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Design of Additional Dissipative Structures for Seismic Retrofitting of Existing Buildings

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Abstract: This paper presents an innovative approach for improving the seismic protection of existing structures by introducing an additional dissipative structure (ADS). The seismic energy impacting the building can be dissipated through the contribution provided by the ADS, thereby reducing the need for the existing building to ensure its seismic capacity. This retrofitting technique is well-suited for structures facing architectural restrictions or challenging-to-update elements. It can help to address foundational issues by applying loads to new external components. This paper describes the design of the ADS and proposes a displacement-based design procedure. The design process involves a nonlinear static analysis and a simple procedure that must be iteratively repeated until the retrofitting target is achieved. This approach is simple and computationally efficient and can also be used for complex and irregular structures. Such structures are frequently encountered, and existing structures often exhibit unusual geometries and materials requiring extensive numerical modeling. The efficacy of this technique was evaluated using the case study of a school building located in central Italy. The results of numerical analyses indicated that owing to the ADS's contribution, the seismic capacity of both buildings was enhanced, addressing the challenges associated with complex foundational interventions.

Keywords: passive damping; dissipative devices; seismic retrofitting

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1. Introduction

In recent years, the European Union's interest in multidisciplinary improvements to the building heritage of its member countries has materialized through substantial funding for activities aimed at enhancing both the energy efficiency and seismic resilience of public and private buildings [1,2]. In Italy, the building stock consists of historical or post-World War II constructions designed without seismic regulations, making the development of retrofitting techniques a matter of strong strategic interest.

The retrofitting of existing concrete buildings aims to reduce the risks associated with failure and damage. Traditional retrofitting strategies aim to increase structural strength to reduce ductility demand. However, in the last two decades, new conceptual approaches have gained prominence, falling into two categories: increasing available ductility and reducing demand. The latter can be achieved by reducing input energy through base isolation or increasing energy dissipation via additional dissipative devices. These devices, like dissipative bracings or external dissipative structures (e.g., dissipating frames) [3], introduce a nonlinear component to the retrofitted structure, altering its behavior and eliciting nonlinear responses.

Various conditions, such as interference with building utilization during the retrofit, can influence the choice of the retrofitting strategy. In some cases, the use of dissipative braces can be disadvantageous, primarily in terms of architectural and functional impacts. As detailed in this paper, these problems can be overcome by creating additional dissipative structures directly connected to existing buildings. Implementing external dissipative structures involves constructing new foundations and placing dissipative devices outside the existing building, often within specially dimensioned framed constructions. Some applications have recently been developed, discussed, and realized [4–6].

Both solutions, dissipative bracing and dissipative frames, imply that the retrofitted structure will include a nonlinear component that can modify the behavior of the structure itself and usually require the evaluation of a nonlinear response. Based on international codes and the scientific literature, the following considerations are made:

None of the existing codes, with the partial exception of FEMA, defines design criteria for additional dissipative systems. FEMA 274 [7] and FEMA 356 [8] highlight the variability in design methods, depending on the type of existing dissipative devices. These devices can be broadly grouped into two major categories: displacement-dependent devices (yielding metallic and friction dampers) and velocity-dependent devices (viscoelastic solids or viscous fluids). Although a wide range of devices has been proposed in the literature, research aims to limit the residual damage induced by seismic events. The inadequacy of conventional structures for repair is a critical issue observed after severe earthquakes [9]. Additional dissipative structures offer significant benefits, including stiffness redistribution, damping, and the attraction of base shear in new foundations. Consequently, these interventions can significantly enhance seismic performance without increasing, in many cases, the base shear and floor accelerations.

Dissipative structures can be equipped with various types of dissipative devices, such as generic hysteretic or viscous dampers, buckling-restrained braces [10], and shear link devices [11].

This paper describes an effective and easy-to-use displacement-based design procedure for the seismic upgrade of existing buildings (*S*) with additional dissipative structures (ADSs) for seismic enhancement. The procedure is derived from Bergami et al. [12] and is based on the capacity spectrum method [13]. It defines the retrofitted building's capacity by considering contributions from the existing structure and the ADS to achieve a desired performance level based on the target displacement. The procedure is iterative, and the capacity curve is determined via pushover analysis at each iteration. In this study, the design of an ADS is discussed, applied, and verified. The primary performance objective is to prevent earthquake-induced damage, ensuring life safety for the retrofitted building (*S+ADS*) and avoiding structural and non-structural element damage. The target displacement is related to the permissible interstory drift value.

2. State-of-the-Art and Innovative Contribution of this Research

Numerous design procedures have been developed by researchers and applied worldwide. The most innovative procedures are displacement based and are intended for installing dissipative devices on additional braces. Among these procedures, a detailed summary is presented below.

Magdalini D. Titirla [14] focuses on passive energy dissipation systems and, more specifically, the dampers that can be positioned in steel braces to increase the absorption of seismic energy and to protect them from buckling.

Davide Bellotti [15] aimed to investigate the effectiveness of energy dissipation devices in enhancing seismic performance and possibly extending the nominal service life of these structures. Seismic retrofitting with two dissipation devices was considered herein, namely, a rotational friction damper and a bracing system with dissipative sacrificial elements.

Laguardia and Franchin [16] presented a risk-based optimal design procedure for retrofit interventions with dissipative bracing systems by further developing this method. The procedure provides the optimal characteristics of braces to obtain a retrofitted structure that respects limits on the mean annual frequency of the exceedance, λ_{LS}, for multiple limit states.

The seismic protection of new or pre-existing buildings with a steel self-centering device (SSCD) was investigated by Braconi et al. [17], who developed, designed, and experimentally validated the device (equipped with replaceable steel dissipative elements and prestressing cables to minimize the residual deformation of the system after a seismic event).

