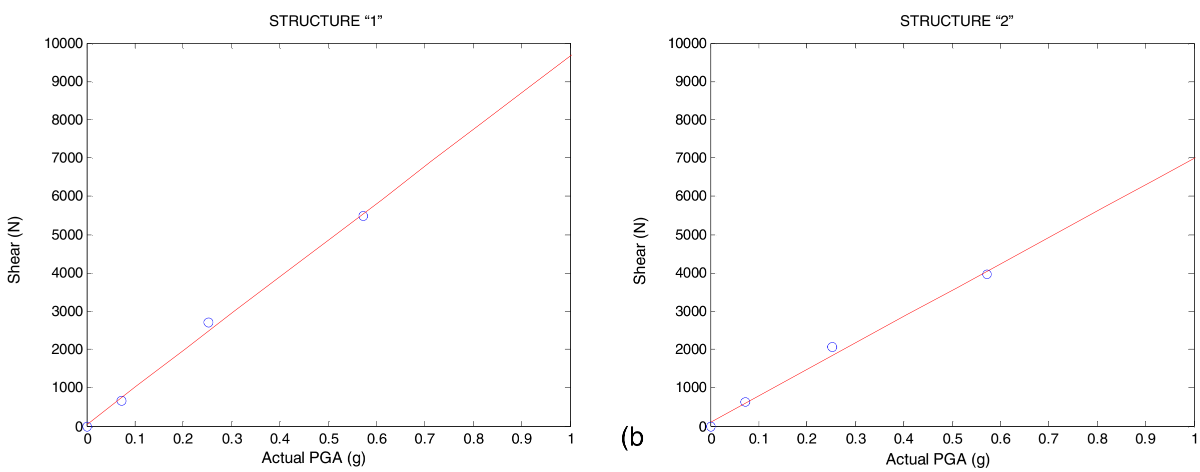
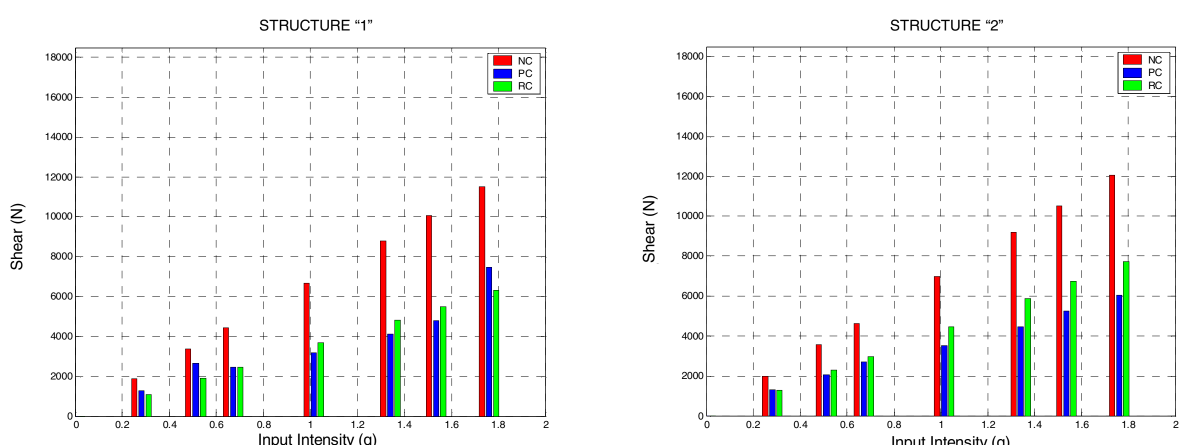
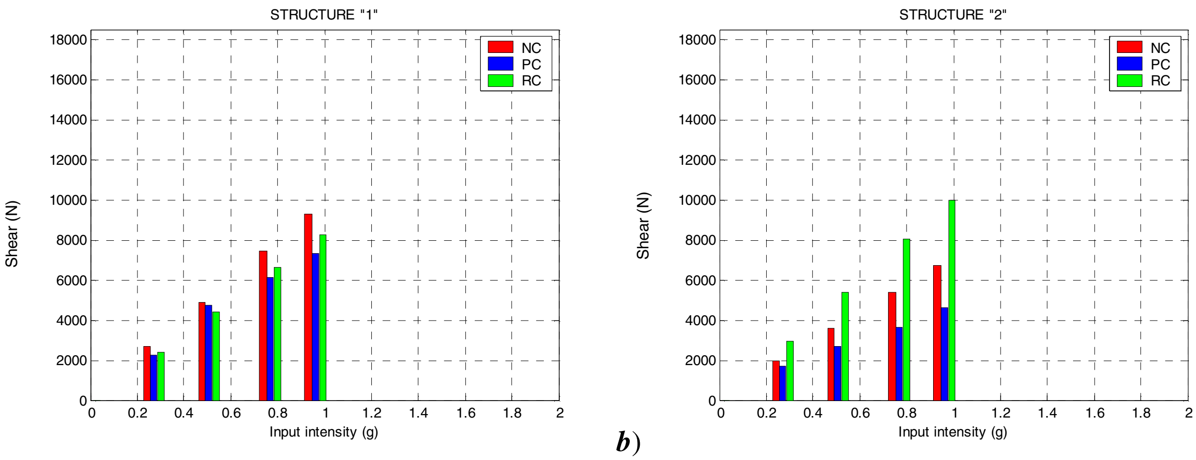


