Paper No. 1531. (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: EARTHQUAKE SIMULATOR TESTS

P. D'ANZI *, A. J. CREWE **, F. C. PONZO *

* DiSGG - Universita' della Basilicata - Potenza (IT)
** Dep. Civil Engineering - EERC University of Bristol (UK)

ABSTRACT

Two 1/4 scale 4-story 2-span r.c. frames were tested on the Bristol University Earthquake Simulator (ES). One of the two identical models, both designed for gravity loads only, was retrofitted by means of steel braces with energy absorbing devices in "upside-down V" configuration. Main goal of the paper is to compare the two test model behaviours in order to verify, by experimental results, the capability of the retrofitting design method sorted out by Braga and D'Anzi to improve seismic performances of "weak buildings". Most design parameters have been focused, analysed and verified, so that the proposed procedure appears to be robust and reliable.

KEYWORDS

Seismic passive control; retrofitting; energy absorbing device; braced frame; steel brace; shaking table; experimental test; scale modelling; yielding bracing system.

INTRODUCTION

Throughout the world there are many reinforced concrete framed structures that have only been designed for vertical loads and, at best, nominal horizontal loads. Many of these structures exist in seismic zones and their collapse is often the major cause of loss of life in earthquakes. Complete replacement of such structures with properly designed r.c. frames is impractical and so there is considerable interest in technically simple and economic ways of strengthening the existing structures. Yielding bracing system, in particular, produces two main advantages:

- increasing lateral stiffness of the r.c. frame affecting, in turn, the seismic forces experienced by the frame;
- getting the structure able to absorb a great amount of energy by the plastic deformation of mild steel devices.

Braga and D'Anzi (1994) developed a procedure that optimised the distribution of bracing stiffness and strength, while allowing for a limited amount of ductility in the r.c. frame itself. They showed analytically that the optimised configuration led to more uniform distributions of ductility demand over the height of the structure than was the case when constant bracing stiffnesses and strengths were used throughout the structure, as for example by Filiatrault and Cherry (1990). In addition, storey displacements were reduced by a factor of about 2.5 with evident decreasing of the overall damages to structural and non-structural elements. Experimental test were thereafter sorted out to corroborate the analytical studies through observing the performance of two carefully scaled model plane frames, with and without the bracing fitted respectively, when subjected to simulated earthquakes on a shaking table. This paper describes details of the performed experiments and the main achieved findings.

GENERAL TEST ARRANGEMENT

Earthquake simulator

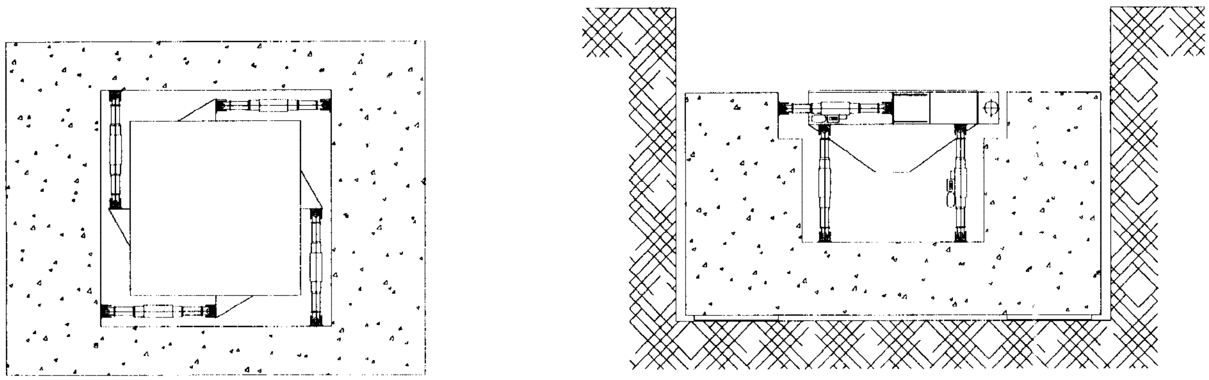

Fig. 1 Bristol University Earthquake Simulator

Dynamic tests were performed using the shaking table at Bristol University (Fig. 1). It is a 6 d.o.f., 3m x 3m steel platform with a maximum dead load capacity of 15 t. Platform motion is induced by eight 50 kN hydraulic actuators with a range of displacement of +/- 150 mm. They are connected in series to a 360 l/min hydraulic pump. The table can produce random and impulsive shakes in the range of 1 to 100 Hz, and reproduce natural and artificial earthquakes with a maximum of 1g peak platform acceleration with a 5 t dead load.

