

SEISMIC RETROFIT OF THE STUDENT HALL OF RESIDENCE OF MESSINA THROUGH BUCKLING RESTRAINED BRACES

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Abstract

This contribution describes a seismic retrofitting intervention designed and currently ongoing in the earthquake-prone area of Messina, Italy, where local strengthening techniques are combined with supplemental energy dissipation devices. This intervention concerns a 5-story building that represents the main student hall of residence of Messina. The structure is made of confined brick masonry in the first four stories, built in the 1930s, with an added fifth story together with a light appendage in reinforced concrete frame, built around 40 years later than the original masonry building. The building has a C shape in plan and exhibits moderate torsional effects, which are undesirable. Buckling restrained braces (BRBs) placed in selected frames of the structure have been adopted as dissipative elements to improve the seismic performance of the building. BRBs reduce torsional effects and increase the dissipation capacity of the building. Furthermore, to reduce the vulnerability of the last-floor reinforced concrete frame, pre-tensioned stainless steel ribbons are used in the beam-column joints of the last elevation, thus enhancing the confinement effect and inducing a beneficial pre-compression state that increases the ductility. The effectiveness of the retrofitting interventions is assessed through pushover analysis on the original and retrofitted structure comparatively. Acceptance tests on the employed hysteretic dampers carried out at the laboratory CERISI of Messina are also described.

Keywords: Seismic retrofit, Buckling Restrained Braces, Hysteretic Dampers, Energy Dissipation, Pre-tensioned stainless steel ribbons, Confined Masonry-RC Building.

1 INTRODUCTION

Several buildings placed in earthquake-prone areas were designed according to past seismic regulations and, consequently, may not comply with current seismic codes in force today. The constantly evolving map of seismic hazard, based on past seismic events, has in most cases increased the peak ground acceleration (PGA) of many installation sites, which makes seismic retrofitting interventions on existing buildings urgent. Furthermore, principles of modern building codes, such as capacity design and ultimate limit states, have been introduced in seismic regulations only in relatively recent times; therefore, it is very likely that existing buildings were originally designed ignoring such principles and concepts. Retrofitting operations are particularly important for those building with high importance class, i.e. strategic because of the underlying social and/or economic implications of an eventual collapse (hospitals, university buildings, schools, etc.). The case-study building discussed in this contribution belongs to this class of strategic buildings and represents the main student hall of residence of the city of Messina, Italy.

Seismic retrofitting of existing buildings can be carried out following two main families of strategies: 1) local strengthening of the structure by increasing the load-carrying capacity of specifically selected structural members, for instance with fiber reinforced polymers [1] or similar ones; 2) modification of the seismic performance of the structure through seismic protection devices, including seismic base isolators [2], [3] or other supplemental energy dissipation devices, like viscous dampers [4], [5], hysteretic dampers, dissipative braces in general [6], [7], or exploiting the advantageous properties of tuned mass dampers [8], [9]. While local strengthening operations are preferable for limited parts of the structure, they may become too expensive for large buildings requiring interventions on many structural members (beams, columns). As a result, in most practical cases these local strengthening operations, applied to critical zones or to specific structural members (deemed to be particularly weak and vulnerable), are conveniently combined with the above-mentioned seismic protection devices for a more effective seismic retrofit of the structure. Such a combined seismic retrofit strategy has also been adopted in the case-study building described in this contribution.

The earthquake events of L'Aquila (Italy) in 2009 led to the dramatic collapse of many public buildings, among which the university hall of residence. This catastrophic and emblematic collapse has induced a number of regional authorities in other parts of Italy to carry out surveys and structural analyses of similar student accommodation buildings, unveiling considerable structural deficiencies in many instances. Indeed, the student hall of residence of the University of Messina is one of such buildings that requires urgent retrofitting interventions. Of particular interest to the present paper, it is worth recalling that the area of Messina strait experienced one of the most disastrous seismic events of modern history, occurred on December 28, 1908 and associated with moment magnitude 7.1 Mw: more than 100000 people died, and around 91% of structures were significantly damaged by the ground motion shaking and concurrent tsunamis. After this tragic event, the city of Messina was reconstructed to a large extent. One of the most popular structural configurations adopted in the reconstruction stage of Messina consists of a confined brick masonry scheme, with confining reinforced concrete (RC) beams and columns casted after the erection of the masonry walls. Originally built in 1930, the student hall of residence of Messina adopted this structural scheme for the first four elevations. The remaining two elevations (fifth floor and a light appendage) were built subsequently (in the early 1970s) with a more modern RC framed structure, which causes a structural heterogeneity in elevation. Moreover, the building has a C shape in plan, with a long front side and two shorter wings, which triggers undesirable torsional effects when subject to horizontal ground motion accelerations. The recent Italian

seismic standards NTC08 [10] have increased the seismic demand parameters and this has required urgent seismic retrofitting interventions.

To this aim, buckling restrained braces (BRBs) are adopted as replacement of some selected masonry walls of the existing building. Besides enhancing the overall dissipation capacity through the incorporated hysteretic dampers, the BRBs are placed in some specific frames that are chosen in an attempt to reduce torsional effects of the building caused by the nonsymmetrical configuration in plan. The BRBs (adopted at all stories of the selected frames) are combined with local strengthening interventions applied to the beam-to-column joints of the last elevation, which were particularly vulnerable to shear actions. In this case, pre-tensioned stainless steel ribbons are employed to create a beneficial triaxial compression stress state and to improve the overall ductility of the RC frame. The effectiveness of the proposed retrofitting intervention is assessed through nonlinear static analysis on the original and retrofitted structure comparatively. Acceptance tests on the employed hysteretic dampers carried out at the laboratory CERISI of Messina are also described.