Figure 5b. Maximum base shear *vs* actual PGA (NC configuration). (Hachinohe)



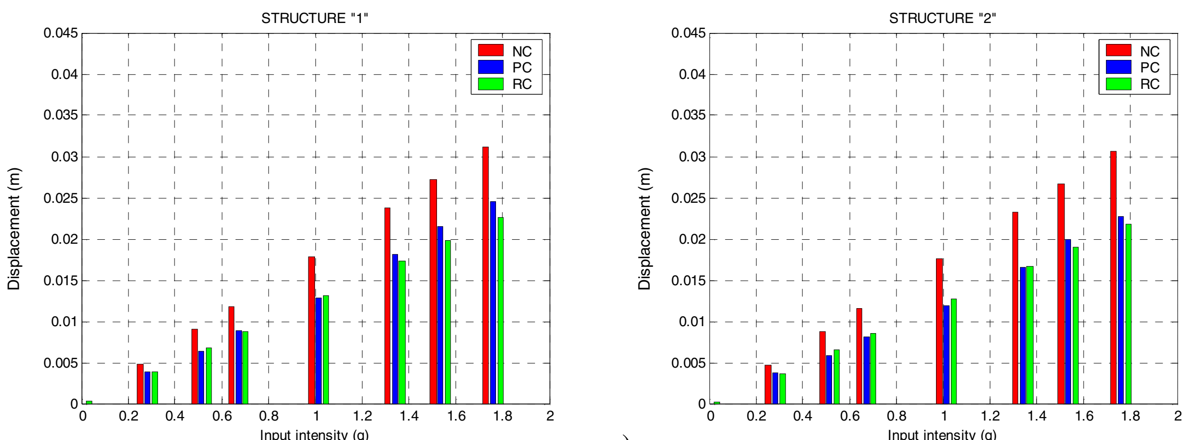
a)

Figure 6a. Maximum total base shear *vs* actual PGA. (El Centro)



