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Seismic upgrading of existing RC buildings by advanced techniques

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ABSTRACT: Reinforced concrete (RC) buildings constitute a significant part of the existing building stock. Many of these buildings have been built in a period of time characterized by either weak code regulations for seismic design or earthquake hazard underestimation at the building site. Consequently, many of them do not meet current criteria of civil protection against damage induced by strong earthquakes. The engineer is currently aware of these lack of safety and is in principle able to reduce the seismic risk by many different technical systems. This paper presents recent experimental results about the use of two systems for seismic upgrading of RC buildings: (i) strengthening the existing RC members by means of composite fiber reinforced materials and (ii) improving the global structure performance by inserting steel dissipative braces.

1 INTRODUCTION

1.1 General

Past earthquakes, such as the last "Abruzzo" earthquake occurred on the 6th of April 2009, clearly prove that even structures designed according to modern codes could suffer severe damage. This is particularly true for RC frames, which have been shown to be significantly vulnerable. Many technical solutions are currently available for the seismic rehabilitation/upgrading. Basically, the following two types could be identified: a) systems based on the addition of new lateral-load resisting elements; b) systems based on the repair/retrofit of existing elements. Typical examples of type a) systems are (i) RC walls, (ii) steel braces, (iii) viscous dampers, (iv) base isolation. Typical examples of type b) systems are (i) steel and/or concrete jacketing, (ii) concrete confinement by composite fiber reinforced materials. Each type of seismic protection system presents its own advantages and disadvantages. For example, in case of a building suffering for an irregular seismic response, type a) systems are particularly suited, because they can eliminate torsion or superior-mode deleterious effects. On the other hand, type a) systems are often invasive, requiring the closure of openings or the change of the architectural aspect. Contrary, type b) systems can often be made without interruption of the functions of the building and are often more economic. However, the use of advanced technologies may permit to keep the benefits of the type a) systems along with the advantages of the local-type intervention systems, i.e. non modification of the aesthetic appearance of the building and minor disruption to the building occupants. Examples of such possibilities are given in this paper, which presents and discusses three types of seismic upgrading techniques:

1. Carbon fiber reinforced polymer (C-FRP) materials.
2. Steel eccentric braces (EB).
3. Steel buckling-restrained braces (BRB).

1.2 Research outline

The research consisted of full-scale tests on two RC buildings (Fig. 1). The buildings were both located in Bagnoli (Naples, Italy), in the area where competent Authorities destined to demolition the plants of the steel mill named ILVA (former Italsider). The first building (Fig. 1*a*) was preliminarily divided into six independent substructures, by removing all non-structural components and cutting the floor slabs. Then, each substructure was equipped with a different seismic upgrading system (Fig. 2) and subsequently tested more than one time. The whole experimental activity is described in Mazzolani (2006). This paper summarizes the tests carried out on those substructures strengthened by composite materials, steel buckling-restrained braces and steel eccentric braces. The second building (Fig. 1*b*) was initially tested two times: (i) in its original conditions, up to a strong damage state, and (ii) after some minor repair of damaged parts of concrete, reconstruction of perimeter masonry infill walls and their strengthening with C-FRP bars placed in the horizontal mortar joints of the masonry infills. A detailed description of the two tests can be found in Della Corte *et al.* (2008). Subsequently, the damaged building was equipped with three novel BRB prototypes, each of which tested up to failure. The main characteristic of the novel braces is the possibility to hide them within the space between the two panels of masonry infill walls commonly used for claddings of RC buildings. Then, the system avoids any change of the architectural appearance of the original building. The tests are here presented per category. This implies that tests on FRP material systems, which have been carried out on two different RC structures, and similarly tests on BRBs, are presented together.

a) *b)*


Figure 1. The first (*a*) and the second (*b*) building in their original conditions before testing.

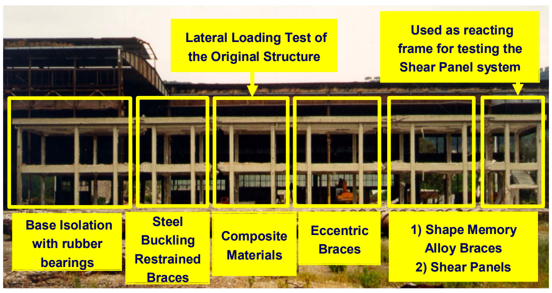


Figure 2. The first building divided into six independent substructures.

