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EXPERIMENTAL STATIC AND DYNAMIC RESPONSE OF A REAL R/C FRAME UPGRADED WITH SMA RE-CENTERING AND DISSIPATING BRACES

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SUMMARY

Many existing R/C framed buildings in seismic areas are designed with no seismic criteria, considering only gravity loads. They are a significant percentage of existing structures and their strengthening is one of the most urgent problems for seismic risk mitigation. New passive control techniques are very effective and to assess their feasibility and effectiveness, an in-situ experimental investigation has been carried out on an existing two storey, one bay R/C building structure, designed for gravity loads only, with no transverse frame. The structure was upgraded in the transverse direction with four re-centring braces based on the super-elastic properties of NiTi shape memory alloys (SMA). Push-over and cyclic tests, as well as dynamic release tests, were carried out on the upgraded structure.

INTRODUCTION

A big number of Italian buildings is characterised by very low seismic resistance, considerably lesser than prescribed for new structures by the current Italian seismic code. This is mainly due to the evolution of the seismic classification and to the inadequacy of the old regulations. In many other seismic countries all over the world, the same problem occurs for the same reasons. The urgent need for expensive retrofit interventions requires accurate investigations on their effectiveness, which cannot be assessed on the only base of numerical investigations.

The results of previous research studies highlighted the peculiar behaviour and the effectiveness of devices based on NiTi shape memory alloys (SMA), invented and implemented within the MANSIDE project (Nicoletti et al. [1]). Several shaking table tests on reduced scale models have been carried out in other research project (Cardone et al. [2], Cardone et al. [3]) after MANSIDE The availability of an existing R/C building to be demolished in the ex-Italsider steel works at Bagnoli-Naples, within the ILVA-IDEM voluntary project coordinated by prof. Mazzolani [4] of the University of Naples, gave the chance to carry out in situ large-displacement tests, on a R/C frame, to evaluate the effectiveness of SMA-based

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braces for retrofit. A concurrent research project called TREMA, aimed at evaluating the effectiveness of new techniques in the seismic upgrading of masonry and r/c buildings, permitted to support the experimental activities. The in-situ tests consisted of push-over and cyclic tests, as well as release tests. The bracing system was based on the super-elastic properties of Ni Ti shape memory alloys (SMA) (Dolce and Cardone [5]), resulting in a strongly re-centring behaviour. The devices used in these tests represent a technological evolution of the devices (Dolce et al. [6]) conceived, designed and tested within the MANSIDE project (Nicoletti et al., [1]).
The tested structure was a two-storeys 3D R/C frame, with one-by-one bay. It was obtained by dividing a one-by-twelve bay structure into six sub-frames or modules, in order to obtain separate structures to test different strengthening systems. One of these units was put at disposal of the University of Basilicata (USB) research group, to test advanced bracing systems.
During the release test some seismic measurements at the soil level were also performed, in order to verify the modality of energy transmission from the structure to the soil. The measurements were made in collaboration with the University of Bologna and the National Geophysics, Seismology and Volcanology Institute (INGV), trough alignments of instrumentation disposed in several directions. The information drawn from this data is particularly important to define the interaction effects between soil motion and structural vibrations, (Mucciarelli et al. [7]).
In the present paper the setting out of the tests and the first analyses of the experimental outcomes, including the seismometric measurements, are shown.

TEST STRUCTURE

The test structure of the ILVA-IDEM Project, is an old R/C building, built in the seventies in the now dismissed industrial area of Bagnoli (Naples). The building had originally two storeys, one span in the transverse direction and twelve spans in the longitudinal direction. The structure was made of two longitudinal bearing frames, with no intermediate transverse frames, which is a quite usual situation in gravity-load designed structures. Other usual characteristics of this kind of structures is the small cross section of columns and their poor reinforcement, especially transverse reinforcement.

a) b)

Figure 1 - a) Test Building Structure and b) module at disposal of University of Basilicata, without infills.

The intestorey height is about 3.0 m for both the 1st and 2nd storey. The span length is 5.60 m in the transverse direction, and varies between 2.80 and 3.80 m in the longitudinal direction. The slabs have 24 cm thickness at the 1st level and 20 cm at the 2nd level (roof). Both slabs have just a couple of transverse T-lintels, in the slab thickness, joining the opposite columns, with the aim of realising a transverse link beam. All the columns have square cross section 30x30 cm. The reinforcement is made of four, 12 mm diameter, corner bars. To avoid any interaction with the structural elements, all internal and external infill

masonry panels were demolished and then the structure was subdivided into 6 similar modules, one of which was put at disposal of the USB research group. Preliminary investigations on the quality of concrete were performed by means of compression tests on cylindrical specimens extracted from the building. The average, failure resistance was about 20 N/mm^2. Other investigations regarded the fundamental frequency of one of the 6 modules. This was performed through artificial vibration induced by a shaker placed on the roof (Spina et al. [8]). The fundamental frequency of the analysed structural module was 1.44 Hz, which was taken into account in the calibration of the numerical model of the building.
The tests described in the present paper were carried out on the module N.5, shown in figure 1 b). The module was strengthened with the SMA bracing system in its weak direction, i.e. the transverse direction.

UPGRADING SYSTEM

The upgrading system was made of four special braces, whose mechanical characteristics are based on the properties of Shape Memory Alloy (SMA) Nickel-Titanium (NiTi) wires. The SMA NiTi wires were used in their austenite phase. Therefore they exhibited superelastic characteristics and great fatigue resistance for large deformations, of the order of 6-8%, (Dolce et al. [5]). The SMA based devices are made in such a way as to transform any deformation of the braces in a tensile deformation of the SMA wires (Dolce et al.[6]). Moreover the wires are prestressed, so that the device in rest condition is compressed and has a high initial stiffness, depending only on the steel truss section. Obviously, when the external force becomes greater than the prestress force, the tangent stiffness of the device depends on the stiffness of the SMA wire group. Therefore, this wire arrangement provides the device with a strongly non linear behaviour and a great re-centring capability, i.e. the availability of large force for small displacement. If the number of wires and their prestress level are suitably calibrated, the structural system (frame+braces) is able to recover its initial undeformed configuration after an earthquake, even if the R/C frame is subjected to significant plastic deformations. Austenitic wires can also dissipate some amount of energy, increasing the global damping by some few percent. However the present investigation is mainly aimed at exploring the capability of the re-centring characteristic alone to limit the effects (i.e. displacements) of a real structures, relying upon the energy dissipation capability of the structure itself to dissipate energy. To increase energy dissipation of the brace, however, additional dissipative elements or devices, based again on SMA or other materials, can be easily included in the braces, as shown in (Dolce [6]).

