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EXPERIMENTAL ANALYSIS OF STEEL DISSIPATIVE BRACING SYSTEMS FOR SEISMIC UPGRADING

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Abstract. Energy dissipating devices, such as metallic ductile dampers, could represent one reliable system for seismic performance upgrading of reinforced concrete (RC) structures. This paper illustrates the significant improvement to the seismic response of RC structures equipped with dissipative bracing systems, such as eccentric braces (EBs) and buckling restrained braces (BRBs). In fact, the results of experimental tests carried out on two similar two-storey one-bay RC structures, respectively equipped with EBs and BRBs, are described. Referring to EBs, 3 lateral loading tests have been performed. Each test is characterized by shear links with bolted end-plate connections. Different design criteria have been applied in the design of the connections. In the first test, capacity design criteria have not been considered. In the second test, a capacity design criterion has been applied, with a link shear over-strength factor equal to 1.5. In the third test, a design criterion similar to the one adopted for the second test has been implemented, but with a larger over-strength factor. In case of BRBs, two types of 'only-steel' braces have been tested: one type was made using two buckling-restraining rectangular tubes that are fully welded together with steel plates; the other type is detachable, being made again with two buckling-restraining rectangular tubes but joined together by means of bolted steel connections. In both cases, the internal yielding core was a rectangular steel plate. The experimental results of both bracing systems are encouraging about the possibility to use these devices for improving the seismic resistance of existing RC structures.

Keywords: buckling restrained braces, capacity design, ductility, eccentric braces, experimental tests, seismic upgrading.

1. Introduction

Existing reinforced concrete (RC) frame buildings with non-ductile detailing represent a considerable hazard during earthquakes. This type of buildings suffered severe damages and were responsible for most of the loss of life during the major Italian seismic events such as the 1981 Irpinia earthquake. Improving the seismic response of this type of construction can be considered as one of the main concern for structural engineers.

Among seismic performance upgrading methods several options are normally available, one of which is to employ energy dissipation devices, such as friction, viscoelastic and metallic dampers etc. Energy input by a strong earthquake is expected to be greatly dissipated by these devices, and if they are damaged, they make the rehabilitation easy after the earthquake, since these devices are designed to be replaceable. In particular, this paper focuses on removable steel Eccentric Braces (EBs) and Buckling Restrained Braces (BRBs) as a seismic upgrading approach for protecting RC buildings from severe earthquake damage. The results of lateral loading tests performed on existing RC structures seismically retrofitted by EBs and BRBs are presented and discussed. The tests have been carried out within the context of a wider experimental research activity, named the ILVA-IDEM project, with the purpose to evaluate several innovative technologies for the seismic retrofitting/upgrading of existing RC structures (Mazzolani 2006).

The experimental part of this research activity started from the exceptional opportunity to carry out tests in the inelastic range of response on a real RC building (Fig. 1a). The building, which is located in Bagnoli (Naples, Italy), was destined to demolition by competent Authority, by the dismantling process of the Italian steel mill ILVA (or Italsider). In order to increase the potential number of specimens for testing different upgrading solutions, slabs were cut at the first and second floor, for dividing the whole building into 6 separate structures to be analyzed (Fig. 1b). Before cutting the slabs, exterior and partition walls, as well as any other non-structural element, were removed in order to get bare RC structures. Multiple tests for each of the investigated systems have been carried out, thus summing up to 15 full-scale tests, including 3 tests on the bare RC structures. The following is the complete list of the tested techniques and the corresponding number of tests:

1. Base isolation with rubber bearings (2 tests).
2. Buckling restrained braces (2 tests).
3. Composite fibre-reinforced materials (2 tests).
4. Eccentric braces (3 tests).
5. Shape memory alloy braces (3 tests).
6. Shear panels (3 tests).

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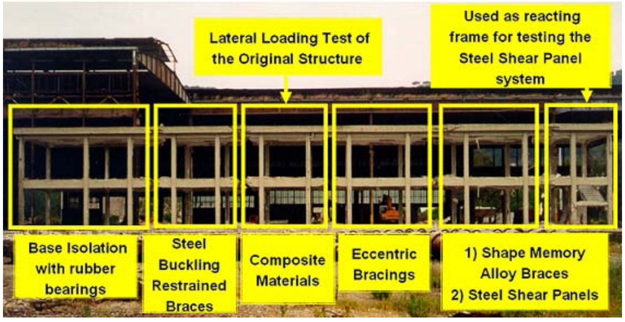
a)



b)

Fig. 1. The tested structure: a) original condition, b) the sub-structured building

A detailed description of the whole experimental activity can be found in Mazzolani (2006) and Della Corte & Mazzolani (2006).

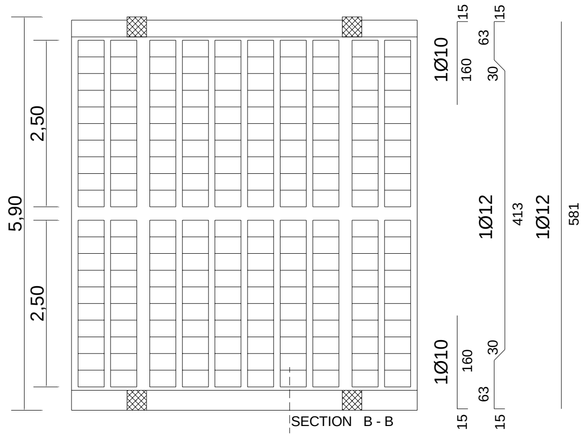
2. Description of the RC structure and the test setup

The generic RC sub-structure subjected to test is essentially constituted by 4 columns sustaining 2 floors (Fig. 2). Columns have a square 300 mm × 300 mm cross-section. The structure of the 2 floors can be essentially described as made of T-section beams going parallel in the transverse direction and supported by 2 longitudinal L-section beams. Column longitudinal steel rebars are in number of four, placed at the section corners and have a diameter of 12 mm. Transverse stirrups have a diameter of 8 mm and are spaced at about 200 mm.