2 COMPOSITE MATERIALS

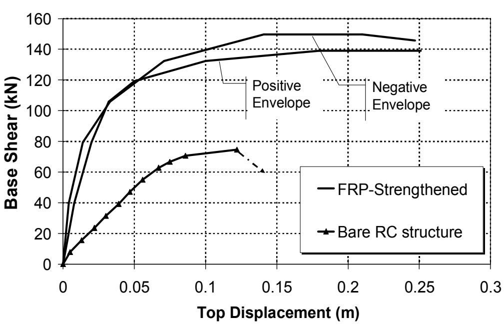
2.1 Experimental results

The bare RC substructure of building No. 1 (third from the left in Fig. 2) was first tested in its original conditions and then repaired, strengthened and tested again. The strengthening system consisted of longitudinal C-FRP strips and transverse C-FRP sheets. Figure 3*a* shows the FRP strengthened structure, illustrating also the test setup. Figure 4 shows a comparison of response between original and upgraded structures, illustrating the column-sway collapse mechanism of the original structure (Fig. 4*b*) and the beam-sway mechanism of the C-FRP upgraded structure (Fig. 4*c*).

The second application refers to the masonry infill panels of building No. 2 (Figs. 3*b*). C-FRP bars were placed into the horizontal mortar joints (Fig 3*c*). The original building was first tested under full load reversals. Subsequently, some damaged portions of RC columns were repaired and perimeter masonry infill panels were rebuilt and strengthened using the structural repointing technique. Figure 5 shows a comparison of response between the original (test #1) and the repaired (test #2) structure. In both cases, a first story column sway mechanism was apparent. When looking at such a comparison, it should be considered that the second test was on a damaged structure, because the staircase and all the interior partitions, as well as the internal RC columns were not repaired. Test results show that the failure mode of masonry panels changed from the diagonal tension cracking mechanism, mainly observed during the first test, to the shear sliding mechanism, which was dominant during the second test (Fig. 6). A detailed description of these experimental results, as well as information about analytical modelling of test structures, can be found in Della Corte *et al.* (2006*a*, 2008) and Mazzolani *et al.* (2007).

a) 

Figure 3. The C-FRP repairing/upgrading systems: *a*) building No. 1; *b*) and *c*) building No. 2.



a)

b)

c)

Figure 4. Base shear vs. roof displacement for the bare and FRP repaired/reinforced structure.

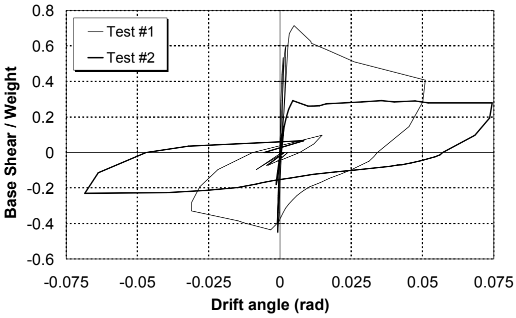
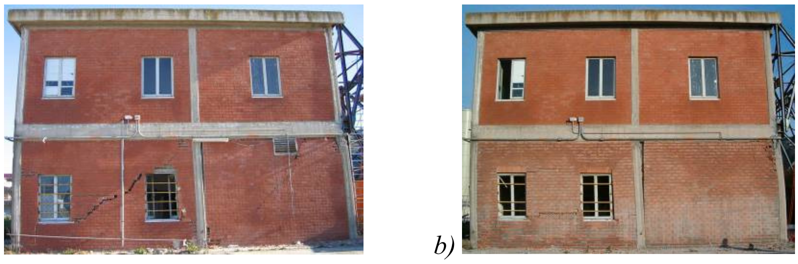


Figure 5. Base shear vs. 1st story drift angle for the original and repaired building.



a) b)

Figure 6. Failure of masonry panels: a) test on the original building; b) test with FRP strengthened panels.