2 DESCRIPTION OF THE CASE-STUDY BUILDING

The case-study building is located in the city center of Messina. With reference to the photographs and sketches reported in Figure 1, it is noted that the structure has a C shape with a long front side and two shorter wings. The building consists of 5 stories (with a floor area of approximately 1300 m² per story) and a light appendage built in a limited zone of the overall floor area. The first four stories (basement, ground floor, first and second level) were constructed in the early 1930s. The structural configuration for this part of the structure is represented by a confined masonry-RC scheme widely adopted in the reconstruction of the city of Messina after the 1908 earthquake, which is illustratively sketched in Figure 2. In this configuration, the masonry brick walls were originally built with courses left staggered along the wall height and reinforcing bars simultaneously prepared for the column casting. However, both RC beams and columns were casted only after the masonry walls. The mutual collaboration between the two systems (toothing between masonry and RC) increased the out-of-plane resistance of the panel and was effectively exploited to prevent the overturning of the masonry walls, which was frequently observed during the 1908 Messina earthquake in most masonry buildings. In this way, the masonry walls were utilized as formwork of the confining RC beams and columns. Beams with variable sections (increasing near the beam ends), commonly employed at the time of construction, were realized by leaving staggered bricks of the walls as depicted in Figure 2. The resulting hybrid masonry-RC scheme presents a box-shaped configuration and an effective structural collaboration of masonry and RC even for small displacements. This structural arrangement was among the schemes recommended by the seismic regulations (R.D. n. 2089) in force at the time of the reconstruction of Messina [11] and extensively adopted until the Second World War in the area of the Messina strait. On the other hand, the fifth story (third floor) and the overlying appendage (marked in red in Figure 1) were realized nearly 40 years later than the original masonry building (early 70s) with a conventional RC framed structure (and internal partitions in hollow brick masonry walls). The steel reinforcement bars of the added story were welded to the upper portions of the bars of the bottom story of the masonry building. Finally, in the internal courtyard there is another structure ("Mensa building"), which was built in more recent times and is separated from the case-study building by a seismic gap (i.e., it is structurally independent from the analyzed building and will be not considered in the sequel of the paper).

The foundations of the building were realized with a bi-directional grid of inverted T-beams (RC strip foundation). The geotechnical properties of the installation site deposit (identified through two continuous surveys, twenty-four standard penetration tests and

multichannel analysis of surface waves) can be classified as being of a soil type B according to Italian seismic regulations NTC08 ($360\text{m/s} < V_{s30} < 800\text{m/s}$).

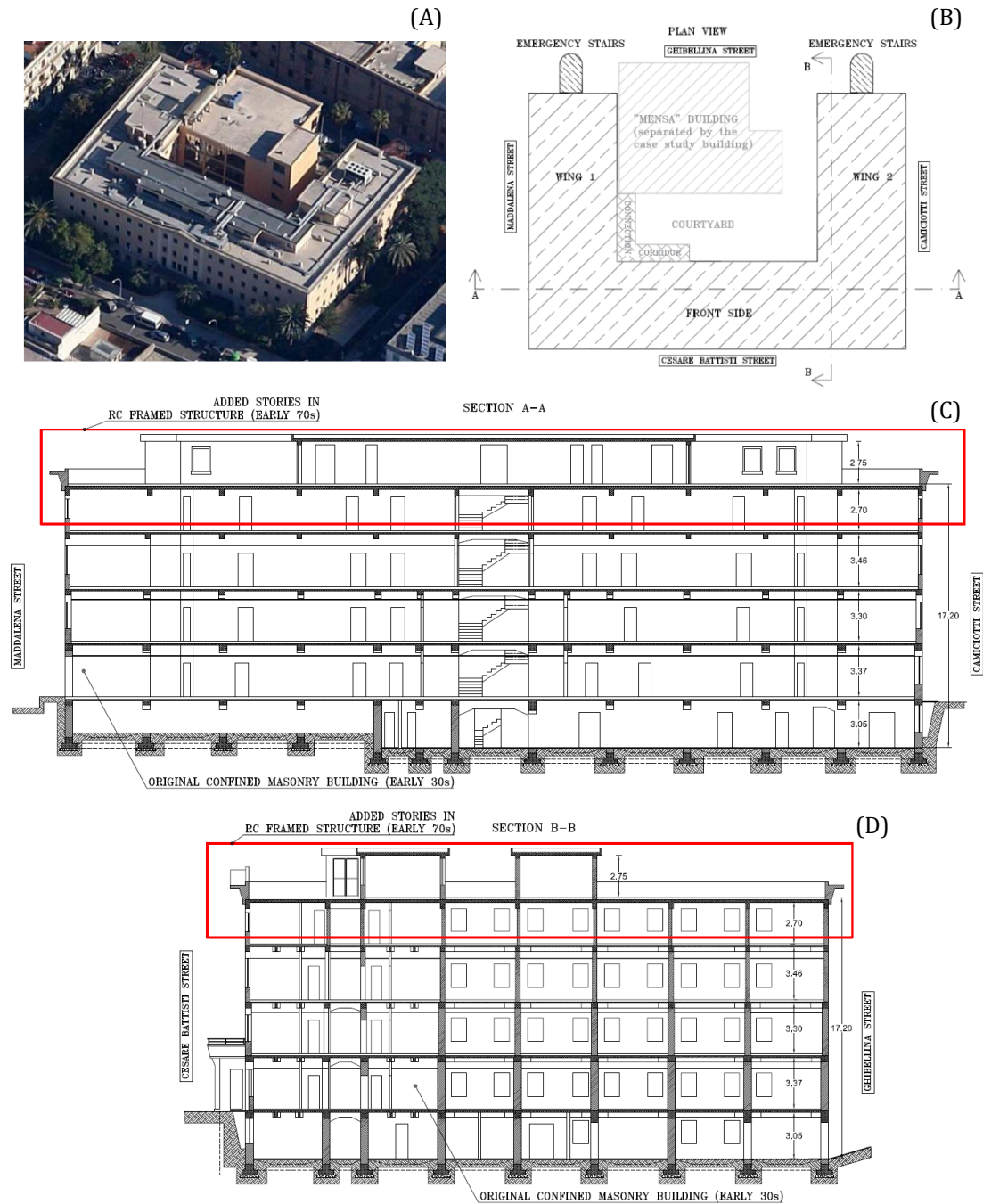


Figure 1 Case-study building: (A) aerial view; (B) plan view; (C) and (D) sections A-A and B-B (front views) with added RC frame highlighted in red

The mechanical properties of the materials were determined through 48 extractions of concrete core cylinder samples, 20 extractions of steel reinforcing bars and 46 pull-out tests (all equally distributed on beam and column members). The average compressive strength of concrete cores R_c in the first four stories was 13.9 MPa for columns and 14.2 MPa for beams, while the yield stress of bars was $f_y = 222.8$ MPa. On the other hand, the analogous quantities

for the added stories of the RC frame were 18.9 MPa, 20.7 MPa and 301.9 MPa, respectively. These values clearly demonstrate the different structural configuration of the two parts of the structure: in the lower part, the RC beams and columns were placed primarily for confining purposes of the masonry walls rather than to represent the main structural system, which instead occurs in the upper parts of the structure (added RC framed structure).