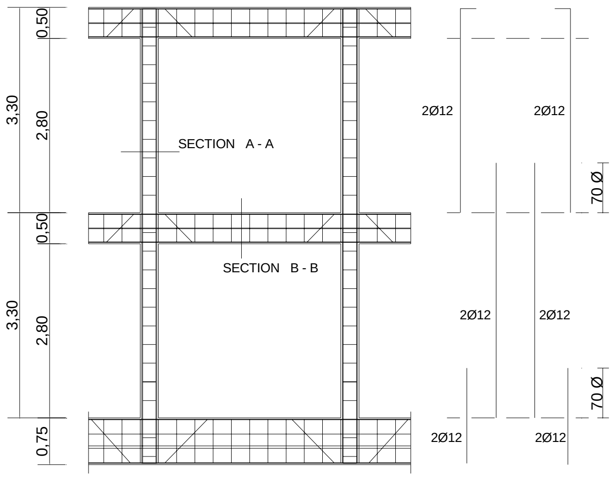
Fig. 2 shows the structural plan (Fig. 2a) and one longitudinal section (Fig. 2b) and also some local details of floor slabs, beams and columns of the generic RC substructure (Fig. 2c). Dimensions other than indicated in Fig. 2 are variable slightly from one substructure to the other, and they will be specified case by case.

The mechanical properties of materials were estimated both in-situ and in the laboratory, by means of tests on specimens, such as steel bars and concrete cores taken from the structure. The average cylindrical strength of concrete resulted to be about 20 MPa, while the average yield stress of longitudinal rebars in columns can be approximately fixed at 440 MPa.

Fig. 3 illustrates the test set-up, showing a global view of the reacting steel frame (Fig. 3a), close-up views of the 2 loading jacks used for applying the load in 2 opposite directions and the supporting steel beam (Fig. 3b, c). In particular, the vertical steel beam (Fig. 3c) was used for distributing the applied lateral force between the 2 stories of the structure to be tested in such a way to



a)



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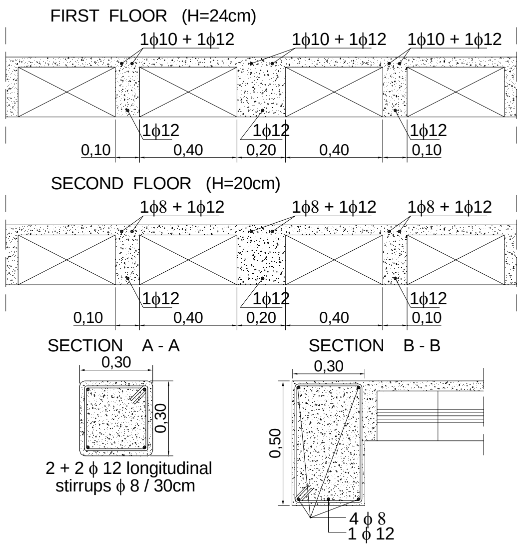

c)

Fig. 2. Local details (a); generic plan (b); vertical section of the generic RC sub-structure (c)

reproduce an inverted triangular lateral load pattern, as often assumed in theoretical pushover studies. The strengthened structure was subjected to a cyclic loading history up to the development of a clear collapse mechanism.

Floor displacements have been measured by means of a special video-camera device (Fig. 3d). This displacement-measuring device proved to give lateral displacements with the same precision of a topographic total station, which was used in a former test on one of the sub-structures.

a)

b)

c)

d)

Fig. 3. Test setup

3. Loading protocols

The loading protocols adopted for the tests can be considered as a compromise between technical operativeness and scientific needs. In fact, starting from the loading protocols suggested by AISC seismic provisions, for the sake of inexpensiveness, the adopted ones have been slightly modified in such a way to reduce the number of cycles. Moreover, the loading protocols have been adjusted in order to properly face the occurrence of unexpected non-linear events trying to preserve the adherence to the displacement history to be applied.

Furthermore, a number of recent studies have shown that the loading protocols suggested by AISC may be excessively severe if compared with the seismic demands deriving from theoretical and numerical investigations (Richards & Uang 2006). In particular, this effect is particularly evident for short links of EBs, whose ductility is actually depending on the loading protocol, as already noted by Malley & Popov (1984).

Referring to the influence of the loading protocol on BRBs performance, at the light of the massive experimental studies available in the scientific literature (Watanabe *et al.* 1988; Black *et al.* 2002; Usami *et al.* 2003; Merritt *et al.* 2003), it can be assumed that this system is capable to sustain large cumulative plastic deformation demands without strength degradation. The low-cycle fatigue life of BRBs needs to be explicitly checked by ad-hoc experimental tests, but this was out of the scope of the activity presented in this paper.

4. Experimental tests on eccentric bracing

4.1. Generality

In eccentric braces, storey shear forces are transferred to the brace members through bending and shear forces developed in the ductile steel link (Roeder & Popov 1978; Hjelmstad & Popov 1983; Popov & Malley 1983). The link is designed to yield and dissipate energy while preventing buckling of the brace members. In case of RC frames, the concrete beams are incapable to perform as a ductile link for the steel bracing system, which is inserted in the frame bays. Consequently, it is impossible to adopt for RC frames the inverted k-brace configuration typically used in steel frames. Hence, the need to adopt a Y-inverted bracing configuration, with a vertical steel link, can be easily recognized (Ghobarah & Elfath 2001). Besides, in this case bolted connections at the link ends are required, what could have the advantage to permit replacement of the dissipative members (links) after a damaging earthquake.

The basic geometry of the tested RC structure and bracing system is summarized in Fig. 4. Further details are given in subsequent sections.

Three experimental tests have been carried out (D'Aniello *et al.* 2005, 2006a Mazzolani 2006). The link cross-section and end-connection details were changed from one test to another, as described in the following.

4.2. Test on EB No. 1

The first eccentric bracing system was designed according to Eurocode 8 (EC8, 2003) prescriptions, but neglecting capacity design criteria. Thus, the base shear strength demand for earthquakes having a 475 years return period has been fixed equal to 117.76 kN. Accordingly, the link cross section (HEA100) and the steel grade (S 275) have been selected. The shear strength capacity was evaluated according to the first-yielding definition given in Popov & Engelhardt (1988). As far as the link length is concerned, it was chosen using the intersection of 3 conditions:

1) to have short links;
2) to achieve a target inelastic link rotation when the first plastic hinge forms in the RC structure;
3) to satisfy inter-storey drift limitations suggested by EC8 (2003) for non-structural damage control under frequent earthquakes.