Model description

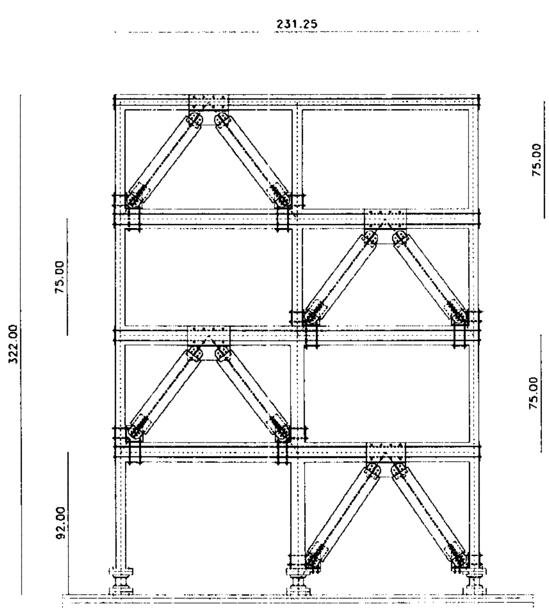

Fig. 2 Test model

One quarter scale r.c. models four-storey by two-bay were used (Fig. 2). Each storey height was .75 m giving an overall model height of 3.0 m. The spacing between the three columns was 1.25 m. The cross section of the external columns measured 62.5 mm wide (i.e. parallel to the plane of the frame) by 75 mm deep (i.e. perpendicular to the plane of the frame) for all storeys. The cross section of the central column in the lower two storeys measured 87.5 mm wide by 75 mm deep and in the upper two storeys 62.5 mm wide by 75 mm deep. The top level beams were 100 mm (vertical dimension) by 75 mm. Other storey beams measured 125 mm by 75 mm.
Each column base of the r.c. frame was attached to the shaking table platform via a heavy duty load cell which measured the 3-D axial, shear and bending moment reactions.
The frame was supported and constrained to move horizontally in its plane by a rigid 3-D steel braced frame bolted to the shaking table platform. Steel arms equipped with roller bearings were cantilevered from the support frame and placed around the beam ends to avoid any out of plane motion. The steel frame had the fundamental frequency in excess of 30 Hz, well above the test frequency of both the models.
According to the dimensional analysis to obtain the correct mass scaling, 168 lead-antimony alloy blocks, weighing in total 4.2 t, were added to each model. These blocks were clamped to the beams in such a way as to minimise any stiffening effects.

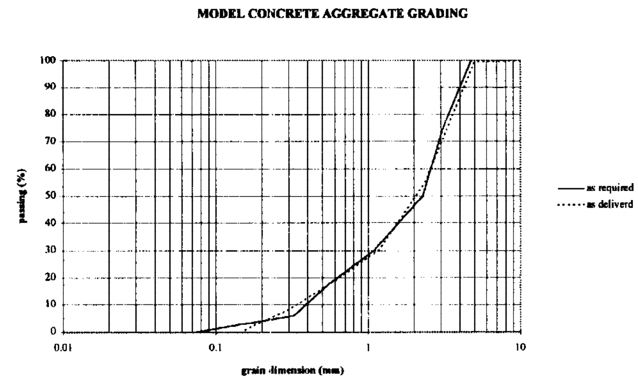

Fig. 3 Aggregate grading

Concrete. The r.c. frames were made from microconcrete. The aggregate was a washed, well graded sand and small gravel with grain size between 425 μ and 4.5 mm and a D_{50} grain size of 2.36 mm. The grading curve of the used aggregate is presented in Fig. 3. Rapid hardening Portland cement was used to minimise the time between casting and testing. The sand/cement ratio was 4.5 and water/cement ratio 0.6, giving an average compressive strength of around 28 N/mm^2.