3 ECCENTRIC BRACES

3.1 Experimental results

Figure 7 illustrates the EB system. Three experimental tests have been carried out. The link cross-section as well as link end-connection details were changed from one test to another. In the first EB system, flexural failure of link end-plate connections occurred (Figs. 8*a*, 8*b*, 8*c*). In the second test, link end-connections were strengthened, increasing the end-plate thickness. The response showed now failure of bolts at the link-to-brace connections (Figs. 8*d*, 8*e* and 8*f*). In the third EB system, a weaker link built-up section was designed and the bolt steel grade was increased. The structure showed significant plastic shear deformation of links (Figs. 8*g* and 8*h*). At large link rotations, shear failure of bolts at the bottom connection was again observed (Fig. 8*i*). Further details as well as an explanation of the reasons for this behavior can be found in Della Corte and Mazzolani (2006*b*) and Della Corte *et al.* (2009*a*, 2009*b*).

a) b)

Figure 7. The RC structure equipped with eccentric braces: global view (*a*) and zoom in to the link (*b*).

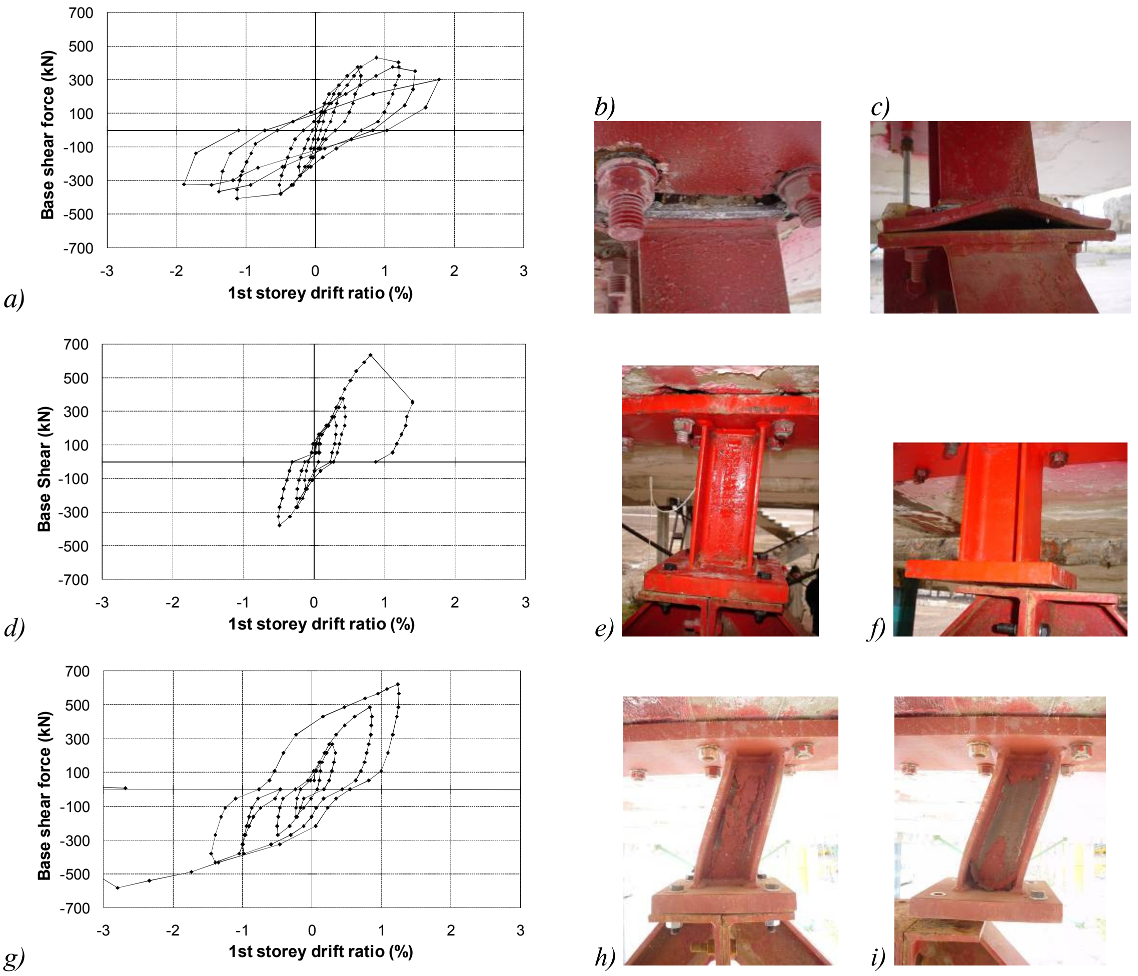


Figure 8. Test results for three EB systems.