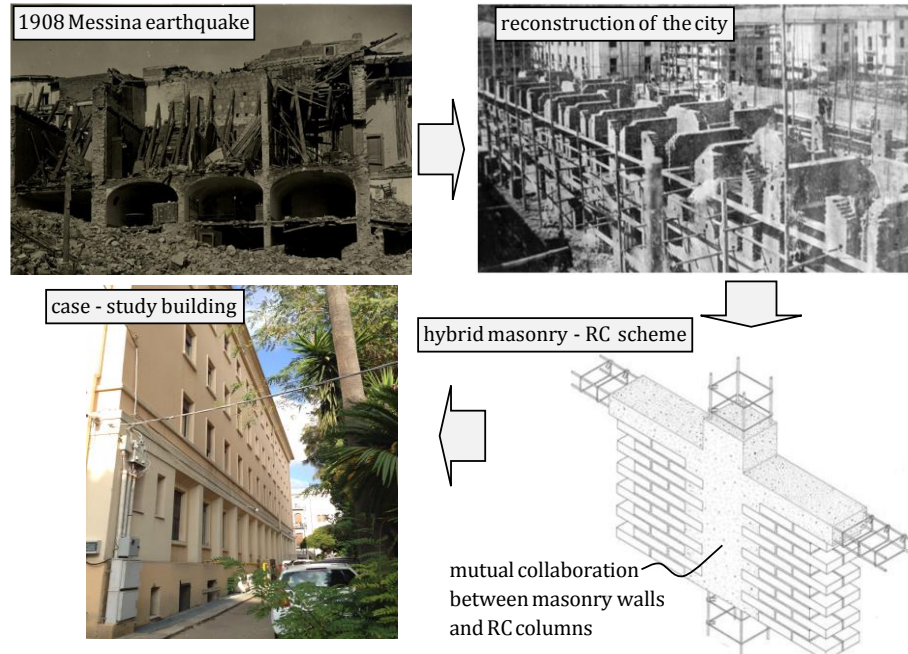


Figure 2 Confined masonry-RC scheme widely used in the reconstruction of Messina after the 1908 earthquake

The cross-section details of RC beams and columns as well as the masonry wall thickness are listed in Table 1. The floor slabs of the first four stories were realized with a RC plate 8cm thick with stiffening beams at every 1.50 m. The floor slabs of the last two elevations (added RC framed structure) were realized with conventional one-way RC slabs with hollow brick as internal lightening elements.

Table 1 Cross-section details of RC beam/column members and masonry wall thickness

Level	RC columns $b \times h$ [cm]	RC beams $b \times h$ [cm]	Masonry wall t [cm]
Foundation	-	50 x 70	-
Basement	50 x 50	50 x 50	50
Ground floor	50 x 50	50 x 40	50
First floor	45 x 45	40 x 45	40
Second floor	35 x 40	30 x 45	30
Third floor (added)	35 x 40; 35 x 30	30 x 60; 50 x 19	-
Fourth floor (light appendage)	30 x 30	30 x 40	-

Since original drawings and documentations of the existing building were found only for the added RC framed structure (last two elevations), a series of simulated calculations have been preliminarily performed to identify reasonable reinforcement arrangement of beams and columns of the first four floors. To this aim, the prescriptions of the seismic code in force at the time of construction, R.D. 2089 [11], were followed for the simulated calculations. The materials considered in the calculations are Aq42 steel (having admissible tensile stress of

1600kg/cm²) and concrete Rck150 (having admissible compressive stress of 50kg/cm²). All the other principal guidelines reported in the seismic code R.D. 2089 [11], here omitted for the sake of brevity, were followed. Two representative frames are considered in the simulated calculations, one on the main front side and one on the wing side of the structure. Assuming the design loads as per R.D. 2089 [11], reinforcement bars are computed for RC columns and beams and subsequently compared to actual reinforcement bar amounts determined through pachometer experimental measurements (rebar locator) and visual inspection after removal of the concrete cover. Selected comparisons are shown in Figure 3. Overall, the simulated calculations lead to results that are in good agreement with actual reinforcement arrangement determined *in-situ*. All the previous surveys and tests have led to a knowledge level LC2 according to the Italian seismic code §8.5.4 [10], which is associated with a confidence factor (FC) equal to 1.20.

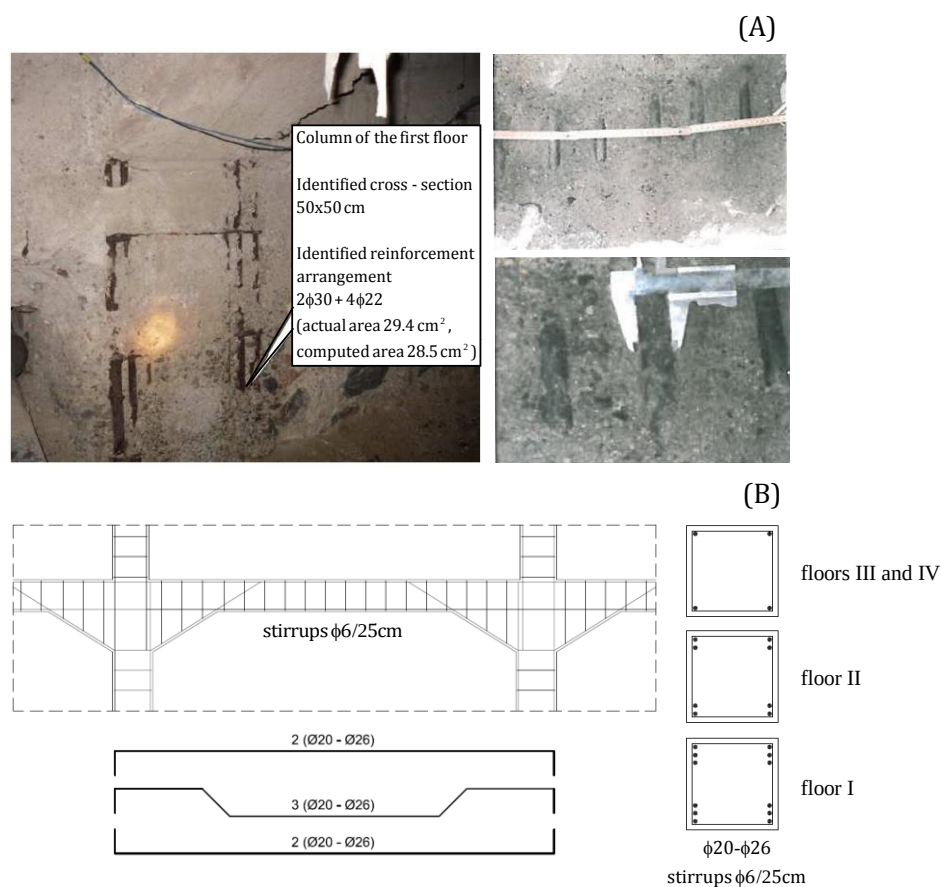


Figure 3 Identification of steel re-bars in RC beams and columns of the first four stories through visual inspection (A) and simulated calculations (B)