Steel reinforcement. Threaded steel studding were used to accurately simulate the bond between concrete and reinforcement bars and, consequently, to achieve the best resemblance in cracking behaviour. Because of the strain hardening due to the mechanical treatment, the steel bars were heat treated to give them back an acceptable ductility. The steel was annealed to a yield strength of approximately 260 N/mm^2. The results achieved by heat treating are presented in Fig. 4. The reinforcement design is characterised by poor transversal steel and weak beam-column joint confinement, this being typical of the non-seismically designed prototype built in Italy in the 1950's and 1960's. Typical beam reinforcement details are shown in Fig. 5.

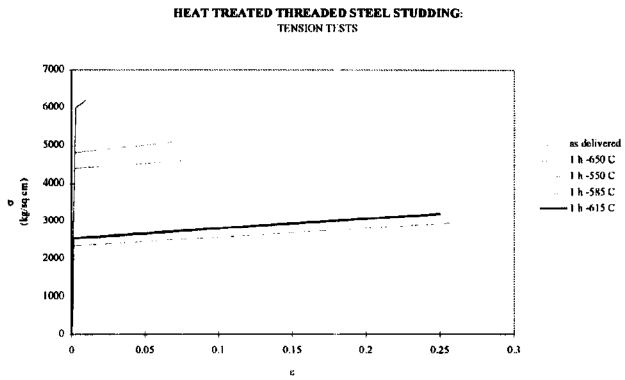

Fig. 4 Heat treating of steel studding

Steel bracing design and arrangement

As far as the retrofitted model is concerned, the characteristics of the bracing system were arranged following the stiffness-strength hybrid design method proposed by Braga and D'Anzi (1994). Brace stiffnesses and device slip-load distributions were assessed to reach the design method main tasks:

- allowing for a limited ductility demand in the original r.c. frame, reducing structural and non-structural damages;
- getting uniform distribution of ductility demand over the height of the structure avoiding any damage concentration;
- preventing any fragile local failure.

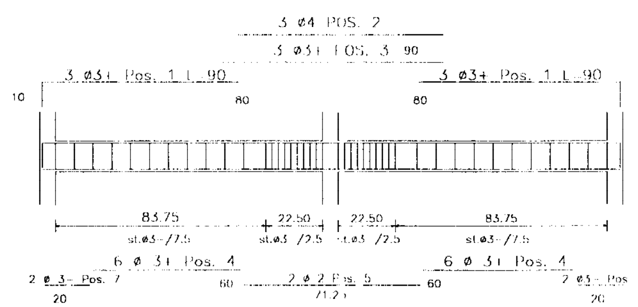

Fig. 5 Typical beam reinforcement

The braces, in "upside-down V" configuration, were staggered in alternate bays over the height of the frame (Fig. 1). They were attached at the column-floor beam corners by steel brackets, which were themselves clamped to the beams and columns by tensioned bars. A similar arrangement is used to attach the upper ends of the braces to the mid-point of the upper floor beam. The brackets are further bonded to the frame using epoxy resin grout.

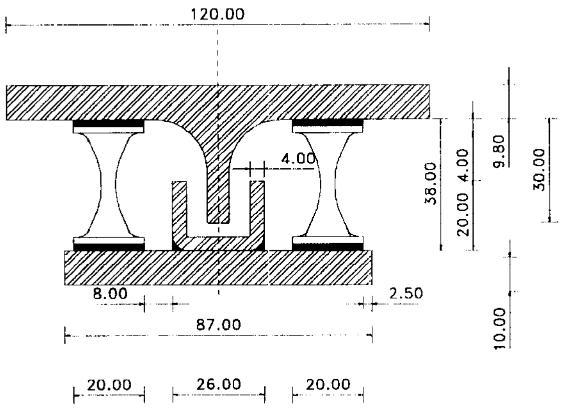

Fig. 6 Device cross section

The braces were fabricated from a pair of steel T-sections. At the upper end of the brace, the T-sections were joined by a rigid link. At the lower end, the sections were attached to a specially designed yielding link, showed in Figs. 6 and 7. Yielding was achieved through the bending of X shaped steel elements connecting the main T-sections to the corner mounting bracket.