4 BUCKLING RESTRAINED BRACES

4.1 Experimental results

Novel types of "only-steel" BRBs have been designed and tested. Two tests were conducted using substructure No. 2 of building No. 1 (Figs. 1*a* and 2). Three more tests were carried out using building No. 2 (Fig. 1*b*).

Figure 9*a* illustrates the RC substructure equipped with BRBs (second from the left in Fig. 2*b*). Two slightly different types of BRBs were tested. In the first type (Fig. 9*b*) two buckling-restraining tubes were connected by welded plates and the gap between the restraining tubes and the yielding core was specified to be 0.5 mm at the design stage. Figures 10*a*, 10*b* and 10*c* show the structure response. There was yielding of the BRB core in both tension and compression, but local compression buckling of the end portion of the core produced large local plastic deformation (Figs. 10*b* and 10*c*). In the second type of BRB (Fig. 9*c*), the gap between the core and the restraining tubes was fixed to 1 mm and bolted connections were used between tubes. Besides, a more gradual tapering was designed for the inner core end portions (Della Corte and Mazzolani 2006*b*, Mazzolani *et al.* 2009) The structure response is shown in Figures 10*d*, 10*e*, 10*f*. The dark part of the BRB core visible in Figure 10*e* highlights the relative displacement between the core and the restraining tubes. Figure 10*f* shows the high-order inelastic buckling of the inner core, which is expected to be a normal response of this system.

Figure 9. The RC structure equipped with BRBs: global view (*a*) and zoom in to the BRB 1 (*b*) and 2 (*c*).

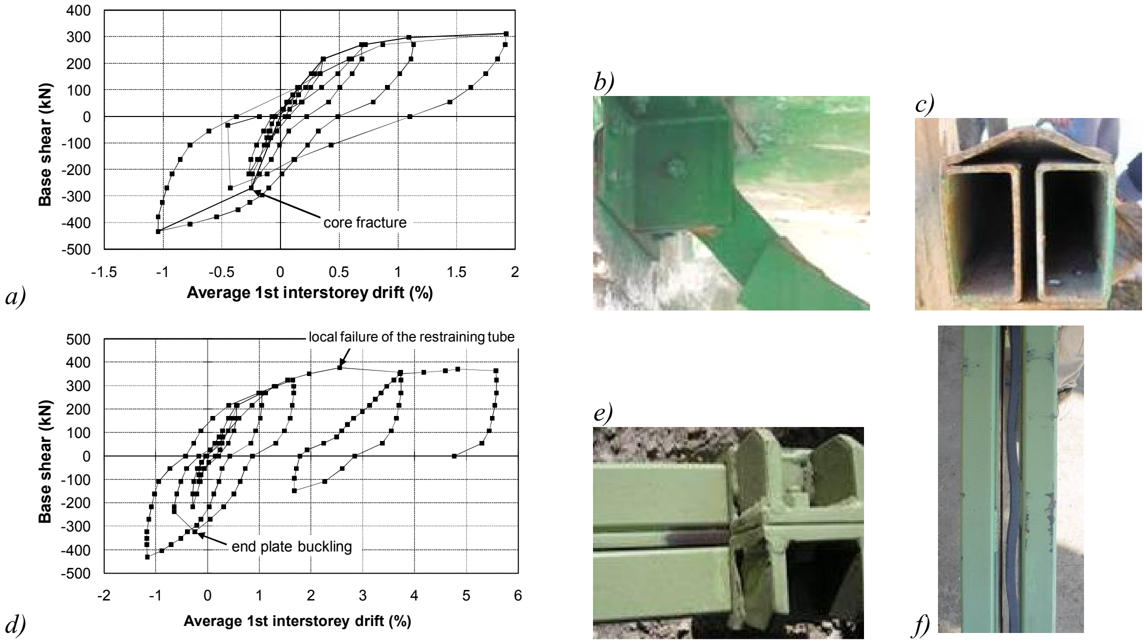


Figure 10. Tests results for two BRB systems.