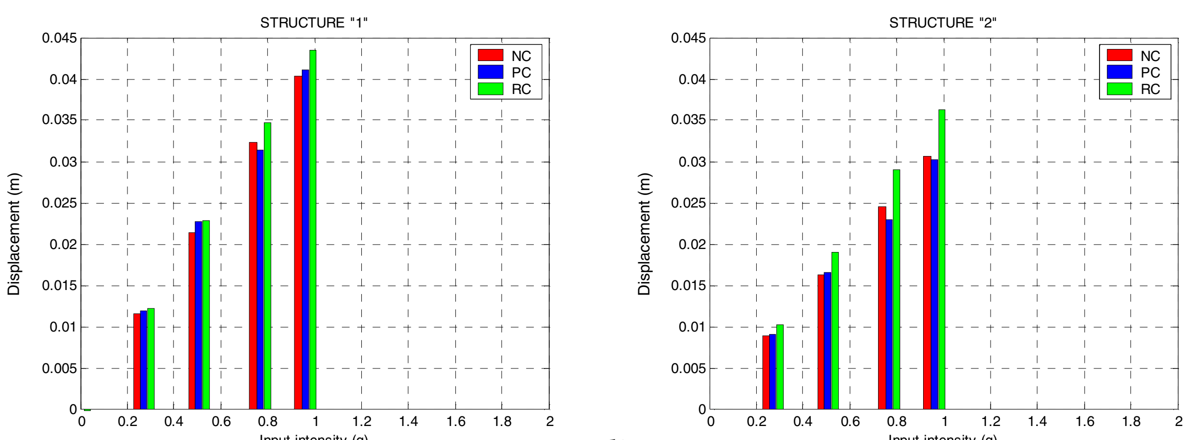
b)

Figure 6b. Maximum total base shear *vs* actual PGA. (Hachinohe)



a)

Figure 7a. Maximum top displacements *vs* actual PGA. (El Centro)



b)

Figure 7b Maximum top displacements *vs* actual PGA. (Hachinohe)

Table 1. Comparison between PC, RC and NC configurations. (El Centro)

Actual PGA / g	0.28		0.50		0.66		0.99		1.31		1.50		1.73	
Configuration	PC/NC	PC/RC	PC/NC	PC/RC	PC/NC	PC/RC	PC/NC	PC/RC	PC/NC	PC/RC	PC/NC	PC/RC	PC/NC	PC/RC
Total Base Shear Structure “1”	0.68	1.17	0.78	1.38	0.56	1.00	0.47	0.86	0.47	0.85	0.47	0.86	0.65	1.19
Total Base Shear Structure “2”	0.66	1.00	0.58	0.90	0.58	0.90	0.51	0.79	0.48	0.76	0.50	0.78	0.50	0.78
Top Displacement Structure “1”	0.78	0.98	0.71	0.94	0.75	1.01	0.72	0.98	0.76	1.05	0.79	1.09	0.79	1.09
Top Displacement Structure “2”	0.79	1.02	0.67	0.91	0.70	0.96	0.68	0.95	0.71	0.99	0.74	1.04	0.74	1.04
Ductility	0.76		1.79		2.54		3.50		4.94		5.88		6.91	

Table 2. Comparison between PC, RC and NC configurations. (Hachinohe)

Actual PGA / g	0.26		0.49		0.75		0.94	
Configuration	PC/NC	PC/RC	PC/NC	PC/RC	PC/NC	PC/RC	PC/NC	PC/RC
Total Base Shear Structure 1	0.84	0.92	0.97	1.08	0.82	0.93	0.79	0.89
Total Base Shear Structure 2	0.85	0.57	0.74	0.50	0.68	0.45	0.69	0.46
Top Displacement Structure 1	1.03	0.98	1.06	0.99	0.97	0.90	1.02	0.94
Top Displacement Structure 2	1.00	0.88	1.01	0.87	0.94	0.79	0.99	0.83
Ductility	0.63		2.15		3.57		5.07	

Table 3. Comparison between PC, RC and NC configurations. (Kobe)

Actual PGA / g	0.28		0.45		0.72		0.96		1.16	
Configuration	PC/NC	PC/RC	PC/NC	PC/RC	PC/NC	PC/RC	PC/NC	PC/RC	PC/NC	PC/RC
Total Base Shear Structure “1”	0.66	1.20	0.62	1.16	0.60	1.13	0.64	1.23	0.73	1.40
Total Base Shear Structure “2”	0.97	0.67	1.05	0.73	1.02	0.72	1.05	0.74	0.90	0.64
Top Displacement Structure “1”	0.86	1.02	0.88	1.05	0.87	1.03	0.89	1.06	0.92	1.10
Top Displacement Structure “2”	0.82	0.93	0.84	0.95	0.82	0.93	0.83	0.94	0.83	0.94
Ductility	0.60		1.45		3.22		5.36		7.25	