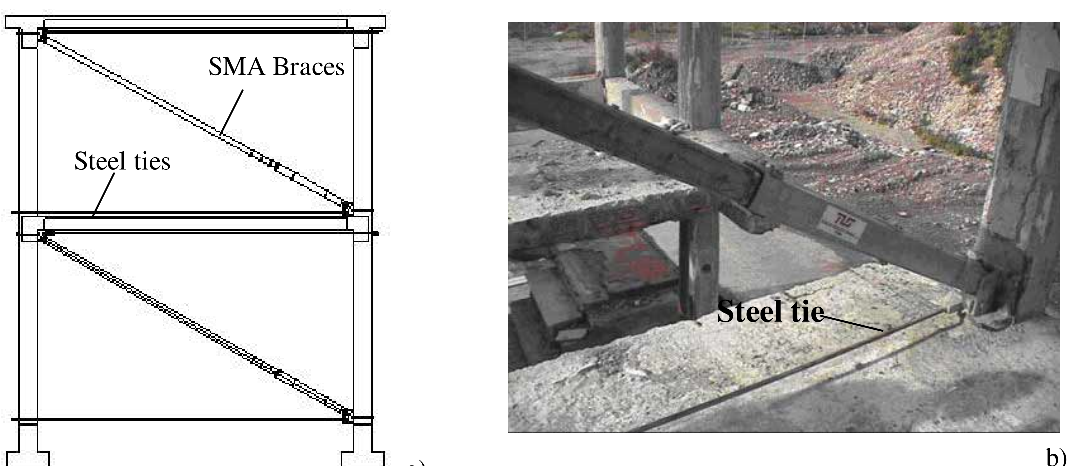


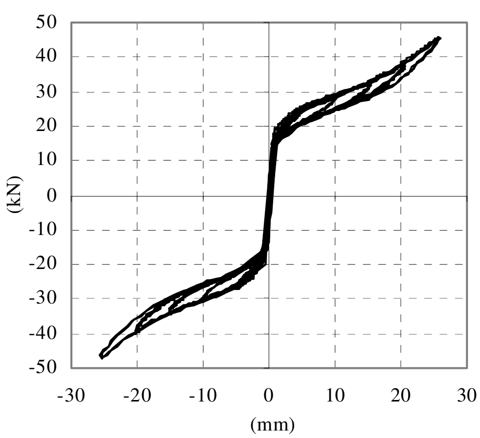
Figure 2. a) Transverse section frame of the building with bracing system arrangement, b) installed brace.

As can be seen in figure 2, the bracing system is completed by a series of connection plates and transverse ties. These latter limit shear variations in columns and avoid or limit tensile forces in the slab.

The wires used had 1 mm diameter, while their number was calibrated, for each couple of brace devices, according to the output of the design procedure.
The retrofit was designed with reference to the Italian zone 1 (PGA=0.35g on firm ground for 475 years return period) and a medium stiff soil (1.25 soil amplification coefficient), with the following purpose:

1. limiting interstorey drift within the code (Ord. 3274/03 [9]) requirement, for the Damage Limit State design action (0.5% of the storey height);
2. providing full re-centering capability to the structure, for the Ultimate Limit State design action;
3. providing adequate safety against collapse also for design actions considerably greater than 475 years return period.

A first approximated evaluation of the wire number in each brace was made on the basis of a pushover analysis carried out with the DRAIN-3DX finite element program, Prakash [10], that provided the stiffness and strength characteristics of the structure under horizontal actions. In the next steps of the design, the number of wires was modified, according to the output of non linear dynamic analyses, in order to meet the previous design objectives. The final number of 1 mm diameter SMA wires in each device was 64.
The design elongation of the braces was 25 mm. With reference to this displacement, and being 8% the maximum deformation limit of the SMA wires to keep them in the superelastic range, the length of the wires was consequently found.
Figure 3b shows the testing set-up at the USB Laboratory of Structures, to carry out the cyclic dynamic tests of the braces. The tests were performed in displacement control. Two different frequencies were investigated: 0.3 and 1.0 Hz. The maximum amplitude displacement of the cycles was taken as ± 25 mm, while intermediate displacements were also carried out with 5-10-15-20 mm amplitude. As can be seen in figure 3a, where the recorded mechanical behaviour is shown.



a)

b)

Figure 3. a) 1.0Hz frequency mechanical behaviour of a brace, b) testing equipment

The force-displacement diagram reported in fig. 3.a clearly shows the strongly non linear re-centring characteristics of the braces, with a high initial stiffness, a large re-centring force, of the order of 20 kN, a small energy dissipation capability, providing an effective damping of the order of 2%. In figure 4, two graphs show, respectively the secant stiffness and the effective damping of one brace, as a function of the displacement amplitude.
Once the braces were tested and characterised, numerical simulations were made on the structural model, including the braces with the exact number of SMA wires. The accelerogram was the Sturno record of the 23.11.1980 Irpinia earthquake, scaled up to 0.45g PGA. The numerical analyses provided 40 mm roof displacement, which is fully compatible with the design displacement of the braces.

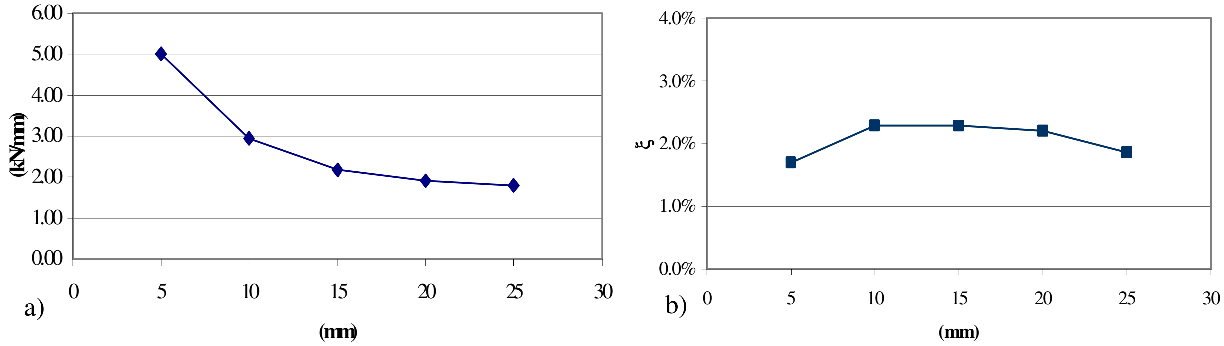


Figure 4 – a) Secant stiffness and b) effective damping of a SMA brace in a 0.3Hz frequency cyclic test.