The tested link and its connections to the RC slab and to the diagonal braces are shown in Fig. 5.

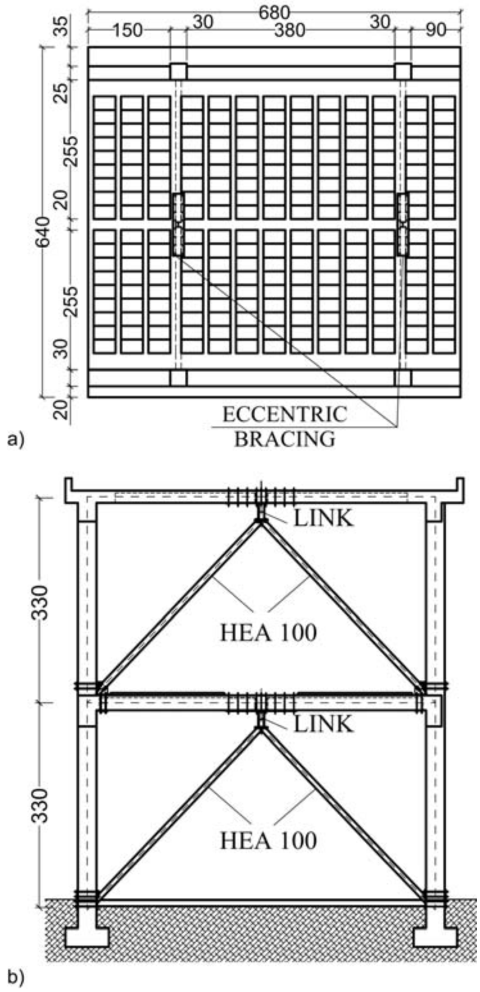

Fig. 4. Geometry of the tested EBs: a) RC structure plan; b) RC structure elevation

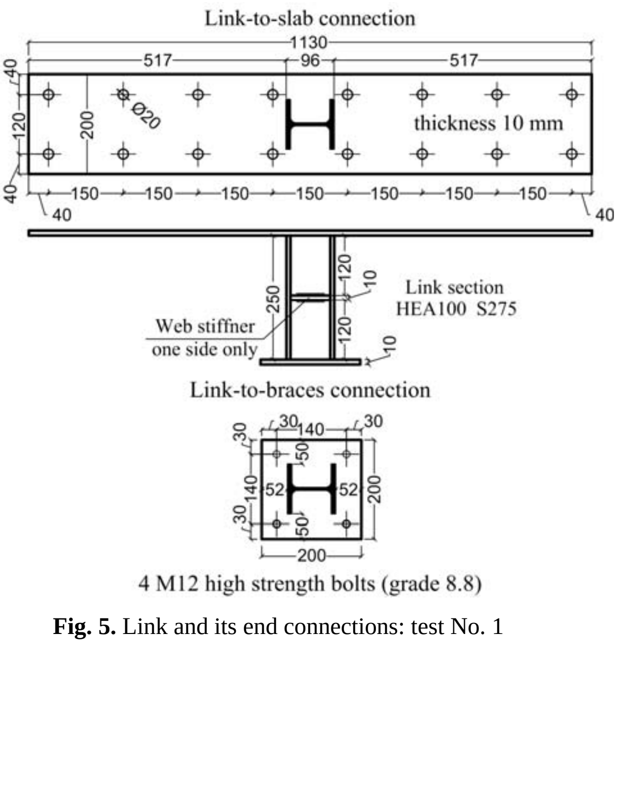

Fig. 5. Link and its end connections: test No. 1

The experimental test has shown that the collapse was due to failure of link end connections. In fact, as illustrated in Fig. 6, the global response curves soften from the peak value, corresponding to failure of the weld connection between the top link end and the 10 mm thick plate connecting the link to the RC slab (Fig. 7a, b). Increasing the imposed interstorey drift, both the failure of fasteners and welds at the bottom connection occurred, along with plastic bending of the corresponding end-plate connecting the link to the diagonal braces (Fig. 7c, d).

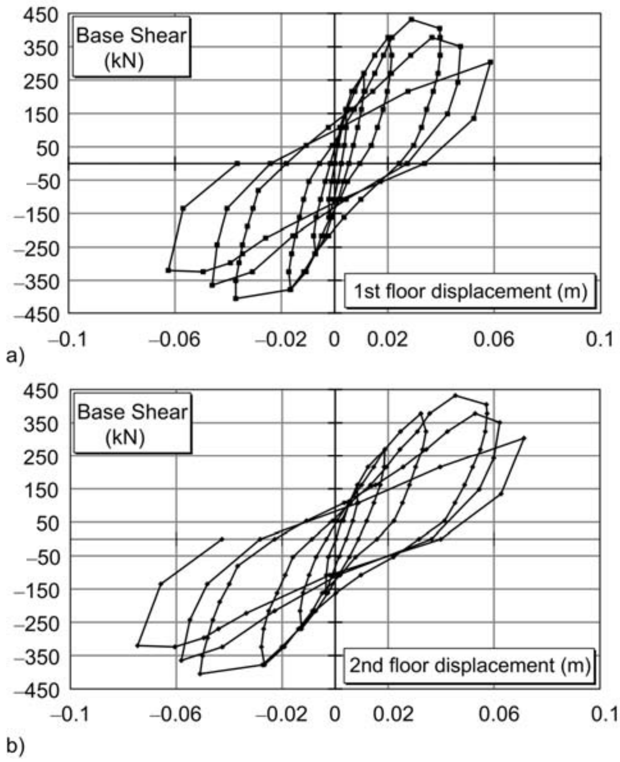

Fig. 6. Test No. 1 on EBs: average cyclic response curves: a) first floor; b) second floor

The maximum first-storey drift was 1.90% of the first-storey height, while the maximum top-storey drift was 0.54% of the structure height. Notwithstanding the undesired localization of damage at the link end connections, considering the values of inter-storey drift angles reached during the test and the large increase of the storey stiffness and strength, the upgrading technique seems to be promising.

