“All-steel” buckling-restrained braces for seismic upgrading of existing reinforced concrete buildings

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ABSTRACT
Buckling-Restrained Braces (BRBs) are a relatively recent development in the field of seismic resistant steel structures. BRBs can be considered a structural system much more efficient than classic concentric braces (CCBs) to resist earthquakes because they exhibit an almost symmetric load-deformation behaviour and larger energy absorption capacity. Results of an experimental campaign consisting of full scale tests on two reinforced concrete (RC) buildings equipped with BRBs are presented and discussed. The experimental activity led to develop a novel “all-steel” BRB, which has been specifically designed for seismic upgrading of RC buildings, without interference with their functions and aesthetics. Indeed, 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.

1 INTRODUCTION
Buckling Restrained Braces (BRBs) represent a very efficient structural system to upgrade the seismic capacity of both new and existing building structures (e.g. Brown et al. 2001, Black et al. 2002, Wada & Nakashima 2004, Xie 2005). BRBs brilliantly solve problems of classic concentric braces (CCBs), which are characterized by a pinched hysteresis loop and a small ductility due to the concentration of strain occurring when braces buckle. In fact, BRBs are designed to avoid the global compression buckling, thus providing the same response in tension and compression. A number of approaches to accomplish this objective have been suggested, such as encasing a ductile metal (usually steel) core (rectangular or cruciform plates, circular rods, etc.) either in a continuous concrete filled tube or between two or more steel tubes. The assembly is detailed so that the central yielding core can deform longitudinally independent from the mechanism that restrains lateral and local buckling. BRBs usually consist of the following four parts (Black et al. 2002): 1) an axial force-carrying unit (usually called “core”); 2) a stiffened transition segment between the core and the connections to the main frame; 3) a sleeve or buckling-restraining unit, whose function is to prevent the core buckling; 4) a separation unit between the core and the buckling-restraining unit, which ensures that the core can freely slide inside the buckling-restraining unit and that the transverse expansion of its cross section can take place. The separation unit can be constituted by layers of some debonding materials in case of “unbonded” BRBs, whose restraining unit is a tube filled with mortar or concrete. Alternatively, the use of a clearance between the core and the restraining unit is typically adopted in case of “all-steel” BRBs, whose restraining unit is entirely made of steel (Tsai et al. 2004a-b, Della Corte et al. 2003, 2005, Della Corte & Mazzolani 2006, D’Aniello et al. 2007).

“All-steel” BRBs have some advantages over the “unbonded” type: (i) they can be designed to be detachable, hence they can be inspected to control their condition after each seismic event and, if necessary, the yielded steel core can be substituted; (ii) they are lighter, implying a technical and economical advantage during the erection; (iii) their cost is smaller, because the poring and curing of concrete are eliminated.

This paper describes an experimental research activity on novel “all-steel” BRB prototypes.
The research consisted of a series of full scale tests on two reinforced concrete (RC) buildings (Figs. 1a,b) equipped with BRBs. The buildings are 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), including the two RC buildings shown in Figure 1.

The first building (Fig. 1a) 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 and subsequently tested more than one time (Fig. 2). The whole experimental activity is described in Mazzolani (2006). This paper summarizes the tests carried out on one of the substructures (the second from the left in Figure 2), which was equipped with BRBs and tested two times (two different BRB prototypes were tested).

The second building (Fig. 1b) 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 and the reconstruction of some perimeter masonry infill walls (Della Corte et al. 2008). Subsequently, it was equipped with three slightly different BRB prototypes, each of which tested up to failure. The BRB prototypes used in this second series of tests were different from those used in the former series.

Therefore, a total of five tests on BRB prototypes applied to real RC buildings has been carried out. The final tested devices are special “all-steel” BRBs, which have been specifically designed for seismic upgrading of RC buildings, without interference with their functions and aesthetics. Indeed, the main characteristic of the 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.

The paper describes the test set-up, the BRB geometrical and mechanical details and the results obtained with the two series of tests.

Design principles and criteria are described in D’Aniello et al. (2009).

