Paper
Topic: Seismic evaluation of concrete structures

TREMA Project: Experimental evaluation of the seismic performance of a R/C ¼ scaled model upgraded with the DIS-CAM system

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1. INTRODUCTION

A new system for the seismic retrofit of R/C frames has been tested within an extensive experimental investigation on shaking table, carried out for the TREMA project (Technologies for the Reduction of seismic Effects on Architectural Manufacts) [1]. The 3D model was a ¼ scaled frame, with three stories, two and one bay in the two directions respectively, originally designed for gravity loads only [1]. It was previously tested, firs with base isolation and then in the fixed base configuration, up to being severely damaged. After repairing, by just restoring concrete in broken joints at the 2nd floor with high strength mortar and injecting large cracks with epoxy resin, the model was upgraded with the DIS-CAM system. This system is an improvement of the CAM system described in [2]. The original CAM system provides confinement to columns using steel angles and steel ribbons, while the new improvement is based on the addition of special steel connecting energy dissipating elements at the beam-column joints. The retrofit determines an increase of strength and ductility of single elements and joints, as well as an increase of the overall ductility, if it is designed according to the capacity design criterion forcing the structure to develop a strong column-weak beam mechanism.
The experimental tests were carried out with the two simultaneous horizontal components of the Umbria-Marche 1997 earthquake, up to 0.52g normalised peak value of the acceleration (NPA as defined in [1]) of the table and with the three simultaneous components of the near-fault Northridge Rinaldi record, up to 0.84g horizontal NPA. The confinement of concrete, the strength improvement and the additional energy dissipation provided by the DIS-CAM system determined a great benefit for the R/C structure. Actually the structure could survive the Northridge earthquake up to a horizontal NPA of 0.84 g, with just some energy dissipating connecting elements broken, mainly because of the fatigue accumulation in the previous 19 tests, carried out with increasing earthquake intensities.
In the paper the main results of the experimental tests are described. The behaviour of the model under the different seismic excitation is discussed.

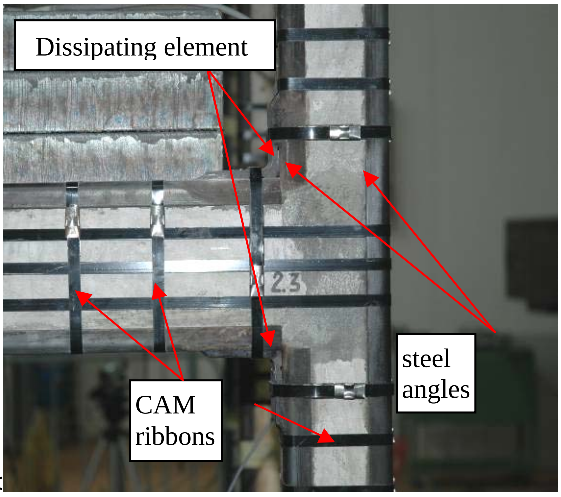
Keywords: Dissipating energy, reinforced concrete, structural models, seismic upgrading, shaking table.

2. REPAIR AND SEISMIC UPGRADING OF THE FRAME

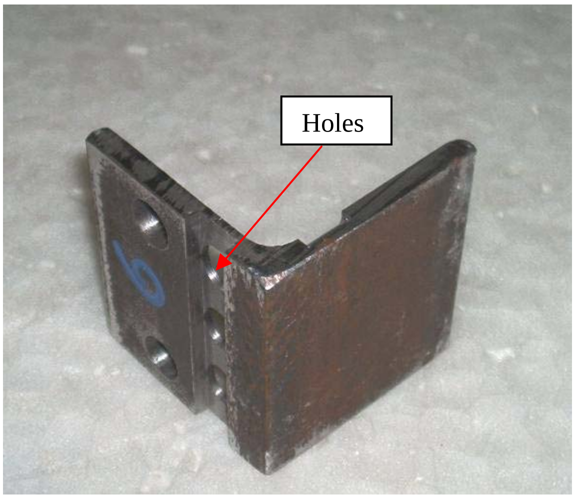
The tests were carried out on the third of the three frame models described in [1]. This frame was severely damaged in the previous tests, up to the near collapse condition. The damage condition was quantified in terms of reduction of its fundamental frequency at the end of the tests on the fixed base configuration with infilled masonry panels. It decreased from 3.3 Hz to 1.6 Hz. Many of the beam-column joints were practically collapsed, while beams and columns were quite undamaged. A detail of the final condition of a broken joint at the 2nd floor is shown in Fig. 1.

Fig. 1 – Detail of a damaged joint of model No. 3 at the end of the tests in the fixed base condition

Before the strengthening operations, the frame was repaired, without increasing local strength. Therefore the crushed or spalled concrete in the severely damaged joints was reconstituted by using high strength mortar, while large cracks were repaired with epoxy injection. No addition or substitution of steel reinforcement was made.
The DIS-CAM system was used to upgrade the model. This system is derived from the CAM system [2], which was originally conceived just for concrete confinement and shear strength improvement of R/C columns. It is made of steel angles running all along the four corners of a column and a suitable number of pre-stressed steel ribbon loops. In order to improve the energy dissipation capacity of a frame, also the beams are endowed with steel angles and confining ribbon loops, while energy dissipating mild steel L-shaped elements are added at the beam-column joints, as shown in fig. 2. The dissipating elements are welded to the longitudinal steel angles of adjacent columns and beams. Besides establishing the continuity of columns and beams across the joint, with the help of additional ribbon loops, they can dissipate energy in their relative rotation around the joint. Moreover, the dissipating elements can be so shaped as to increase the flexural strength more in columns than in beams, thus fulfilling the capacity design criterion aimed at realising the strong column – weak beam mechanism [4,5].
The upgrading design was carried out with reference to the new Italian seismic code [4], which is consistent with the Eurocode 8 [5]. The building was assumed to be placed in seismic zone 2. Consequently, the design PGA on bedrock was assumed to be 0.25g. Intermediate stiff soil (class B) was assumed, resulting in an actual design PGA equal to 0.3125 g and in a corner point of the spectrum at 0.5 secs.
The capacity design criteria were systematically applied to each structural element and joint, so that flexural failure anticipates shear failure and plastic hinges occurs in beams rather than in columns, except for the bottom end of the columns at the 1st level [7]. Therefore the structure could be treated as a high ductility class (class A in [4]) frame.

< 

a)



b)

Fig. 2. a) Components of the DIS-CAM seismic upgrading system, b) detail of a dissipating element.

The design of the upgrading system was carried out using static linear analysis. The horizontal seismic forces were derived from the design spectrum referred to a behaviour factor q = 5.85, according to the regularity characteristics of the model and the high ductility class assumption. The dimensions of the dissipating elements and the number of CAM ribbon loops around the joints required the evaluation of the additional flexural strength needed in the beams to withstand the design seismic action, starting from the existing reinforcement in the beam section. This additional resisting moment is then decomposed in the couple of force, according to the mechanism shown in fig. 3a. The compressive force is absorbed by concrete while the tensile force is absorbed by the dissipating elements. In order to get the required strength and stiffness characteristics the dissipating elements are locally weakened, by reducing thickness and, eventually, making holes, as shown in figs. 2 and 3.