4.3. Test on EB No. 2

For the second test, link end-connections were strengthened using capacity design. Namely, an ultimate shear strength of links equal to 1.5 times their yielding strength was assumed (Ricles & Popov 1994). This implied that the end-plate thickness increased from 10 mm to 25 mm. In particular, according to Kasai & Popov (1986a, b), Popov & Engelhardt (1988) and Engelhardt & Popov (1989), the ultimate bending moments transferred by links can be conservatively evaluated through static balance. Link end connections have been designed to resist these flexural actions.

a)

b)

c)

d)


Fig. 7. Test No. 1 on EBs: failure of link connections

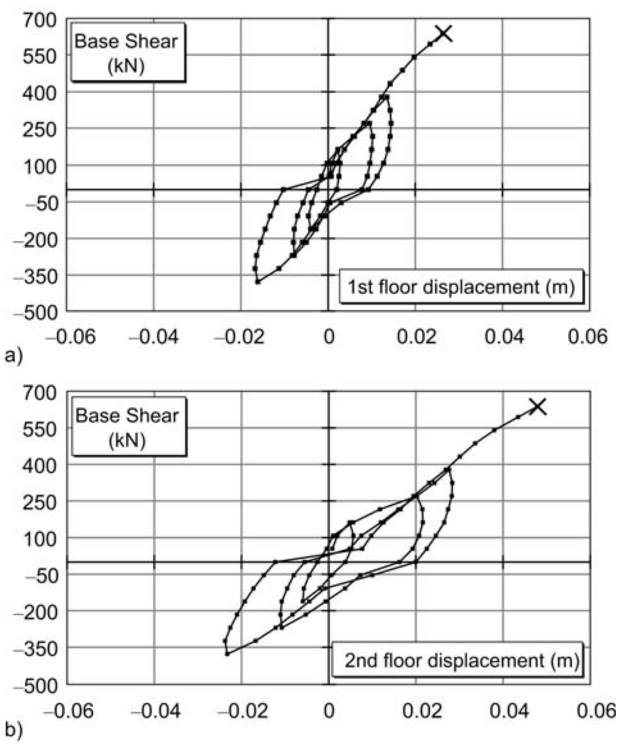

Fig. 8. Test No. 2 on EBs: average cyclic response curves: a) first floor; b) second floor

The second test showed shear-dominated failure of the bolts connecting the links to the braces. In fact, as shown in Fig. 8, the global response curves stop suddenly at a base shear value corresponding to the failure of link-to-brace connections. The maximum first-storey drift was 0.80% of the first-storey height, while the maximum top-story drift was 0.65% of the structure height.

Fig. 9 shows that the plastic bending of end-plate connections was now completely avoided, while a moderate plastic engagement of links along with a strong plastic deformation of bolts at the link-to-brace joints was observed in the form of shear hinging.

a)

b)

c)


Fig. 9. Test No. 2 on EBs: brittle shear failure of link-to-brace connection bolts

4.4. Test on EB No. 3

The third link was designed in order to increase the system ductility by forcing plastic deformation to be confined within links. In fact, the previous test revealed link over-strength larger than that expected, as testified by the shear failure of bolts.

Because it was impossible to modify the geometry of link-to-brace joints, a steel built-up section was now designed for the links, in order to have shear strength of connections at least 2 times larger than the average yielding strength of links. This led to the need of increasing the bolt steel grade. Indeed, four high-strength bolts (grade 10.9) with a diameter of 12 mm resulted to be sufficient for this purpose, having a link built-up section constituted by 90 mm × 10 mm rectangular plates for flanges and 80 mm × 4 mm rectangular plate for the web

(Fig. 10). Moreover, contrary to the previous cases, link web stiffeners were not adopted. The flexural resistance of the built-up section was chosen to be similar to the one of HEA100 (which is the section used for test No. 1 and No. 2), while the shear resistance (consequently the web area) was lesser than in the previous cases.

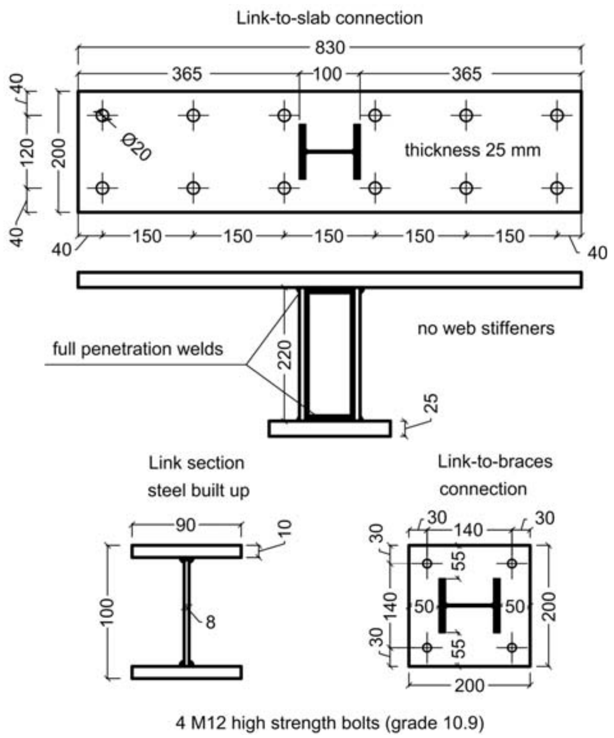

Fig. 10. Link and its end connections: test No. 3

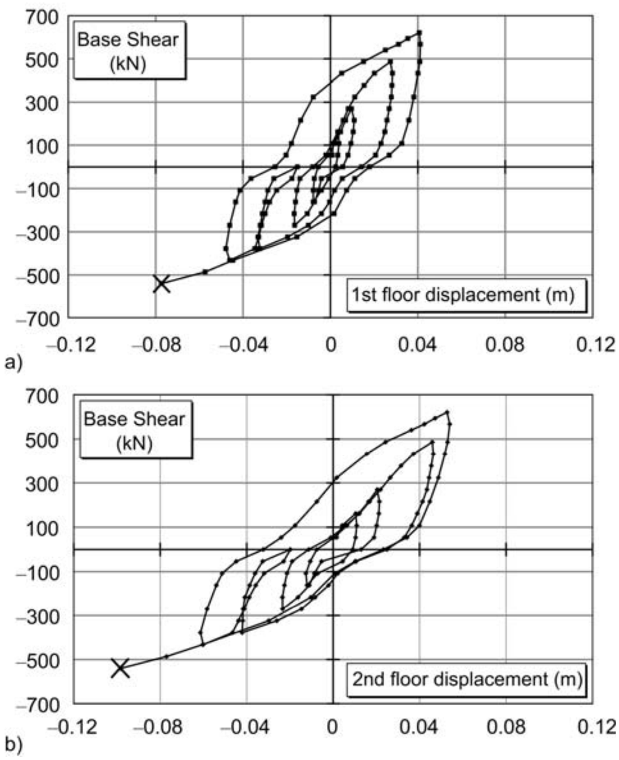

Fig. 11. Test No. 3 on EBs: average cyclic response curves: a) first floor; b) second floor