3 SEISMIC PERFORMANCE OF THE EXISTING BUILDING

The seismic performance of the building in its original configuration (prior to any retrofitting operation) was evaluated through static nonlinear analysis (pushover analysis – N2 method) [12]. The design loads were established according to the Italian seismic code NTC08 [10], assuming a reference life of the building $V_R = V_N \times C_U = 50 \times 1.5 = 75$ years (relevant to an importance class III, $C_u = 1.5$). Both damage limit states (SLD in Italian seismic code) and ultimate limit states (SLV in Italian seismic code) are considered, which are related to

probability of exceedance equal to 63% and 10% in the reference life of the building, respectively. The associated PGA values are 0.102g and 0.296g, respectively.

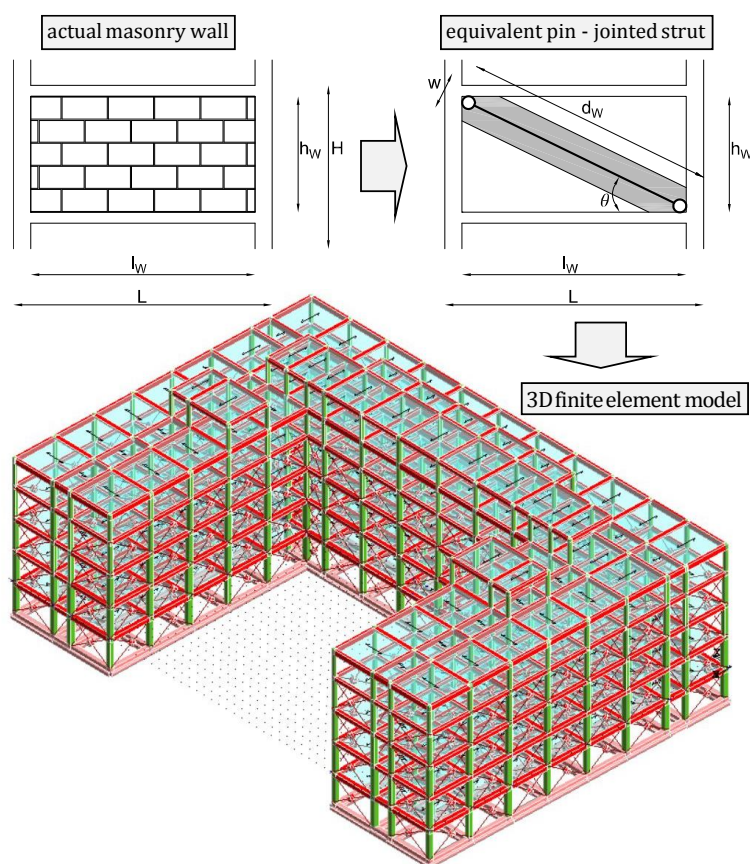


Figure 4 3D finite element model of the original building with masonry walls modelled through the equivalent pin-jointed strut (macro-modelling approach)

A 3D finite element model (FEM) has been developed to simulate the structural behaviour of the existing building, which is shown in Figure 4. After a preliminary assessment of the in-plane stiffness of the existing floor slabs, it has been verified that the conditions of the Eurocode 8 §4.3.1 about the rigidity of the diaphragm are satisfied. Therefore, diaphragm constraints have been applied at each level to simulate the behaviour of a rigid floor slab, which significantly reduces the computational effort of the overall model. The contribution of the brick masonry is incorporated in the model through a macro-modelling approach, by introducing equivalent pin-jointed struts related to the actual geometry and mechanical characteristics of the masonry walls. There exists a broad variety of empirical expressions in the literature to determine the dimensions of the equivalent strut in a consistent manner [14]-[16]. In the present paper, the simplified approach incorporated in the Italian guidelines [17] is used, whereby the nonlinear response of the masonry wall is modelled through an elastoplastic idealized behaviour. The shear strength of the masonry without any applied vertical load f_{vk0} , the Young's modulus of the masonry E_w and the wall thickness t_w represent the input parameters of the adopted model to determine the ultimate strength of the strut, the lateral stiffness and the yield and ultimate displacement of the elastoplastic model [17].

A plastic hinge approach has been adopted to incorporate the nonlinear behaviour of RC beams and columns, accounting for the confinement effect due to the presence of stirrups as per Italian regulations NTC08 [10]. The moment-curvature relationship is determined through

conventional analysis of the flexural plastic behaviour of the section. The curvature capacity of the plastic hinges is evaluated in accordance with the Italian code, assuming the definitions of the deformation capacity in terms of the chord rotation as reported in § C8A.6.1 [10].

According to the pushover method, the structure is subject to vertical loads and a set of monotonically increasing lateral loads. Two load distributions are considered, the mode distribution and the mass proportional distribution. From the base shear versus roof displacement curve of the multi-degree-of-freedom (MDOF) system, energy equivalence is applied to determine an equivalent single-degree-of-freedom (SDOF) model with a bilinear idealization [12]. Then, seismic performance of the building is assessed by comparing seismic demand (from the response spectrum) with seismic capacity (from the resulting SDOF pushover curve). A series of 16 different combinations are considered (four directions of the ground motion, two profiles of lateral loads and two additional eccentricities), from which the verification is carried out in terms of $PGA_{\text{capacity}} / PGA_{\text{demand}}$ (ratios lower than one indicate that the structure is not safe). For damage limitation requirement (SLD, return period 75 years) the lowest ratio was 0.536; for no-collapse requirement (SLV, return period 712 years) the lowest ratio was 0.20. As a result, in either case the structure is not safe, meaning that the displacement demand exceeds its displacement capacity. Consequently, the structure needs retrofitting interventions.

