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# AN INNOVATIVE SEISMIC PROTECTION SYSTEM FOR EXISTING BUILDINGS: EXTERNAL DISSIPATIVE TOWERS

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## Abstract

A common technique for the retrofit of existing structure is based on the installation of dissipative devices connecting adjacent storeys of buildings in either diagonal or chevron brace configurations. This type of damping system may present some disadvantages like the increment of axial forces in columns, which may leads to premature local failures, or some feasibility limits on the strengthening of the existing foundations at the base of the bracing system. In addition, indirect costs related to the interruption of the building utilization during execution of the retrofit can be very demanding, in particular for strategic buildings, such as hospitals or schools. Most of previous problems can be overcome by placing the dissipative bracings and the relevant foundations outside the building; systems with external dampers can be grouped into three main categories, depending on the kinematic behaviour of the system, which is a function of the arrangement of dampers and bracings. Recently, some applications have been developed by proposing a new configuration exploiting the rocking motion of a stiff external tower. This paper deals with this innovative system for seismic protection of existing buildings, especially strategic ones, patented as “Dissipative Towers”. The protection system is based on the structural coupling of the building with new steel truss towers constructed externally and then rigidly connected to the building floors by means of steel elements; the towers are erected over a rigid r.c. thick base plate that is restrained to the foundation cap with a spherical hinge located in central position of the base slab. The towers are equipped with dissipative devices connecting the corners of the two plates; the effectiveness of the dampers is enhanced by the use of articulated quadrangles which amplify the vertical displacements of the devices. The efficiency of the system is so high that usually it is designed to satisfy the immediate occupancy limit state even for high intensity earthquakes. The above system is applied to the retrofit of an existing school building in Italy, constituted by different blocks made of reinforced concrete frames. The seismic rehabilitation is obtained by suitably positioning external dissipative towers and eliminating expansion joints between adjacent blocks. The towers allow a high level of seismic protection at the ultimate limit state, with a significant reduction of horizontal displacements and accelerations. Moreover also the shear actions resisted by the existing frame are significantly reduced by the "Dissipative Towers". The seismic protection is achieved with a moderate economic impact due to the elimination of indirect costs related to the arrangement of internal spaces, interruption and/or relocation of activities.

*Keywords: Seismic Retrofitting; Steel Dissipative Towers; Strategic Buildings; Viscous Dampers*

# 1 Introduction

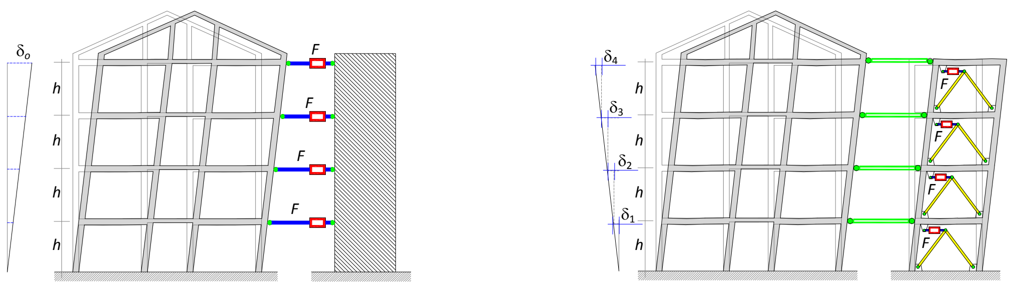
The seismic protection of both new and existing buildings, especially strategic ones, is a current issue that involves not only structural but also economical and functionality aspects. Among the others, passive control systems have proven to be very efficient solutions for the seismic design and retrofit of existing structures [1, 2].

With reference to existing structures, a traditional retrofitting technique consists in the installation of dissipative devices (*e.g.* viscous dampers or hysteretic devices) within a building frame in either diagonal or chevron brace configurations connecting adjacent storeys. In this framework an interesting application is constituted by the use of viscous dampers, for which studies concerning both the dynamic properties of the damped system and design methods are available in the literature [*e.g.,* 3, 4]. However, this type of damping system may also present some disadvantages like the increment of axial forces in columns, which may leads to premature local failures [5], or some feasibility limits on the strengthening of the existing foundations at the base of the bracing system. Also, the indirect costs related to the interruption of the building utilization during execution of the retrofit can be very demanding, in particular for strategic buildings, such as hospitals or schools.

These problems can be overcome by placing the dissipative bracings and the relevant foundations outside the building. Systems with external dampers can be grouped into three main categories depending on the kinematic behaviour of the system, which is a function of the arrangement of dampers and bracings. Whichever is the configuration, they all permit to control both the total amount of the dissipated energy and the frame deformation at the different storeys. A possible configuration is obtained by placing dampers horizontally at the floor level, between the frame and an external structure, which can be a new stiff structure [6] or an adjacent building [7] (Fig. 1a). In this case dampers are activated by the floor relative horizontal displacements and the system efficiency is strongly related to the dynamic properties of the connected structures. An alternative solution can be obtained by coupling the frame with an external shear deformable bracing structure equipped with dissipative devices; in this case the two structures are rigidly connected at the storey levels and the dissipative devices are activated by the interstorey displacements, as in the more traditional case of bracings placed within the existing structure (Fig. 1b).