A novel retrofitting technique for precast concrete industrial frame buildings, based on tension-only monolateral dissipative devices mounted on steel braces, was proposed by Bruno Dal Lago [18]. These braces act as energy fuses and can provide additional stiffness and relevant hysteretic damping to the structural behavior during low-drift cycles.

Kim and Choi [10] proposed a design procedure to provide the required effective damping using additional buckling-restrained braces (BRBs) to achieve the desired target displacement. Ponzo et al. [9] introduced an energy equivalence criterion for dimensioning the bracing system based on the ultimate frame displacement capacity.

Durucan and Dicleli [11] put forward an energy-based iterative design procedure for retrofitting existing RC frames using steel braces with shear links, demonstrating its effectiveness in achieving both operational and life safety performance levels.

Bergami and Nuti [9,19] defined a comprehensive design procedure for dissipative braces, encompassing the design of the braces' stiffness, yielding force, and metallic components for seismic retrofitting. An optimization procedure is also included, based on static nonlinear analysis, enabling a useful comparison between standard and innovative pushover procedures and considering the influence of higher-mode contributions [20–22].

Mazza and Vulcano [23] developed a design procedure based on defining a target displacement and iteratively determining the properties of an equivalent damping system. The assumptions proposed in the procedure characterize the equivalent damping system in terms of the equivalent stiffness and independently determined yielding force.

Moreover, many researchers have focused their attention on the possibility for reducing the seismic vulnerability of existing infilled frame buildings, using dissipative systems to prevent damage to infills and partitions controlling the interstory drift. The first applications of this approach were proposed by Bergami et al. [19] and, more recently, by Terenzi et al. [24–27].

The techniques described above are now well-known and have been applied. However, it should be noted that these approaches have some disadvantages, such as the increment in axial forces in columns, which may lead to premature local failures [28] or overloads on existing foundations that are usually difficult to reinforce. Other relevant limits are related to architectural and functional compatibility. Some dissipative braces may require significant space within the structure. This could impact interior layouts and limit space design options and the interventions that may entail the temporary shutdown of the building. Moreover, in dealing with strategic buildings, the indirect costs (social and economic) related to the interruption of the buildings' utilization can be very demanding. These problems can be overcome, as discussed and proposed in this paper, by placing the dissipative bracings and the relevant foundations outside the buildings. Therefore, the objective and innovation of this study are to discuss the interesting possibility of this intervention, namely, the adoption of additional dissipative structures for the seismic retrofitting of existing buildings.

Indeed, the use of external additional dissipative structures (ADSs) allows for minimizing the impact of the function and functionality of buildings. The characteristics of this intervention are further elaborated in the following section.

3. General Aspects for Retrofitting Using Additional Dissipative Structures

3.1. Additional Dissipative Structures

Among the various retrofitting approaches, additional dissipative structures have gained popularity owing to their undeniable advantages. These structures divert seismic forces to a new structure with fresh foundations (as depicted in Figure 1), and most of the construction work occurs outdoors. Consequently, the existing structure can continue to operate. This intervention typically requires the ability to create a new volume. However, it is worth noting that the tower can often replace existing external structures, such as emergency stairs, with minimal impact on the building's architecture.

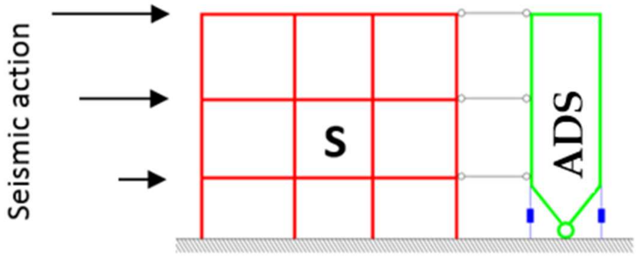


Figure 1. Building *S* retrofitted with the ADS: distribution of the base shear.

Dissipative towers are typically constructed using steel, though in cases requiring high stiffness, they may be made of reinforced concrete (R.C.). These towers must be connected to the existing building and equipped with dissipative devices. These devices can exhibit displacement-dependent behavior (e.g., yielding metallic and friction dampers) or velocity-dependent behavior (e.g., viscoelastic solids or viscous fluids). These devices can be installed in various configurations (as illustrated in Figure 2). This study considered the use of dissipative devices, such as BRBs [10], at the base of the tower (Figure 2a) because this is deemed to be the most cost-effective and practical solution.

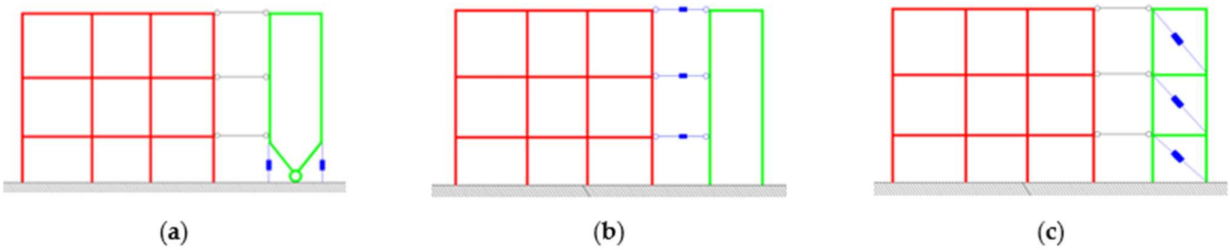


Figure 2. Possible configurations of the dissipative devices of the ADS (**a**) at the hinge of a tower, (**b**) at each connection between S and the contrast structure, and (**c**) as dissipative braces inside the framing system.

