Retrofit hypotheses of a pre-normative steel school building by fluid viscous damper-based technologies

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Abstract

Two advanced seismic protection technologies, and relevant hypotheses of application to the retrofit of an Italian pre-normative steel school building, are examined in this paper. The two technologies consist in a dissipative bracing system, and a damped cable system, both incorporating pressurized fluid viscous dampers as protecting devices. The essential characteristics and performance of the dampers and the two systems, along with their analytical/computational modelling criteria, are recalled in the first part of this paper. The dimensions, layouts and locations selected for the constituting elements of the two rehabilitation solutions are then discussed. A synthesis of the nonlinear dynamic analyses and performance-based evaluations carried out on the case study building in original and protected conditions is finally proposed.

Introduction

Several Italian steel buildings designed in the Sixties and early Seventies included no seismic provisions, due to the lack of reference Technical Standards at that time. In the last few years, a new public policy aimed at a seismic evaluation and retrofit of this stock of potentially unsafe structures was started, so as to improve their seismic performance substantially. These retrofit interventions are still generally based on conventional rehabilitation and strengthening solutions (e.g., addition of reinforced concrete walls and/or traditional steel bracing systems, extensive enlargement of columns and beams, jacketing, etc). As an alternative, supplemental energy dissipation-based seismic protection strategies were recently adopted, by extending to steel structures the advanced technologies already applied to pre-normative reinforced concrete buildings.

Two special technologies belonging to this class, and namely the dissipative bracing (DB) system and the damped cable (DC) system, which incorporates pressurized silicone fluid viscous (FV) spring-dampers as protective devices, have been studied by the authors of this paper for several years, also in the frame of European Commission-funded Research Projects ("ECOLEADER", "DISPASS" and "NEFOREE" Projects – DB; "SPIDER" Project – DC). Numerical and analytical modelling; experimental characterisation and verification; definition of design procedures; demonstrative applications; and technical implementation of the two technologies, were particularly developed in these studies (Molina et al. 2004, Sorace and Terenzi 1999, 2003a, 2004, 2006 – dissipative braces; Sorace and Terenzi 2003b, 2003c – damped cables). The essential aspects of these activities are summarized in the following three sections. Then, two retrofit hypotheses of an Italian pre-normative steel school building

– well representative of the above-mentioned stock – are offered as demonstrative potential applications of the two rehabilitation strategies. Details of the final design solutions, as well as of the structural analyses and performance-based evaluations developed on the reference building in original and retrofitted configurations, are also provided.

Pressurized fluid viscous devices

As illustrated in the schematic layout in Figure 1, a pressurized silicone FV spring-damper includes an internal casing filled with a compressible silicone fluid, pressurized with a static pre-load F_0; a piston moving in this fluid; an external casing; and two terminal plates that connect the device to the supporting structural elements. When a dynamic action is applied, the fluid flows through the annular space between the piston head and the internal casing, and generates a nonlinear damping reaction force that can be expressed as a fractional power α of velocity (ranging from 0.1 to 0.2). When this action is completed, the piston recovers its initial position due to the pressurization of the fluid. More details on the mechanical behavior, manufacturing configurations, and damping performance of this class of anti-seismic devices are presented in Terenzi (1999), and Sorace and Terenzi (2001a, b).

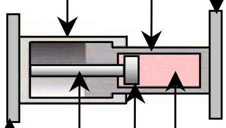

Figure 1. Cross section of a pressurized silicone FV spring-damper

The $F_d(t)$ damping and $F_{ne}(t)$ nonlinear elastic reaction forces of these devices can be expressed analytically as follows (Peckan et al. 1995, Sorace and Terenzi 1999, 2001a):

$$F_d(t) = c\,\mathrm{sgn}(\dot{x}(t))\left|\dot{x}(t)\right|^{\alpha} \tag{1}$$

$$F_{ne}(t) = k_2 x(t) + \frac{(k_1 - k_2)\,x(t)}{\left[1 + \left|\frac{k_1 x(t)}{F_0}\right|^R\right]^{1/R}} \tag{2}$$

where c = damping coefficient; sgn(·) = signum function; |·| = absolute value; k_1, k_2 = stiffness of the response branches situated below and beyond F_0; and R = integer exponent, set as equal to 5 for pressurized FV devices. The model defined by (1) and (2) satisfactorily reproduces the experimental response of all types of spring-dampers belonging to this class (Sorace and Terenzi 2001a). Moreover, relations (1) and (2) have been incorporated in finite element programs in widespread use within the technical community, such as the SAP2000NL code (CSI 2006), which allows easily modelling FV devices in design analyses.