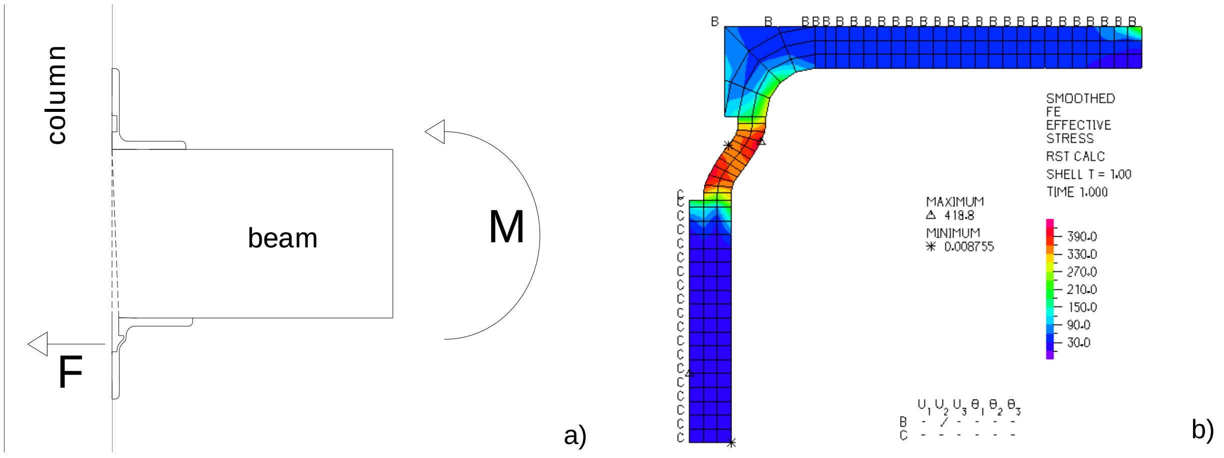


Fig. 3. Dissipating element: a) yielding mechanism; b) stress state by finite element analysis

The proper thickness of the dissipating elements and eventual holes in its weakened part were determined by means of finite element nonlinear numerical analyses on a 2D model, using ADINA [6]. Fig. 3b shows the stress distribution obtained through the numerical simulation.
Cyclic tests on some dissipating elements were then made, in order to verify the expected force threshold and the stability of the hysteretic loops. The testing set-up and the cyclic behaviour of the specimen are illustrated in figure 4. The test was made on a couple of elements in a symmetrical arrangement. The elements were welded to the horizontal and vertical plates, so as to restrain it and allow for the desired deformation only. The dissipating elements were welded in a similar way to the longitudinal steel angles of columns and beams of the R/C structure, as shown in fig. 2.

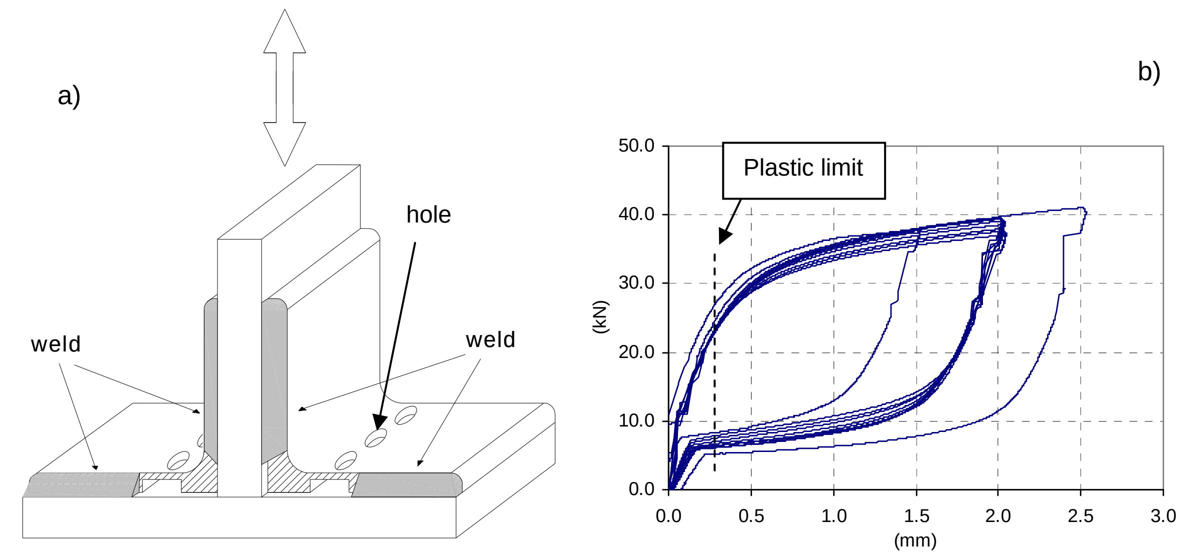


Fig. 4. a)Test set-up and b) Experimental behaviour of a dissipating element in a cyclic test.

3. TEST AND SENSOR SET-UP

The dynamic tests on the frame model were carried out on the 4x4m 6-dof shaking table facility of the Enea-Casaccia (Rome). The table is able to move in a range of frequencies between 0 and 50 Hz, with a maximum PGA of 3g.

The dynamic behaviour of the frame model was monitored using several groups of sensors, to measure base and floor accelerations, inter-storey displacements, joint deformations, for a total of about 90 sensors. The project partners provided, for this test, an integrated acquisition system constituted by:

- 15 accelerometers for the table and model accelerations (ENEA);
- 16 accelerometers for the table and model accelerations (Dept. Of Civil Protection-USSN, DPC-USSN);
- 2 accelerometers for the roof accelerations (University of Basilicata, UNIBAS);
- 14 displacement transducers for the joint deformations (UNIBAS);
- 6 displacement transducers for some inter-storey displacements (DPC-USSN);
- 3 optical transducers for strain measurements of some ribbons and angles (ENEA)

After some checks of consistency with the measurements obtained with the other sensors, only the records of the 15 accelerometers provided by ENEA have been used to analyze the overall response of the model in the present work. In fig. 5 their layout is shown. The accelerometers are placed at every floor of the model including the foundation basement and the table. By suitably processing the accelerations, it was possible to obtain other quantities useful in the evaluation of the seismic behavior of the frame: absolute and inter-storey displacements, frequency of the fundamental vibration modes, base shear forces.