Table 4. Comparison between PC, RC and NC configurations. (Northridge)

Actual PGA / g	0.32		0.46		0.72		0.94		1.4		1.54		1.70	
Configuration	PC/NC	PC/RC	PC/NC	PC/RC	PC/NC	PC/RC	PC/NC	PC/RC	PC/NC	PC/RC	PC/NC	PC/RC	PC/NC	PC/RC
Total Base Shear Structure “1”	0.98	1.51	0.93	1.48	0.82	1.33	0.82	1.36	0.87	1.46	0.93	1.56	1.11	1.87
Total Base Shear Structure “2”	0.71	0.84	0.54	0.64	0.65	0.77	0.61	0.74	0.59	0.72	0.66	0.81	0.88	1.07
Top Displacement Structure “1”	0.84	1.10	0.82	1.10	0.87	1.19	0.92	1.28	0.97	1.37	1.02	1.43	1.06	1.49
Top Displacement Structure “2”	0.79	1.03	0.79	1.06	0.80	1.07	0.81	1.10	0.82	1.12	0.85	1.16	0.88	1.20
Ductility	0.91		0.80		1.85		3.14		5.66		6.50		6.94	

Tables 1, 2, 3 and 4 present, for each seismic input, and for the different values of the measured PGA, a comparison between the PC configuration response and the corresponding response for the NC and RC cases, by reporting the ratios of the significant structural response values, (max base shears and top displacements), obtained in the PC case, with respect to the ones obtained for the NC and RC configurations. The tables present also the maximum ductility of the connection device, obtained during the PC tests, a response parameter which gives a measure of the energy dissipation activated: it may be noted that the best performances of the control system, (in terms of response reductions), are observed, for all the seismic inputs, when the maximum reached ductility is about 3 ÷ 4; this occurs, for the different earthquakes, at PGA which are in the range 0.7-1.0 g, in good accordance with the design assumptions used for the device.

Finally, Figures 8 and 9 present, for El Centro PGA=0.99g and Hachinohe PGA=0.75g, (max ductility equal to 3.0 and 3.6 respectively), an example of the force-displacement cycles of the connection device (PC configuration), and the details of the floor shear distribution in the two structures for NC, RC and PC configurations. It may be observed that the response of the control device, in spite of a certain “slip” at zero force, (of about ± 0.5 mm), due to realization imprecision, shows a good and stable hysteretic behaviour with satisfactory energy dissipation capability. The shear distributions, along the height of both structures, show the uniform reduction obtained for the PC configuration, with respect to non-connected and rigidly-connected configurations.

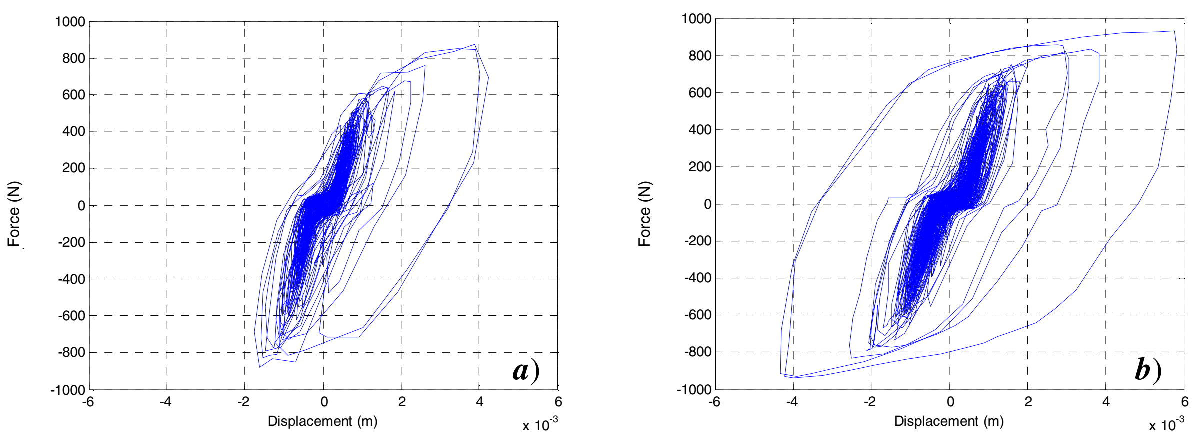


Figure 8. Force-displacement cycles: *a*) El Centro, PGA = 0.99g; *b*) Hachinohe, PGA=0.75g