3.2. Retrofitting with an ADS

In this context, where *S* represents the existing structure, *ADS* denotes the dissipative structure under design, and *S+ADS* represents the retrofitted building (as shown in Figure 3), the designer can simplify the capacity curve of the final configuration as the sum of the capacity curves of the S and ADS. Hence, this study evaluates the behavior of the ADS by subtracting the contribution of the S from the overall response of the *S+ADS*.

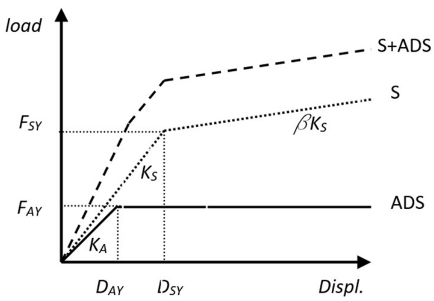


Figure 3. Interaction between the structure (*S*) and ADS expressed in terms of horizontal components of the force–displacement relationship.

The capacity curves of both the S and ADS, if deemed useful for streamlining the design process, can be approximately elastoplastic according to well-established procedures, making the *S+ADS* curve trilinear.

Following the capacity spectrum method, seismic action is expressed in terms of the response spectrum. Once the capacity curve is defined, the structural response can be assessed. By evaluating the equivalent viscous damping ($\xi_{eq,S+ADS}$) associated with each point on the capacity curve, the structural response can be succinctly described by a specific performance indicator defined by a displacement value and the corresponding base shear.

The force–displacement behavior of the ADS can be modeled using a simple bilinear law characterized by the horizontal elastic stiffness (K_A), the horizontal yield strength (F_{Ay}), and the horizontal displacement corresponding to the devices' yielding (D_{Ay}).

K_A depends on the structural solution of the ADS (including the geometry, material, and configuration) and the stiffness of the installed dissipative K_A devices. F_{Ay}, D_{Ay}, and β_A depend on the mechanical properties of the dissipative devices.

The design process is finalized to evaluate the following:

1. The geometry and stiffness of the ADS (e.g., a tower) that influences the deformed shape of the building in the elastic range;
2. The stiffness (K_A) of the ADS;
3. The yielding limit of the ADS (D_{Ay}, V_{Ay}), which is the point beyond which the system becomes dissipative (e.g., the plastic limit of the dissipative devices installed inside the ADS).

The designer has the flexibility to employ various approaches in determining the necessary stiffness and strength of the tower. This is essential to ensure that the building response remains within the desired range. To achieve this, the designer can refer to different damage indices, such as the top displacement, interstory drift, or base shear.

It is evident that if the *ADS* yields before the existing structure *S* ($D_{Ay} < D_{Sy}$), the effectiveness of the intervention will be enhanced. Therefore, this assumption is fundamental and will be considered.

Now, it is valuable to express each limit state of interest in terms of displacement, denoted as D^*. The same D_i^* value can be achieved through the implementation of different combinations of retrofitting in terms of stiffness, strength, and, consequently, dissipation.

The first parameter to be determined will be the tower's stiffness (additional stiffness).

4. Energy Dissipation Capacity

According to an existing procedure [15,18] developed for the dimensioning of dissipative additional systems to be installed inside buildings (dissipative bracings), the energy dissipated by the *S* and ADS can be evaluated at each deformation value (Figure 4), and, according to A.K. Chopra [29], it can be evaluated by calculating $\xi_{eq,S}$, that is, the equivalent viscous damping of the structure (as a function of the displacement, *D*); it can be expressed as follows:

$$\xi_{eq,S} = \frac{1}{4\pi}\frac{I_{D,S}}{I_{S,S}} \tag{1}$$

Equation (1) can be solved by determining all the necessary quantities from the capacity curve, where

D is the displacement reached by the control joint;
$F_s(D)$ is the force (base shear) corresponding to D;
D_{sy} is the displacement at yielding;
F_{sy} is the yielding force (base shear at yielding);
$I_{D,S}$ is the energy that is dissipated (cycle of D amplitudes);
$I_{S,S}$ is the elastic strain energy at D.

In a simplified approach, an equivalent bilinear capacity curve (BCC) can be easily used. The BCC can be determined (according to one of the methods available in the literature or technical codes) from the "real" capacity curve (output of the pushover analysis).

In this way, the terms of Equation (1), considering an ideal elastoplastic hysteretic cycle, are determined as follows:

$$I^{id}_{D,S} = \left(F_{sy}D - D_{sy}F_s(D)\right)/0.25 \tag{2}$$

$$I_{S,S} = DF_s(D)/2 \tag{3}$$

The hysteretic cycle of a real structure, which differs from the mathematically evaluated ideal cycle, can be evaluated according to specific corrective coefficients c_S (for the structure) and c_A for the ADS (c = 1 for ideal elastoplastic behavior).

Therefore,

$$I_{D,S} = c_S I^{id}_{D,S} \quad (4)$$

$$I_{D,A} = c_A I^{id}_{D,A} \quad (5)$$

with $I^{id}_{D,A}$ being the energy dissipated by the ideal hysteretic cycle of the ADS (elastoplastic behavior defined by the elastic stiffness, yielding limit, and hardening ratio).

Then, c_S can be determined through specific analysis or by simply referring to the provision's technical codes or the scientific literature (e.g., [30]); according to the author's experience, the assumption of $c_A \approx 1$ can be considered as reasonable, and the force–displacement relationship of the *ADS* can be idealized as a bilinear curve.