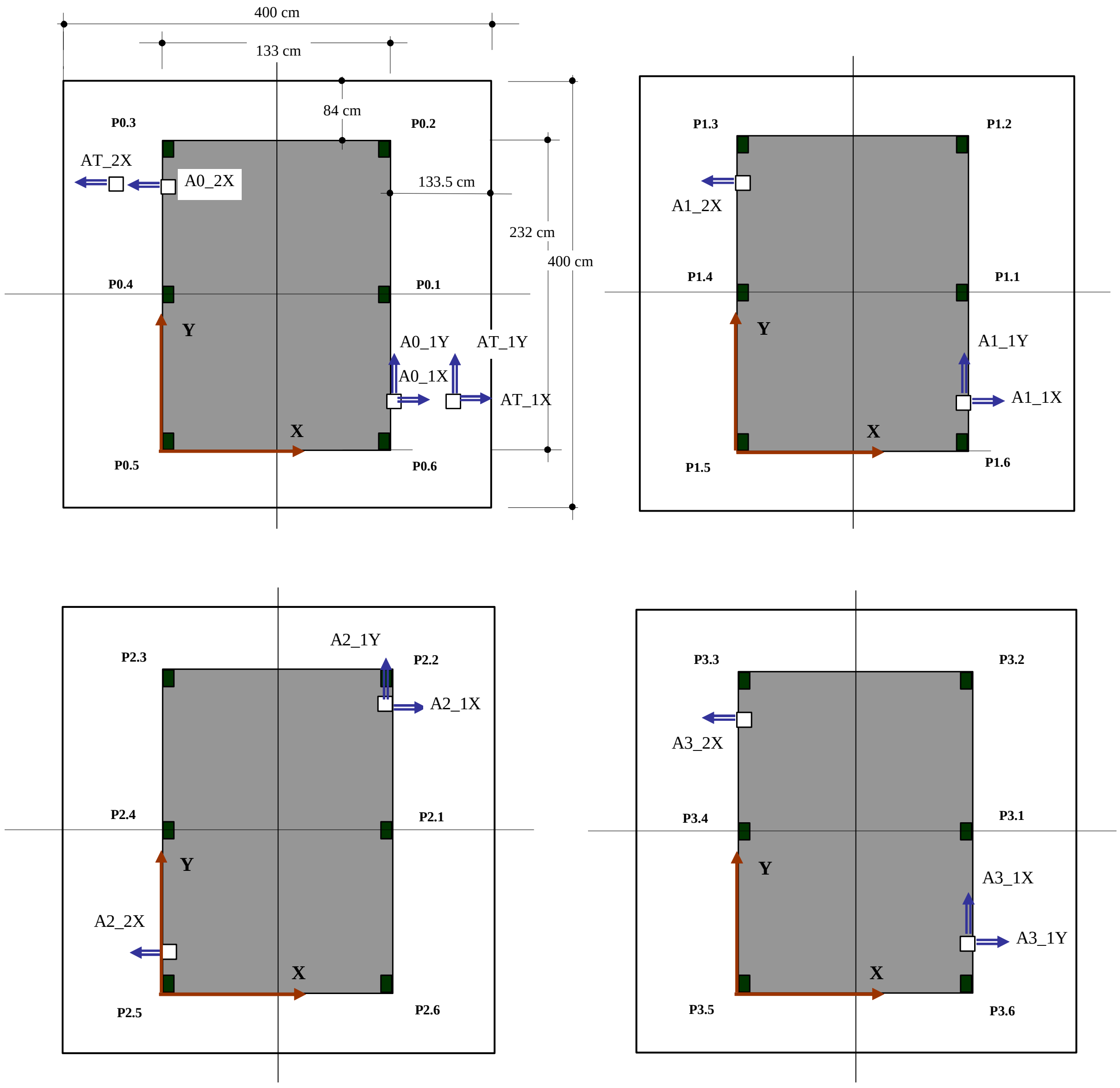


Fig. 5. Layout of the ENEA accelerometers

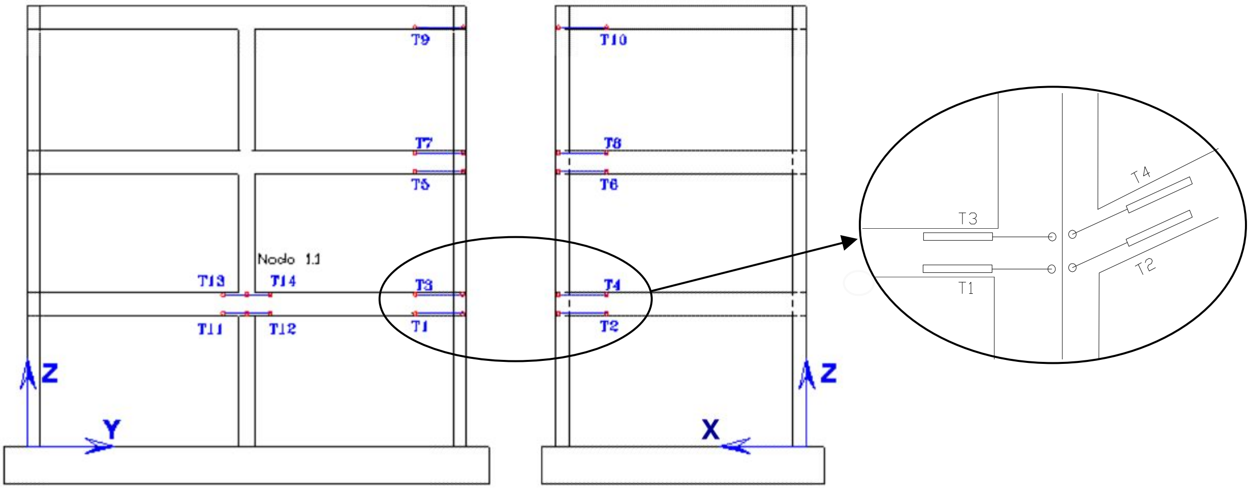


Fig. 6. Positioning of the UNIBAS displacement transducers

The local behaviour of the joints and of the energy dissipating elements have been analyzed using the 14 displacement transducers provided by UNIBAS. They were positioned in the most presumably stressed joints of the frame. Four beam-column joints were monitored, as illustrated in fig. 6, to measure the relative displacements between the joints and the adjacent beams at the upper and lower edge of the beam, and then the relative rotations and the deformations of the dissipating elements.

SEISMIC INPUT

Two natural input records were used in the dynamic tests of the DIS_CAM strengthened model: the Colfiorito 1997 earthquake replied consecutively three times, to compare its response to the other cases that were excited in the same way [1], and the Northridge 1994 near-fault record, to test its ability to withstand near-fault earthquakes with large pulsed and high vertical accelerations. Both records were scaled in time by a factor 2, derived as the square root of the geometric scale of the model SL=4. The first 11 tests were made with the Colfiorito earthquake, the remaining tests with the Northridge Rinaldi Station record. Moreover, random tests were performed at the start of the dynamic test sequence and between two following tests, in order to verify the frequency decay of the model and, then, the damage progress. The main characteristics of the input records are shown in fig. 7, where their signal and normalized response spectra (PGA=1.0g) are illustrated.