4 SEISMIC RETROFIT OF THE BUILDING

The most critical aspects of the original building detected by the previous analysis concerns the premature shear failure of the T-beams in the foundation, the moderate torsional behavior due to the non-symmetrical configuration of the building in plan (along the x axis) and the unsatisfactory dissipation capacity of the structural members due to a generalized lack of steel reinforcement amounts, especially in the critical zones (beam-column joints). In essence, the structure does not comply with capacity design principles.

To retrofit the foundation, the cross-sections of the inverted T-beams were enlarged to increase the stiffness and strength. This has involved the following steps: removal of the concrete cover, introduction of new reinforcement bars connected to the existing T-beams through rheoplastic resin, and final casting of concrete. Details of these retrofitting phases are here omitted for the sake of brevity and also because they follow conventional retrofitting procedures of existing RC members and are not of particular interest to this contribution.

What is more interesting to discuss here is the introduction of BRBs (incorporating hysteretic dampers) to improve the seismic performance of the building by reducing torsional effects and increasing the overall dissipation capacity. The underlying theoretical principles of BRBs date back to the early 1970s, although devices with a stable force-displacement curve were developed and tested in the following years [18]. BRBs dissipate energy by hysteresis (plastic deformation and yielding) of an internal steel core element, placed within a concrete-filled steel tube. The risk of buckling of such steel element for high compressive loads is prevented by the surrounding concrete or grout, which ensures a stable (symmetric) cyclic force-displacement curve in both tension and compression. However, low-friction coating materials are introduced in the device to prevent the transmission of axial force from the steel core to the surrounding concrete/grout. In this way, the steel core is axially disconnected from the filling material. After attainment of the yielding deformation of the steel core, BRBs undergoes large deformations without decreasing strength. Nowadays, BRBs have been extensively applied worldwide, especially for steel structures in Japan [19] and in the United States [20], [21].

The BRBs were inserted as replacement of the original confined masonry walls at all stories of the building (from ground floor to the top floor). The adoption of BRBs was motivated by the limited flexibility of the masonry-RC structure (first natural period equal to 0.4s). In this regard, the use of alternative supplemental energy dissipation devices like viscous dampers [22]-[26] might be not effective because they may not be fully engaged due to the small interstory drifts experienced by such stiff structures. On the other hand, the plastic deformation of BRBs is activated even for relatively small displacements (the yield displacement of the internal steel core may be of few millimeters), which renders these devices particularly effective for stiff structures, as in the masonry-RC building here analyzed.

The BRBs were introduced in the 3D FEM as shown in Figure 5 through an idealized elastoplastic constitutive behavior, based on the force-displacement characteristics identified in the experimental tests, which will be described below. A series spring model incorporating the brace stiffness k_b along with the damper stiffness $k_d = F_y / u_y$ is assumed. The position of the BRBs in plan has been carefully chosen so as to align the center of mass (CM) and the center of stiffness (CS). In Figure 6 the old CS (of the existing building) and the new CS (of the retrofitted building with BRBs) are depicted against the CM. Since the building is symmetric with respect to the y axis but not with respect to the x axis, BRBs were mostly concentrated near the two wings of the building to compensate for the asymmetrical stiffness distribution on that side.

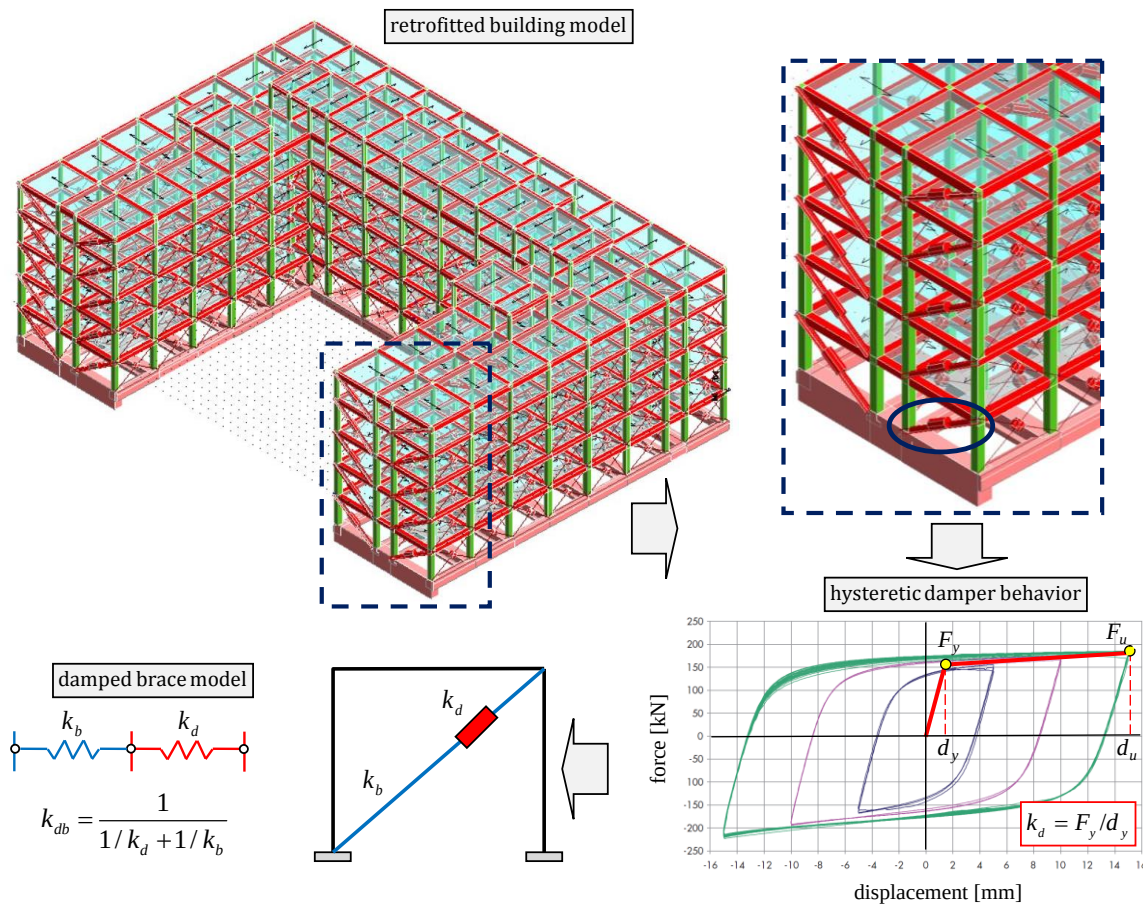


Figure 5 3D finite element model of the retrofitted building with BRBs modelled with elastoplastic behavior

The parameters of the hysteretic dampers were preliminarily estimated through a direct displacement-based design procedure [27], [28], combining pushover analysis of the real MDOF structure with response spectrum analysis of an equivalent SDOF system. For practicality reasons, only one family of hysteretic dampers has been considered, having yield displacement $d_y = 1.63$ mm , corresponding force $F_y = 737$ kN , ultimate displacement $d_u = 15$ mm and corresponding force $F_u = 1065$ kN . More details of the adopted hysteretic dampers will be provided below. As already said, the yield displacement of the hysteretic dampers is lower than 2 mm, which ensures that such devices are fully engaged even for small interstory displacements (as in the case study building), provided the terminals of the device are effectively attached to the existing structure with the necessary torque moment in order to avoid any loosening effect.