Recently, some applications have been developed by proposing a new configuration exploiting the rocking motion of a stiff truss tower, known as "Dissipative Towers" [8], hinged at the foundation level [9, 10] and connected to the existing building at floor levels (Fig. 2). The dampers are located, in vertical position, at the tower base and are activated by displacements induced by the tower base rotations.

This paper presents the application of the latter system to the retrofit of an existing school building in Italy, constituted by different blocks made of reinforced concrete frames. The seismic rehabilitation is obtained by suitably positioning external dissipative towers and eliminating expansion joints. The towers allow a high level of seismic protection at the Ultimate Limit State (ULS), with a significant reduction of horizontal displacements and accelerations. The seismic protection is achieved with a moderate economic impact due to the elimination of indirect costs (arrangement of internal spaces, interruption and/or relocation of activities).




a) b)

Fig. 1 - Seismic retrofitting systems with external structure: dissipative devices activated a) by absolute floor displacements, and b) by interstorey drifts

## 2 Description of the "Dissipative Towers" retrofitting system

The “Dissipative Towers” system is based on the coupling of the existing structure with new steel truss towers (Fig. 3a) that are built externally and equipped with dissipative devices.

Each tower is erected on a r.c. thick base plate that is centrally pinned to the thick plate of the pile foundation cap by means of a spherical support (Fig. 3b). The truss tower is characterized by high stiffness and is connected to the building floors by horizontal steel braces so that a horizontal displacement of the building induces a rotation of the tower around its base hinge. Braces connecting towers to the building are bolted to thick steel plates that are anchored to lateral beams of the building; sometimes, to avoid the concentration of the interaction forces on few frame elements, horizontal steel rods are used to transfer a certain amount of the interaction forces to the beams of the internal frames.

The dissipative devices employed are generally nonlinear viscous dampers whose response is usually described by an exponential constitutive law of the form [3]

$$F = c|v|^{\alpha} \quad \mathrm{sgn}(v) \tag{1}$$

where $F$ is the device viscous force, $c$ is the damping coefficient, and $\alpha$ is a parameter which defines the nonlinear behaviour ($\alpha$=1 for the linear viscous damper). Viscous dampers are located between the base plate and the foundation plate, close to the vertices, and are mounted in vertical position so that the rigid rotation of the base plate, due to horizontal displacements of the building, activates simultaneously all the devices. In order to increase the device efficiency, dampers are inserted into an articulated quadrangle (Fig. 3b) that amplifies the device motion (displacement and velocity) with respect to the vertical displacement of the base slab vertices.

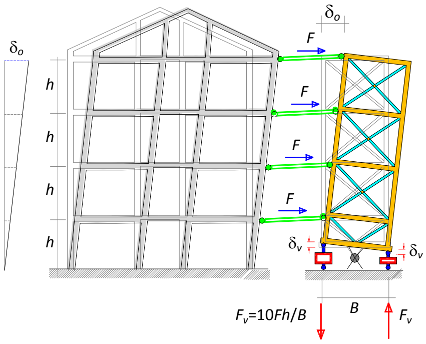



Fig. 2 - System with Dissipative Tower

a) b)

Fig. 3 - a) Building coupled with Dissipative Towers; b) Detail of the tower base: spherical hinge and viscous dampers on lever mechanisms

The bracing system influences both the stiffness and the damping properties of the coupled system, while its contribution on the mass is negligible. The stiffness of the truss tower can be properly designed to make nearly uniform the interstorey drift of the building so that the Serviceability Limit State (SLS) verifications are satisfied even at the lower storeys (devices may be designed to avoid damages of non structural elements even in the case of high intensity earthquakes). As regard damping, dissipative devices are activated by the rocking motion of the tower whichever is the input direction, differently from traditional systems where dissipative devices work for one specific direction.

The effectiveness of the "Dissipative Towers" system is very high compared with other solutions; with regard to the ones shown in Fig. 1, by considering non linear viscous damper with a very low exponential coefficient $\alpha$ (as usual for practical applications) and by assuming a uniform interstorey drift $\delta$, a roughly estimation of the dissipated energy within a cycle ($E_D$) is provided by the following expressions for the system of Fig. 1a and Fig. 1b, respectively:

$$E_D/4 = (4+3+2+1)F\delta = 10F\delta \tag{2}$$

$$E_D/4 = (1+1+1+1)F\delta = 4F\delta \tag{3}$$

where $F$ is the dampers viscous force at each level. On the other hand, energy dissipated by the "Dissipative Towers" system (Fig. 2) can be roughly estimated through

$$E_D/4 = 2F_v\delta_v = 2(4+3+2+1)Fh/B{\times}B\delta/2h = 10F\delta \tag{4}$$

Previous equations demonstrate that the proposed system has potentially the best dissipative performance among previous ones, as dampers displacements are amplified by the lever mechanisms. Thanks to its very high effectiveness, the system may be designed to satisfy the Immediate Occupancy Limit State (IOLS) even for high intensity earthquakes. Consequently, the building remains in the elastic range and this: *i*) allows the refocusing of the system after a seismic event thanks to the elastic restoring force of the building; and *ii*) avoiding structural damages. Finally dissipative elements constituting the protection system do not need to be substituted.

On the other hand, high precision metallic carpentry and bolted joints, as well as mechanical leverage with very low tolerances, more typical of mechanical engineering than civil engineering, are required to guarantee the activation of damping devices even for the low values of the building displacements required by the verifications at the IOLS. Furthermore, interaction forces acting through the connecting braces can be very high and particular attention must be paid to the connection design.