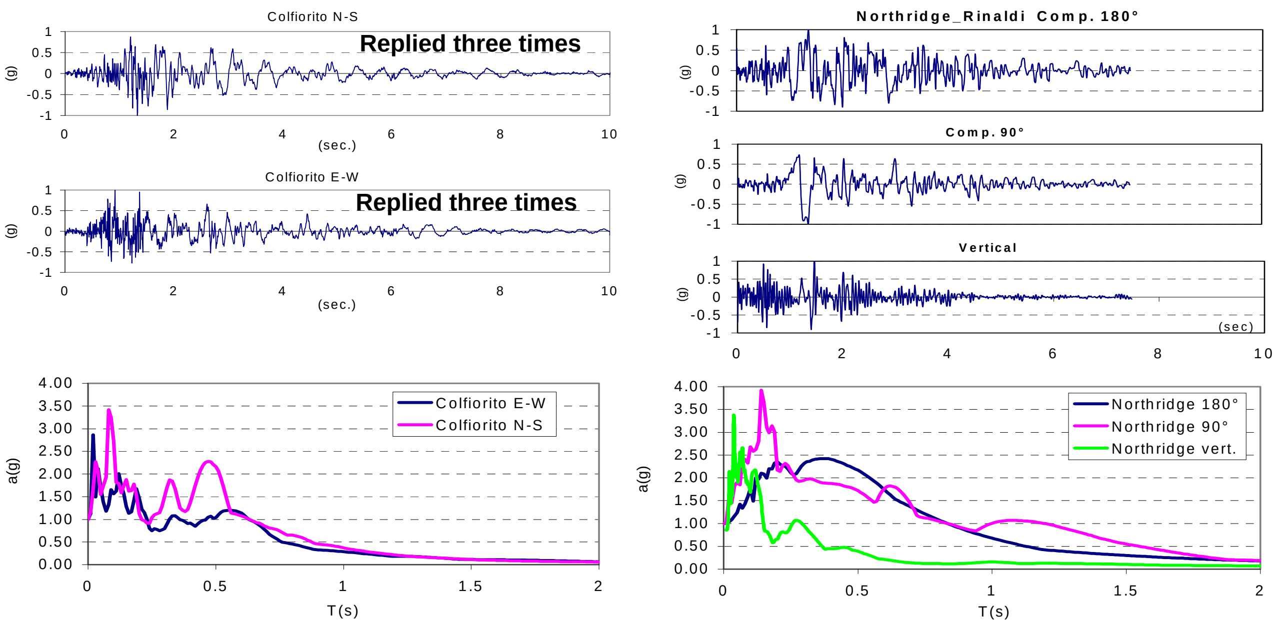


Fig. 7. Normalized accelerograms and relevant 5% damping response spectra of the Colfiorito and Northridge acccelerograms already scaled in time for shaking table tests.

Because of some high frequency noise recorded on the table during the tests, that altered the shape and amplitude of the input acceleration. This lead to inconsistent values of the peak acceleration of the table (PGA_{TAB}). In order to better characterize the table records in terms of destructive potential for the tested model, a procedure of normalization was applied. This consisted of the cleaning of the signal with a 30 Hz lowpass filter, a range of frequency of no interest for the model, and then on the normalization of the maximum acceleration based on the Housner intensity calculated in the range of periods between 0.11 and 1.0 sec. The normalised value of the table peak acceleration NPA is then obtained by equating the Housner intensities of the original signal and the filtered table signal.
In Figs. 8 and 9, the NPA values are reported as a function of the table PGA, for all the dynamic tests. The NPA during the tests was 0.52g in the Y-direction with Colfiorito and 0.84g in the X-direction with Northridge. Generally speaking, the normalised peak acceleration (NPA) is between 40% and 60% the peak acceleration actually recorded on the table (PGA_{TAB}), with some few exceptions.

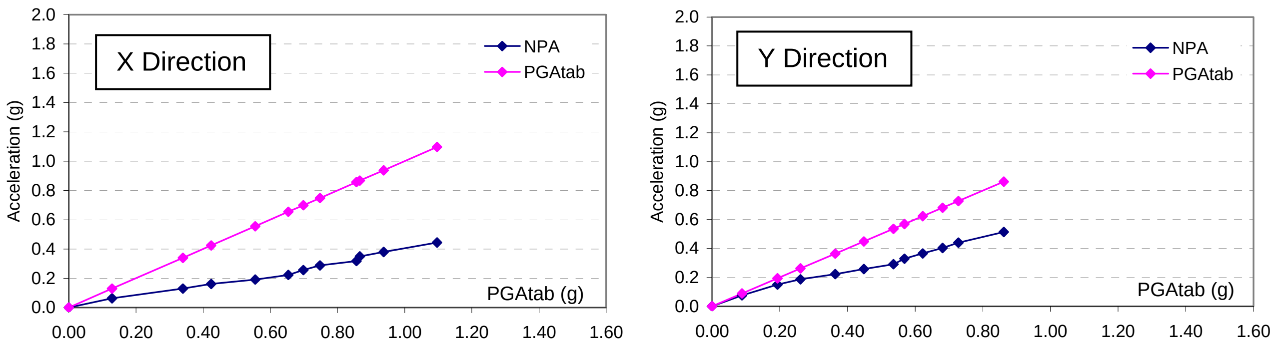


Fig. 8. Comparison between NPA and PGA_{TAB} for the colfiorito dynamic tests.

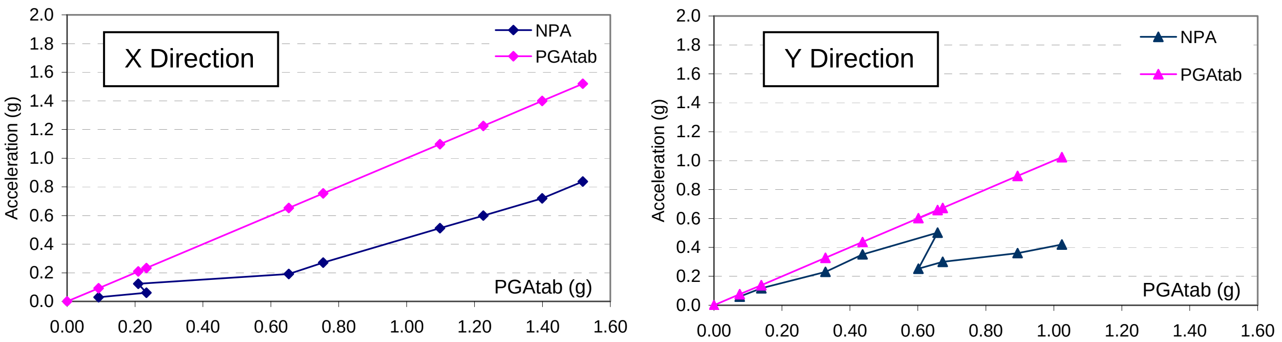


Fig. 9. Comparison between NPA and PGA_{TAb} for the Northridge dynamic tests.