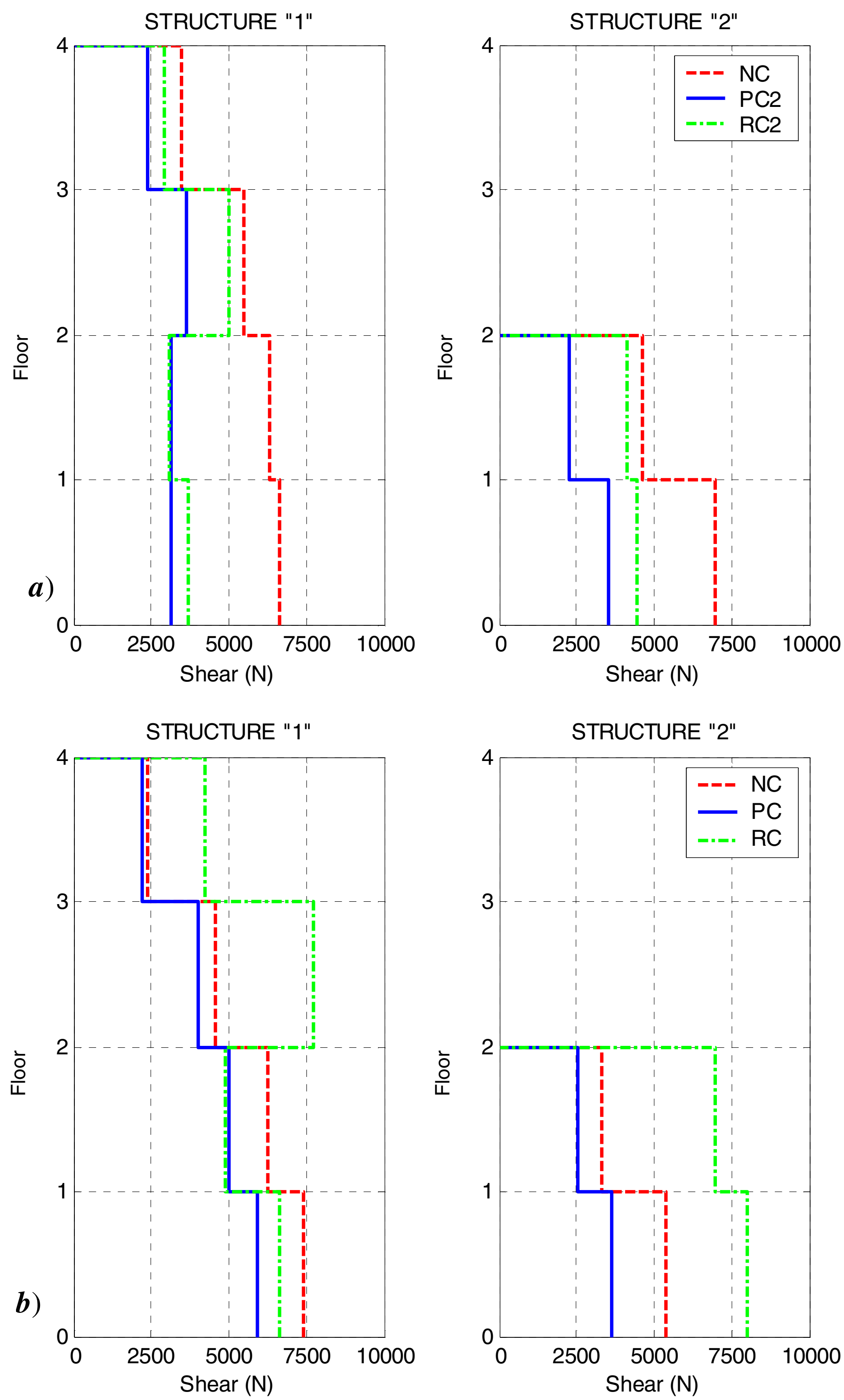


Figure 9. Shear force distribution. ***a*****) El Centro, PGA=0.99g,** ***b*****) Hachinohe, PGA=0.75g**