With reference to the second building, three BRB prototypes were tested (BRB No. 3, No. 4 and No. 5). The novel BRBs were designed to be hidden in the inner space between the two panels of masonry claddings (Fig. 11). The performance of BRB No. 3 was not satisfactory. Indeed, at an interstorey drift ratio equal to about 1.25% a local-distorsional buckling failure of the unrestrained end portion of the brace occurred, corresponding to a peak of the lateral strength (Figs. 12*a*, 12*b*). Buckling out-of-plane, the brace pushed against the external facing wall, leading to its complete collapse (Fig. 12*c*). BRB type 4 showed a satisfactory performance, with stable and symmetric hysteresis for interstorey drift ratios in the range ±1.5% (Fig. 12*d*). When the maximum displacement capacity of the tested device was achieved (Fig. 12*e*), two secondary failure mechanisms occurred: (i) local buckling and related plastic bending of the steel plates constituting the restraining sleeve (Fig. 12*f*); (ii) global brace buckling (only for one among the four braces, not shown in Figure). BRB type 5 showed an excellent response (Fig. 12*g*) with a fully operational and symmetric response up to an interstorey drift ratio equal to ± 3% (Fig. 12*h*). Figure 12*i* shows the typical high-order buckling of the restrained core.

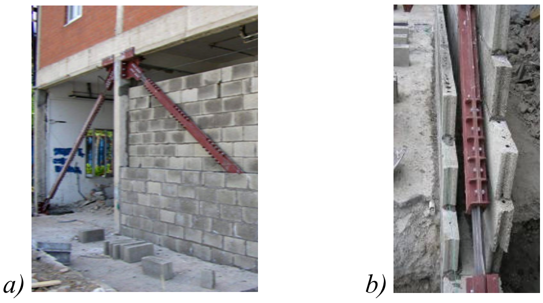


Figure 11. Building no. 2 equipped with BRBs: global view (*a*) and zoom in to the BRB (*b*).