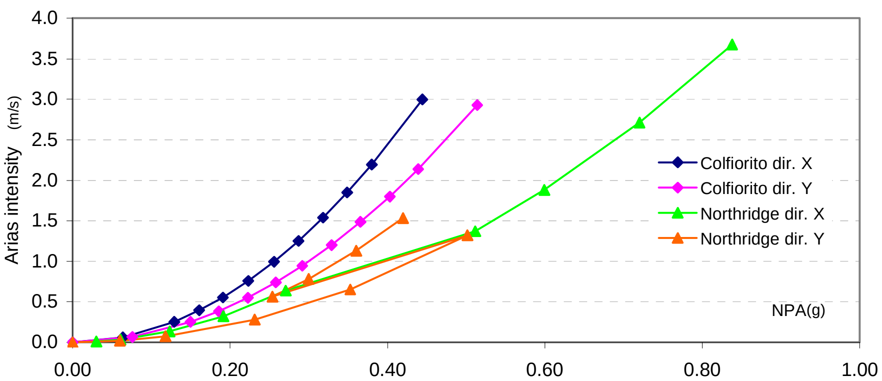


Fig. 10. Arias intensity as function of NPA.

The Northridge earthquake was applied in the first 5 tests with the 90° component in the X direction and the 180° component in the Y direction, while in the last four tests, the horizontal acceleration components were inverted, in order to stress severely the upgrading system also in the X-direction, as shown by fig. 9.
In order to consider also the duration of the motion in the intensity evaluations of the several acceleration records, also the Arias intensity has been calculated. Fig. 10 shows the relationship between Arias intensity and NPA. Obviously, this evaluation is conditioned by the three times replication of the Colfiorito earthquake in each test. For this reason, for a given value of NPA, the Colfiorito produces a greater value of the Arias intensity with respect to the Northridge earthquake. If the single Colfiorito earthquake is considered, its intensity would turn out to be considerably less than the Northridge earthquake.

EXPERIMENTAL RESULTS

As shown in tab. 1, the test sequence started with the application of a low intensity (0.05g) random signal to characterize the initial dynamic behavior of the frame. Then the Colfiorito earthquake was applied with increasing intensity. After the test with NPA equal to 0.19g some cracks in columns and beams, close to the joints, were noticed. For NPA=0.37g a relaxation of the bolt fixing the base dissipating elements to the R/C basement of the model was observed. Therefore they were fastened and the test was replied with the same acceleration. After the Colfiorito test at NPA=0.44g several dissipating elements appeared yielded according to design. Also this test was replied after fastening The most damaged energy dissipating elements were found at the base and at the top of the 1st level columns, as can be seen in the pictures of figure 11, where some failure was noticed. The Colfiorito's tests were carried out up to NPA=0.51 g.

a)

b)

Fig. 11. Damage occurred in the dissipating elements of the 1st level column at a) top and b) base

Test n.	Acceleration	NPA - X (g)	NPA - Y (g)	
1	Random	0.05	0.05	
2	Colfiorito	0.06	0.08	
3	Colfiorito	0.13	0.15	
4	Colfiorito + Random	0.16	0.19	
5	Colfiorito + Random	0.19	0.22	
6	Colfiorito	0.22	0.26	
7	Colfiorito	0.26	0.29	
8	Colfiorito	0.29	0.33	
9	Colfiorito	0.32	0.37	
10	Random + Colfiorito + Random	0.32	0.37	rep.
11	Colfiorito + Random	0.35	0.40	
12	Colfiorito + Random	0.38	0.44	
13	Random + Colfiorito + Random	0.38	0.44	rep.
14	Colfiorito + Random	0.44	0.51	
15	Northridge + Random	0.03	0.06	
16	Northridge + Random	0.06	0.12	
17	Northridge + Random	0.12	0.23	
18	Northridge + Random	0.19	0.35	
19	Northridge + Random	0.27	0.50	
20	Northridge + Random	0.51	0.25	
21	Northridge + Random	0.60	0.30	
22	Northridge + Random	0.72	0.36	
23	Northridge + Random	0.84	0.42	

Tab. 1. Test Program (rep. stands for replication)

The dynamic tests with the Northridge records started newly from NPA=0.06g. In this case, due to the big number of tests and in order speed up the testing program, for each dynamic test the random signal followed directly the earthquake acceleration. The 90° component was actuated along the Y direction of the model (long direction) while the 180° component along the X direction. The components were inverted after the test with NPA=0.5g along Y, in the last 4 tests.

At a NPA equal to 0.6g along X a complete check on the structure was made and other failures in the dissipating elements were found. Seven dissipating elements were completely failed while six more elements lost at least 25% of their resistant section. All the failures occurred in dissipating elements acting along the Y direction of the model. Moreover, a progress of the concrete cracks near the joints and a slight relaxation of the bolts connecting the dissipating elements to the base of the model were observed. The test series continued up to NPA=0.84g. After this last test thirteen (**Conferma Giuseppe**) dissipating elements were completely failed along the X direction also. Thirteen elements partially failed. As expected, no failures in the steel angles and in the CAM ribbons were found. The concrete cracks had dimensions of few millimetres.
The absolute displacements of the slabs were calculated by double integration of the bandpass filtered acceleration data of the three accelerometers placed at each level of the model and on the table. The absolute displacements were then used to calculate the displacements in the two directions and the rotation of each floor, assuming in-plane infinite stiffness. These displacements and rotations were then used to determine the drift in the four external frames of the model. In figures 12 and 13, the maximum values of the inter-story drift during the dynamic tests are shown for each external frame.
As can be seen, the drift of parallel frames results to be somewhat different, because of some torsion effects mainly due to a rotational acceleration component of the table motion.
In the Colfiorito tests the trend of the drift is quite regular. Generally speaking, the drift increases at all levels while increasing NPA. The highest value of drift occurred at the first level, being of the order of 3% in the X direction and slightly greater than 2% in the Y direction, for NPA=0.5g.
For the Northridge tests the drift increases up to 8% in the X direction frames for NPA equal to 0.84g, while in the Y direction the maximum drift (4.3%) is attained for NPA equal to about 0.5g, then decreasing because of the inversion of the horizontal components of the earthquake.
The distribution of the drift along the height of the structure is consistent with the desired strong column - weak beam mechanism. Significant inelastic excursions occurred at all stories, mainly at the first and second ones, resulting in a considerable energy dissipation.