A consequence of the bracing system during seismic loading is that the floor beams are placed into axial tension. Horizontal steel bars are therefore attached externally to the beams to provide sufficient compressive prestress to counteract the induced tensile forces. After being tensioned the horizontal prestressing chains are grouted to the beam using epoxy resin.

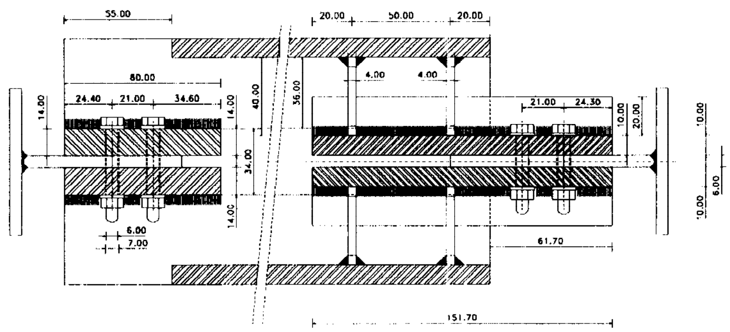

Fig. 7 Device longitudinal section

Construction sequence

The r.c. frames were cast on their side, off the shaking table, in a special, rigidly braced steel formwork. To vibrate the concrete during and after casting an eccentric mass engine was clamped to the formwork. When hardened, the frame was moved onto the shaking table with the main part of the formwork still attached, thereby imparting sufficient stiffness and strength to prevent the r.c. frame from bending or breaking during transit. When the r.c. frame was securely bolted to the shaking table and fastened to the steel support frame mentioned above, the steel formwork was removed. Next, for the braced model only, the mounting brackets for the diagonal braces were attached and grouted. Finally the lead blocks were placed and clamped to the beams, followed by inserting the diagonal braces.

TEST PROGRAMME AND RESULTS

Unbraced frame

Input motions. All the models tested on the earthquake simulator were shaken with one horizontal acceleration component in the plane of the frame. The table motion was tailored to envelope the Eurocode 8 elastic response spectrum for soft soil (type C). The acceleration time history, having an overall duration of 10 sec., was numerically generated in such a way to have a trapezoidal envelope with an ascending branch of 2.5 sec. at the beginning and a descending one of 2.5 sec. as well at the end. Figs. 8 and 9 show the strongest testing shake used for the unbraced model and its response spectrum. Before that the model was shaken several times with gradually incremented peak table acceleration ranging from 0.023g to 0.33g (Tab. 1). This was done partly to establish the onset of yielding in the model and partly to provide a degree of seismic ageing of the frame. As a matter of fact prototype buildings are likely to have experienced a number of small earthquakes prior to suffering a bigger one. As it can be easily seen from Figs. 8 and 9 the response amplification corresponding to the fundamental frequency of the model is close to one.

Tab. 1 - Peak platform acceleration of the performed shakes

shake n.	peak platf. accel. (a/g)
I	0.02344
II	0.05496
III	0.12817
IV	0.16257
V	0.22577
VI	0.27774
VII	0.32632