This system presents all the advantages of the external retrofitting systems, namely, it enables the retrofitting works to be carried out without interrupting the activities inside the building since the construction of towers, foundations and the connection with the existing structure do not interfere with building activities. Furthermore, the external retrofitting system can host elevators or emergency stairs, thereby providing accessory benefits and allowing the building to be upgraded to safety and accessibility standards. Finally, the external system is easily removable, and permits to restore the building to its original state.

The above mentioned aspects lead to a considerable economic saving than traditional retrofitting techniques, particularly in the case of schools and hospitals, where interfering with internal spaces and interrupting building functionality (for retrofitting works, structure recentering, devices substitution, local damage repair works) may lead to high costs, both from economic and social points of view.

# 3 Case study: school building in Avezzano

## 3.1 Description of the building

The High School B. Croce in Avezzano town, not far from L’Aquila (Italy), is a 4-story r.c. building constructed in the 60’s, which needed to be seismically retrofitted to meet the recent Italian seismic regulations [11]. The innovative system "Dissipative Towers" was adopted to carry out retrofitting works without interrupting the activities inside the building, which is composed of 3 main 4-story blocks (A, G, and D) placed around a 1-story block (C-AM). Other two 1-story blocks (B and D) are located laterally to block D. Fig. 4 shows a plan view of the entire building with the dissipative towers, while Fig. 5 illustrates sectional elevations of the block A.

In the sequel the retrofit of block A will be presented; this has a plan dimension of about 13 × 48 m, in transverse ($y$) and longitudinal ($x$) direction, respectively. The first floor is located about 1.3 m above the ground level, the interstorey height is 3.5 m and the last floor has a medium height of about 1.5 m. The concrete frame structure has 2 spans of 6.6 m and 2.8 m, respectively, in the transverse direction ($y$) and 12 spans of 3.9 m in the longitudinal ($x$) direction. Columns have 300 × 600 mm cross sections, with the greater dimension oriented in the transverse ($y$) direction, beams carrying vertical loads have 300 × 600 mm cross sections whereas secondary beams have 300 × 450 mm or 450 × 160 mm cross sections.

## 3.2 Building retrofitting

The seismic retrofit of the buildings has been obtained with six external dissipative towers connected to each floor, excluding the first one (Fig. 3a and Fig. 5). The main blocks A and G are protected with two steel towers per block, located at the back side. Blocks B, D and F, originally separated by expansion joints, are protected with two dissipative towers, which reciprocally connect the three blocks (Fig. 4). Towers protecting blocks A and G have been used to locate a lift (TA) and an emergency stairwell (TS). Eight dissipative devices (two devices per vertex) are located between the base plate and the foundation plate: these are constituted by nonlinear viscous dampers with a damping coefficient $c$ = 320 kNs/m and a damping exponent $\alpha$ = 0.15.

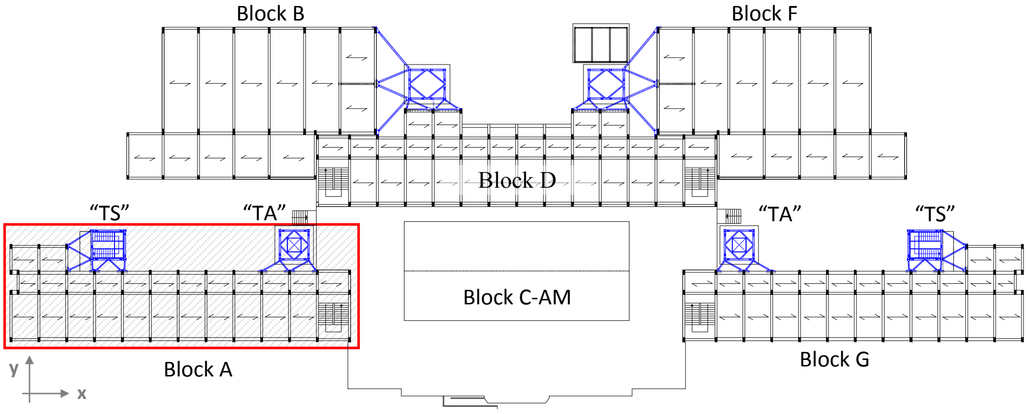



Fig. 4 - First floor plan view

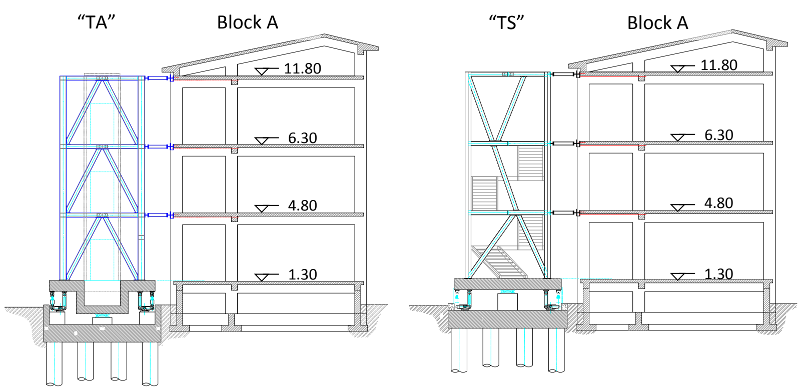



Fig. 5 - Sectional elevations of block A with dissipative towers in the transverse direction

## 3.3 Seismic response

This section reports some results concerning the seismic response of the system, before and after the retrofit. The seismic action is constituted, for each investigated Limit State, by three groups of artificial earthquakes, each one constituted by two signals acting in the principal directions of the structure. Accelerograms have a total duration of 25 s and are characterised by a stationary part duration of at least 10 s, as required by the Italian Standards [11]. Each signal is generated so that its acceleration response spectrum matches the code one for a reference period $V_R$ of 75 years, soil category B and topographical category T1.