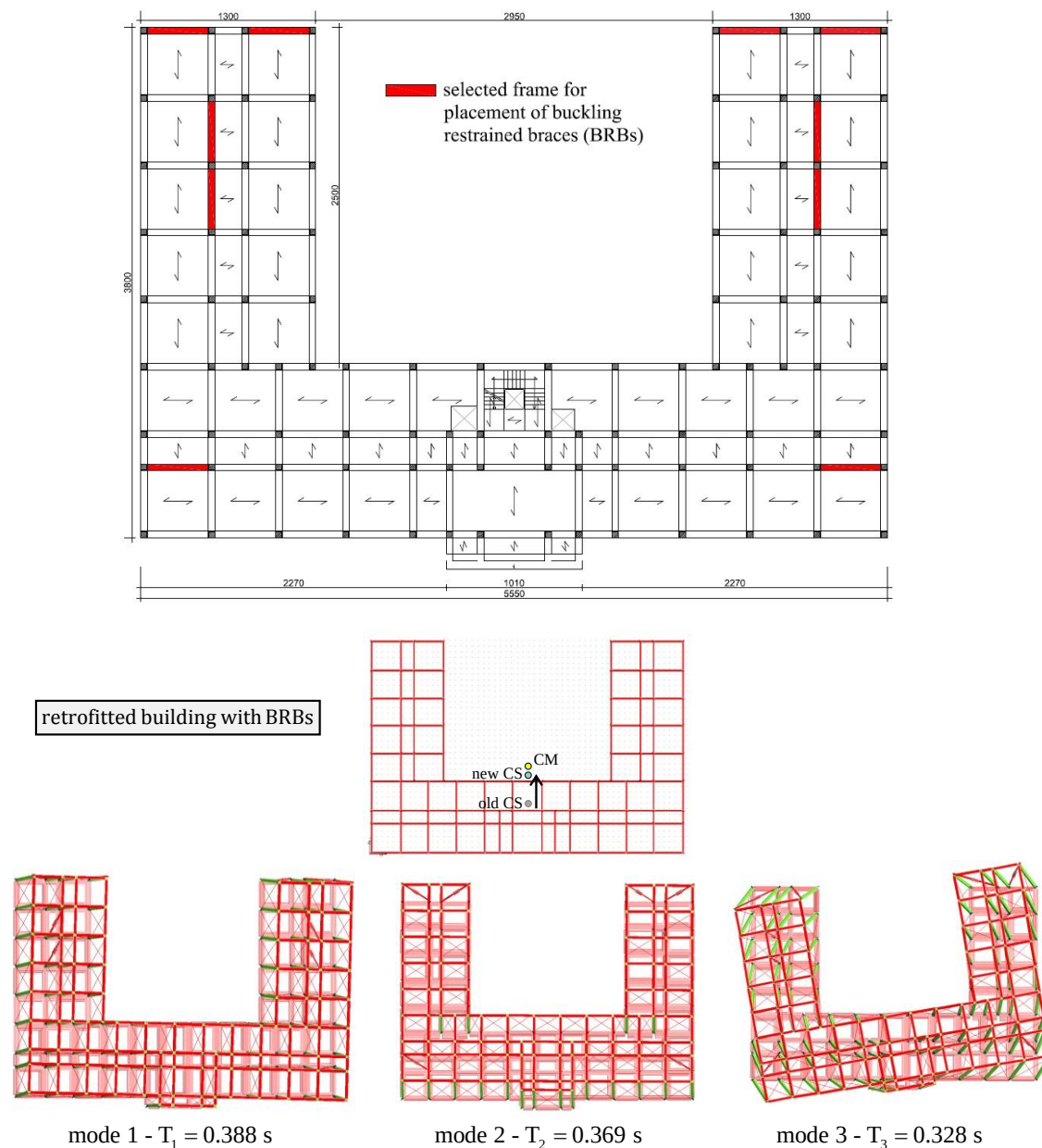


Figure 6 Selected frames for the introduction of BRBs (to reduce torsional effects of the existing building)

However, a preliminary pushover analysis of the retrofitted structure with BRBs shed light on the vulnerability of the beam-column joints of the last elevation. This is motivated by the different flexibility characteristics of the two portions of the structure, namely the bottom confined masonry-RC structure and the upper RC framed structure. Evidently, the upper portion of the building has lower stiffness than the lower part, thus behaving as a more flexible SDOF system appended at the top of the underlying confined masonry-RC building. Consequently, large plastic deformations are concentrated in the beam-column joints of the last elevation, which represents the principal cause of collapse due to the low amount of steel stirrups. Due to such premature (brittle type) shear failure of the beam-column joints, the structure with BRBs still did not meet the requirements of the Italian code, as the seismic demand exceeded the seismic capacity.



Figure 8 Strengthening of beam-column joints of the last elevation through pre-tensioned stainless steel ribbons

Therefore, in addition to the installation of BRBs, the seismic retrofit of the building also involved pre-tensioned stainless steel ribbons applied to the beam-column joints of the last elevation to increase the shear capacity. The pre-tensioned stainless steel ribbons, applied for an overall length of 100 cm above the beam-column joint, offer a beneficial triaxial compression stress state and act as external stirrups – see design drawing and photographs in Figure 8. Design and verification of the strengthening system was carried out using a macro-element software package. Without going into details of the calculations, the shear capacity of the beam-column joints increased of 230% and 260%, passing from 86kN and 75kN to 191kN and 165kN in the two main directions, respectively.