TEST SETUP

For the experimental in situ tests, a reaction structure was designed and made-up for the ILVA-IDEM Project. The steel made reaction structure, shown in figures 5 (a, c), was able to undergo both pull and push forces, up to about 300 kN. A steel vertical beam was tied to the R/C frame at both stories, to connect a hydraulic jack contrasted against the reaction frame at an intermediate level.

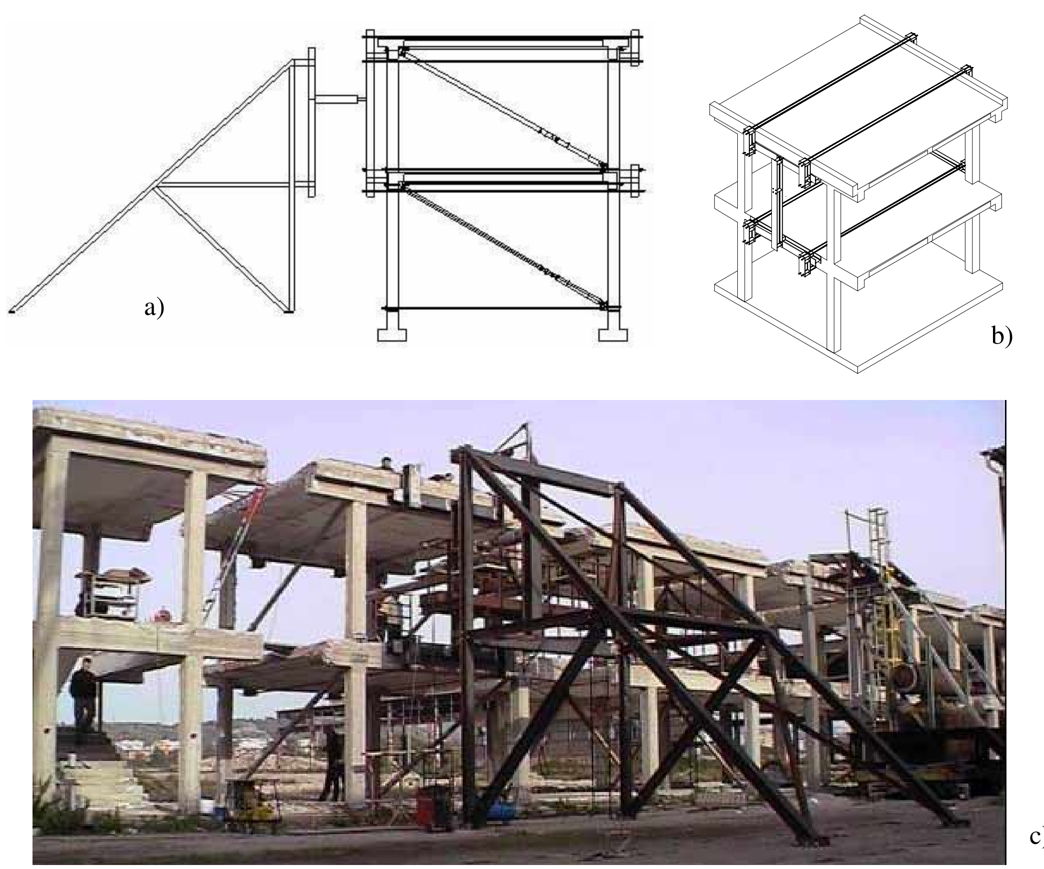


Figure 5 – a) Scheme of the reaction structure, b) the linking system and c) overall view

Thus the applied force simulates a force distribution derived from a linearly increasing acceleration distribution along the height. The vertical beam was linked to the slabs with hinge restrains, in order to allow independent displacement of the two floors and perform both pull and push tests.
The push-pull and the release systems were designed and realised by the USB group. It consisted of a double effect Enerpac hydraulic jack, with a force capacity of ±570 kN and 256 mm maximum elongation. For the cyclic test the jack was provided with a double orthogonal hinge at the top and bottom, linked to the model with a steel extension, as in figure 6. With only few operations there was the possibility to change the extension and to insert the device for the release dynamic tests. As shown in figure 7, this device was made of a steel 45 mm diameter threaded bar, connected to two circular plates, which were rigidly connected to the head of the jack and to the loading beam of the structural model.

Figure 6. Push-pull system.