MODELING THE EXPERIMENTAL RESULTS

In order to formulate a mathematical model of the structures, useful in view of a full understanding of the observed dynamical behaviour and for response prediction purposes, an identification procedure has been set up, based on the recorded experimental data.

By considering a simplified plane model for the adjacent structures, in which the masses are lumped at the floors, and by assuming as degrees-of-freedom the horizontal floor displacements, the model of the first structure has 4 dof and the model of the second structure has 2 dof. By using ERA/OKID algorithm, De Angelis [20], the frequencies and the modal damping factors have been identified, and are shown in Table 5.

Table 5. Identified frequencies and damping factors in NC configuration. ERA/OKID algorithm

	STRUCTURE "1"				STRUCTURE "2"	
Frequencies (Hz)	$f_{1,1}$	$f_{2,1}$	$f_{3,1}$	$f_{4,1}$	$f_{1,2}$	$f_{2,2}$
	7.3	24.6	41.1	53.4	16.1	45.7
Damping factors (%)	$\xi_{1,1}$	$\xi_{2,1}$	$\xi_{3,1}$	$\xi_{4,1}$	$\xi_{1,2}$	$\xi_{2,2}$
	3.89	1.60	1.44	1.99	1.23	0.80

By modifying, for the seismic case, an identification procedure set up for the case of forces directly applied to the degrees-of-freedom, De Angelis [20], and forcing the mass matrices to be diagonal, the following mass and stiffness matrices for the non-controlled structures have been also identified:

$$\boldsymbol{M}_1 = 125\begin{bmatrix} 1 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 \\ 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 1 \end{bmatrix} \text{ (kg)} \qquad \boldsymbol{K}_1 = \begin{bmatrix} 7.24 & -3.54 & 0.23 & 0.16 \\ -3.54 & 6.12 & -3.45 & 0.15 \\ 0.23 & -3.45 & 6.90 & -3.51 \\ 0.16 & 0.15 & -3.51 & 3.13 \end{bmatrix} \times 10^6 \text{ (N/m)} \tag{4}$$

$$\boldsymbol{M}_2 = 125\begin{bmatrix} 1 & 0 \\ 0 & 1 \end{bmatrix} \text{ (kg)} \qquad \boldsymbol{K}_2 = \begin{bmatrix} 8.10 & -3.82 \\ -3.82 & 3.42 \end{bmatrix} \times 10^6 \text{ (N/m)} \tag{5}$$

Finally, by using a modified elastic-plastic Bouc-Wen model, and a mean square error minimization procedure, also the parameters of the connection device have been identified. Figure 10 shows, for the El Centro input, both experimental and numerical responses. Hysteretic cycles of the connection device and floor accelerations of both structures may be directly compared, demonstrating that the identified numerical model of the coupled structures satisfactorily reproduces the experimental data.