The equivalent viscous damping ($\xi_{eq,S+T}$) of the *S*+*ADS*, to be added to the inherent damping (ξ_I—usually, ξ_I = 5% for r.c. structures, and ξ_I = 2% for steel structures) can be evaluated using the following expressions:

$$\xi_{eq,S+A} = 0.25\left[\frac{c_S I^{id}_{D,S}}{I_{S,S+A}} + \frac{c_A \sum_j I^{id}_{D,A,j}}{I_{S,S+A}}\right] 1/\pi \quad (6)$$

$$\xi_{eq,S} = c_S 0.25\frac{I^{id}_{D,S}}{I_{S,S+A}}\pi;\ \xi_{eq,A} = c_A 0.25\frac{\sum_j I^{id}_{D,A,j}}{I_{S,S+A}}\pi \quad (7)$$

where $I^{id}_{D,A,j}$ is the energy dissipated by j ADSs connected to the structure (e.g., in practical applications, one or more dissipative towers can be designed).

Note that $\xi_{eq,S}$ and $\xi_{eq,A}$ are obtained by dividing the dissipated energy, determined from the capacity curves of the *S* and *ADS*, respectively, by the elastic strain energy of the retrofitted building, which is determined from the curve of the *S*+*ADS*.

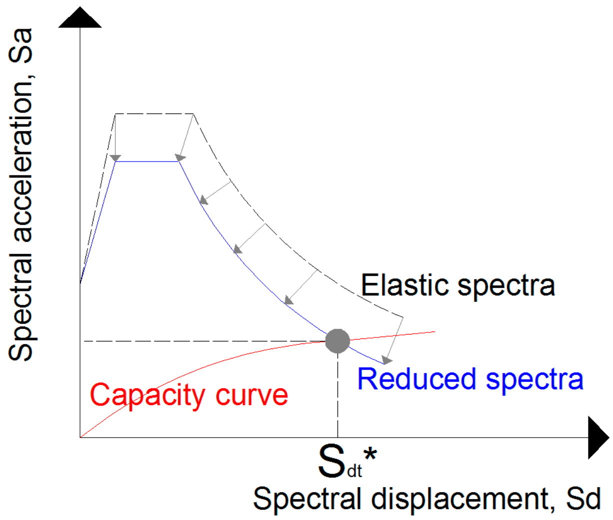


Figure 4. Evaluation of the equivalent viscous damping needed to achieve the target performance point.

5. Proposed Design Procedure

The previous sections discussed the key aspects for evaluating the seismic response of a structure with an ADS. This section provides a detailed explanation of the proposed procedure.

The proposed procedure is based on the capacity spectrum method (CSM), and the design objective is expressed in terms of a displacement limit. It is crucial to emphasize that existing buildings, often designed without seismic considerations, tend to be irregular and sensitive to higher modes. This condition can significantly affect the effectiveness of a capacity-spectrum-based design procedure, such as the one presented herein. Therefore, when deemed suitable for a specific application, the use of a standard pushover analysis can be more efficiently replaced by alternative approaches, such as incremental modal pushover analysis (IMPA) [23]. IMPA extends the well-known modal pushover analysis (MPA) [29,30] to obtain a multimodal capacity curve, which proves valuable for seismic assessment or design implementation.

As dissipative towers alter the structural response of the original building, the procedure is inherently iterative. The capacity curve must be continually updated to reflect the characteristics of the new coupled structure (building + tower).

According to the CSM, considering the energy dissipated by the ADS (in addition to the dissipative capacity of the structure, which is computed from the capacity curve of the original structure), the structural response is obtained by reducing the design spectrum based on the damping (ξ_{tot}) of the *S+ADS*.

$$\xi_{tot} = \xi_I + \xi_{eq,S+A} \tag{8}$$

To execute this procedure, the designer must define the desired performance. Because this is a displacement-based procedure, the definition is based on a target displacement, typically corresponding to a chosen limit state under specific seismic conditions. Subsequently, the total effective damping required to match the actual maximum displacement and the target displacement can be determined. The additional damping provided by the ADS (e.g., a dissipative tower) is estimated as the difference between the total damping and the hysteretic damping of the original structure. The characteristics of the ADS are then determined to meet the required additional damping. Although the procedure is iterative, it converges after only a few iterations. The key steps are outlined below.

Step 1. The seismic action was defined in terms of the elastic response acceleration spectrum ($T - S_a$).

Step 2. The target displacement was selected (e.g., the top displacement, D_t*) according to the desired performance (limit state).

Step 3. The capacity curve for the retrofitted structure, *S+ADS*, considering the top displacement (D_t) and base shear (F_b), was established through a pushover analysis. A pushover analysis can be conducted by adopting one of the various force distribution methods outlined in the building codes and literature. It is advisable to employ a multimodal procedure. When a modal pushover analysis is performed, it is important to note that the modal shape is influenced by the interaction between the building and the tower. Consequently, at each iteration (in Step 3, from iterations 1 to n), the load profile must be adjusted to match the modal shape of the current braced structure. It is worth noting that during the initial iteration, the existing building is considered, and the capacity curve obtained at this stage is crucial for assessing the contribution provided by the existing framing structure.

Step 4. The capacity curve obtained in Step 3 can be approximated by a simpler bilinear curve that is completely defined by the yielding point (D_S + $A_{DS,y}$, $F_{S+ADS,y}$) and the hardening ratio (β_{S+ADS}). (At the first iteration, these parameters correspond to $D_{S,y}$, $F_{S,y}$, and β_S of the existing building). This step can be avoided using a specific software (such as MATLAB or other calculation tools), and the evaluation of the energy can be performed using the real capacity curve from Step 3.

Step 5. The MDOF system is converted to an SDOF system by transforming the capacity curve to a capacity spectrum ($S_{dt} - S_{ab}$).

$$S_{dt} = \frac{D_t}{\Gamma \phi_t}; S_a = \frac{F_{S+T}}{\Gamma \cdot L} \tag{9}$$

where Γ is the participation factor of the modal shape ϕ (Γ = $(\phi^T MI)/(\phi^T M\phi)$), and $L = \phi^T MI$.