2 EXPERIMENTAL TESTS – BUILDING # 1

2.1 Test setup

Figure 3 illustrates the substructure of building #1 equipped with BRBs. The diagonal braces were directed in alternate way, in order to highlight the effect of differences between the tension and compression response of BRBs, if any. The RC structure was made of two floor slabs sustained by four columns. Each column had a square cross section, reinforced with 4 longitudinal bars (diameter = 12 mm) placed at the section corners and stirrups (diameter = 8 mm) spaced at about 200 mm. A more complete description of geometry, structural details and mechanical properties of the bare RC structure can be found in Mazzolani (2006).

Lateral loads have been applied by means of two hydraulic jacks working alternatively in two opposite directions, in order to apply load reversals. The height on the ground of the two loading jacks (hence the height of the load center) has been fixed in order to obtain a linear height-wise distribution of horizontal forces. This loading arrangement is shown in Figure 4, which illustrates: a) the steel structure erected to react to the lateral forces applied onto the RC structure; b) a close-up view (from the inside of the RC structure) of the vertical steel beam used to distribute the horizontal force between the two
stories along with the horizontal steel members used to sustain and attach the loading jacks; c) a view from the bottom side of the two loading jacks and d) one of the two video-cameras installed on each floor slab to measure horizontal displacements.

Figure 3. The RC frame equipped with BRBs.

Figure 4. Setup for the first series of experimental tests.

The strengthened structure was subjected to a cyclic loading history up to failure of the BRBs. The loading protocols can be considered as a compromise between technical feasibility and scientific needs. In fact, for the sake of inexpensiveness, the protocols are characterized by a reduced number of cycles per each deformation level. The loading protocol may have an effect on the energy absorption capacity of the device. However, available experimental studies (e.g. Watanabe et al. 1988, Black et al. 2002) show that well-designed BRBs can sustain large cumulative plastic deformation demands. Anyway, the effect of the cyclic loading protocol on the response of the BRB prototypes was out of the scope of this research.

2.2 Description of the BRB specimens

The buckling-restraining unit of the first two prototypes was made by two rectangular steel tubes (100 mm x 50 mm x 5 mm), while a rectangular steel plate was used for the yielding core (25 mm x 10 mm). The steel grade was the European S 275 (average measured yield stress of about 319 MPa). The BRB system was characterized by a ratio between the Euler buckling load \( N_E \) of the two tubes and the yield force \( N_y \) of the internal steel core \( N_E/N_y = 2.1 \).

Figure 5. Geometry and cross section details of BRB type 1 (a) and type 2 (b).

As highlighted in Figure 5, the two tested BRB prototypes (henceforth called type 1 and type 2) differ in some aspects, such as the restraining unit.
concept, the inner clearance between the core and the sleeve and the tapering of the inner core. Indeed, the tubes constituting the sleeve of prototype 1 have been welded together along the sleeve length (Fig. 5a). Contrary, the sleeve of type 2 has been designed to be detachable. In fact, the restraining tubes have been joined together by means of bolted steel connections (Fig. 5b), allowing the BRBs to be opened for inspection and monitoring at the end of the test. In both cases, the restraining effect was given by the flexural stiffness of the tube walls in one direction (horizontal direction in Figure 5), while in the perpendicular direction two small steel bars were welded to the tubes. In addition, this detail was arranged with two different clearance sizes between the core and the buckling-restraining units. Indeed a gap of 0.5 mm per core side was selected for the BRB type 1, while 1 mm per core side was chosen for the BRB type 2.

In order to prevent the instability of the tubes and to permit the core inelastic deformations, both sleeve ends were detailed so that the core had no possibilities of bearing on them. This has been obtained by interposing an interior reserve space equal to 100 mm per each sleeve end (Fig. 5a). Moreover, the inner core of type 2 was tapered in a more gradual manner in order to provide extra flexural stiffness for larger buckling strength (Fig. 5b).

2.3 Experimental results

2.3.1 Prototype 1

Figure 6a highlights the relative displacement between the internal yielding core and the restraining tubes when the braces were subjected to tension. The brace ductility was limited by the local buckling of the unrestrained end portion of the core, near the end tapering. The buckled core bear on the end closing plates, which were used to weld together the restraining tubes, producing strong flexural deformation of the plates (Figs. 6b,c). On the opposite side, the end tapering plates punched the welded closing plates, as highlighted by the white circle in Figure 6d. 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 (Fig. 6e). Hence, this strong flexural plastic engagement of the core at its ending portion led to its premature fracture (Fig. 6f).