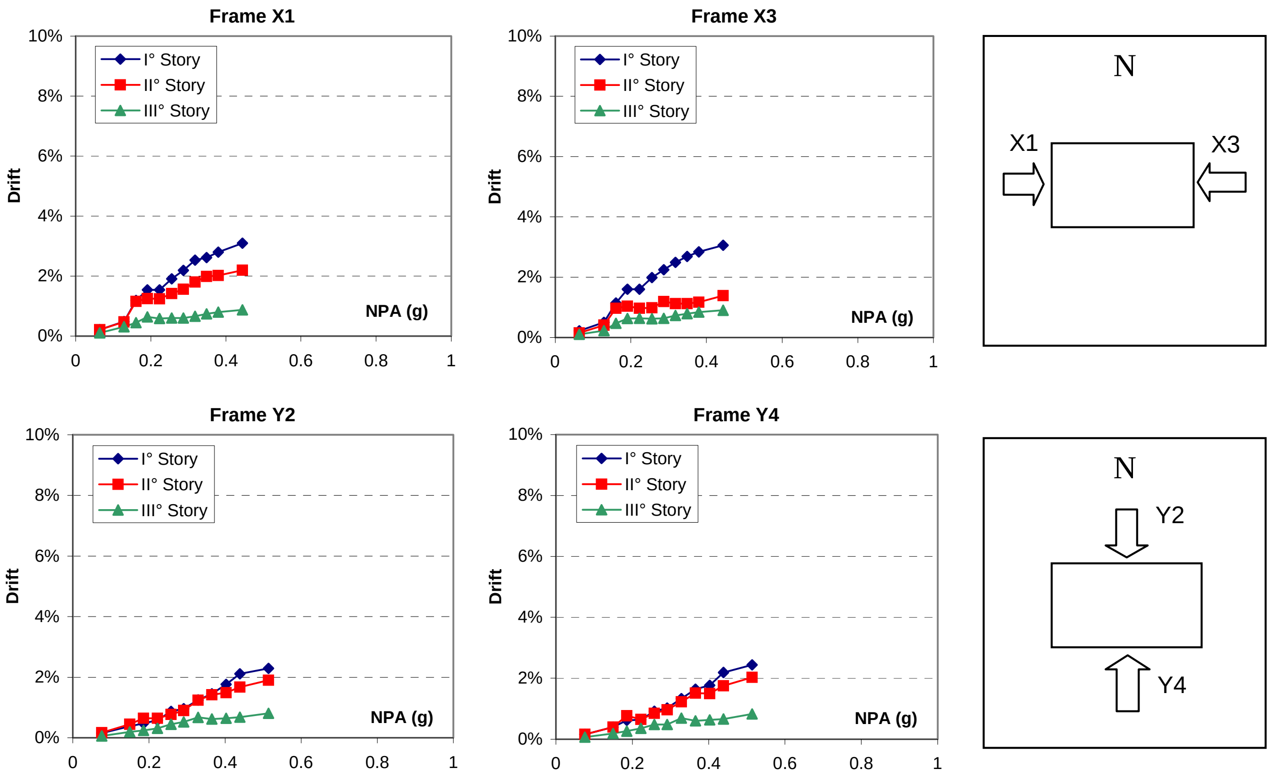


Fig. 12. Maximum inter-storey drift of the external frames for the Colfiorito dynamic tests

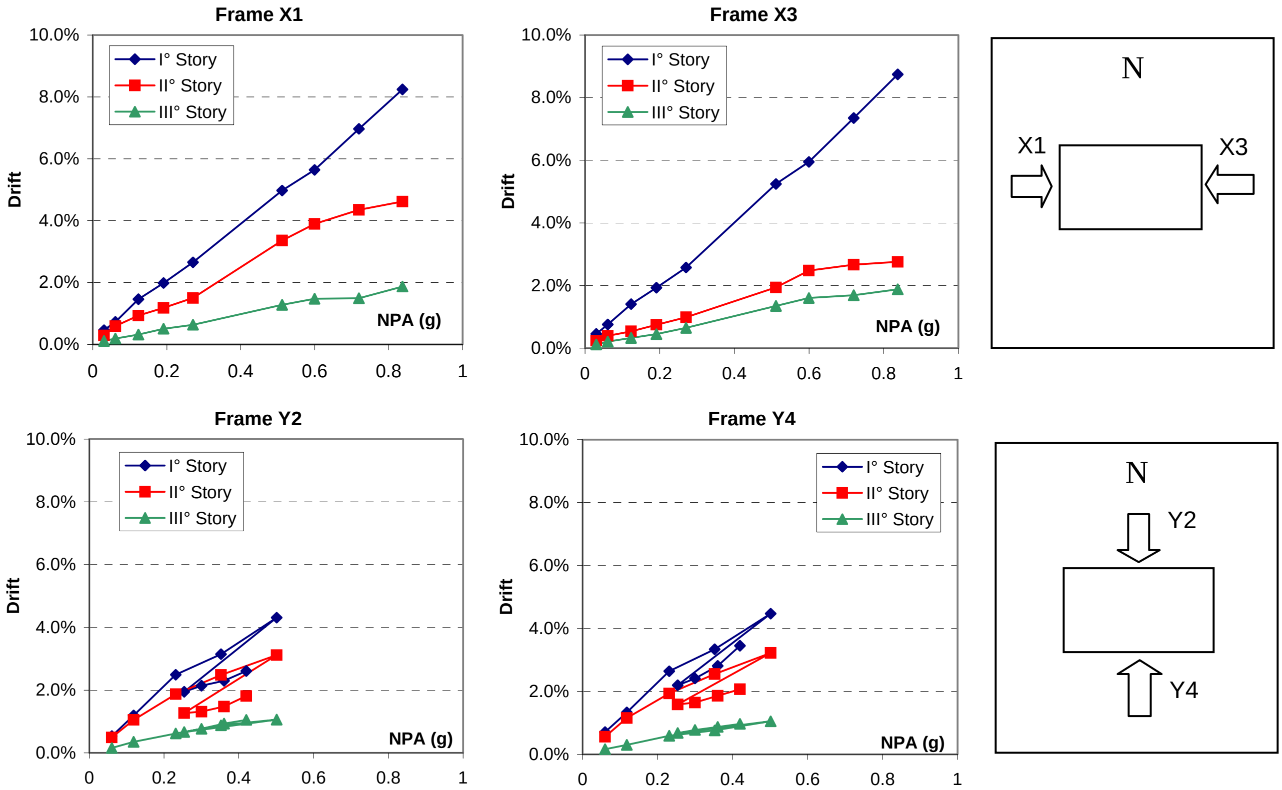


Fig. 13. Maximum inter-storey drift of the external frames for the Northridge dynamic tests

Even with the Northridge earthquake, when some energy dissipating elements were severely damaged or broken, a good distribution of the inelastic excursions along the height of the structure was found. Considering that, at the end of the tests the structure exhibited almost no residual displacement and a residual lateral force capacity, in spite of as high values as 8% of drift, it can be said that the DIS-CAM system was able to provide the structure with an outstanding ductility capacity. This capacity is partly due to the high local ductility capacity of the energy dissipating elements and to the capacity design effectiveness in redistributing plastic hinges all over the structure.

In the graphs of figures 14 and 15 the maximum accelerations calculated at the centres of mass for each floor are plotted as a function of NPA. For Colfiorito earthquake, as expected for a fixed base structure and for low values of NPA, the accelerations increase from the bottom to the top of the model. For high values of NPA the amplification of acceleration along the frame height reduces considerably, because of the structural damage that caused the shift of the vibration period (fig. 7). In the case of Northridge, this effect is more evident, leading to some reduction of acceleration with respect to the table, due to the progress of damage. In fig. 15 some abnormal values of the peak acceleration, especially at the 3rd floor of the model, are visible in the Y direction. This effect can be ascribed to some undesired movements of the additional steel masses placed on the roof of the model and occurred during some tests, after which the masses were fastened again.