The seismic performance of the retrofitted structure has been re-assessed through pushover analysis. In Figure 9 we report the results in acceleration-displacement response spectrum (ADRS) format for the two main directions and two load profiles. Comparing the seismic capacity with the seismic demand, it emerges that the retrofitted structure satisfies the requirements of the Italian seismic code [10] for both the SLD (damage-limitation requirement) and SLV (no-collapse requirement). The progressive formation of the plastic hinges was quite in line with a (desired) global collapse mechanism, with development of plastic zones in the beams prior to those in the columns. The ratio of the capacity PGA to the demand PGA exceeds one for all the 16 design scenarios analysed (the lowest ratios were 1.089 for SLD and 1.021 for SLV). Instead the safety index with respect to brittle failure modes (shear failure of beams) was found to be greater than 1.25 for all the combinations.

The hysteretic dampers of the present structure were realized by the Italian manufacturing company C.M.M. F.lli Rizzi s.r.l.. In line with the prescriptions of the European regulations for antiseismic devices EN 15129 [29], factory production control (FPC) tests were performed before installation *in situ*. These dampers are classified as displacement-dependent devices and, according to [29], they were tested under 5 fully reversed cycles with amplitude

$d_u/4 = 3.75$ mm, 5 fully reversed cycles with amplitude $d_u/2 = 7.5$ mm and 15 fully reversed cycles with amplitude $d_u = 15$ mm. The FPC tests were performed at the laboratory Eurolab of the CERISI – Centre of Excellence Research and Innovation of Large Dimensions Structures and Infrastructures, whose main hydraulic and mechanical characteristics can be found in [30]. We limit ourselves to recall the load capacity up to 3100 kN, the stroke of the actuators up to ± 550 mm and the maximum allowed velocities up to 1100 mm/s in the main direction (x axis) of the testing equipment.

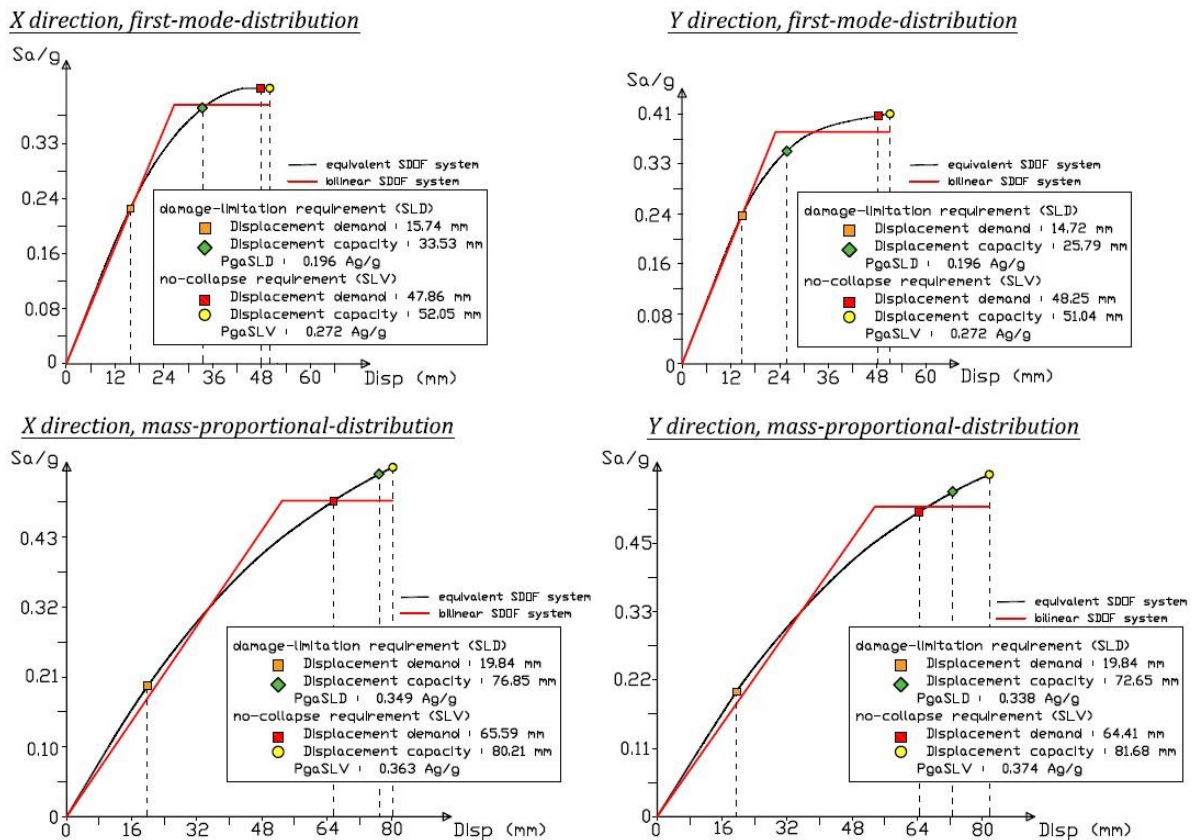


Figure 9 Pushover curves of the retrofitted structure for different loading scenarios

A sketch of the hysteretic damper, some photographs of the testing equipment of the CERISI, and experimental force-displacement curves for the three above-mentioned tests (at the three increasing displacement amplitudes) are illustrated in Figure 10. The damper is more than 1 m long, with a rectangular cross-section of 290mm x 390 mm. Considering the relatively limited displacement (yield displacement of 1.63mm), it was necessary to install linear variable displacement transducers (LVDTs) at the two ends of the device to measure the actual axial deformation of the internal steel core. Indeed, the displacement measures from the hydraulic actuators were found to be significantly influenced by loosening effects of the bolt connections and, thus, affected by a series of additional spurious displacements. These spurious displacements had to be totally eliminated from the force-displacement curves for a correct assessment of the hysteretic characteristics. This also implies that the effectiveness of this kind of devices is strongly related to careful installation operations that allow the transfer of the interstory displacement from the two terminals of the braces to the internal steel core. By inspection of the force-displacement curves shown in Figure 10, the experimental values

of the hysteretic parameters (d_y, F_y, d_u, F_u) were found to be in reasonable agreement with the theoretical design parameters adopted in the calculations.