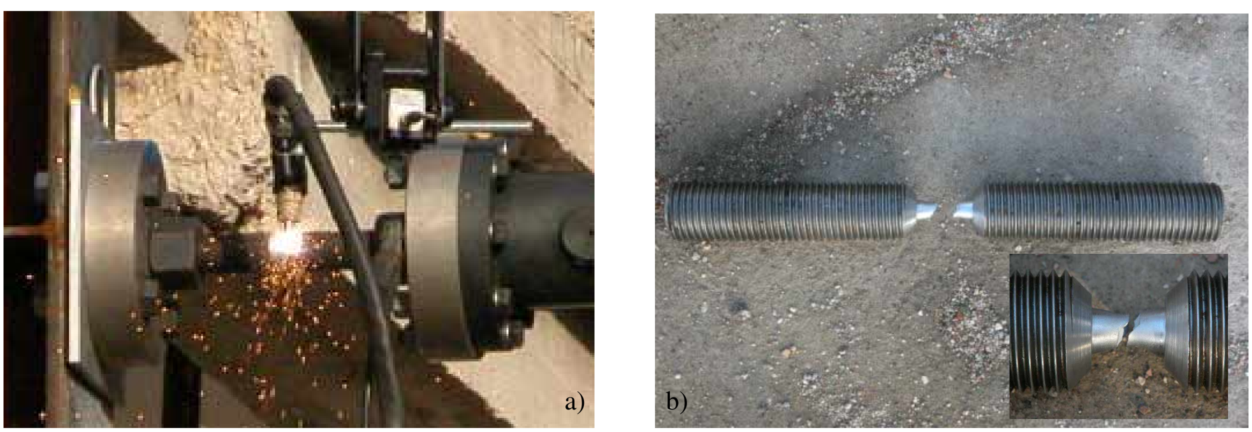
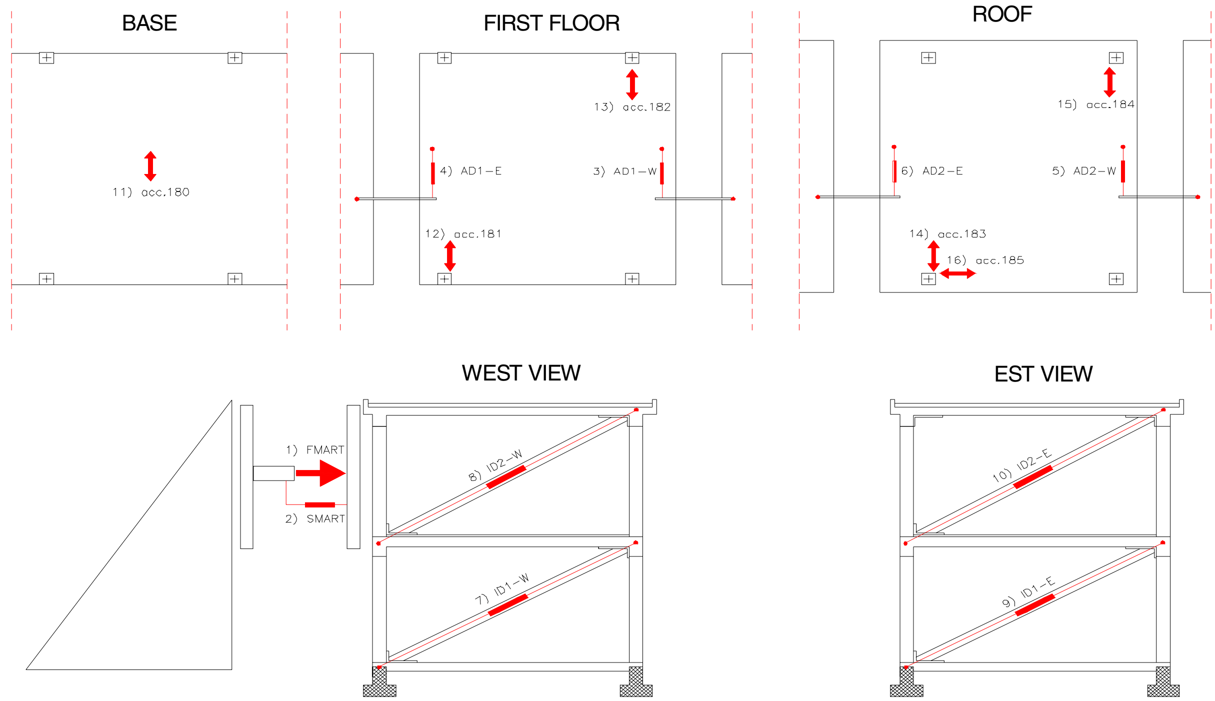


Figure 7. a) Cutting of the steel release bar and b) Release bar after failure.

Tensile failure tests were carried out on a series of steel threaded bars, in order to evaluate accurately the steel strength and the area of the restricted section to facilitate the failure and then the release for the desired values of the total force of the jack. This force, for each deformation level of the test, was determined from the cyclic tests performed on the model before the release dynamic tests. The force was then increased by about 10% and the corresponding restricted section area calculated, in order to avoid an early failure and unexpected release. Obviously the test was carried out by pulling the structure up to the target displacement was reached. Then the steel bar was cut by a gas-based cutting system (figure 7a), to release the structure and let it move freely. To set up again the system for a new test, just the change of the steel bar was needed, requiring only few minutes.

ACQUISITION INSTRUMENTS

The structural response was recorded during the tests by means of a series of instruments mounted on the floor slabs of the frame, on the braces and on the hydraulic jack. As can be seen in figure 8, four Celesco wire transducers (two on each floor), fixed on the near structural models, were used for the acquisition of the floor displacements.



\# 4 wire transducers for the floor displacement acquisition ±127 mm
\# 4 staff transducers for the braces deformation acquisition ±125 mm
\# 1 transducer for the hydraulic jack displacement ±125 mm
\# 1 load cell for the acquisition of the applied load ± 570 kN
\# 6 Servo-accelerometers ±1 g sensibility

Figure 8 – Arrangement of the acquisition instruments.

The displacements of the braces were recorded by means of potenziometric transducers. The jack was instrumented with an external wire transducer and a load cell. The acceleration of the model during the release tests were monitored by 6 servo-accelerometers with ±1g sensibility, arranged according to the scheme of figure 10.

EXPERIMENTAL PROGRAM

The first part of the experimental program included quasi-static cyclic tests on the structural model retrofitted with the SMA braces. The tests were carried out, by making three complete cycles for each

value of the target displacement, referred to the displacement on top of the structure. In table 1 there are reported the target displacement and the number of cycles for each test. It can be noted that the maximum value of the displacement was 90mm. Despite of the manual control of the hydraulic jack, the displacements were reached with good precision.
The aim of these tests was to check the actual cyclic behaviour of the structural system and of the braces, the effectiveness of the braces in their re-centring function, as well as to test their deformation limits, beyond the 25 mm design displacement.
After these cyclic tests, a series of dynamic release test with a start displacement of about 70mm, slightly varied in the different tests, were made.
Two release tests on the bare frame were also made, in order to evaluate the difference of behaviour with respect to the condition of braced structure, although in only one of these tests exploitable data were obtained, due to the malfunctioning of some instruments.
After these dynamic tests, a new series of quasi-static tests were made on the bare frame, bringing the structure near to the collapse condition. The maximum displacement reached at the 1st floor was 125 mm, leaving the structure with large residual deformations and a permanent inclination of the vertical structural elements.

Table 1 - Near static cyclic test program.

Test #	# of cycles	Maximum dipl. (mm)	Arrangement
1	3	±12	Braced structure
2	3	±30	Braced structure
3	3	±50	Braced structure
4	3	±70	Braced structure
5	1	±90	Braced structure
6	3	±90	Braced structure

Table 2 – Dynamic release test program.