The modal characteristics of the braced structure may change at every iteration owing to new brace characteristics. Therefore, ϕ, Γ, and L must be updated with the current configuration.

Step 6. The equivalent viscous damping ($\xi^*_{eq,S+ADS}$) of the $S+ADS$, which is necessary for obtaining the match between the displacement of the equivalent SDOF system and the target spectral displacement (S^*_{dt} = D_t*/($\Gamma\phi^T$)), was evaluated by imposing the equivalence of the target displacement and performance displacement. According to the capacity spectrum method, the demand spectrum was obtained by reducing the 5% damping response spectrum by multiplying by the damping correction factor (η), which is a function of ξ_{tot}, as follows:

$$\eta = \sqrt{2 + \frac{1}{\xi_{tot} \cdot 10}} \tag{10}$$

From Equation (10), one obtains ξ^*_{tot}, the damping needed to reduce displacement up to the target, S^*_{dt}.

$$\xi^*_{tot} = 0.1\left(\frac{S_{5\%}}{S^*_{dt}}\right)^2 - 0.05 \tag{11}$$

Step 7. The damping provided by the structure ($\xi^*_{eq,S}(D^*_t)$) can be determined using Equation (7), where D_t* is the top displacement corresponding to $I_{D,S}$, $I_{S,S+ADS}$ is the energy dissipated by the S, and the elastic strain energies, $S+T$ $I_{D,S}$ and $I_{S,S+ADS}$, are determined from the capacity curves of the S and $S+ADS$, respectively.

Step 8. Given ξ^*_{tot} from Equation (11), the equivalent viscous damping required to be supplied by the tower ($\xi^*_{eq,ADS}(D^*_t)$) (the additional equivalent viscous damping contribution owing to the tower) is evaluated from Equations (6) and (8) as follows:

$$\xi^*_{eq,ADS}(D^*_t) = \xi^*_{tot}(D^*_t) - \xi^*_{eq,S}(D^*_t) - \xi_I \tag{12}$$

Step 9. Once the additional equivalent viscous damping ($\xi^*_{eq,ADS}(D^*_t)$) (to be provided by the tower) was evaluated using Equation (12), the stiffness and yielding strength required to achieve the desired additional damping can be determined using the same procedure previously adopted for the structure (Step 7). Therefore, the dissipative tower can be designed (e.g., according to the configuration in Figure 1, the extension to the other configurations in Figure 2 is very simple). The energy dissipated by the tower can be expressed as follows:

$$I^{id}_{D,ADS} = \left(F_{Ay}D - F_{Ay}F_A(D)\right)/0.25 \tag{13}$$

D was obtained from the pushover analysis according to the control joint (where D is the top displacement, D_t).

D_{Ay} is the top displacement corresponding to the yielding of the dissipative devices: D_{Ay} can be reasonably assumed as being $D_{Ay} \leq 0.25D^*_t$. F_{Ay}, once D_{Ay} has been defined, is consequently determined.

A dissipative system usually consists of a dissipative device or a group of devices characterized by K_d and F_{dy}, which are the stiffness in the elastic range and the yielding force of the system, respectively. The tower, except for the dissipative devices, has to be designed to remain elastic and to be as stiff as possible; therefore, the following suggestions should be considered.

Designing the tower structure helps to calibrate the stiffness of the dissipative elements; this has to be conducted after the definition of the global parameters of the additional dissipative system. The ADS can be considered as a series of springs: the dissipative system (with flexibility f_d = $1/K_d$) and an elastic structure (with flexibility f_e = $1/K_e$).

$$f_A = f_d + f_e \tag{14}$$

Assuming that the dissipative system is perfectly elastoplastic,

$$F_{Ay} = F_A(D > D_{Ay}) \tag{15}$$

and

$$F_{Ay} = D_{Ay}/f_A \tag{16}$$

Therefore, using Equation (14), f_A can be evaluated from Equation (6); consequently, by selecting a reasonable value for D_{Ty} (e.g., $D_{Ty} \leq 0.25D_{Sy}$), the dissipative system is defined.

Using Equation (14), which is for a dissipative tower, such as a series of dissipative devices $(f_d; D_{Ty})$ and an elastic structure (f_e), the dissipative devices can be selected or designed according to the desired D_{Ty} and V_{Ty}, and the stiffness required for all the components follows, resulting in the evaluation of f_e.

The flowchart of this procedure is presented in Figure 5.

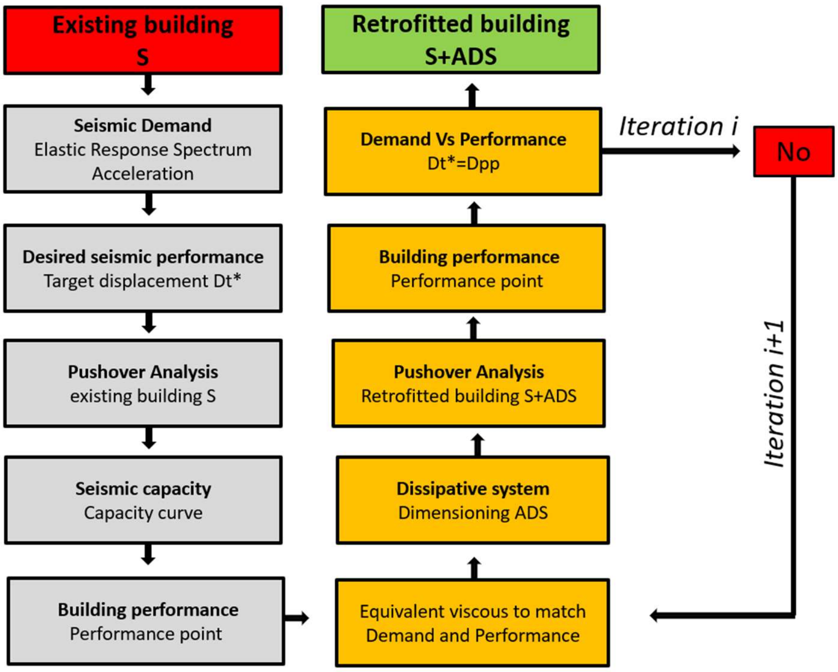


Figure 5. Flowchart for the design procedure of additional dissipative structures (ADSs).