The response curves (Fig. 11) show that the behaviour of the retrofitted structure was now characterized by larger energy dissipation capacity. In particular, as in test No. 2, plastic bending of connection end-plates is now avoided, while significant plastic shear deformation of links was observed (Fig. 12). However, at larger link rotations, shear failure of bolts connecting the links and the diagonal braces was again observed. It is also noted that no shear buckling occurred in the link webs, even at relatively large link rotations and in absence of transverse stiffeners.

Fig. 12. Test No. 3 on EBs: ductile shear hinging of links

4.5. Discussion of test results on EBs

The lateral strength achieved by the tested EBs was significantly larger than the expected one. The reason can be found in the shear over-strength exhibited by the tested steel links. In fact, the maximum shear developed by the links can be approximately estimated by taking the value of the measured peak base shear minus the shear force carried by the RC columns. The latter value changed from test to test at a given lateral storey displacement, because of strength deterioration taking place in the RC columns. In particular, at the end of the test No. 3 the RC structure appeared to be strongly damaged, with a measured peak strength after failure of the braces of about 30 kN. Therefore, a range has been adopted to schematize the response of the RC columns when going from the first to the third test. In this way it was possible to define the range of the possible link shear over-strength. An upper bound of the response of the bare RC structure has been obtained by a refined finite element model developed considering the RC structure without any initial damage state. A lower bound of the RC frame response has been obtained by scaling the numerical RC frame response down to its lateral resistance (about 30 kN) finally measured after 3 loading tests. In this way, the maximum link shear force developed during tests No. 2 and No. 3 may range in the following intervals: $V_{\text{link,max}} = (2.77 \div 3.07)V_p$ for the test No. 2, and $V_{\text{link,max}} = (4.06 \div 4.5)V_p$ for the test No. 3, where V_p indicates the plastic shear strength of the link web.

These large link shear over-strength can be attributed to the following cooperating causes:

1) no shear buckling occurring in the link web;
2) large link rotations (up to 0.30 radians);
3) large flange over web area ratio of the link cross-sections;
4) tension axial forces developing at large link rotations, as a consequence of the axial restraints exerted by the RC slab and the diagonal braces;
5) cyclic hardening effects associated to the loading history.

In particular, as far as the item 4 is concerned, it is contended that large link rotations may produce significant axial tension forces in the link. Tension axial forces are expected to increase ductility, since they delay the occurrence of shear buckling because of second order geometric effects. Finite element numerical simulations of the shear response of links in a fixed-fixed configuration show that axial forces develop because of the axial restraint given to the link (Della Corte *et al.* 2007, 2008). Such axial forces appreciably contribute to the link post-yield stiffness, thus leading to increase the peak strength at large link rotations.

5. Experimental tests on buckling restrained bracing

5.1. Generality

BRBs are special devices that solve the problem of the limited ductility of classic concentric bracing. In fact, the axial strength is de-coupled from the buckling resistance; the axial load is confined to the steel core, while the buckling restraining mechanism resists overall brace buckling and restrains the high-mode steel core buckling (rippling).

Several studies have proposed a large number of different types of BRBs, but all of them based on the basic concept to use tubes for restraining lateral displacements while allowing axial deformations of the core. In the most classical form, the restraining tube is filled with concrete and an unbonding layer is placed at the contact surface between the core plates and the filling concrete, thus this version is called 'unbonded brace' (Fujimoto *et al.* 1988; Watanabe *et al.* 1988; Nagao & Takahashi 1990; Clark *et al.* 2000; Black *et al.* 2002; Carden *et al.* 2004; Tremblay *et al.* 2004; Wada & Nakashima 2004; Iwata *et al.* 2000, 2006).