Test #	Initial displ. (mm)	Arrangement	Date
1	71.28	Braced structure	04/12/03
2	70.96	Braced structure	10/12/03
3	77.72	Braced structure	10/12/03
4	74.74	Braced structure	15/12/03
5	86.13	Braced structure	15/12/03
6	70.57	Bare structure	16/12/03

MAIN RESULTS OF THE QUASI-STATIC CYCLIC TESTS

In figures 9 and 10 there are shown the force-deformation relationship of the entire structure and of the brace system, at the two stories. The total base shear considered in the diagrams of fig. 9 corresponds to the total hydraulic jack force. The forces acting on the braces at the two stories in fig. 10, are evaluated by considering the arrangement and the position of the hydraulic jack. Therefore the force at the 2nd floor and the 1st floor were considered to be, from equilibrium conditions, respectively 2/3 and 1/3 of the total base shear. In the following graphs the force-displacement behaviour in terms of 1st and 2nd level forces and displacement, are presented, with reference to test n# 3.
Some irregularities in the force-displacement curves, due to dynamic effects, can be noted, caused by the lack of control of the oil flow in the hydraulic jack during the unloading phase. However, the small energy

dissipation is apparent, due to both the characteristics of the SMA devices and the great flexibility of the R/C frame, in spite of the high imposed displacement. The overall behaviour of the structure shows an excellent re-centering capacity, with practically no residual deformation when the load is removed. This feature is better shown in figure 10, which represents the force-displacement curves of the re-centring devices of each couple of braces at the two stories.

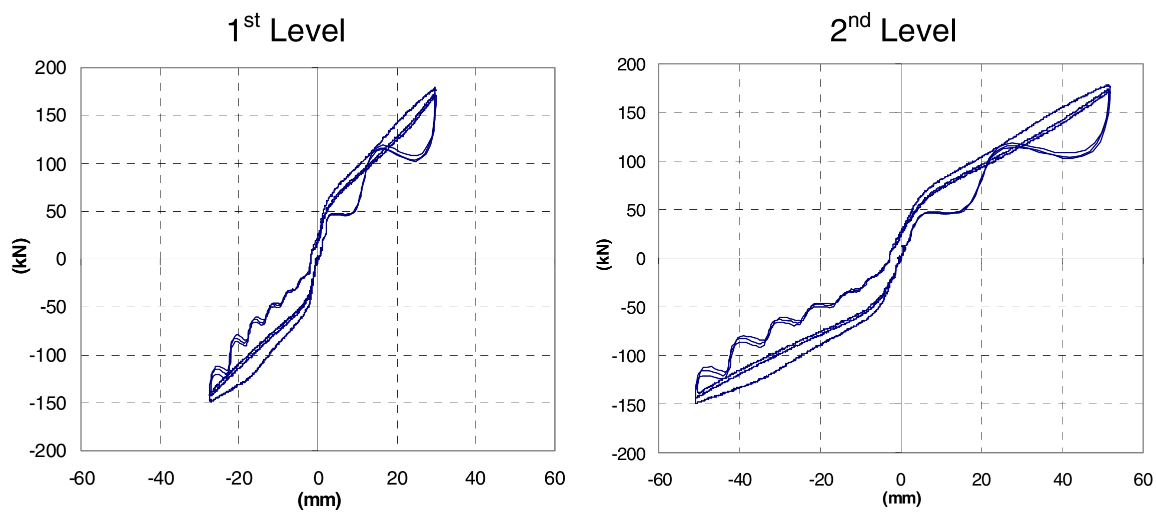


Figure 9 – Force-displacement behaviour of the structure during the cyclic test #3.

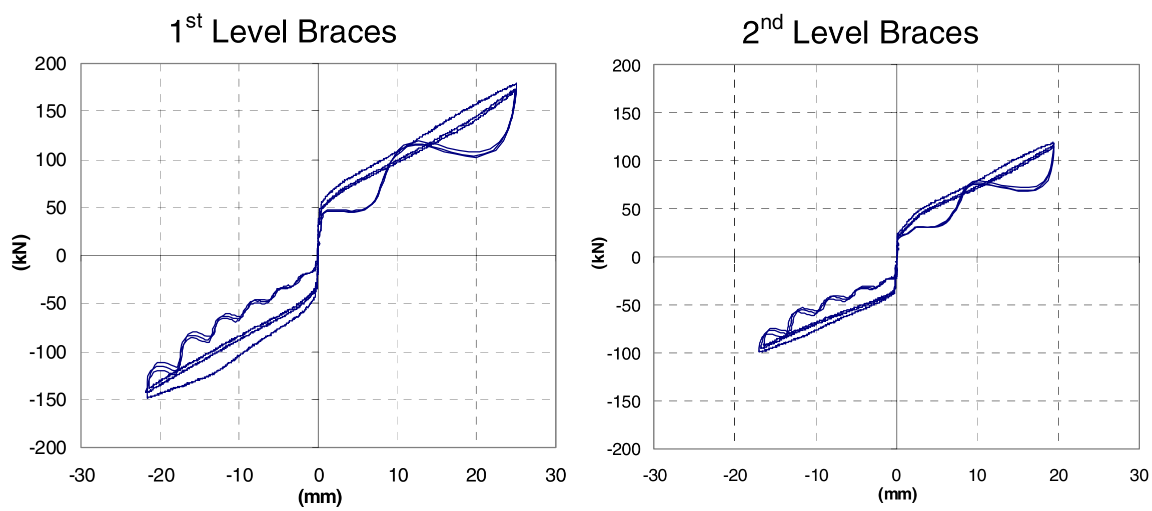


Figure 10 –Force-displacement behaviour of the couple of braces during the cyclic test #3.

During test #5 at 90 mm displacement, the failure of few SMA wires in one of the 1st level braces occurred. The difference in the total recorded force before and after the wire's failure indicates that about 50% of the wires of one brace became ineffective. It was decided to go on with the tests, also to verify the effectiveness of the damaged protection system. Therefore, a further cyclic test was made at 90mm amplitude, giving a good behaviour, although the 1st level static re-centring capacity was reduced.