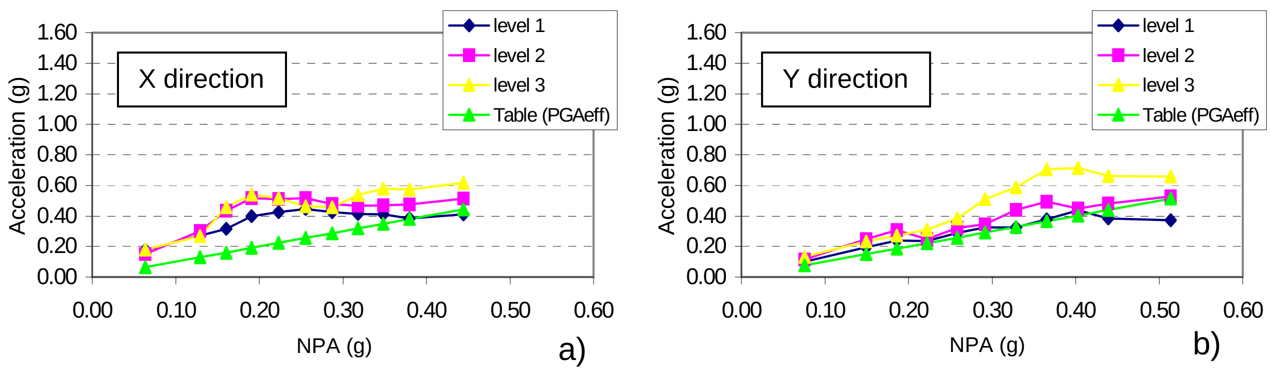


Fig. 14. Accelerations along the height of the structure in the Colfiorito's tests

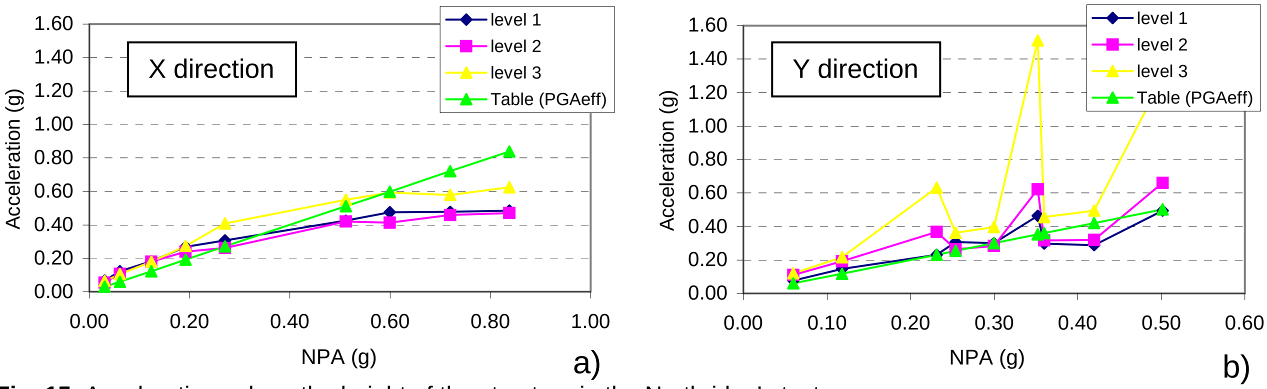


Fig. 15. Accelerations along the height of the structure in the Northridge's tests

In the graphs of fig.16a the frequency decay of the model caused by the cumulated structural damage in both directions X and Y during dynamic tests is shown. The frequencies of vibration were determined by analyzing the random tests records by means of a transfer function of the Fourier transform between the roof and the basement accelerations. As can be noted, most of the damage occurred in the Colfiorito tests. In fact, as already said, after the Colfiorito test series, some dissipating elements were collapsed and several cracks in the concrete members had occurred. In the following Northridge tests, most of the structural stiffness was provided by the dissipating element and the CAM system, which were subject to lesser degradation.

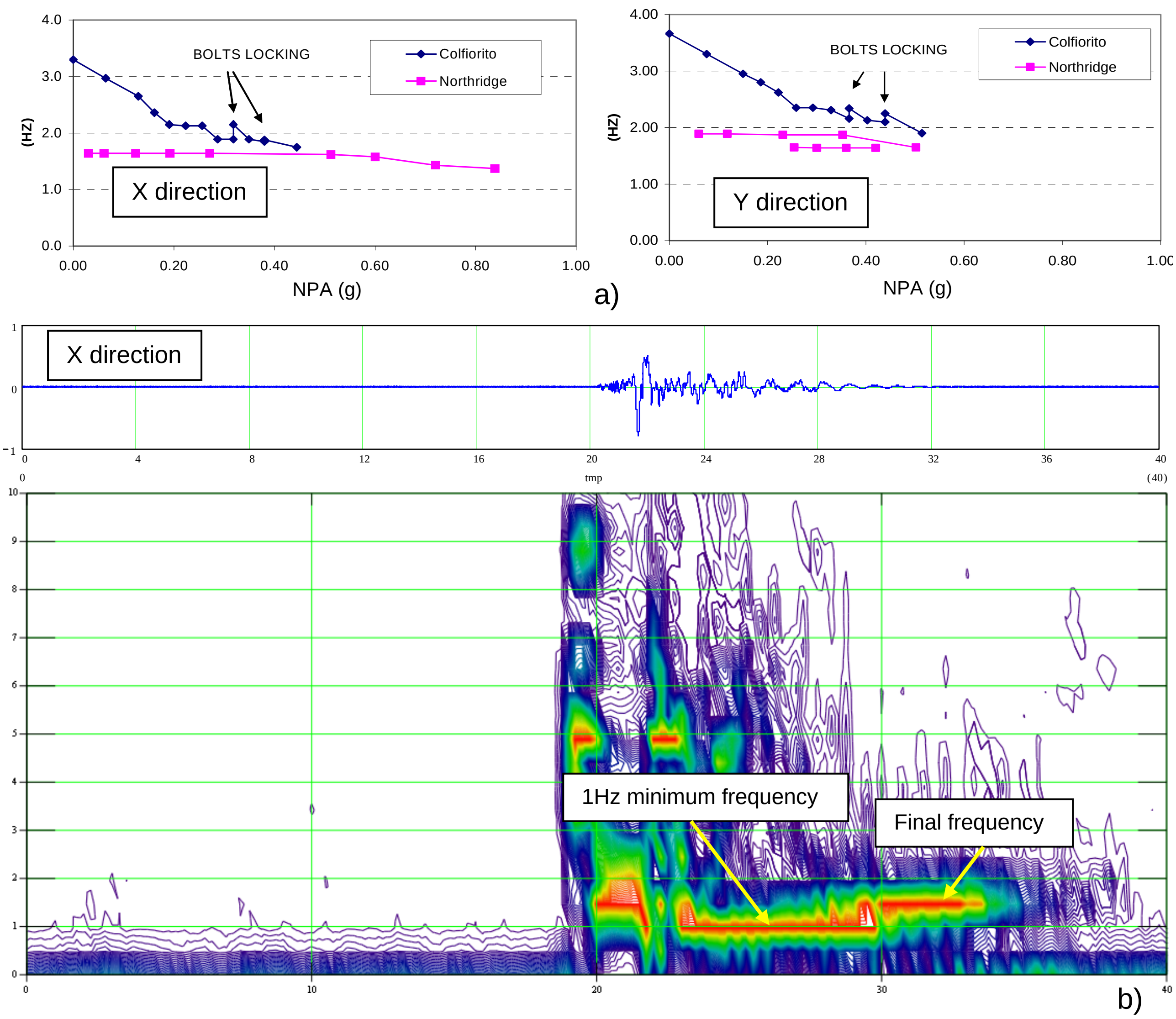


Fig. 16. a) Frequency decay during the dynamic tests; b) Gabor transform of the X direction acceleration.