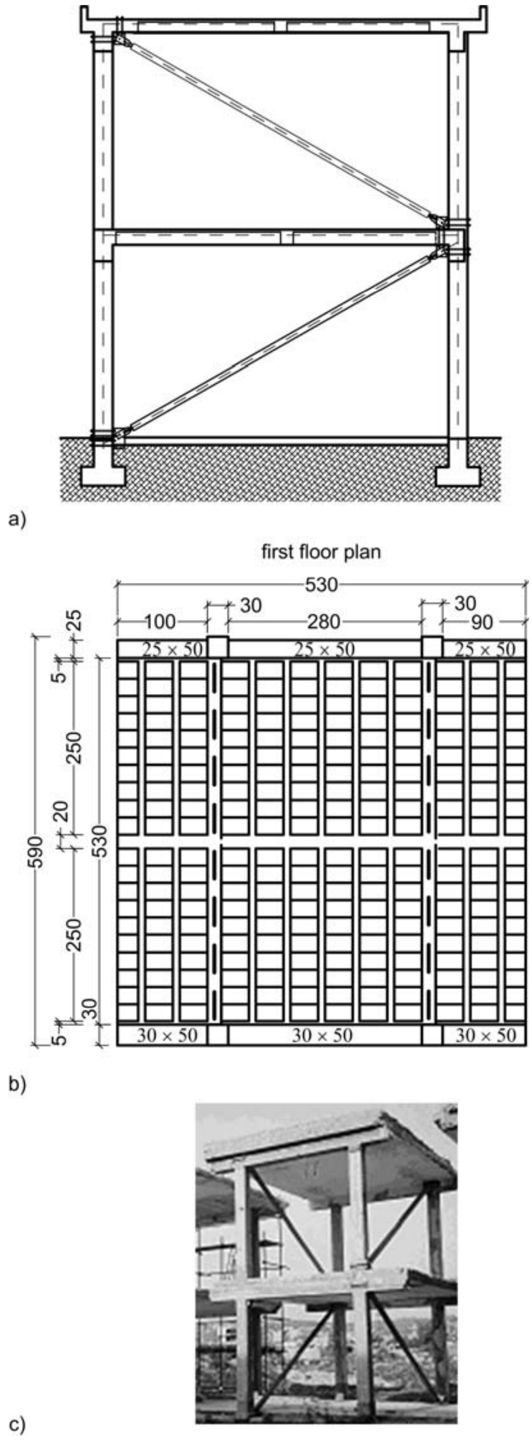

Fig. 13. Configuration of the tested BRBs

'Only-steel' solutions have also been proposed, with two or more steel tubes separated by a small gap with the yielding steel plates. Contrary to the "unbonded" type, the ‘only-steel’ BRBs can be designed to be detachable. This aspect implies that it is possible to design these systems in such a way to be inspected, so that to control their condition after each seismic event and to allow for an ordinary maintenance during the life-time. To do this, the restraining tubes should be connected by bolted steel connections (Tsai *et al.* 2004a, b). Moreover an ‘only-steel’ BRB is lighter than an ‘unbonded’ one; this implies a technical and economical advantage during the construction.

These considerations led to study a special only-steel detachable BRB to be used for improving the seismic response of RC buildings. In particular, two different types of this special device have been applied on one of the RC substructures of the ILVA-IDEM project.

The diagonal braces were directed in alternate way, in order to evaluate the response of the studied braces in tension and compression. In particular, the location of these braces is shown in Fig. 13a, b, c.

Two experimental tests have been carried out (Della Corte *et al.* 2005; D’Aniello *et al.* 2006b; Mazzolani 2006). Each BRB system has been tested under lateral cyclic loads as in the case of EBs.

5.2. Test on BRB No. 1

Figs 14 and 15 give the fundamental geometry of the first type of BRB tested. The yielding steel core is a rectangular plate (25 mm × 10 mm), made of the European S 275 steel. The actual average yield stress of the core was measured to be 319 MPa (i.e. 1.16 times the characteristic value).

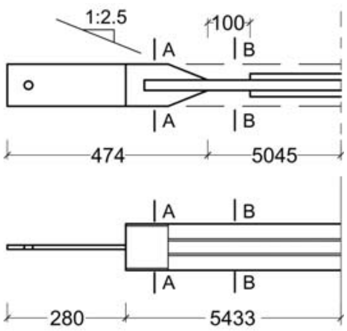

Fig. 14. BRB type 1: geometry

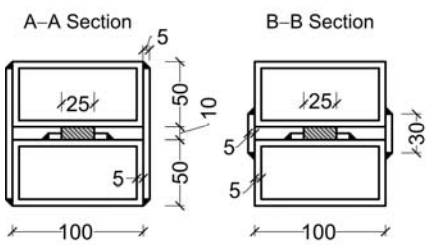

Fig. 15. BRB type 1: cross-section details

The buckling-restraining action is given by 2 rectangular steel tubes (100 mm × 50 mm × 5 mm), with a ratio between the Euler buckling load (P_E) of the 2 tubes and the actual yield force (P_y) of the internal steel core $P_E/P_y = 2.1$. As it can be seen in Fig. 15, the restraining effect is given by the flexural stiffness of the tube walls in one direction (vertical direction in Fig. 15), while in the perpendicular direction 2 small steel bars were designed to be welded to the tubes with a total clearance with the core of about 1 mm (0.5 mm per side).

a)
b)
c)
d)
e)
f)

Fig. 16. Test No. 1 on BRBs: damage pattern

The experimental evidence showed a good response of the brace when it is in tension, with the expected relative displacements developing between the internal yielding core and the restraining tubes (Fig. 16a). But, the compression brace ductility was limited by the local buckling of the core, near the brace ends. This buckling produced strong flexural deformation of the closing plates, which were welded for joining the tubes at their ends (Fig. 16b, c). Moreover, increasing the external load, the deflected end-tapering plates punched the welded closing plates, as highlighted by the white circle in Fig. 16d. Because of their flexural failure, the end closing plates were unable to restrain the end portion of the brace core. This localization of damage ultimately led to a significant plastic engagement at the transition section between the reduced core and the end tapering. Hence, this strong flexural plastic engagement of the core at its ending portion led to its premature fracture (Fig. 16e). Damage in the RC structure is shown in Fig. 16f, where flexural cracking is visible.

The measured base shear vs. first and second storey lateral displacement relationships are plotted in Fig. 17a, b. At each floor two measures of lateral displacement were taken, approximately symmetric with respect to the loading axis. The difference between the two displacements at each floor (hence the floor rotation) was small, with a maximum of about 15% of the average displacement in the inelastic range. The maximum first-storey drift was 1.9% of the first-storey height.

The global storey ductility (μ) reached a maximum of about 4.75. In fact, the yielding value of the first storey drift angle (which corresponded to yielding of BRBs at first storey) was equal to about 0.004 rad, hence $\mu = 0.019/0.004 = 4.75$.

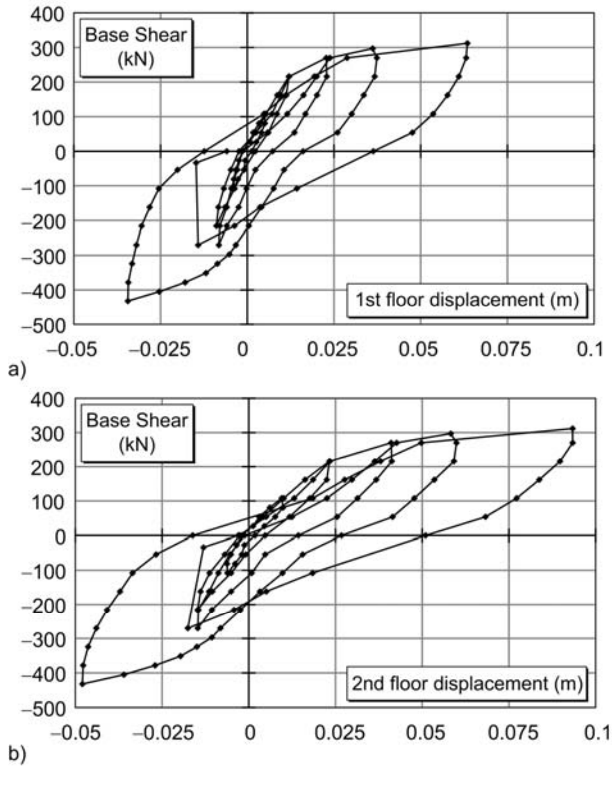

Fig. 17. Test No. 1 on BRBs: average cyclic response curves: a) first floor; b) second floor