The design of the dissipative system is based on nonlinear analyses perfermod with a finite element model, developed with the structural analysis program SAP2000 [12]. Structural members are modelled with 2-node beam elements while sheel elements are used for floors and the towers base plates. The protected structure is assumed to behave elastically and a 5% structural damping is introduced in terms of Rayleigh damping, calibrating stiffness and mass coefficients on the first two vibration periods of the protected structure. A-posteriori checks are performed to evaluate the correctness of the previous assumption. Viscous dampers are introduced exploiting a built-in library nonlinear link based on the Maxwell model. A pictorial view of the developed structural model is shown in Fig. 6

Fig. 7a, b and Fig. 8a, b show the time-histories of displacements at the third level (+11.80 m) of block A, measured in the two main directions, respectively, for the three bidirectional seismic excitations at the ULS before and after the retrofit. The violet dashed line plotted in the graph represents the displacement limit for the IOLS provided by the Italian Standards [11] for frames with infilled walls rigidly connected to the frame, obtained by considering an allowed maximum interestorey drift equal to $0.0033$. The reduction of the maximum displacement with respect to the bare frame is nearly 61% in the longitudinal direction and nearly 41% in the transverse one. Moreover, displacements measured in the retrofit configuration are very close to the maximum allowable ones for the IOLS; this means that even for high intensity seismic actions the building behaves well with regard to non-structural components.

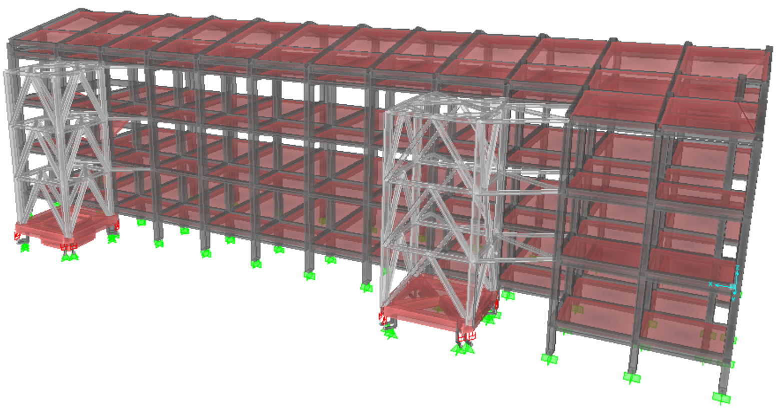


Fig. 6 - Extruded view of the structural model

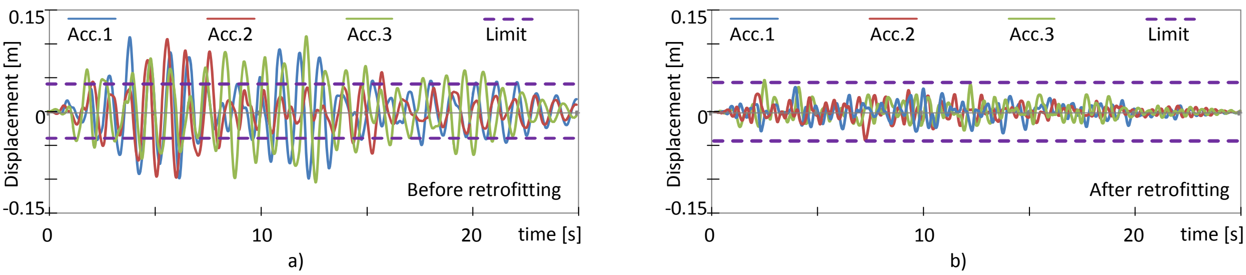



Fig. 7 - Time histories of displacements in longitudinal direction: a) before retrofitting; b) after retrofitting

Fig. 9a, b and Fig. 10a, b show the time histories of the absolute accelerations in the two principal directions, respectively, at the ULS, observed at the 3$^{rd}$ floor (+11.80m) before and after the retrofit. The reduction of the maximum acceleration is nearly 14% in the longitudinal direction and 30% in the transverse one. With respect to the bare frame configuration, the coupled system presents a significant reduction of the maximum absolute acceleration; this result is noteworthy especially for all the building contents that could be hosted in a structure.

Fig. 11a, b and Fig. 12a, b show the time histories of the base shear along the two principal directions, respectively, at the ULS observed before and after the retrofit; in the latter case the base shear is the sum of the one resisted by the frame and the one resisted by the towers. The reduction of the maximum base shear is nearly 23% in the longitudinal direction and 25% in the transverse direction.