MAIN RESULTS OF THE DYNAMIC RELEASE TESTS

The model was moved up to the release start position, 71.3 mm for the first release test and 70.6 mm for the final test, then the cut system was activated and in few seconds the 17mm calibrated bar section was cut to produce the instantaneous failure and the release of the frame.
In the dynamic tests, the main check point was the top of the structure. As already said, there were two displacement transducers placed on the roof. The results are then reported as the mean values of the two displacement time histories.
In figure 11 the responses of the braced frame and of the bare frame, respectively test #1 and test #6, are compared. The main effect of the bracing system is a faster reduction of the displacement amplitude.
Another interesting outcome is represented by the peak roof acceleration reached during the dynamic test (figure 12). In test #1 and test #6 the peak roof accelerations were respectively 0.42g and 0.27g respectively. Also comparing the accelerations and corresponding displacements in the following peaks, it turns out that the mass accelerations that can cause the same structural displacement, and then damage to structural and non structural elements, is much larger in the case of the braced frame.

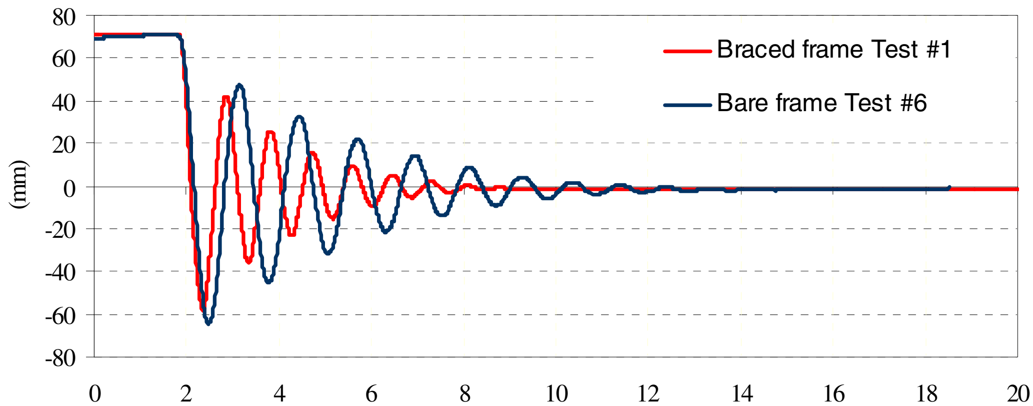


Figure 11 – Roof displacement comparison between braced and bare frame.

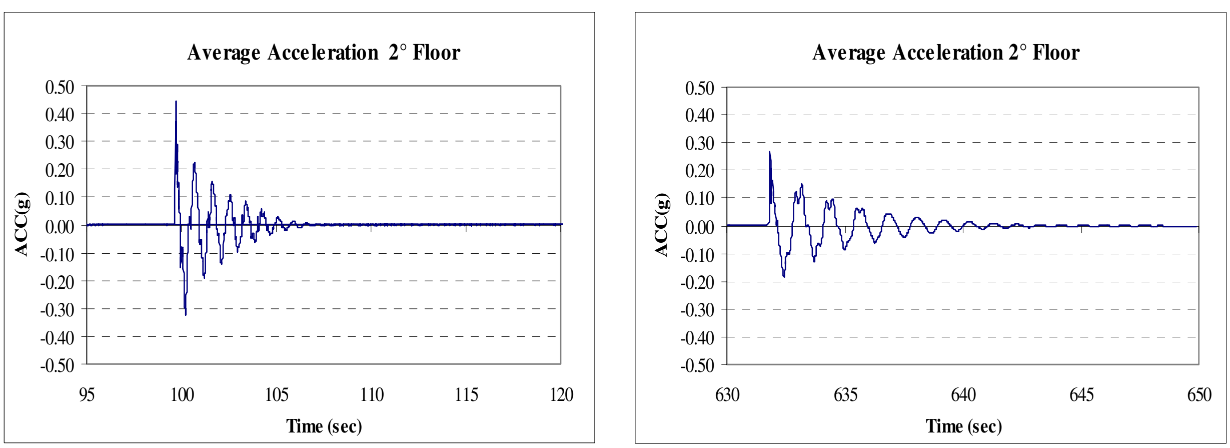


Figure 12 – Roof acceleration comparison between braced and bare frame, a) test #1 Braced frame, b) test #6 bare frame

Based on the recorded data, also the effective structural damping was calculated, by assuming that the structure was a SDOF system, in which the considered degree of freedom was the roof displacement in the release direction.

The diagram of figure 13 shows the effective total damping exhibited by the system structure and braces for all the release tests, computed using the well known formula:

$$\xi = \frac{1}{2\pi} \ln \frac{v(t)}{v(t+T)}$$

that produce the evaluation of the effective damping amount through the peak displacement of the roof, *v(t)* and *v(t+T),* at the times t and *t+T*, where *T* is the fundamental vibration period. In each time history, the first five cycles were considered and the greatest displacement for every test, that can be seen in figure 13, is the second relative peak of the roof displacement. Apart from test #5, which is the one relevant to the greatest reached displacement and present an effective damping value of about 14% at large displacement, all the other test show damping values of the order of 6-8%, with decreasing values as damage progresses.

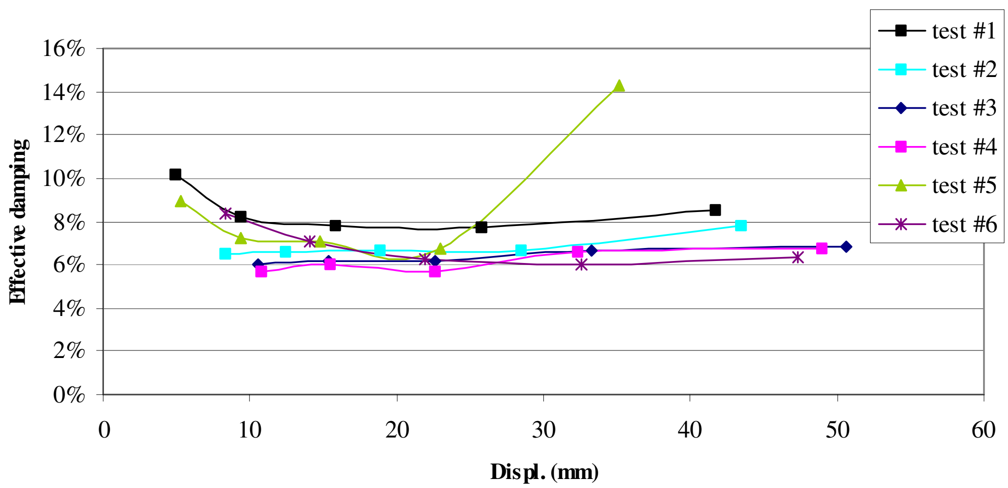


Figure 13 – Roof displacement comparison between braced and bare frame.