It is known that a small difference between the tension and compression response of BRBs is to be expected. In this test, the maximum difference between the displacements at two points of each floor (hence the floor rotation) was equal to about 15% of the average displacement. This indirectly proves that the brace response was sufficiently symmetric in tension and compression. Then, the average floor displacements are shown in the following.
The calculated value of the first storey yield drift ratio \( \theta_y \) (corresponding to yielding of BRBs at the first storey) is equal to about 0.004 radians. Then, the global storey ductility \( \mu \) (expressed as the ratio \( \theta_{\text{max}} / \theta_y \)) reached a maximum of about 4.8.

2.3.2 Prototype 2

The experimental response of BRB type 2 showed a significant improvement of performance over the previous type in terms of ductility and global deformation capacity. Figure 8 summarizes the damage pattern evidenced during the test. The dark part of the BRB core visible in Figures 8a,b highlights the relative displacement between the internal core and the restraining tubes, developed when the BRBs were either in tension (Fig. 8a) or in compression (Fig. 8b).

Figure 8 shows the inelastic high-order buckling mode of the inner core, which was expected as a normal response of this system. This phenomenon became very apparent at the maximum first-story drift ratio of 5.6% reached during the test (Fig. 8d). Figure 8c illustrates a detail of the local/distortional buckling failure of one BRB at the first storey. This unexpected and undesired phenomenon occurred at only one location and it may be attributed to some damage produced in the corresponding gusset plates during erection of BRBs. In fact, the same gusset plates used for the first test were also utilized for this second test. During the erection of the braces, it was clearly noted that the gusset plates were forced and deformed, thus introducing geometric imperfections and losing some parts of the restraining effect against out-of-plane rotation of the BRBs. Figure 8f shows the localization of plastic flexural deformation at the transition section between the reduced core and the end tapering. This localization is essentially due to the combined effect of sliding friction between the core and the restraining tubes and the out of plane deformability of the tube walls.

Figure 9 shows the base shear vs. average first storey drift ratio. Similarly to the previous test, the difference between the tension and compression behaviour of the BRBs was within the expected range of behaviour, originating relatively small torsion of floors.

The local strain concentration at the end of the yielding zone occurred at an interstorey drift of 3.8% and a corresponding ductility of 12.7 (\( \theta_y = 0.3\% \)). The maximum interstorey drift ratio (5.6%) corresponded to a global storey ductility \( \mu = 18.7 \).

Figure 9. BRB type 2: base shear vs. 1st interstorey drift ratio.

3 EXPERIMENTAL ACTIVITY – BUILDING #2

3.1 Test setup

Figure 10 shows the plan layout and one vertical section of the second building equipped with BRBs. The building is rectangular in plan (18.50 m x 12.00 m), on two floors, with first and second floor heights equal to 4.60 m and 8.95 m. Figure 10a shows the plan location of the braces (highlighted with the dashed lines). Figure 10b shows the vertical location of the BRBs, which were applied only at the first floor, which is the one strongly damaged during the first two tests on building #2. Indeed, as previously mentioned, this building has initially been tested two times: (i) in the original conditions and (ii) after some improvements were made.
repairing. In particular, it was pushed in the Y-direction (Fig. 10a) by lateral loading up to severe damage of both infill walls and structural frame members. Both tests showed the formation of a weak story at the first floor. Detailed information about the first two tests can be found in Della Corte et al. (2008). After these tests, the structure has been partially repaired and three different BRB systems have been designed and tested separately and subsequently.

The BRBs were designed to be hidden in the inner space between the two panels of masonry claddings. In order to demonstrate the feasibility of hiding the device into the claddings, for the first test of this experimental series one of the walls has been reconstructed (Figs. 11a, b and c).

The test set-up consisted of a reacting steel structure with a push-pull system made of six hydraulic jacks used for applying the load in two opposite directions (Fig. 12a). The total applied horizontal force was distributed between the first and second floor of the building by using a triangulated steel structure, which was fixed to the two floor slabs of the building. The load center was fixed in such a way to approximately obtain a triangular load distribution.