Dissipative bracing system

The DB system considered herein features an inverse-chevron brace configuration, where a pair of interfaced devices are placed, in parallel with the floor-beam axis, at the tip end of each couple of supporting steel braces. A design drawing of a devices-braces-beam connection is shown in Figure 2, where the corresponding finite element model is also displayed. Therein, in addition to a dashpot ("D" element) and a spring ("S"), whose reaction forces are expressed by relations (1) and (2), two further elements are placed in parallel for each dissipater: a "gap" ("G") and a "hook" ("H") elements. The latter elements are adopted to disconnect the device when stressed in tension, and to stop it when the maximum stroke is reached, respectively.

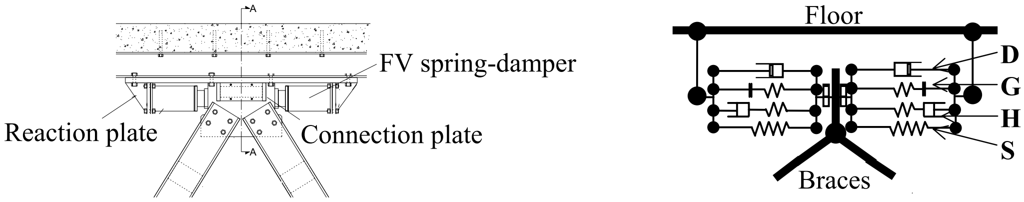

Figure 2. Drawing and finite element model of a devices-braces-beam connection in a DB system

Extensive experimental activities were carried out on this technology (Sorace and Terenzi 2003a, 2004, 2006; Molina et al. 2004), and they evidenced that high seismic performance levels of tested structures are attained after retrofit. Moreover, these activities allowed developing a thorough calibration of the formulated analytical and numerical models, as well as of the proposed design methodology of the DB system (Sorace and Terenzi 2003a, 2006).

Damped cable system

As opposed to a dissipative bracing technology, the DC system considers structural response according to a global approach. As shown in Figure 3, this system includes pre-stressed high-grade steel cables whose lower extremities are coupled with FV spring-dampers fixed to the foundation. The unbounded cables have sliding connections (contact being ensured by pre-stress) with the floor slabs, to which they are joined by steel curved-deviators specially designed for the purpose. Since the cables extend following the horizontal building deflection, a DC system basically exploits the displacements that occur along the complete height of the building, or a significant portion of it (when the upper end is linked to an intermediate floor, rather than to the top one). Based on this operational principle, only one spring-damper for each cable – and thus a small total number of devices – must be incorporated into the building. The global deformation-related operation of a DC system also prompts its application to a large class of existing buildings that, although not complying with new seismic Standard requirements, are not flexible enough to obtain significant benefits from interstory drift-governed protective technologies, including DB systems.

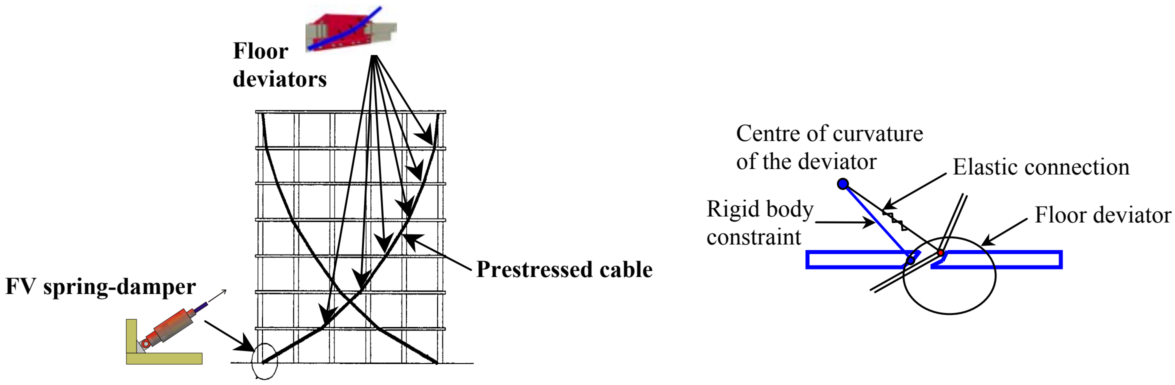

Figure 3. Scheme of a DC system and finite element model of a cable-floor contact