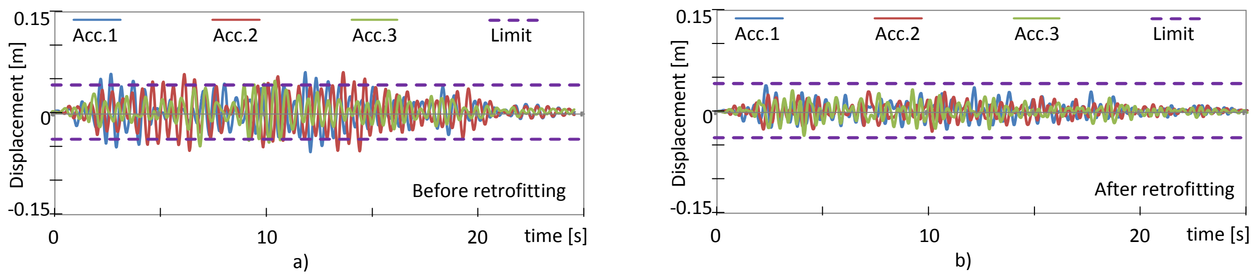



Fig. 8 - Time histories of displacements in transverse direction: a) before retrofitting; b) after retrofitting

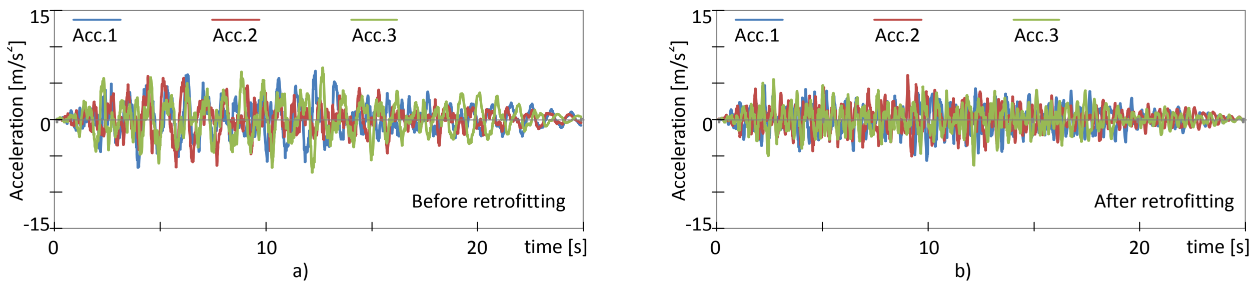



Fig. 9 - Time histories of accelerations in the longitudinal direction: a) before retrofitting; b) after retrofitting

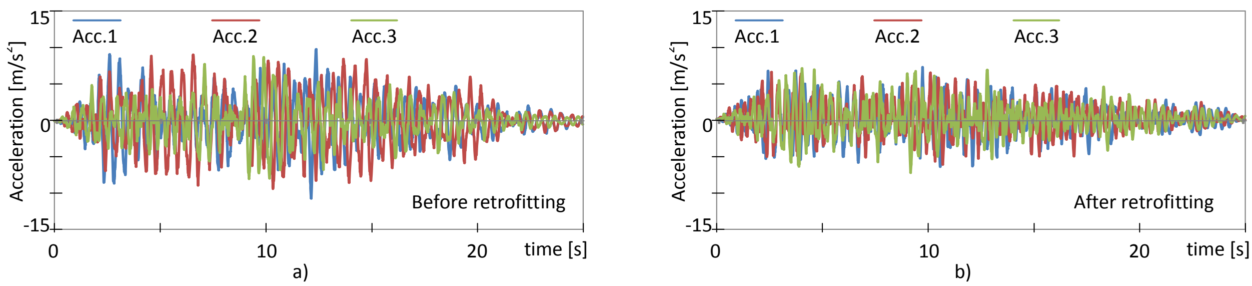



Fig. 10 - Time histories of accelerations in the transverse direction: a) before retrofitting; b) after retrofitting

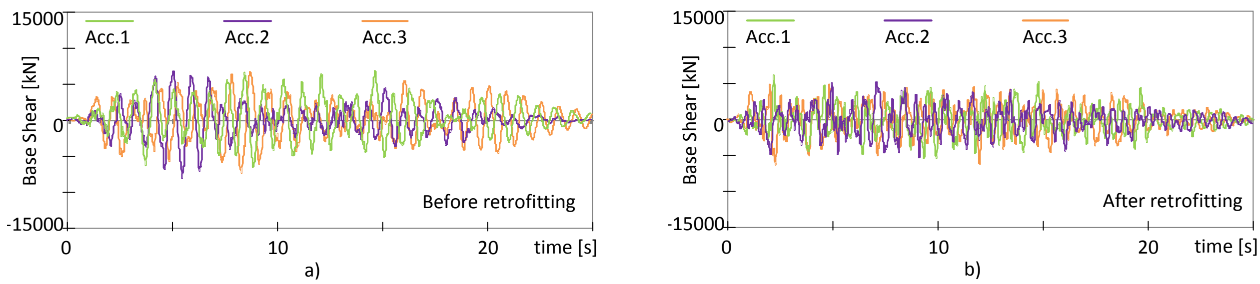



Fig. 11 - Time histories of the base shear in longitudinal direction: a) before retrofitting; b) after retrofitting

Fig. 13a, b compares the total shear force acting at each floor in the two principal directions of the building, respectively, resisted by the existing frame at the ULS before and after the retrofit. Results refer to one of the trhee groups of accelerograms (Acc.1). It is observable that the shear acting on the existing frame after the retrofit is considerably smaller if compared with the building as is, especially at the first two levels, in both the main directions. This result is a consequence of the combined effect of global reduction of displacements and of the interstorey drifts regularization induced by the towers, which at the lower storeys oppose to the higher displacements of the frame structure.