The fundamental frequency reduced in both directions more than 50%, from the first to the last test. Actually, the first random test gave 3.30 and 3.66Hz frequency, respectively in the X and Y direction, while the last one gave 1.37 and 1.64 Hz. This results in about 75% stiffness reduction. Figure 16b shows the reduction of the main frequency during the maximum intensity of the earthquake application, obtained by processing the acceleration signal with the “Short Time Fourier Transform” (STFT) [3]. Due to the plastic behaviour of the dissipating system and of the frame as a whole. the minimum fundamental frequency attained 1Hz.

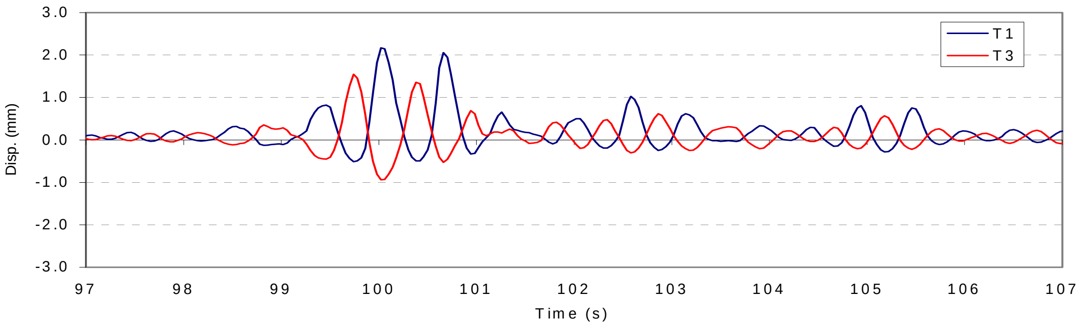


Fig. 17. Displacements of the corner plates near the transducers T1 and T3 in the Colfiorito test NPA=0.5g.

COLFIORITO earthquake					
Displacement Transducers (Fig. 6)	Max. deformation	Min. deformation	Max. rotation	Min. rotation	Cumulated displacement
1	2.17 mm	-0.51 mm	0.0164	-0.0109	522.1 mm
2	1.28 mm	-0.39 mm	0.015	-0.012	477.4 mm
3	1.55 mm	-0.94 mm	0.0164	-0.0109	560.8 mm
4	1.70 mm	-1.39 mm	0.015	-0.012	653.2 mm
5	0.91 mm	-0.34 mm	0.010	-0.006	279.9 mm
6	0.63 mm	-0.23 mm	0.008	-0.009	211.9 mm
7	0.64 mm	-0.78 mm	0.010	-0.006	369.4 mm
8	1.10 mm	-0.72 mm	0.008	-0.009	440.2 mm
9	0.02 mm	-0.04 mm	0.000	0.000	7.6 mm
10	0.15 mm	-0.05 mm	0.001	0.000	27.3 mm
11	0.33 mm	-0.57 mm	0.000	-0.003	219.4 mm
12	0.21 mm	-0.40 mm	0.002	-0.006	16.8 mm
13	0.20 mm	-0.49 mm	0.000	-0.003	28.6 mm
14	0.63 mm	-0.30 mm	0.002	-0.006	101.1 mm

Tab 2. Cumulated plastic displacement in the energy dissipating elements during the Colfiorito tests

As said above, the behaviour of the joints and, therefore, the deformations of the corner dissipating elements were monitored during the tests with the displacement transducers shown in fig. 6. The displacements of the transducers can be approximately converted into the dissipating element deformations, making the assumption of in-plane deformation of the beam sections and considering the distance of the dissipating part of each element from the transducers. In fig.17 the time histories of the dissipating elements corresponding to transducers T1 and T3 (see fig. 6) are shown. It can be observed that for the Colfiorito earthquake with NPA equal to 0.5g, both dissipating elements, at the bottom and top edge of the beam, deformed well beyond the elastic threshold, as can be seen by comparing the recorded displacement values with the force-displacement diagram of fig. 4. Although the maximum deformation of all dissipating element was always lesser than the failure value (about 4mm), a conspicuous damage accumulation occurred in many elements due to the great number of cycles experienced at the end of the last test.
By processing the experimental data, the total plastic displacements in an element during a test and in the entire sequence of tests was evaluated, in order to get an estimate of the energy dissipated. This latter quantity was obtained by assuming a force-displacement relationship based on the kinematic elastic-plastic law approximating the experimental behaviour shown in fig. 4b. Following this criterion, the energy dissipated by the above mentioned couple of elements was found to be about 4.7% of the total input energy due to the single earthquake component in the Colfiorito test with NPA=0.5g. This can give an idea of the important

contribution to the total energy dissipation of the structural system, although the elements in the upper stories underwent smaller deformations, as shown in tab. 2, and had less strength, then, giving a smaller contribution to the dissipation. Further analyses of the huge amount of data, along with well calibrated numerical simulations, are necessary to compute the global contribution of all elements to the energy dissipation of the structural system.

CONCLUSION

A new upgrading system, expressly devised for existing R/C framed buildings designed without seismic provisions, has been applied on a previously damaged ¼ scale R/C model. The system, named DIS-CAM, makes use of three types of steel elements, among which the dissipating elements are the key components to get the desired ductile behaviour of the frame. The model was subjected to severe shaking table tests up to reaching deformations well beyond the deformation which normally determine the collapse of a R/C frame.
The shaking table tests demonstrated that the structural system can benefit of a great energy dissipation capability, due to both the excellent performances of the single parts of the strengthening system, especially the energy dissipating elements, and of the whole system, due to the application of the capacity design criterion to get a ductile strong column – weak beam collapse mechanism.
The performances of the upgraded model went well beyond the design objectives, which were referred to a design PGA equal to 0.31g. Actually the model was subjected to a long sequence of increasing intensity natural earthquakes, namely the Colfiorito (Italy) 1997 and the Northridge (California) 1994 earthquakes. This latter is a well-known near fault earthquake, with strong horizontal velocity pulses and very high vertical accelerations.
With the Colfiorito earthquake as high normalised peak acceleration as 0.51 g were reached, producing a maximum 2% drift approximately. With the Northridge earthquake, as high accelerations as 0.84 g, i.e. the same as the real earthquake PGA, were withstood by the model without collapsing. In this latter case, the maximum interstory drift was about 8%. The residual lateral strength of the model at the end of the tests demonstrate the noticeable ductility that the DIS-CAM system is able to confer to a R/C frame, even if it was originally designed for gravity loads only. This high ductility capacity justify the assumption of high ductility class made in the design and, then, the possibility of assuming high values of the behaviour factor for the design spectrum.

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