Figure 3 also illustrates the finite element model of the cable-floor sliding connection set-up in previous studies. This model is obtained by putting an elastic link between the cable joint in contact with the floor and the centre of curvature of the deviator, and assigning the same joint a rigid-body constraint that makes it moving along a trajectory defined by the deviator shape. As for the DB technology, high-level performance of the DC system emerged from the experimental programmes carried out. Moreover, satisfactory correlations of test data with the predictions of the analytical and numerical models, as well as with the target response fixed by the formulated design procedures, was observed (Sorace and Terenzi 2003b, c).

Case study building

The pre-normative structure examined in this study was built in Florence in the late Sixties. It consists of a two-story central body, two four-story wings, and an adjacent gym, separated from the main structure by a technical joint. The building was designed for vertical and wind loads, and included no seismic provisions, according to the Italian Technical Standards of the time. The structural skeleton consists of moment-resisting frames with semi-rigid flanged connections. No bracing systems or shear walls are included. The remarkable asymmetries in plan and elevation of the building make it very sensitive to torsional components of seismic response. Indeed, the first vibration mode of the main structure is purely rotational around the vertical axis z, with effective modal mass (EMM) equal to 23.4% of the total seismic mass. The second and third modes consist in a translational/torsional mix. EMMs equal to 77.7% (x axis) and 24.8% (around z) are found for the second mode; and equal to 79% (y axis) and 31.1% (around z), for the third mode. Figure 4 shows two photographic views of one four-story wing during the demolition of the building infills carried out at the first stage of its architectural refurbishment and seismic retrofit works.

Figure 4. Views of the building during the demolition phases of the infills

A performance-based seismic assessment analysis was developed by the finite element model of the structure in original conditions, for the following reference seismic hazard levels: basic design earthquake – BDE (with a 10% probability of being exceeded over 50 years), and serviceability earthquake – SE (50%/50). According to the new Italian territory classification, the building site, like the whole city of Florence, is situated in seismic zone 2. The peak ground acceleration assigned to BDE for this zone is equal to 0.25 g, for rock-soil conditions ("A"-soil type). The foundation soil of the building is of "B" type, for which an amplifying coefficient of 1.25 must be applied to the "A"-soil acceleration. Moreover, an importance factor of 1.2 must be considered to account for the use of the building as a school. In total, a BDE peak amplitude of 0.375 g was adopted in the analyses. The SE amplitude was obtained by dividing the BDE amplitude by a factor of 2.5. Five artificial accelerograms generated from the response spectrum of the new Italian Seismic Standards, scaled at the above-mentioned peak accelerations, were used as inputs. The results of the assessment enquiry, elaborated in mean terms over the five ground motions, showed that: (a) the building does not meet the requirement of collapse prevention under BDE, along its weakest direction in plan (parallel to the x axis); (b) for the same direction, only a "limited safety" structural performance level, as defined in FEMA 356 (2000), is obtained for SE. The latter descends from values of the interstory drift ratio (i.e., the ratio of interstory drift to story height) exceeding 2% for the second and third stories of the building wings. A slightly better, although still very low performance came out for the y direction, with attainment of near-collapse conditions, for the BDE level, and a "life safety" performance level, for SE. It can be noted that non-structural performance levels were not considered in this analysis, since partition and infill panels not interacting with the structural skeleton were included in the architectural restoration project.

DB-based retrofit hypothesis

The plan distribution of the dissipative braces incorporated in this retrofit solution is sketched in Figure 5, where the positions of the eighteen DB systems (fourteen in the main building and four in the gym) are outlined as black rectangles.

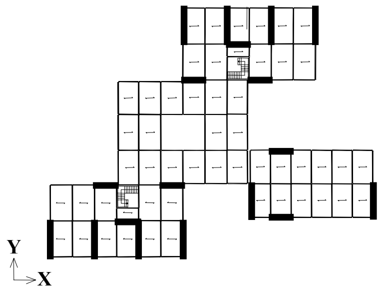

Figure 5. Plan distribution of the DBs included in the first retrofit hypothesis

This distribution was aimed firstly at reaching the lowest architectural impact of the retrofit intervention inside the building and on the façades. Then, within the limits imposed by this non-structural objective, some of the existing distances between centre of masses and centre of stiffness were reduced at each story, so as to constrain the torsion effects featuring the response of the original structure. As a result, the first mode in protected conditions is still rotational around the vertical axis, with EMM decreased to 21% of the total seismic mass. The second and third modes, again a translational/torsional mix, have EMMs equal to 81.4% (y axis) and 18.2% (around z); and equal to 81.1% (x axis) and 23.6% (around z), respectively. A global view of the computational model of the building incorporating the dissipative braces, and a cross section of one of the two wings, are shown in Figure 6.