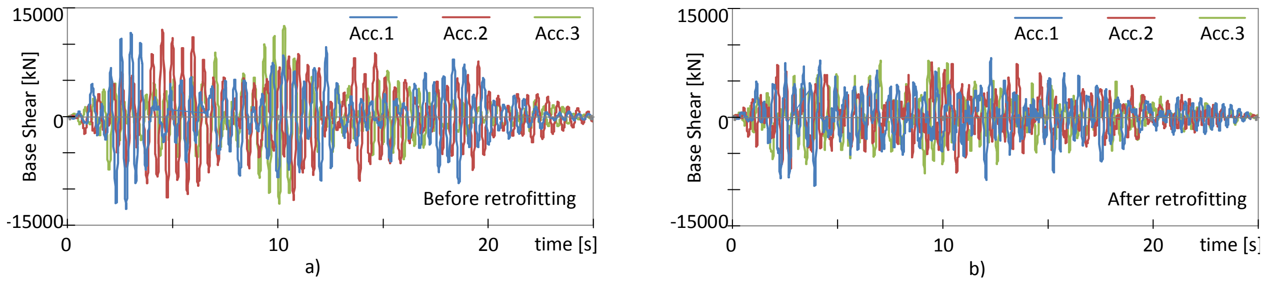



Fig. 12 - Time histories of the base shear in transverse direction: a) before retrofitting; b) after retrofitting

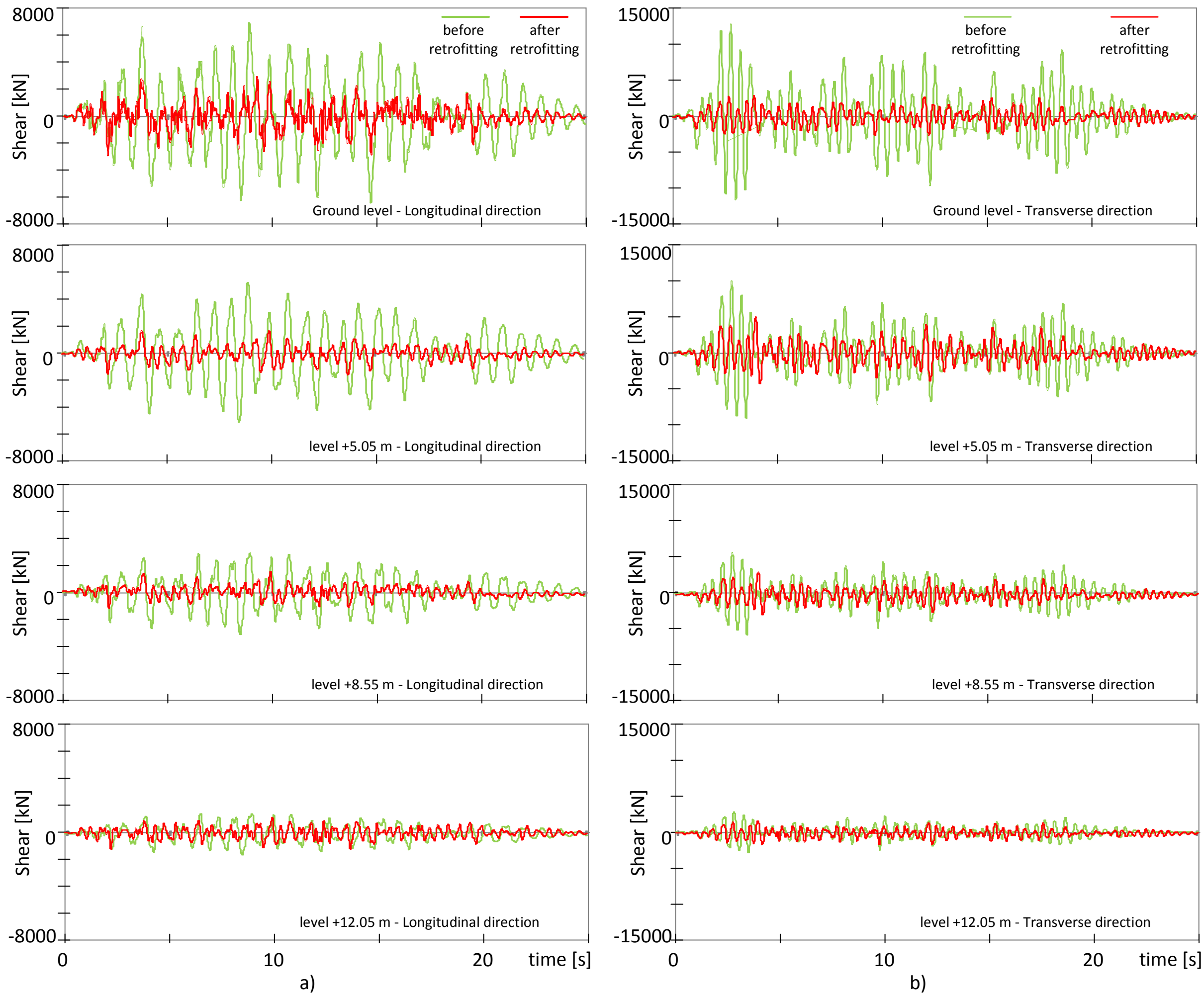



Fig. 13 - Time histories of the shear resisted by the existing frame: a) longitudinal direction; b) transverse direction