6. Application of the Proposed Procedure to an Existing Building

The proposed design procedure was applied to the retrofitting of a real building designed according to the 1964 Italian Code (Figure 6: ante operam; Figure 7: post operam) to test a real case characterized by real materials and geometric boundary conditions.

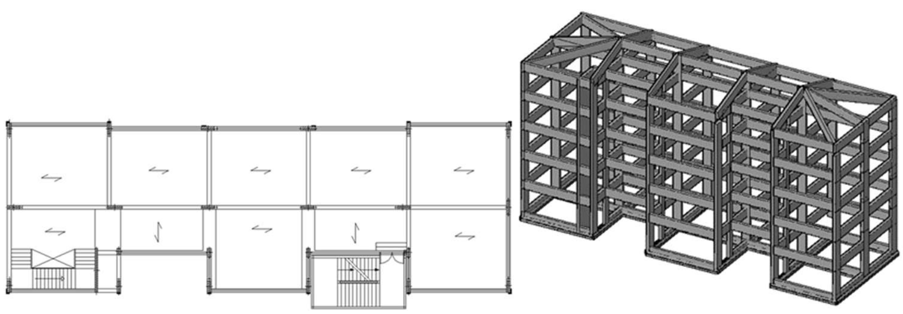
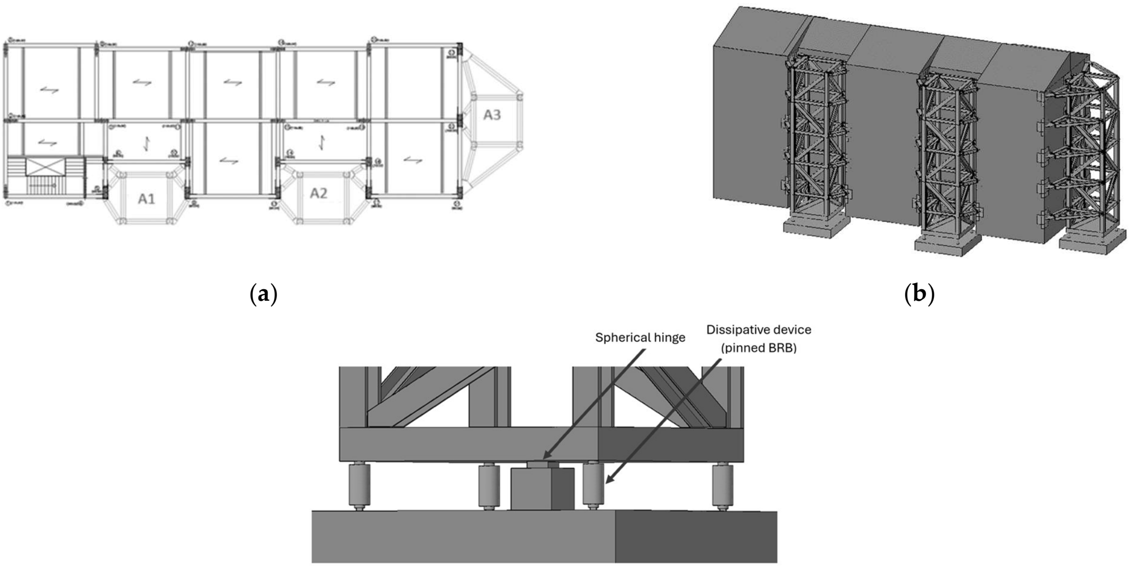


Figure 6. Planar and 3D views of the existing building.



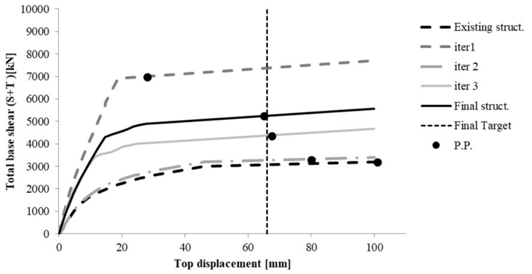
(**a**) (**b**) (**c**)

Figure 7. (**a**,**b**) Planar and 3D views of the retrofitted building (A_1, A_2, and A_3 are the designed ADSs); (**c**) detail of the base of an ADS.

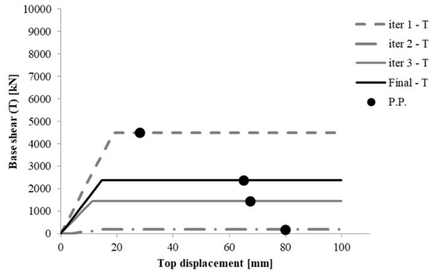
The structure is a regular seven-story RC-framed building, and retrofitting was conducted to achieve a seismic upgrade of up to 60% of the seismic demand required for a new building, with the same function to be realized at the same site, according to Italian NTC 2018 (Italian technical code D.M. 2018, currently in effect in Italy) [30].

In accordance with the proposed approach, incremental modal pushover analyses have been conducted to derive capacity curves and assess the structural responses in both the longitudinal and transverse directions. This paper presents the longitudinal analysis for concision, as this analysis holds the most significance.