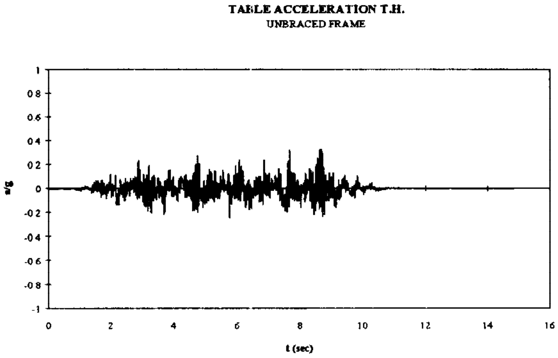

Fig. 8 Table acceleration time history

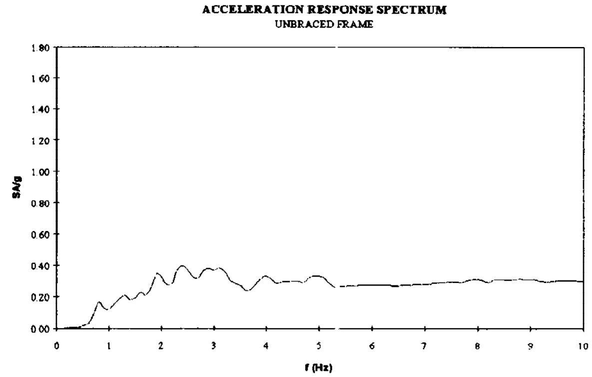

Fig. 9 Table acc. response spectrum (d=5%)

This effect is due to the great amount of energy absorbed by the model plasticization, occurred at that level of loading, from the table input which has not any real time prediction-correction motion control.

Specimen instrumentation. The test model was instrumented to measure frame sway accelerations and displacements, and column base reaction forces. Other instruments were used to measure platform accelerations and displacements. One more accelerometer was used to check the out of plane accelerations of the top level.

Test results. The unbraced model, subjected to seven earthquakes with incrementing peak acceleration (Tab. 1), showed initial damages for a peak table acceleration of 0.13g. The damage concentrated at the first floor, and four main cracks formed at both side of each first floor beam. The dominant frequency after this shake went down from the initial 3.125Hz to 3.0Hz. Tab. 2 shows the main frequency of the model after each performed seismic test.
The unbraced frame effectively failed at a peak table acceleration of 0.33g when the first storey beam collapsed and was caught by the safety rope connected to the steel support frame avoiding the overall crash of the model. Fig. 10 shows the envelope of maximum storey displacement during the last shake. Storey drift registered at the first floor was 2.30%.

Tab. 2 -Fundamental frequencies of the unbraced frame

shake n.	fund. frequency (Hz)
I	3.125
II	3.125
III	3.000
IV	2.750
V	2.375
VI	2.125
VII	1.625

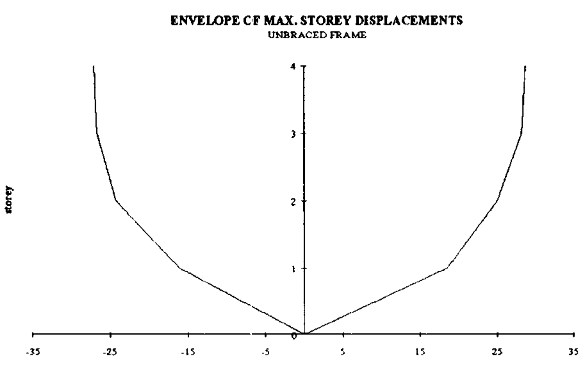

Fig. 10 Unbraced frame max. displacements

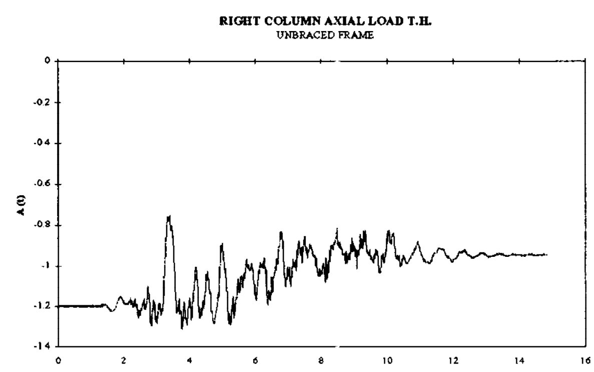

Fig. 11 Axial load t.h. at rx. column base

It was instead of only 1.10%, 0.40% and 0.07% for the second, third and fourth floor respectively. The difference in drift over the height of the model confirms the bent of non seismically designed structures for damaging at lower floors.
Fig. 11 shows the time history of the axial load measured at the base of the right column during the last test earthquake. Complete collapse occurred after around four seconds of shaking. The right end of the first floor beam came off from the joint and remained hung up to the support steel frame. Corresponding to the occurred failure the axial load at the base of the right column decreased.