Fig. 14a and b depict the time evolution of chord rotations around the principal axes for a representative column of the first elevation (+1.3 m) of the building before and after the retrofit. These are obtained considering displacements measured at points highlighted with red dots in Fig. 15 for all the analyses performed at the ULS. The chord rotation capacity ($\square_{yield}$) is obtained considering a conservative biaxial domain, starting from the elastic rotation capacity of the column cross section in the two principal directions (black dashed line) defined in accordance with the Italian Standards for the SLS [11]. It can be observed that despite the bare frame presents moderate plastic excursions, the retrofit was required by the Authorities to guarantee the immediate occupancy of the strategic building after a distructive event.

Analogously, Fig. 16a and b compares the evolution of chord rotations of the above mentioned column, for the three time history analyses performed at the Collapse Limit State (CLS), before and after the retrofit, respectively. The chord rotation capacity at collapse $\square_u$, is defined in accordance with the Italian Standards and is reported with a red continuous line. It is worth nothing that, after the retrofit chord rotations reduce to such an extent that the structures is able to elastically withstand the CLS seicmic action.

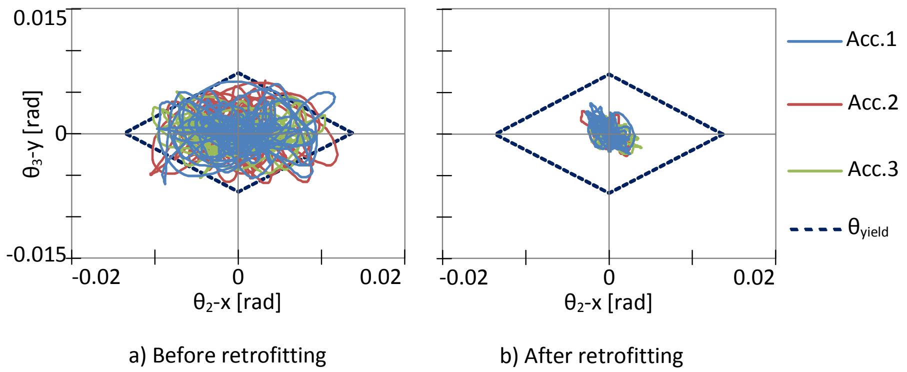



Fig. 14 - Chord rotation demand and capacity ($\square_{yield}$): a) before retrofitting; b) after retrofitting

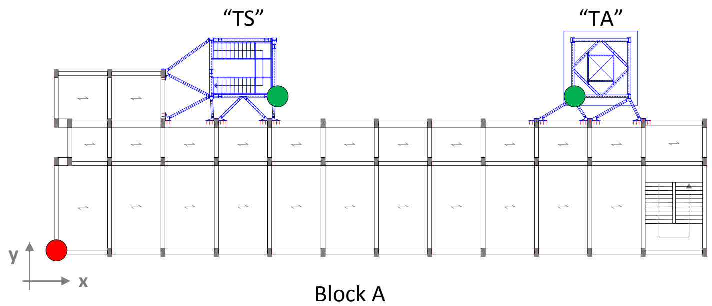



Fig. 15 - Block A first floor plan view

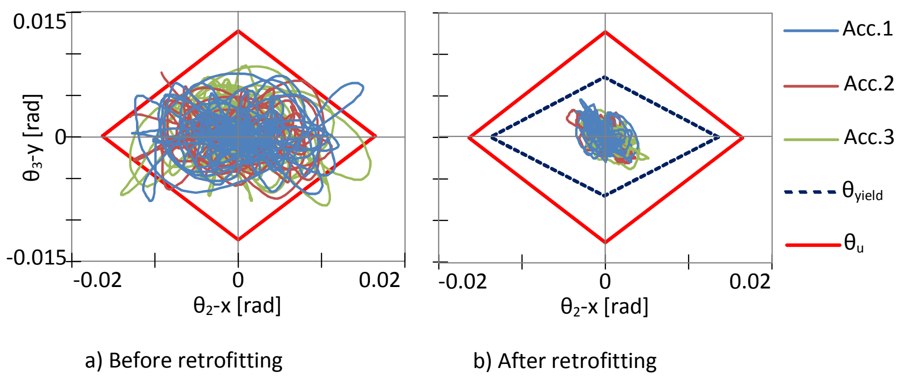



Fig. 16 - Chord rotation demand and capacity ($\square_u$): a) before retrofitting; b) after retrofitting