The chosen target displacement, denoted as D^*, in the ADS design procedure corresponds to achieving an interstory drift not exceeding 0.005 times the interstory height ($D_{0.005}$, where h_i represents the interstory height). This interstory limit is reached before the collapse of the top displacement, $D_{s,u}$, is attained ($D^* = D_{0.005} < D_{s,u}$ = 70 mm). This procedure converged after four iterations. As mentioned, the choice of the target displacement (D^*, the limit displacement of the selected control point) was made by identifying the condition under which the interstory-drift limit ($D_{0.005}$) was reached at any level of the building. It is emphasized that such a condition may be influenced by the boundary conditions chosen between the dissipative structure and the existing building, as well as by their respective stiffness relationships. The modeling that was performed, was updated at each iteration, automatically taking into account what has been said, and, therefore, the value of the target displacement could be redefined at each iteration based on a different evolution of interstory drifts. Similarly, the use of modal or multimodal pushover analysis allows for an evaluation that is related to the evolution of the structural system corresponding to each iteration. In this case, the connection between the tower and the building was achieved using pinned struts designed to transfer only tensile–compressive forces. The dissipative system was installed at the base of the tower, where, thanks to the presence of a spherical hinge, the vertical translations of the base perimeter nodes of the tower itself are utilized (A detailed discussion of this or other possible solutions will be presented in further studies.). As depicted in Figure 8, the performance point before the retrofitting is $D_{S,pp}$ = 100 mm (while the collapse displacement is $D_{s,u}$ = 70 mm), with a base shear of $V_{S,pp}$ = 3200 kN. In contrast, for the retrofitted structure, at the end of the fourth iteration, the performance point corresponds to $D_{S+T,pp,4}$ = 65 mm and $V_{S+B,pp,4}$ = 5250 kN. Figure 9 provides a comprehensive illustration of how the retrofitting system enhances the building's safety. Not only does the performance point align with the desired target but also the base shear absorbed by the tower's system significantly reduces the seismic forces absorbed by the existing foundations, which are often challenging to retrofit. The overall increase in the base shear at the performance point is from 3200 to 5250 kN, distributed as follows: 2367 kN in the existing foundations (a 28% reduction) and 3899 kN in the ADS foundations of the three towers (refer to Figure 9). Therefore, the retrofitted building meets the desired performance level (interstory drift containment) and simultaneously reduces the seismic action affecting the existing foundations, which commonly represent a critical aspect in seismic retrofit interventions. In terms of damping, the equivalent viscous damping in the final configuration is ν_S = 0.21, and $\nu_{S+T,4}$ = 0.43.



(**a**)



(**b**)

Figure 8. (**a**) Variations in the global responses of the S+ADS (cumulative base shear–top displacement) and (**b**) contributions of the dissipative towers (Ts) (tower base shear–top displacement).

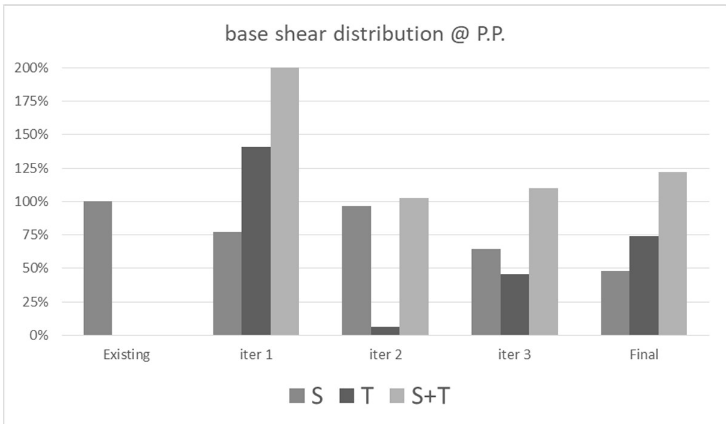


Figure 9. Distribution of base shear between S and T according to the different stages of the design procedure (The base shear of the existing structure at the performance point is 100%).

To validate the previous analysis, a comparison between the time-history analysis (Figure 10; Table 1) and nonlinear static analysis was conducted. For the time-history analysis, a set of 7 accelerograms, compatible with the site response spectrum (Italian technical code 2018; soil class: C; topographical category: T1; nominal life: 50 years; functional type: 3; limit state: SLV; Lon.: 13,4397°; Lat.: 42,0412°) has been generated using the software Rexel [30–32]. The average value of the response derived from the THs (Figure 11) confirms that the results obtained from the pushover analysis can be considered as being representative of the "real" seismic response of the retrofitted building.