Braced frame

Input motions. As far as the braced model is concerned, the overall test procedure was identical to that used for the unbraced model testing. The frame was shaken with ten earthquakes ranging in peak platform acceleration from 0.06g to 0.92g (Tab. 3).
Figs. 12 and 13 show the time history of the table acceleration for the last test shake and its response spectrum respectively. As for the unbraced model the response amplification corresponding to the fundamental frequency is close to one. In this instance, however, that is mainly due to the energy absorbing effect of the devices.

Specimen instrumentation. The braced model was instrumented in the same way of the unbraced one. Yielding of diagonal braces were measured by mounting four additional displacement transducers to measure the relative movement of the braces to their anchor brackets.

Tab. 3 - Peak platform acceleration of the performed shakes

shake n.	peak platf. accel. (a/g)
I	0.06384
II	0.20331
III	0.30975
IV	0.40564
V	0.43042
VI	0.47717
VII	0.56805
VIII	0.59900
IX	0.74750
X	0.91614

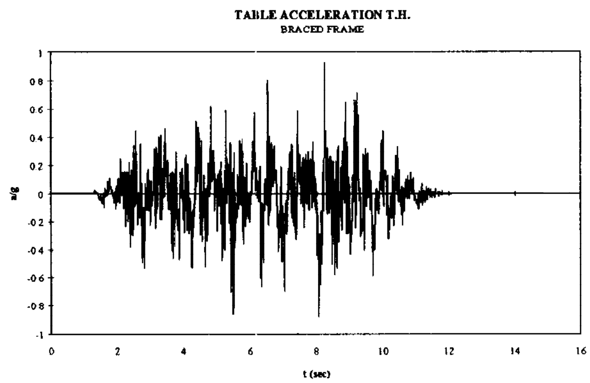

Fig. 12 Table Acceleration time history

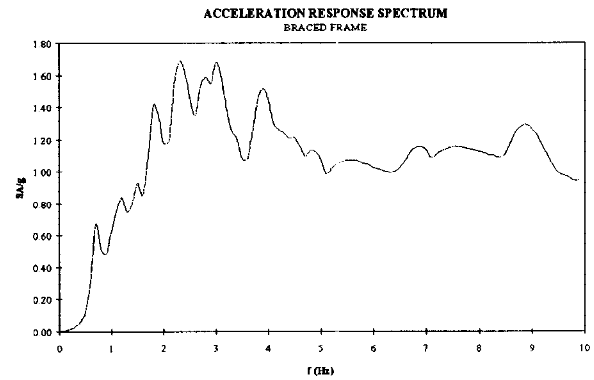

Fig. 13 Platform acc. response spec. (d=5%)

Test results. First yield of the model was observed during the shake with a peak table acceleration of 0.43g. Corresponding to that the fundamental frequency of the model moved from the initial 6.625Hz to 6.375Hz. As predicted by the used design method (Braga and D'Anzi, 1994), there wasn't any concentration of damage. As a matter of fact a "corrective" effect on the natural bent of the frame due to the brace design characteristics was registered.
Tab. 4 shows changes in the dominant frequency of the model ranging from 6.625Hz to 5.9375Hz. The decreasing of around 10.4% in fundamental frequency, that is a decreasing of around 19.7% in horizontal stiffness, confirmed the extremely limited damaging occurred in the model. In Tab. 5 storey drifts after the last earthquake are presented, and besides envelope of maximum storey displacements during that shake are showed in Fig. 14.

Tab. 4 - Fundamental frequencies of the braced frame

shake n.	fund. frequency (Hz)
I	6.625
II	6.625
III	6.625
IV	6.625
V	6.375
VI	6.3125
VII	6.125
VIII	5.9375
IX	5.9375
X	5.9375

Tab. 5 - Storey drifts registered after the X-shake

storey n.	drift
I	0.63%
II	0.28%
III	0.27%
IV	0.19%

By far the most remarkable observation was that after the last shake, with a peak table acceleration of 0.92g, only fine horizontal cracks were evident at the tops of the right column at ground and first storey level. The frame remained mainly intact and could have taken higher acceleration still, but it was decided to repeat the tests with the braces removed to achieve indication of the strength of the frame with the steel chains still mounted.