Storey displacements were monitored by using a diastimeter (Zeiss-Trimble S10) and six reflecting prisms (i.e. six measuring points), three per each floor (Fig. 12b). These measuring points allow obtaining the average storey translation and rotation about the vertical axis. In addition, the axial deformations of one BRB have been measured in the last test of this series, by means of a linear potentiometer spanning the brace ends (Fig. 12c).

The loading protocols were based on AISC (2005), but slightly modified to reach a compromise between technical feasibility and scientific needs.
3.2 Description of the BRB specimens

The new tested BRB prototypes (henceforth called type 3, type 4 and type 5) have also been designed to be detachable, but they differ in some aspects from both the previous ones and among them.

One difference with the former BRBs was in the buckling-restraining unit, as it can be easily recognized by comparing Figures 5 and 13. In this second series of BRBs, the buckling-restraining unit was made by two omega-shaped built-up profiles, which were joined together by means of a discrete number of bolted connections. Two longitudinal bars on each side were used to stiffen the two omega-shaped sleeves, providing the required restraining action to the core. This arrangement permitted to reduce the transverse size of the sleeve thus allowing to hide the brace between the two panels of masonry claddings commonly adopted for RC buildings. Indeed, the transverse size of the sleeve reduced from 130 mm (first series, type 2) to 94 mm of BRB type 3 and type 4, up to 92 mm of BRB type 5. Notwithstanding the sleeves were still characterized by a ratio $N_e/N_y$ equal to 2.06, practically coincident with the value characterizing the first series of devices.

One important difference between BRBs type 3 and type 4 was the length of the unrestrained end-portion of the core, which determines the range of interstorey drift ratios that can be sustained by the device. This length was reduced from 180 mm of BRB type 3 to 50 mm of BRB type 4. For the last device, BRB type 5, this length was again increased to 70 mm. Besides, stiffener details of the unrestrained end portions of the core were slightly different from one BRB to another (Fig. 13).

The ratio between the core length ($L_c$) and the total BRB length ($L$) was 0.39 for type 3, 0.33 for type 4 and 0.40 for type 5.

BRB type 3 has been designed with an inner clearance between the yielding core and the restraining sleeve equal to 1 mm per core side, while for BRBs type 4 and type 5 the clearance was set equal to 2 mm per core side.

It must be noted that, in the last test, the RC building has been equipped with the novel BRBs (type 5) in one bay and classic concentric braces (CCBs) in the adjacent bay. In such a way, it was possible to compare directly the performance of BRBs and CCBs and to highlight the benefits related to the use of BRBs. The CCBs have been designed to have a tensile axial strength as close as possible to the one of the tested BRB device and a normalized slenderness $\lambda$ not larger than 2.

The selected brace was a circular tube (101.6 mm x 2 mm), characterized by a cross section of class 2 according to Eurocode 3 (CEN 2004), and a normalized slenderness $\lambda = 1.8$.

![Figure 13. Geometry and local details of BRBs type 3 (a), type 4 (b) and type 5 (c).](image-url)
3.3 Experimental results

3.3.1 Prototype 3

In case of BRB type 3 the tested structure reached a maximum interstorey drift ratio of about 1.25% (Fig. 14a), which corresponded to a local-distorsional buckling failure of the unrestrained end portion of the steel core. The corresponding ductility was equal to 6.94 (Fig. 14b). For larger displacement demand, the local-distorsional buckling of the unrestrained length of the core produced strength degradation.

![Graph](image)

**Figure 14.** BRB type 3: system response.

![Images](image)

**Figure 15.** BRB type 3: damage pattern.

Figure 15 summarizes the observed damage pattern: a) shows the tensile elongation of BRBs; b) shows the collapse of the external facing wall caused by buckling of the unrestrained end portion of the BRB; c) shows the unrestrained end portion of the brace in its final buckled configuration and the failure of welds between the stiffener plates and the tapered core plate; d) illustrates one half of the sleeve, which was opened at the end of the test and exhibited no sign of damage. It is interesting to note that, after having detached the BRBs after the test, no local buckling waves were discovered along the core. This suggests that the gap size was too small to activate this mechanism prior to the local-distortional buckling mode shown in Figure 15c.