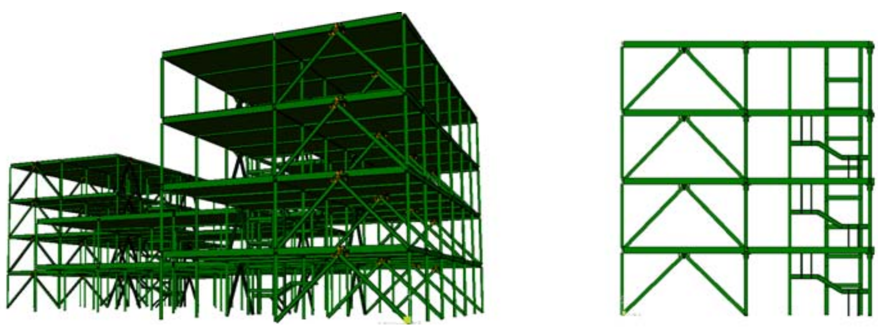

Figure 6. Global view of the finite element model of the DB-retrofitted structure and cross section of a four-story wing

The performance objectives of this first retrofit hypothesis, as well as of the one based on the use of a DC system, consisted in reaching: (1) a "damage control" level for BDE, with some plasticization of few beams and columns, and 1.5% maximum interstory drift ratios for each story; (2) "immediate occupancy" for SE, with 0.7% maximum drift ratios. The FV spring-dampers selected by means of the energy-based design method discussed in Sorace and Terenzi (2003a, 2006) have the following mechanical characteristics: nominal energy dissipation capacity E_n equal to 14 kJ; F_0 = 130 kN; stroke equal to 80 mm; and maximum attainable damping coefficient c_{max} = 79 kN(s/m)$^{\alpha}$ (with α = 0.15). The optimal performance of the system is finally obtained by assigning a mutual c value of 30 kN(s/m)$^{\alpha}$ to each device.

DC-based retrofit hypothesis

Similarly to Figure 5, the plan distribution of the damped cables adopted in this retrofit solution is illustrated, by grey rectangles, in Figure 7.

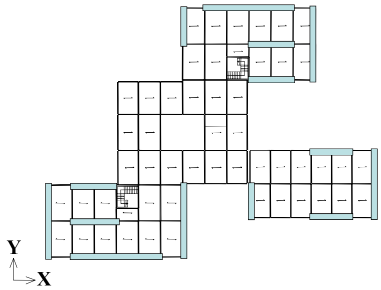

Figure 7. Plan distribution of the DCs included in the second retrofit hypothesis

Also in this case, the location of the protective system was aimed at obtaining the best compatibility with the architectural characteristics of the building, as well as reducing the torsional components of seismic response evaluated for the unprotected structure. For this solution, the first torsional mode disappears. The new first and second modes are mixed translational/torsional, with EMMs equal to 80.3% (y axis) and 29.8% (around z) of the total seismic mass; and equal to 71.2% (x axis) and 39.6% (around z), respectively. Two views of the finite element of the retrofitted structure zoomed on a four-story wing are shown in Figure 8. The image on the left shows that the cables parallel to the x axis are installed over external steel frames connected to the main structure; this avoids any obstruction of the windows on these façades. The remaining cables, instead, are mounted directly on the perimeter frames of the building.