Results with braces removed. The frame was shaken, with the braces removed, with eight earthquakes ranging in peak acceleration from 0.038g to 0.45g. The frame effectively failed at around a peak table acceleration of 0.45g. Plastic hinges formed at the top and bottom of each column at second storey level forming a mechanism.

Figs. 15 and 16 show the acceleration response spectrum and the envelope of storey displacements for the last test shake.

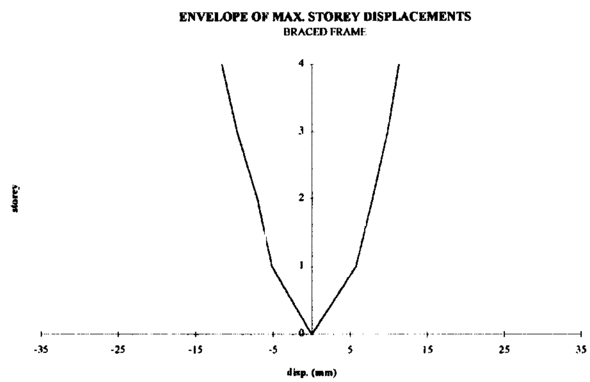

Fig. 14 Maximum storey displacements

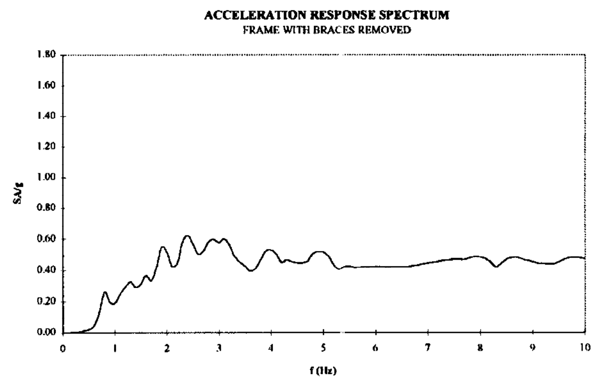

Fig. 15 Platform acc. response spectrum (d=5%)

CONCLUSIONS

The interpretation of the results of the performed tests showed a good agreement with all the theoretical assumption. The used method for the bracing system design performed very well reaching all its main tasks. As a matter of fact the ductility demand in the original r.c. frame was sensibly cut off, and distributed uniformly over the height of the model. Moreover not any fragile local failure was registered.

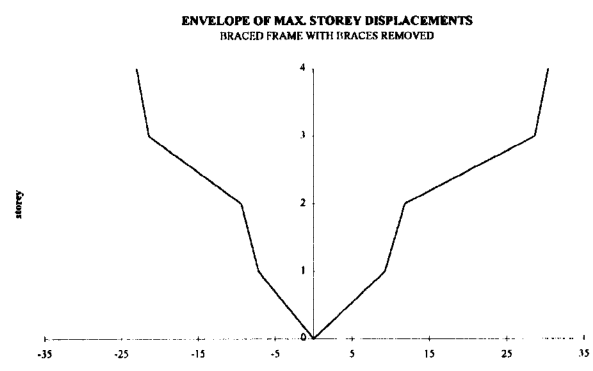

Fig. 16 Max. storey disp. (braces removed)

As far as the energy absorbing devices are concerned, the measured ductility demands were, also for the last testing shakes, quite low compared to their available ductility resources. In fact not any damage or hardening effect was registered confirming that there is no need to repair or replace them even after a strong earthquake.

The absence of any relative displacements of the brace to the r.c. frame confirmed the effectiveness of the connection to the mounting bracket realised via friction bolts.

Finally experimental results stressed the importance in using pretensioned steel chains to counteract the induced tensile forces and to exert a beneficial effect of confinement on beam-column joints. And that is a fully innovative one through the strategies used for the seismic retrofitting of r.c. frames.