The reasons generating the undesired buckling failure of BRBs type 3 may be found in the negative synergy of three combined events: (i) the actual yield stress for the steel of the core plate was appreciably larger than the expected value; (ii) the inner clearance between the yielding core and the restraining sleeve was measured equal to 0.5 mm, which is lower than the design value of 1 mm per core side; (iii) the fillet welds between the stiffeners and the tapered unrestrained end portion of the core were erroneously fabricated as discontinuous, contrary to the design prescription. Notwithstanding these fabrication mistakes, it was deemed interesting to test the devices.

3.3.2 Prototype 4

BRB type 4 showed a satisfactory performance, characterized by stable and symmetric hysteresis loops. As it can be noted in Figure 16a, the tested device showed a symmetric and stable response for interstorey drift ratios in the range ±1.5%, with a minimum ductility capacity of about $\mu = 10.5$ (Fig. 16b).

For interstorey drift ratios in the range ±1.5% the inner core yielded both in tension and in compression (Figs. 17a,b) and the sleeves did not show any sign of damage. Then, the sleeve started to exhibit localized deformations induced by the buckled inner core. Afterwards, when the maximum displacement capacity of the tested device was achieved, two different secondary failure mechanisms occurred:

1) local buckling and related plastic bending of the steel plates constituting the restraining sleeve (Fig. 17c);

2) global brace buckling due to the transmission of compressive forces to the sleeve when the working stroke was exceeded (Fig. 17d). In particular, among the four tested braces the latter mechanism occurred in only one of them.

Finally, at the end of the last loading cycle the tensile fracture of the inner core was recognized in the braces that globally remained stable and the experimental test was completed.
loss of strength and stiffness because of their global buckling in compression, it is worth to note that the cyclic response was satisfactory stable. The degrading behaviour of CCBs may be recognized by comparing the envelope curves of the cyclic response. Indeed, the positive envelope curve, which represents loading phases with BRBs in compression and CCBs in tension, shows structural strength and stiffness larger than the ones shown by the negative envelope curve, which instead represents loading phases with CCBs in compression and BRBs in tension (Fig. 19). The tested structure reached a maximum interstorey drift of about 3.0%, with a minimum ductility capacity of about $\mu = 20$ (Fig. 19b). After the achievement of the target interstorey drift ratio of 3%, the test continued applying fully reversed storey displacements in the range of $\pm 1.5\%$ of the first storey height, up to the core fracture. Three and one half cycles were necessary to bring the core to fracture.

3.3.3 Prototype 5

BRBs type 5 showed an excellent response, characterized by the complete efficiency as a ductile fuse up to their maximum design deformation range. As highlighted in Figure 19a, the tested prototype showed a fully operational and symmetric response into the design interstorey drift range ($\pm 2\%$) and allowed to reach in a stable manner the maximum allowable interstorey drift of 3%.

Notwithstanding the presence of CCBs, which affected the global system response in terms of
3. The European Commission: PROHITECH project “Earthquake Protection of Historical Buildings by Reversible Mixed Technologies”.

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The damage pattern corresponding to the maximum drift applied during the test is shown in Figures 19 and 20. At the positive peak of 3% of interstorey drift ratio, the two BRBs achieved the end of their working stroke (Fig. 19a), while the separation between their sleeve components became very apparent (Fig. 19b). At the same time, CCBs were not able to straighten in tension, showing a residual buckled shape. On the other hand, at a negative interstorey drift ratio equal to -3% both BRBs perfectly behaved in tension (Fig. 20a), while the buckled CCBs showed the formation of plastic hinges at both mid span and the ends (Fig. 20b).

4 CONCLUSIONS

Two novel types of “all-steel” BRBs have been tested. The second type is characterized by the possibility to hide the brace inside masonry claddings. Based on test results the following main conclusions can be drawn:

- Wrong connection details usually leads to undesired local-distortional buckling modes of failure, which may impair the correct performance of the device.
- Properly designed, fabricated and erected BRBs showed excellent cyclic inelastic response.
- Even when undesired failure modes occurred, RC frames equipped with BRBs showed superior performance over bare RC frames or frames with classic braces.

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Figure 20. BRB type 5: damage pattern, interstorey drift ratio = -3%.