Another consideration can be made with regard to differences between large and small displacements. The contribution of the braces to the total effective damping is greater at large displacement due to the little hysteresis loops. The absence of this contribution at low displacement is more than recovered by the damping of the r/c members due the crack closure.
Figure 14 shows the Gabor transform (Gabor [11]) of the acceleration signal recorded at the second level in the release direction. This representation of the structural frequencies permits to appreciate the variation of the fundamental frequency during the free oscillation. The fundamental frequency of the braced frame for large deformation was about 1.2 Hz, as a consequence of the non linear behaviour of the structure and of the bracing system, increasing up to 2.4 Hz for very small oscillations.
After the release tests, the 4 braces were dismounted, however leaving the connection devices, which somehow modify the stiffness and strength characteristics of the columns. The bare frame was the subjected to further cyclic tests, in order to verify its mechanical characteristics, without overcoming the maximum previously reached displacement. Then, a release test was made, giving the fundamental frequency variation shown in figure 15. The minimum value of frequency, recorded in correspondence of the maximum displacement, was about 0.75 Hz, while the final frequency increases up to 1 Hz. The comparison of figures 14 and 15 confirm the strong influence of the braces on the overall behaviour of the model, as well as their strongly non linear behaviour, resulting in a much larger variation of fundamental frequency during free oscillation.

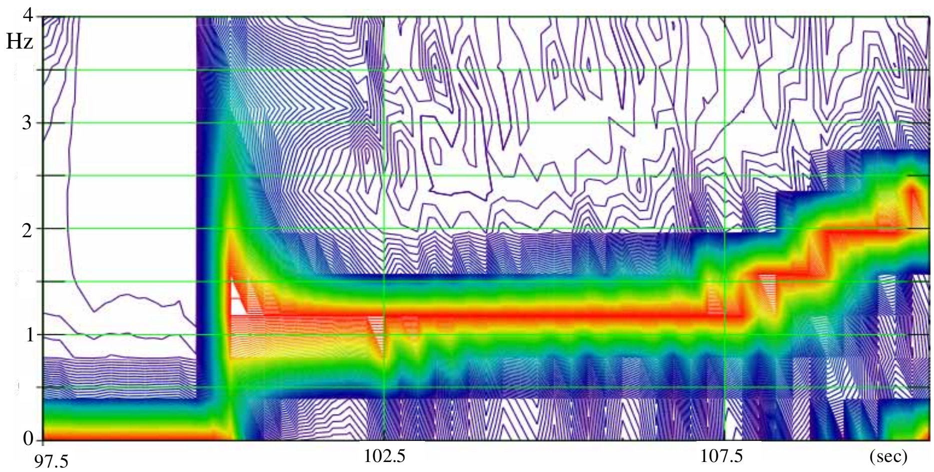


Figure 14 – Gabor Transform of the braced frame release test #3.

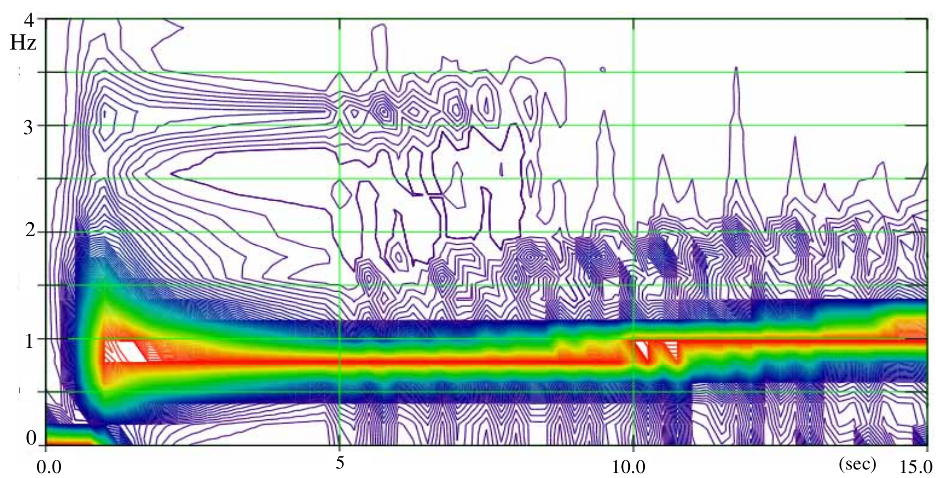


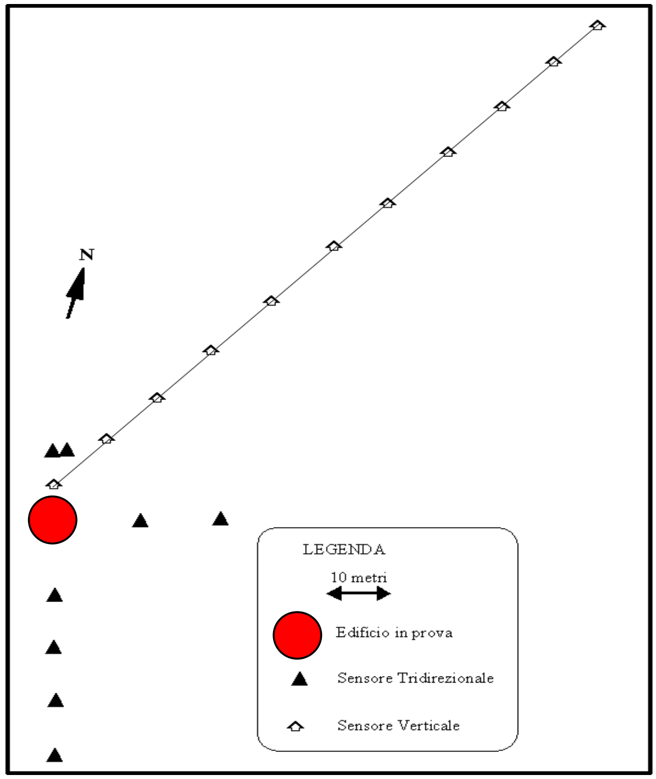
Figure 15 – Gabor Transform of the bare frame release test #6.