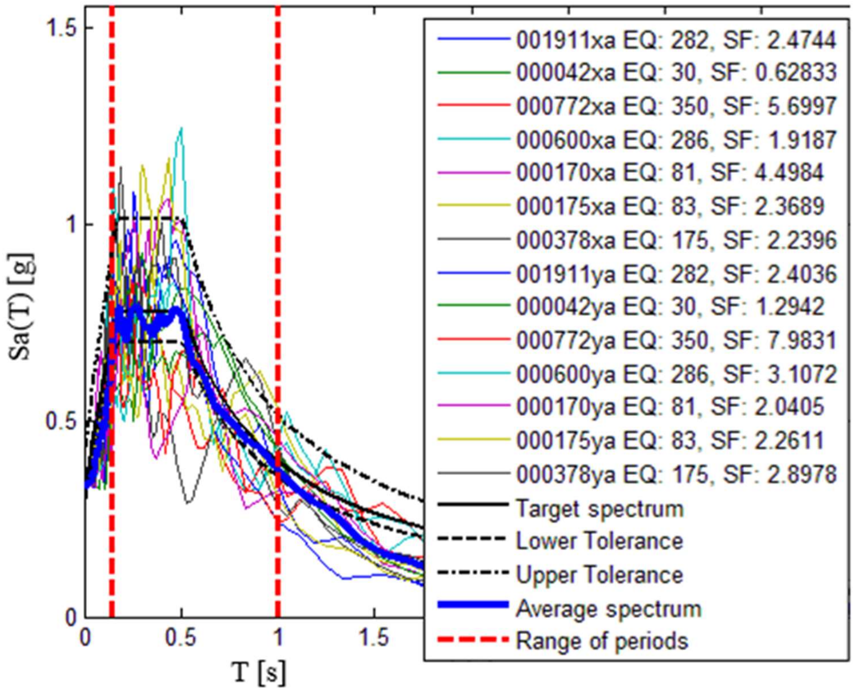


Figure 10. REXEL data selection: selected-record response spectrum.

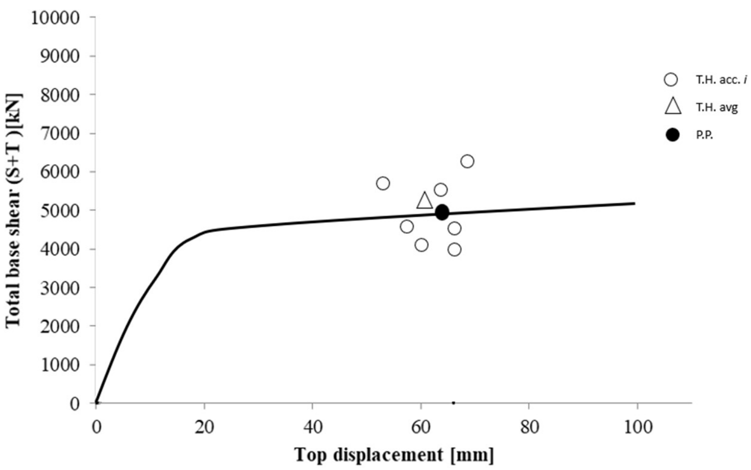


Figure 11. Global response of the S+ADS (cumulative base shear–top displacement) from pushover analysis (PP) and from time-history analysis using a set of 7 accelerograms ($I = 1, \ldots, 7$; TH avg. is the average value of the set TH*acc i*).

Table 1. Selected record: European strong-motion data provided by REXEL.

Waveform ID	Earthquake ID	Station ID	Earthquake Name	Mw	PGA_X (m/s^2)	PGA_Y (m/s^2)
1911	282	ST126	Komilion	5.4	1.3067	1.3452
42	30	ST8	Ionian	5.8	5.1459	2.4983
772	350	ST223	Umbria Marche	5.3	0.5673	0.405
600	286	ST223	Umbria Marche	6	1.6852	1.0406
170	81	ST46	Basso Tirreno	6	0.7188	1.5846
175	83	ST50	Volvi	6.2	1.3649	1.43
378	175	ST152	Lazio Abruzzo	5.9	1.4437	1.1158
Mean:				5.8	1.7475	1.345642857

7. Conclusions

This paper presents and discusses a displacement-based procedure for designing ADSs for the seismic rehabilitation of existing buildings.

The primary objective of this procedure is to achieve a specified target displacement, thereby limiting deformations and interstory drift while enhancing dissipation. A closely related benefit is a significant reduction in the base shear in existing foundations.

The proposed procedure, which involves determining the stiffness and yielding force of the dissipative system, is relatively straightforward as it relies on static (nonlinear) analysis. However, it requires several iterations to reach convergence. Additionally, it can adapt to various challenging situations by working with existing structures, including irregularities in plans and elevations, low plastic limits, and other characteristics.

This approach distinguishes itself from others by considering the contributions of the existing structure. Furthermore, it meticulously evaluates the contribution of the dissipative system to meet the required performance of the new global system: the existing building + the ADS. This procedure originates from a revision of a method for designing dissipative braces, and its effectiveness, based on nonlinear static analysis, has been demonstrated through the application discussed herein.

The limitations of the proposed procedure are, mainly, in the assessment of structural capacity through the use of the pushover analysis. Therefore, the most relevant limitations are related to the simplification of dynamic effects (Pushover analysis simplifies dynamic effects by applying lateral forces incrementally along the height of the structure.) because although this approach is useful for capturing the global response of the structure, it may not accurately represent the local dynamic behavior, such as torsional effects, pounding between adjacent structures, or soil–structure interaction effects.

It is important to emphasize how the designed intervention aims to achieve a structural system that is less susceptible to torsion and soil–structure interactions (in the existing component of the building).

Moreover, because the accuracy of pushover analysis results depends on the selection of input parameters, such as lateral load patterns, the distribution of lateral forces, material properties, and boundary conditions, inaccuracies or uncertainties in these parameters can lead to unreliable analysis results. According to this, the connection between the S and the ADS is finalized to move the "relevant" nonlinear behavior (dissipation) in the dissipative device of the ADS, keeping the existing structure mostly elastic and regularized in terms of the stiffness distribution.

Therefore, despite these limitations, the pushover-based procedure remains a valuable tool for the preliminary seismic retrofitting design of structures with ADSs. The proposed procedure provides engineers with a simplified, yet insightful, approach.

Subsequent developments and applications will be the subjects of future work. However, the author believes that the proposed approach represents a substantial advancement in displacement-based design for retrofitting with dissipative systems, especially given the limited discussion on the use of dissipative towers in the existing literature. This approach is both theoretically simple and straightforward in execution, making it suitable for professional applications without requiring expertise in complex nonlinear dynamic analysis. Only common static pushover analysis is necessary.

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