5.3. Test on BRB No. 2

Figs 18 and 19 illustrate the geometry of the second type of tested BRB. Two main changes were made with respect to test No. 1. Firstly, the inner core was now tapered in a more gradual manner (compare Figs 14 and 18), in order to provide extra flexural stiffness for higher buckling strength and to elastically transfer the axial yield force of the core. Moreover, the tapered end-part was restrained by two parallel bars welded to the tubes, so that the flexural deformation of the terminal parts was avoided. The slope of tapering portions has been assumed of 1 to 2.5. Secondly, the two restraining tubes were now joined together by means of bolted stiffened elements (Fig. 19), allowing the BRB to be opened for inspection and monitoring at the end of the test. However, analogously to the previous case, in BRB type 2 the buckling-restraining action was obtained by the same 2 rectangular steel tubes, with a similar ratio between the Euler overall buckling load (N_E) of the 2 tubes and the yield force (N_y) of about 2. In fact, the yielding steel core was again a rectangular plate (25 mm × 10 mm), made of the European S 275 steel, with a measured average yield stress equal to 295 MPa.

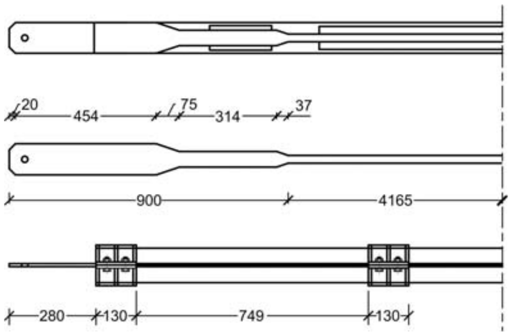

Fig. 18. BRB type 2: geometry

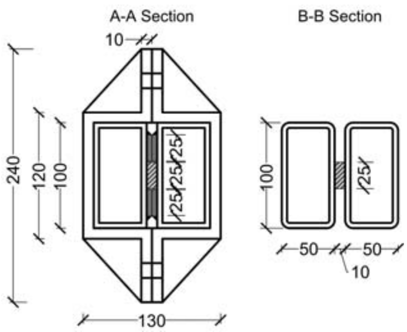

Fig. 19. BRB type 2: cross-section details

Fig. 20 summarises the damage pattern evidenced during the test. The dark part of the BRB core visible in Fig. 20a, b highlights the relative displacement between the internal core and the restraining tubes, developed when the BRB was either in tension (Fig. 20a) or in compression (Fig. 20b). Fig. 20c shows the inelastic buckling mode of the inner core, which was expected as a normal response of this system because of the presence of an inner clearance of 1 mm per each side of the yielding core. This phenomenon became very clear at the maximum first-storey drift reached during the test, corresponding to the end of the core free length working stroke. In Fig. 20d the large deformation of the internal core of one BRB placed at the first storey at the maximum inter-storey drift reached during the test (about 5.6% of the storey height) is shown. Fig. 20e illustrates the local buckling failure of one end plate connection during compression of one BRB at the first storey. This unexpected phenomenon occurred at just one location and it may be attributed to some damage produced in the gusset plates during the mounting of that BRB. In fact, in that occasion the gusset plates were forced and deformed, thus introducing geometric imperfections and losing some part of the restraining effect against out-of-plane rotation of the BRB. This implied that the local buckling length of the BRB end portion significantly increased and consequently its local buckling capacity rapidly decreased. Another confirmation of this consideration is the fact that the local buckling phenomenon occurred during the third cycle in compression. Then, it could be argued that initial imperfections grew up during the first 3 loading cycles, finally leading to the local buckling mode illustrated in Fig. 20e. It is important to highlight that all the other braces had a stable response in compression up to the very large interstorey drift of 5.6% imposed at the first floor. Finally, Fig. 20d shows flexural cracking occurring in RC columns at the first storey in correspondence with the peak values of storey drift.

a)
b)
c)
d)
e)
f)

Fig. 20. Test No. 2 on BRBs: damage pattern

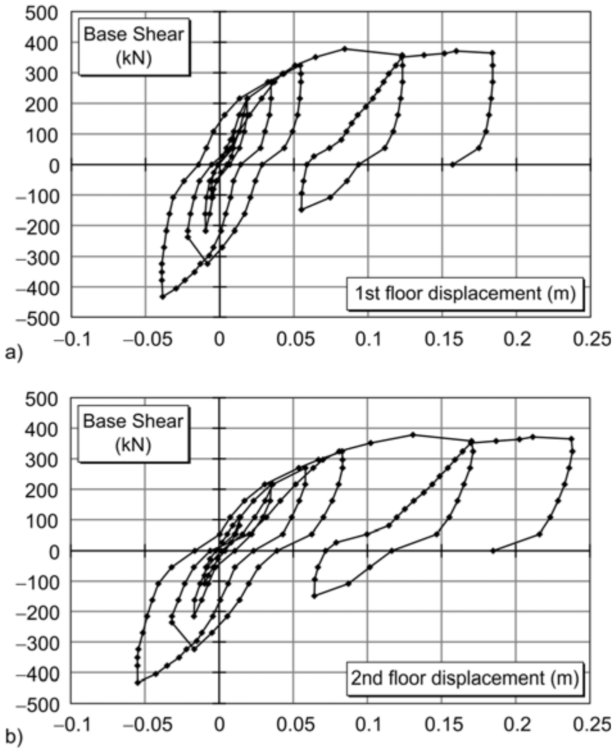

Fig. 21. Test No. 2 on BRBs: average cyclic response curves: a) first floor; b) second floor

Fig. 21a, b show the base shear vs. first-storey and top-storey lateral displacements. Also in this second test, the difference between the tension and the compression behaviour of the BRBs originated relatively small torsion of floors. Obviously, this torsion became larger when local buckling of one end-plate affected the compression response of one BRB at the first storey. Notwithstanding some localization of rippling at one end of the internal steel core of the compressed BRBs, the global storey ductility (μ) was quite large, reaching a maximum of about 14. This value can be computed assuming the yielding value of the first storey drift angle conventionally equal to 0.004 rad, which approximately corresponds to a base shear equal to 40% of its maximum value, hence $\mu = 0.056/0.004 = 14$.