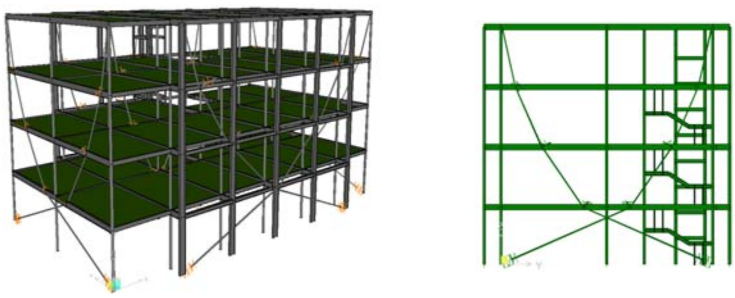

Figure 8. Detailed views of a four-story wing of the finite element model of the DC-retrofitted structure

In order to meet the performance objectives mentioned in the previous section, the design process carried out by the criteria defined in Sorace and Terenzi (1999, 2003b) caused to select FV spring-dampers with the following characteristics: E_n equal to 150 kJ; F_0 = 640 kN; stroke equal to 180 mm; and c_{max} = 376 kN(s/m)$^{\alpha}$. Several different damping coefficient values were selected for the dissipaters belonging to the various cables, ranging from 130 to 205 kN(s/m)$^{\alpha}$. The same applies also to the pre-stressed cables, made of seven-wire strands, for which a total number of constituting strands varying from 4 to 23 was fixed.

Synthesis of design and performance-based evaluation analyses

The seismic analyses were carried out with a nonlinear dynamic approach by the complete finite element model of the structure. The responses obtained before and after retrofit in terms of maximum interstory drift profiles, and for the weakest direction x, are shown in comparison in Figure 9, where mean values calculated over a set of five input accelerograms are plotted, for the two considered earthquake levels.

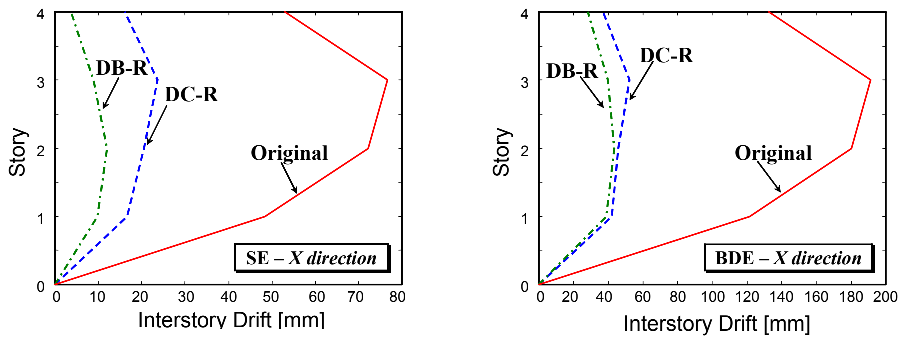

Figure 9. Maximum interstory drift profiles obtained over the set of five input accelerograms (mean values)

Starting from maximum drifts of 76.7 mm, and corresponding drift ratios of 2.19% for the original structure at SE, 12.1 mm (0.35%) and 23.6 mm (0.67%) drifts come out for the DB and DC-based retrofit solutions (indicated as DC-R and DB-R in Figure 9), respectively. Both values are below the drift ratio limit of 0.7% set for the immediate occupancy level, assumed as a target performance for SE. Concerning BDE, the 195.6 mm maximum drift (5.59%) obtained in unprotected conditions is constrained within 43.6 mm (1.24%) – DB, and 51.7 mm (1.47%) – DC, both being below the 1.5% reference threshold for damage control level. Moreover, no plastic activity of beams and columns is remarked in the response to BDE, which highlights a very satisfactory performance also in terms of local stress states. The comparative graphs in Figure 9 show very similar responses of the two protective systems for BDE, whereas performance improves further in the DB-based solution under the serviceability earthquake. This is the consequence of earlier activation of the FV spring-dampers incorporated in the dissipative braces, characterised by considerably smaller dimensions and static pre-loads as compared to the devices included in the damped cables. This effect has virtually no impact on the response to BDE, as it implies complete activation of all the devices incorporated in both retrofit schemes since the initial phases of a seismic action. Figure 10 illustrates the base shear time-histories obtained from the most demanding among the five input accelerograms, when applied along x, for both retrofit hypotheses. In these graphs, the first index of the T base shear force refers to the earthquake input direction, whereas the second refers to the projections of the force on the two axes. It can be drawn from the response time-histories that the T_{xy} orthogonal component of shear is totally negligible – DB –, or no greater than 6% as compared to the parallel one – DC. This underlines the considerable restraint of torsional response effects ensured by both rehabilitation solutions.