Further studies are needed to explore the behaviour of the bracing system on spatial models. Moreover, full scale pseudo-dynamic testing would seem to be a worthwhile last step in evaluating certain scale effects of the model materials in comparison with the prototype ones.

REFERENCES

Aiken, I. D. and J.M.Kelly (1990). Earthquake simulator testing and analytical studies of two energy-adsorbing systems for multi-storey structures. *Report UCB/EERC 90/03*, Berkeley.

Aiken, I. D. *et al.* (1990). The application of viscoelastic dampers to seismically resistant structures. In: *Proceeding of Fourth U.S. National Conference on Earthquake Engineering*, Vol. 3, Palm Springs.

Bracci, J.M. *et al.*(1992). Seismic resistance of reinforced concrete frame structures designed only for gravity loads. *Technical Report NCEER 92/0027 State University of New York at Buffalo,* Buffalo.

Braga, F., P.D'Anzi (1994). Steel braces with energy absorbing devices: a design method to retrofit reinforced concrete existing buildings. In: *Strengthening and repair of structures in seismic area* (V. Davidovici and D. Benedetti, ed.), pp. 146-154. Ouest Éditions Presses Académiques, Nice.

Braga, F. *et al.* (1995). Prove dinamiche su tavola vibrante di un telaio multipiano in c.a. dotato di controventi dissipativi. In: *L'ingegneria sismica in Italia* (A. Borri and F. Braga, ed.), Vol. 1, pp. 341-350. Collegio degli Ingegneri della Toscana, Siena.

Cherry S., A.Filiatrault (1993). Seismic response control of building using friction dampers. *Earthquake Spectra*, *9*, 447-466.

Ciampi, V. (1991). Use of energy dissipating devices, based on yielding of steel, for earthquake protection of structures. In: *Protezione sismica degli edifici.* Ancona.

Ciampi, V. *et al.*(1993). Sul progetto sismico di controventi dissipativi per la protezione sismica delle strutture. In: *L'ingegneria sismica in Italia* (A. Borri and A. Parducci, ed.), Vol. 1, pp. 131-140. Centro Stampa Università degli Studi di Perugia, Perugia.

Ciampi, V. *et al.* (1995). Sperimentazione su tavola vibrante di un telaio con controventi dissipativi. In: *L'ingegneria sismica in Italia* (A. Borri and F. Braga, ed.), Vol. 1, pp. 311-320. Collegio degli Ingegneri della Toscana, Siena.

D'Anzi, P. *et al.*(1995). Shaking table tests of a yielding bracing system for strengthening non-seismically designed reinforced concrete frames. In: *European seismic design practice - Research and application* (A.S. Elnashai, ed.) pp. 271-277. A.A. Balkema/Rotterdam/Brookfield, Chester (UK).

Dolce, M. and B.Filardi (1995). Analisi del comportamento globale e locale di due tipi di dispositivi dissipativi. In: *L'ingegneria sismica in Italia* (A. Borri and F. Braga, ed.), Vol. 2, pp. 729-738. Collegio degli Ingegneri della Toscana, Siena.

Filiatrault, A. and S.Cherry (1990). Seismic design spectra for friction damped structures. *J. of Structural Engineering, 116/5*, 1334-1355.

Petkovski, M. and P.Waldron (1995). Seismic behaviour of reinforced concrete frames with energy dissipating cladding. In: *European seismic design practice - Research and application* (A.S. Elnashai, ed.) pp. 487-494. A.A. Balkema/Rotterdam/Brookfield, Chester (UK).

Vulcano, A. and G.Guzzo Foliaro (1993). Criteri progettuali di strutture intelaiate con controventi dissipativi. In: *L'ingegneria sismica in Italia* (A. Borri and A. Parducci, ed.), Vol. 1, pp. 131-140. Centro Stampa Università degli Studi di Perugia, Perugia.

Vulcano, A. and F.Mazza (1995). Indagine parametrica sulla risposta sismica non lineare di strutture intelaiate con controventi dissipativi. In: *L'ingegneria sismica in Italia* (A. Borri and F. Braga, ed.), Vol. 2, pp. 729-738. Collegio degli Ingegneri della Toscana, Siena.