5.4. Discussion of test results on BRBs

The efficiency of the first type of BRB (test No. 1) was impaired by the flexural failure of the end closing plates, which were unable to restrain the end portion of the brace core. This produced a strong flexural plastic engagement of the core at its ending portion. Hence, ductility of the system was quite limited, even if the strength and stiffness of the upgraded RC structure met the expected improvement.

The second type of BRB (test No. 2) showed instead large ductility, being able to adequately restrain the core from buckling, though some additional improvements may be required in the design. In fact, local buckling of one end plate was observed during this second test, even if the end plate satisfied a capacity design criterion. The local buckling Euler load (conservatively estimated by the assumption of column-type behaviour and assuming a buckling length equal to the distance between the bolt centre-line and the starting section of the restraining tubes) can be computed to be 3.8 times larger than the maximum expected compression load. Hence, buckling of the end plate (occurred at just one location) may be attributed to: (i) strong local geometric imperfections, which grew up during cyclic loading; (ii) a flexible end-restraint, which produced an increase of the buckling length and also some coupling of lateral and torsion effects. Anyway, the maximum storey-drift angle reached during test No. 2 (5.6% of the story height) is appreciably larger than the maximum values commonly applied in the past testing of BRBs.

Finally, it is worth to note that the surplus value of these experimental results consists in having underlined the influence of local details and the quality control in the manufacturing process.

6. Comparison of test results

Both bracing systems presented in the previous sections demonstrated to be effective in improving the seismic performance of existing RC structures. In Fig. 22 the lateral-load response of all the tested bracing systems is compared in terms of envelope curve corresponding to the positive loading direction. Besides, the behaviour is also compared with the results of a previous pushover test, which was carried out on a bare RC structure very similar to the one tested with the bracing systems.

All the tests showed a significant increase of lateral stiffness and strength in respect to the original unbraced RC structure. In particular, in case of EBs it was observed an increase of the lateral strength capacity from 5.65 to 8.34 times, while in case of BRBs the increase was from 4.08 to 4.95 times. However, such an increase of strength and stiffness is strictly related to the type of the tested RC structure. Indeed, in order to obtain similar results for different structures, the tested systems should be re-designed and adapted to the specific seismic demands.

Generally speaking, BRBs were characterized by lower stiffness and strength than EBs, but they provided a larger displacement capacity. In fact, referring to the studied cases, short shear links should supply a rotation

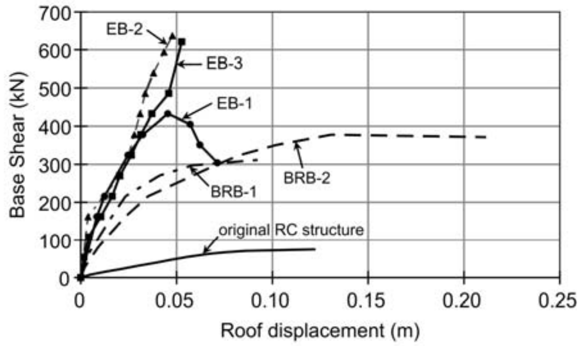

Fig. 22. Comparison of response curves of tested bracing systems

of about 0.60 radians in order to provide the same displacement capacity of the tested BRB type-2.

7. Conclusions

Among the possible systems to retrofit an existing structure, bracing systems appear to be simple and effective, especially when storey drifts need to be limited. The idea is to design systems that are strong enough to resist the seismic forces and light enough to keep the existing structural elements far from needing further reinforcement. Furthermore, these systems could be installed quickly and then eliminate the need to disrupt the occupants of existing structures. In this sense, both EBs and BRBs may be a viable solution. In fact, they provide high elastic stiffness and strength and stable inelastic response.

In case of eccentric braces, link end-connections exhibit a key role in determining the system ductility, especially if bolted connections are selected for removable links. Experimental test results clearly highlight this aspect, emphasizing large over-strength of short links with respect to the first yielding shear and the consequent danger of connection failure. In particular, it has been shown that short links may exhibit over-strength largely in excess of that suggested by current code provisions.

Respect to EBs, BRBs revealed to provide a more complete structural performance, since they can largely improve not only the lateral stiffness and strength capacity but also the displacement capacity of the structure. In fact, test results on 2 different types of “only steel” BRBs showed very good ductility of this system.

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STIPRINAMŲ PASTATŲ SEISMINIUS POVEIKIUS SLOPINANČIŲ PLIENINIŲ RYŠINIŲ SISTEMŲ EKSPERIMENTINIAI TYRIMAI

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Santrauka

Didinant gelžbetoninių pastatų atsparumą seisminiams poveikiams, gali būti naudojami energiją sugeriantys metaliniai slopintuvai. Straipsnyje parodoma, kaip tokios slopinančios sistemos, kaip necentriniai ryšiai ir klupimą suvaržantys ryšiai, gali iš esmės gerinti pastato seisminę elgseną. Pateikiami dviejų panašių vienos angos gelžbetoninių pastatų su minėtaisiais ryšiais eksperimentiniai tyrimų rezultatai. Tyrimai parodė, kad abi suvaržymo sistemos gerina eksploatuojamų gelžbetoninių pastatų atsparumą seisminiams poveikiams.

Reikšminiai žodžiai: klupimą varžantys ryšiai, laikomosios galios skaičiavimas, stamantrumas, necentriniai ryšiai, eksperimentiniai bandymai, atnaujinimas dėl seisminių poveikių.

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