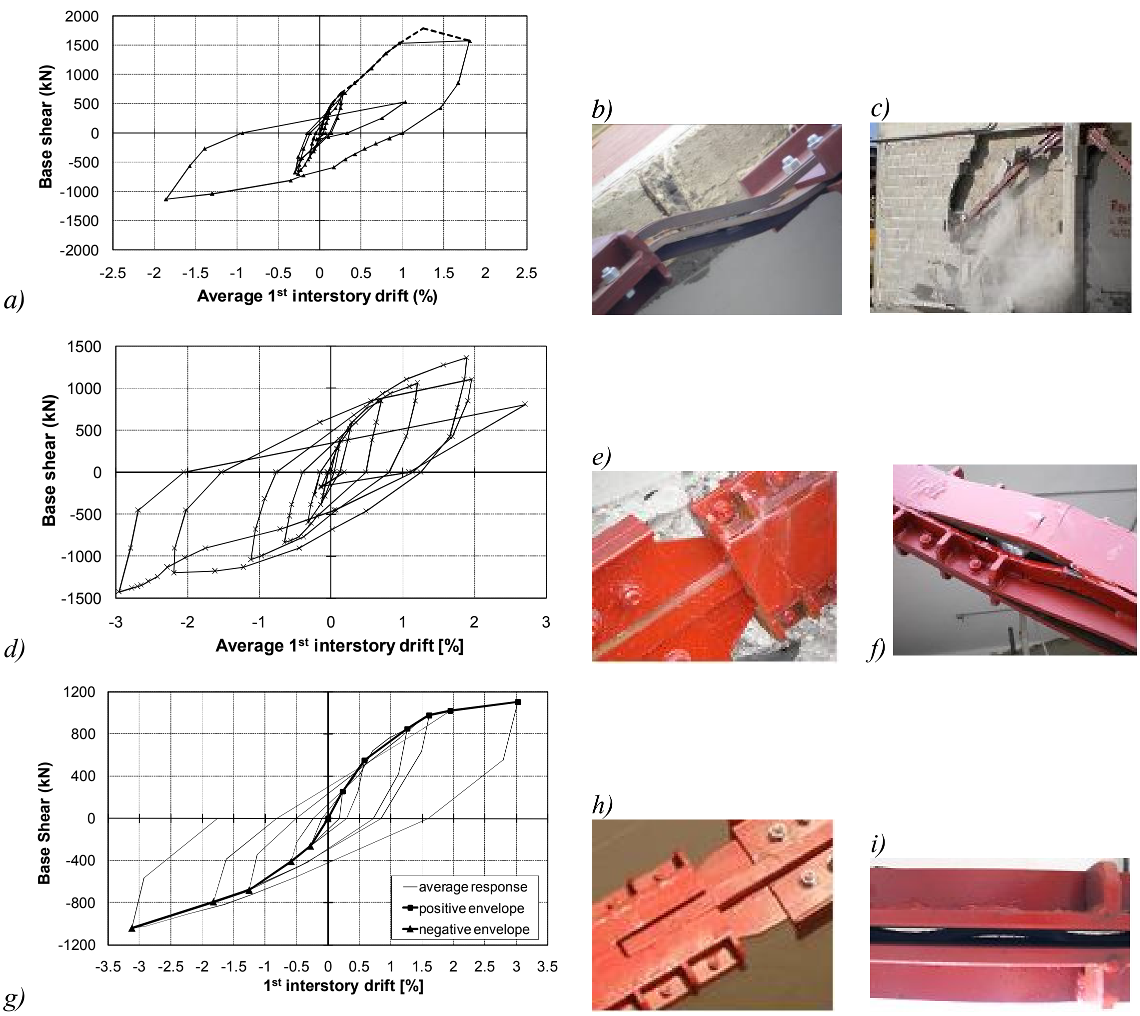


Figure 12. Test results for three BRB systems.

5 CONCLUSIONS

Figure 13*a* shows in one single plot the envelopes of base shear vs. roof displacement relationships obtained from tests on the first building. The C-FRP strengthening system appreciably improved the stiffness and strength, but its main peculiarity is the large increase of deformation capacity. With steel eccentric braces a very large gain of stiffness and strength can be obtained, but the deformation capacity is quite limited. Buckling restrained braces emerges as a compromise between the two systems. Results obtained with the second building equipped with BRBs are compared in Figure 13*b*. The Figure highlights that buckling still occurred in case of BRBs No. 3 and No. 4, at large deformation demand, but outside of the design deformation range. A completely stable response was obtained for BRB No. 5 up to failure by low-cycle fatigue effects. Results reveal that BRBs performance was always acceptable, though they need to be appropriately detailed to obtain a fully predictable dissipation mechanism.

a) *b)*

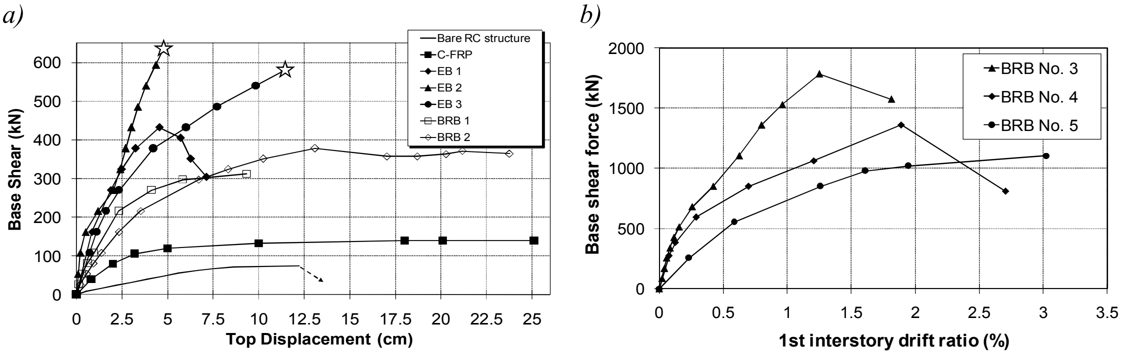


Figure 13. Comparison of test results.

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