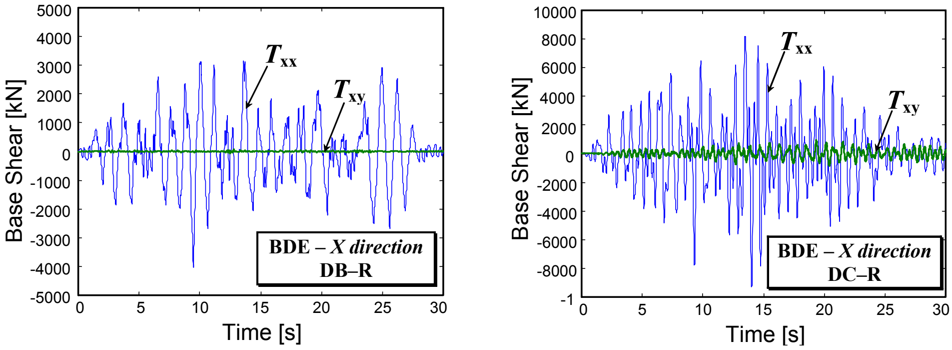

Figure 10. Base shear time-histories obtained from the most demanding input accelerogram

Concluding remarks

The two advanced seismic retrofit hypotheses formulated for the steel school building examined in this paper allowed reaching target design performance levels with reasonably small-sized components in both technologies. This guarantees acceptable architectural impact and competitive costs as compared to traditional seismic design strategies. The realistic design simulations developed herein offer interesting perspectives for practical application of DB and DC systems to the Italian stock of pre-normative steel buildings with similar characteristics to the case study.

References

CSI (2006). *SAP2000NL. Structural Analysis Programs - Theoretical and Users Manual*, Version No. 10.10, Computers & Structures Inc., Berkeley, CA.

FEMA 356 (2000). "Prestandard and commentary for the seismic rehabilitation of buildings." *Building Seismic Safety Council*, FEMA, Washington, DC.

Molina F.J., Sorace S., Terenzi G., Magonette, G., and Viaccoz, B. (2004). "Seismic tests on reinforced concrete and steel frames retrofitted with dissipative braces." *Earthquake Engineering and Structural Dynamics*, 33, 1373-1394.

Pekcan, G., Mander, J. B., and Chen, S. S. (1995). "The seismic response of a 1:3 scale model R.C. structure with elastomeric spring dampers." *Earthquake Spectra*, 11, 249-267.

Sorace, S., and Terenzi, G. (1999). "Iterative design procedure of fluid viscous devices included in braced frames." *Proc., EURODYN '99 – 4th European Conference on Structural Dynamics*, Prague, Czech Republic, A.A. Balkema, Rotterdam, 169-174.

Sorace, S., and Terenzi, G. (2001a). "Non-linear dynamic modelling and design procedure of FV spring-dampers for base isolation." *Engineering Structures*, 23, 1556-1567.

Sorace, S., and Terenzi, G. (2001b). "Non-linear dynamic design procedure of FV spring-dampers for base isolation – Frame building applications." *Engineering Structures*, 23, 1568-1576.

Sorace, S., and Terenzi, G. (2003a). "Large-scale experimental validation of a design procedure for damped braced steel structures." *Proc., STESSA 2003 – 4th International Conference on the Behaviour of Steel Structures in Seismic Areas*, Naples, Italy, A A Balkema Publishers, 657-662.

Sorace, S., and Terenzi, G. (2003b). "An advanced seismic protection technology: the damped cable system." *Proc., ASSCCA'03 – International Conference on Advances in Structures*, Sydney, Australia, A. A. Balkema Publishers, 1185-1192.

Sorace, S., and Terenzi, G. (2003c). "Verification of damped cable system in the seismic rehabilitation of buildings." *Proc., ERES '03 – 4th Conference on Earthquake Resistant Engineering Structures*, Ancona, Italy, WIT Press, 283-292.

Sorace, S., and Terenzi, G. (2004). "Comparative experimental investigation on a R/C structure with/without damped braces." *Proc., 13th World Conference on Earthquake Engineering*, Vancouver, Canada, Mira Digital Publishing, Paper No. 3461, CD-ROM.

Sorace, S., and Terenzi, G. (2006). "Seismic protection of frame structures by fluid viscous damped braces." To appear in the *Journal of Structural Engineering ASCE*.

Terenzi, G. (1999). "Dynamics of SDOF systems with nonlinear viscous damping." *Journal of Engineering Mechanics ASCE*, 125, 956–963.