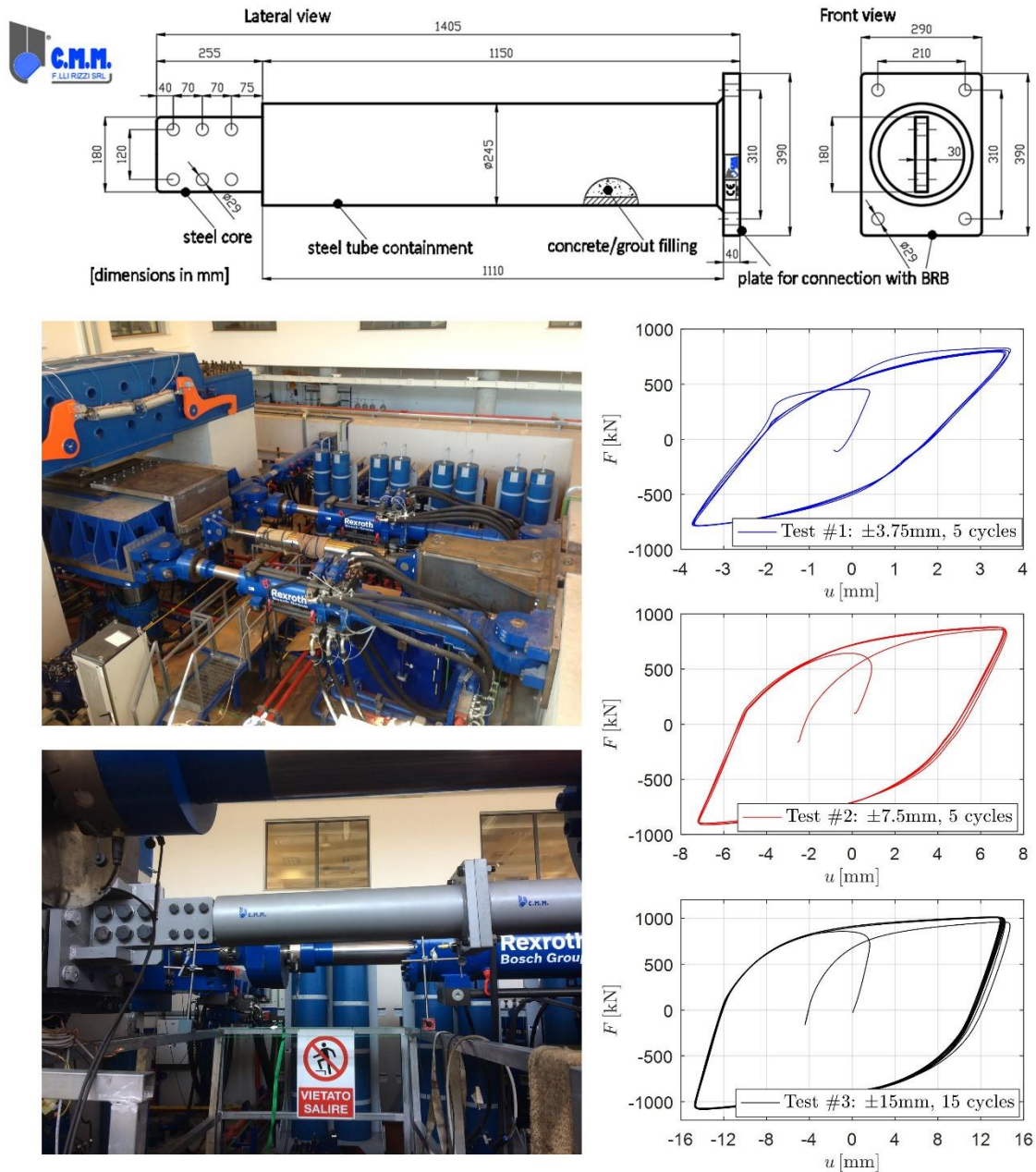


Figure 10 FPC tests according to EN 15129 [29] of hysteretic dampers performed at the laboratory EUROLAB of the CERISI, Messina, Italy

5 CONCLUSIONS

This contribution has summarized a seismic retrofitting intervention designed and currently ongoing in the earthquake-prone area of Messina, Italy. After the tragic collapse of the university hall of residence of L'Aquila during the recent seismic events of 2009, similar student accommodation buildings in other Italian cities were studied to investigate their structural safety against seismic loads. The analyzed case study building represents, indeed,

the main student hall of residence of Messina. The area of Messina strait experienced one of the most catastrophic seismic events of modern history in 1908, with more than 100000 deaths. Despite this, many public buildings are still highly vulnerable to seismic actions and need to be retrofitted urgently. The case study building was structurally deficient because of a number of reasons among which we mention: 1) inadequate foundations; 2) moderate torsional behavior due to the C-shape in plan; 3) heterogeneity of the structural configuration in elevation, with a RC framed structure added on the underlying confined masonry-RC structure around 40 years later than the original construction; 4) unsatisfactory dissipation capacity, due to low transverse reinforcement (especially in the critical zones such as beam-column joints) and absence of capacity design principles.

The retrofitting interventions have considered these critical aspects. More specifically, foundation structures have been strengthened through a set of additional RC plates connected to the existing inverted T-beams (although this was not discussed extensively in this contribution for the sake of brevity). A series of BRBs were added in specifically selected frames of the structures at all the floors, as replacement of the brick masonry walls (at the first three elevations) and the masonry infills (at the last two elevations) in order to minimize torsional effects as well as increase the dissipation capacity of the structure. The frames where the BRBs are installed were selected in an attempt to align the center of mass with the center of stiffness as close as possible. The choice of hysteretic dampers (in place of other supplemental energy dissipation devices) was motivated by the nature of the existing building, which has low flexibility characteristics (quite stiff). Since the yield displacement of such devices is of few millimeters, they are engaged even for small interstory drifts experienced by the structure under design seismic loads. Finally, pre-tensioned stainless steel ribbons were applied to the beam-column joints of the last elevation in order to increase the shear capacity and to compensate for the lack of adequate amount of steel stirrups in such critical zones. These zones were found to be particularly critical with high concentration of plastic deformations due to the high flexibility characteristics of the added RC frame that behaves as a flexible mass oscillating on the underlying masonry building. FPCs tests on the employed hysteretic dampers were also described.

The above set of retrofitting interventions has improved the seismic performance of the structure significantly. The seismic capacity of the retrofitted building, evaluated through the pushover analysis, now exceeds the seismic demand imposed by the Italian seismic code, both for SLD (damage limitation requirement) and for SLV (no-collapse requirement) for all the design combinations analyzed. Although limited to the presentation of a particular case study building, the authors think that the retrofitting interventions proposed here are applicable to other buildings having similar structural configurations and heterogeneity of materials in elevation, which may be the case of several structures constructed around 50 years ago in other cities.

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