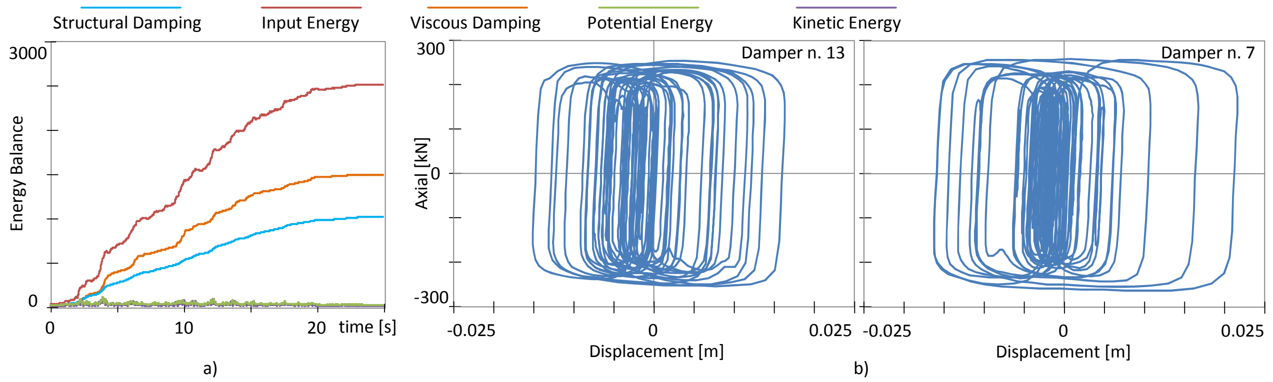



Fig. 17 - a) Energy balance and b) force-displacement cycles of viscous dampers

With reference to the retrofitted structure, Fig. 17a shows the energy balance obtained from the analysis at the ULS, considering group 1 (Acc. 1) of accelerograms. Overall, the energy dissipated by viscous dampers at the base of the towers (orange line) is about 60% of the input energy (red line). Furthermore, energy dissipated through structural damping is shown with a blue line. Fig. 17b and c show, for the same structural analysis, force-displacement cycles of viscous dampers n.13 and n.7, located at the base of towers "TS" and "TA", respectively, as indicated in Fig. 15 with green dots.

## 4 Conclusions

The application of an innovative seismic protection system, patented as “Dissipative Towers”, to the seismic retrofit of a strategic building has been presented. The system is based on the structural coupling of the existing building with new steel truss towers constructed externally and rigidly connected at the floor levels by means of steel braces. Towers, erected over a rigid r.c. thick base plate restrained to the foundation cap with a spherical central hinge, are equipped with dissipative devices that connect the vertices of the tower base plate and the foundation cap. In addition, articulated quadrangles are adopted to amplify vertical displacements of the devices, induced by the tower rocking. The application demonstrates that the efficiency of the system is very high: the “Dissipative Towers” allow a high level of seismic protection at the ultimate limit state, with a significant reduction of horizontal displacements and accelerations. Moreover also shear forces resisted by the frame appear considerably reduced after the retrofit, as a consequence of the displacement reduction. In addition, the seismic protection is achieved with a moderate economic impact due to the elimination of indirect costs related to the arrangement of internal spaces, interruption and/or relocation of activities.

## 5 References

[1] Soong TT, Dargush GF (1997): *Passive Energy Dissipation Systems in Structural Engineering*. Wiley.

[2] Christopoulos C, Filiatrault A (2006): *Principles of Passive Supplemental Damping and Seismic Isolation*. IUSS Press Pavia, Italy.

[3] Whittle JK, Williams MS, Karavasilis TL, Blakeborough A (2012): Comparison of Viscous Damper Placement Methods for Improving Seismic Building Design. *Journal of Earthquake Engineering,* **16**(4), 540-560.

[4] Hwang JS, Lin WC, Wu NJ (2013): Comparison of distribution methods for viscous damping coefficients to buildings. *Structure and Infrastructure Engineering: Maintenance, Management, Life-Cycle Design and Performance,* **9**(1):28-41.

[5] Freddi F, Tubaldi E, Ragni L, Dall'Asta A(2012): Probabilistic performance assessment of low-ductility reinforced concrete frames retrofitted with dissipative braces. *Earthquake Engineering and Structural Dynamics*, **42(7)**, 993-1011.

[6] Trombetti T, Silvestri S (2007): Novel Schemes for Inserting Seismic Dampers in Shear-Type Systems Based Upon the Mass Proportional Component of the Rayleigh Damping Matrix. *Journal of Sounds and Vibrations*, **302**, 486-526.

[7] Gattulli V, Potenza F, Lepidi M (2013): Damping performance of two simple oscillators coupled by a dissipative connection. *Journal of Sound and Vibration*. **332**(26), 6934-6948.

[8] Balducci A Dissipative Towers. Application n. EP2010074723820100831, WO2010EP62748 20100831, International and European classification E04H9/02 – 2005; Italian concession n 0001395591.

[9] Roia D, Gara F, Balducci A, Dezi L (2014): Ambient vibration tests on a reinforced concrete school building before and after retrofitting works with external steel" dissipative towers". *Proceedings of the 9$^{th}$ Int. Conf. on Structural Dynamics, EURODYN 2014.* June 30-July 2, Porto, Portugal.

[10] Roia D, Gara F, Balducci A, Dezi L (2013): Dynamic tests on an existing r.c. school building retrofitted with "dissipative towers". *11$^{th}$ Int. Conf. on Vibration Problems*. September 9-12, Lisbon, Portugal.

[11] Nuove Norme Tecniche per le Costruzioni, D.M. Infrastrutture 14 gennaio 2008, Circolare 02 febbraio 2009 n. 617/C.S.LL.PP. 2009 (in Italian).

[12] SAP2000, Structural Analysis Program, Computers and Structures, Inc., 1995 University Ave., Berkeley, CA.