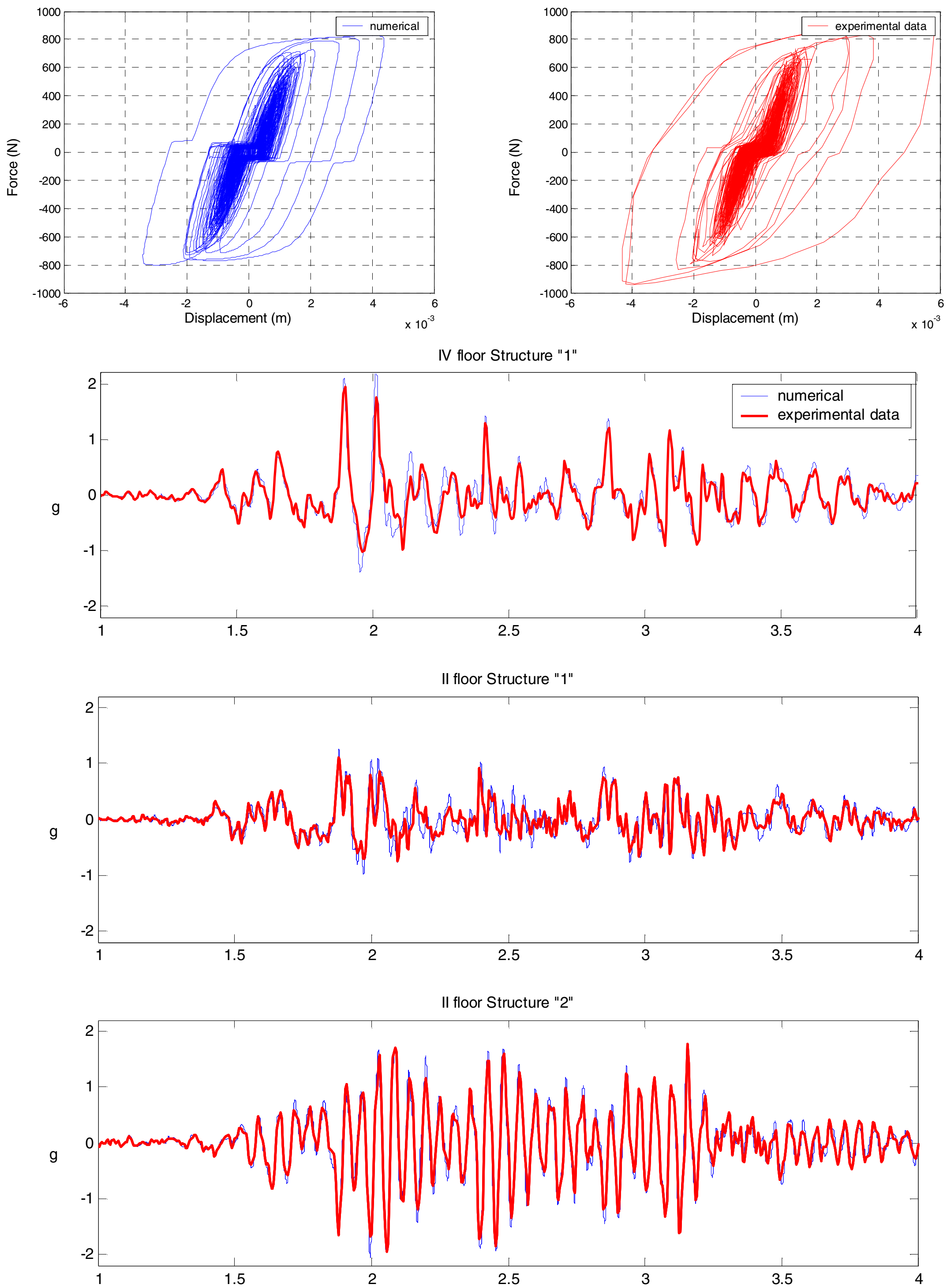


Figure 10. Analitical model *vs* experimental results. El Centro, PGA 0.99g

CONCLUSIONS

The paper approaches the problem of passive control of adjacent structure by means of dissipative elasto-plastic connections. In particular, it describes the design and realization of a shaking-table experimental study, carried out on a 1:5 scaled structural model of two (4-storey and 2-storey) adjacent structures, connected at the second floor level.

The analysis of the experimental results clearly shows the good performance of the proposed control approach which permits significant response reduction, up to 40%, for both structures.

Finally an identification process of both the structures and the connection device has been carried out, which has led to a reliable analytical model of the controlled structure; this is useful for interpreting the experimental data, for predicting the controlled response and for performing parametrical studies.

Work is in progress for studying the effectiveness of semi-active magneto-rheological connections between the structures.

ACKNOWLEDGEMENTS

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