SOIL INDUCED SEISMIC WAVES

Within the same experimental program, the propagation in the soil of the seismic waves due to the vibrations of the structure during the dynamic tests was measured, in order to evaluate the possible interactions between close structures during earthquakes (Dolce [12]). In collaboration with the National Institute of Geophysics and Vulcanology (INGV) and the University of Bologna, MANY seismometric sensors were positioned around the test building, with the aim of understanding the way in which the structure transfer its energy to the ground and the attenuation of it with the distance. The acquisition instruments, shown in figure 16, were:

1. In the near field (5 m) one accelerometer Etna Kinemetrics and one velocimeter Nakagrilla;
2. One array L shaped composed by 7 three-directional sismometers, 6 of which at 1s (Mark L4C) and 1 at 5 s (Lennartz), the distance between the seismometers was 15 and 10 m in the longitudinal and transverse directions respectively.
3. One linear array composed by 11 vertical geophones at 11 Hz, equidistant at 10 m from 5 to 105m;

The more interesting results up to now elaborated, evidenced by recorded data, is the peak to peak acceleration in the near field, that was equal to about 0.10g in the dynamic test no.5, in which the initial displacement was greater than 85mm.



16 – Plan arrangement of the seismometric acquisition instruments.

As can be seen in figure 17, probably due to the wavelength of the signal, the maximum value has been recorded at 10 m rather than at 5 m. Further confirmations of this fact, that can increase the evaluation of the induced PGA, can be obtained by the analysis of the three-directional seismometers array.

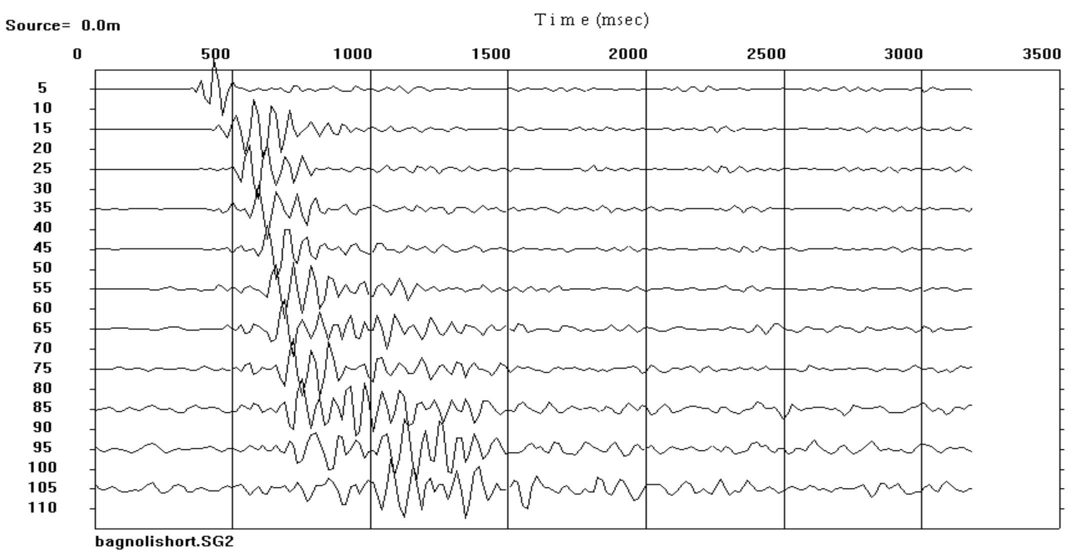


Figure 17 - Seismic waves recorded by the geophones .

CONCLUSION

Within the research projects TREMA and ILVA-IDEM, both aimed at studying and testing new techniques to retrofit existing buildings, some experimental tests have been carried out on an existing old 2 storey, one-by-one bay, R/C structure to be demolished, which was designed in the seventieth for gravity loads only. The experimental tests were aimed at assessing the cyclic behaviour of the retrofitted structure. The retrofit design was based on the use of special braces, whose re-centring characteristics was determined by the superelastic properties of Shape Memory Alloy austenitic NiTi wires, which were the main components of the device included in each brace. The installation of the braces has proved to be easy and fast. Quasi-static cyclic tests and release tests were carried out on both the bare and the braced frame. Preliminary design dynamic non linear simulations showed that the upgrading system is able to allow the structure to withstand the design intensity earthquake with interstorey displacements of the order of 25 mm. This was the target displacement assumed in the design of the SMA-based devices. During the cyclic tests, displacements up to twice the design displacements were reached, producing the failure of part of the wires of one brace. The subsequent release tests were therefore carried out with the retrofit bracing system partially ineffective at the first floor.

The first results here presented demonstrates that the passive protection system based on the use of SMA wires behaves as expected, and is therefore able to provide the tested R/C structure with a strong re-centring capability, increased safety at the Ultimate Limit State and displacement control at the Damage Limit State. It should also be remarked that in the direction of the test there was no frame and that the retrofitting system alone was able to fully protect the structure, with no unpredicted parasite effects.

Further processing of the experimental data will provide more detailed information on the behaviour of the tested structural system and of the single braces. However the first results show that the real behaviour well corresponds to the predicted one, also due to the preliminary experimental tests carried out directly on the devices.

The results of the tests and of the preliminary numerical analyses need to be extended and generalized, by making numerical parametric analyses on the structural model, set up on the base of the experimental results. Actually several seismic inputs and different retrofitting strategies shall be considered. In the present case the retrofitting strategy was based just on the strong re-centring capability of the braces, whose energy dissipation capacity is very low. Energy dissipation to limit displacements is then relied upon some inelastic excursion of the R/C structure. However, further energy dissipation capability can be easily provided to reduce displacements, by simply adding some dissipative elements or devices to the brace system. It will be interesting to check the behaviour of the structure under study, being very flexible and having little energy dissipation capacity.

A specific concern is deserved by the seismometric acquisitions, which have confirmed that the structural vibrations can induce propagation of further seismic waves in the ground and, then, produce non negligible interactions among nearby buildings. Further processing of the huge amount of records will considerably improve the knowledge of these phenomena, whose detailed experimental evaluation is rarely feasible.

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