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Analysis and comparison of two different configurations of external dissipative systems

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Abstract

This paper deals with the seismic protection of existing buildings, especially r.c. frame ones, by means of external passive dissipative systems. These type of systems provide larger flexibility in controlling the structural behavior, and some feasibility advantages, but their efficiency in terms of performance still need to be proven. In particular, this study analyzes and compares the performance of two external solutions using linear fluid viscous dampers (FVDs) for the seismic upgrading of an existing benchmark structure, the Van Nuys building. The first arrangement is a recent solution, known as "Dissipative Tower", which exploits the rocking motion of a steel truss hinged at the foundation level for the dampers activation; the second one consists in coupling the building with an external stiff contrasting structure, where the dampers are located horizontally at the storey level. First, a state space formulation of the problem, based on the assumption of linear elastic behavior for both the existing frame and the external dissipative structures, is presented in general terms. The proposed formulation, suitable for both the external arrangements, allows to evaluate the influence of the dissipative solutions on the system modal properties. Successively, the performance of the two proposed external passive structures, is evaluated and compared with that of the bare existing frame, by considering important engineering demand parameters (EDPs) such as interstorey drifts, absolute accelerations and shear actions resisted by the frame and by external systems.

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Keywords: Passive Seismic Protection; Linear Viscous Dampers; External Seismic Retrofit; Rocking Motion; Mass Proportional Damping.

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1. Introduction

Traditionally, passive damping devices are installed within a building frame in either diagonal or chevron braces connecting adjacent storeys. This type of damping configuration, investigated extensively in the literature (e.g. [1]-[6]), may present some disadvantages, especially when employed for the retrofit of existing buildings. Among these, the increase of internal actions in the columns with induced premature local failures [7], the need of localized strengthening of foundations, and the indirect costs related to the downtime during the retrofitting operations. For these reasons, the use of external passive control system is becoming more and more frequent thanks to the minimized interferences with the existing frames. Such type of solution provides a larger flexibility of structural behaviors and some feasibility advantages, while its efficiency in terms of performance still need to be proven.

A possible retrofit configuration, called external dissipative rocking system, permits to gain the seismic protection of an existing frame building by exploiting the rocking motion of a stiff brace hinged at the foundation level to activate fluid viscous dampers (FVDs), located at the base, for the energy dissipation (Fig. 1 (a)) [8], [9]. The rocking configuration can be used either in planar and spatial arrangements and recently some applications have been developed [10], [11] and are known as "Dissipative Towers", which are a patented solution [12]. Among the external intervention possibilities there is also the case of FVDs coupling two adjacent structural systems, which has also been analyzed in many studies [13]-[17]. Fig. 1 (b) depicts the case where dampers are placed horizontally at the storey level, between the frame and an external stiff contrasting structure; this way, the links are activated by the floor absolute displacements [14]. A similar configuration can be obtained by placing the dampers between adjacent buildings exhibiting different dynamic properties [15]-[17].

This work analyzes the dynamic behavior and the seismic performance of a benchmark building retrofitted by employing the abovementioned external damping configurations. A problem formulation is presented in general terms, providing also an insight into the modal properties of the coupled systems. Successively, the seismic response of the bare building is compared against that of the building retrofitted with the two alternative configurations, assuming for both the cases an added damping ratio ξ_{add} =30%.

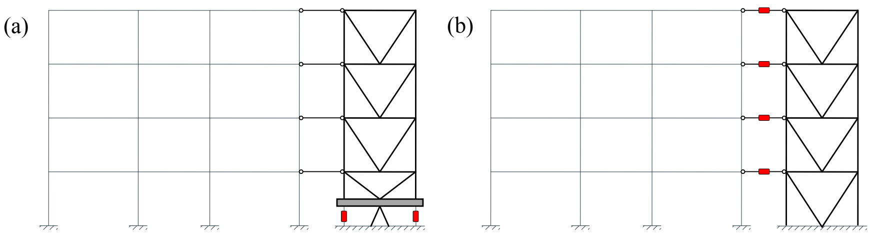

Fig. 1. (a) Dissipative rocking system (b) FVDs used for the coupling with external stiff contrasting structure

2. Problem Formulation

The reported formulation, suitable for both the two external arrangements, refers to the plane problem and only the horizontal ground motion component is taken into account. By assuming a linear elastic behavior for the dynamic system, the equation of motion can be expressed as:

$$\mathbf{M}\ddot{\mathbf{u}}(t)+\mathbf{C}\dot{\mathbf{u}}(t)+\mathbf{K}\mathbf{u}(t)=\mathbf{M}\mathbf{p}a_g(t) \tag{1}$$

where $\mathbf{u}(t)\in R^l$, is the vector of nodal displacements, the dot $(\cdot)$ denotes time-derivative; $\mathbf{p}\in R^l$ is the load distribution vector, l denotes the total number of degrees-of-freedom, and $a_g(t)$ is the external scalar loading function describing the seismic base acceleration. The time invariant matrices $\mathbf{M}$, $\mathbf{K}$, $\mathbf{C}$ describe the mass, stiffness and damping operators $R^l \rightarrow R^l$; they result from the sum of the contribution due to the existing frame and the one coming from the external dissipative bracing system. The inherent damping of the bare frame is modeled by a Rayleigh damping matrix. Generally, the external bracing system notably influences the stiffness and damping operators while it contributes only marginally to the mass operator. The displacement vector $\mathbf{u}(t)$ collects both the displacements describing the frame response and the displacements involving the bracing deformations.

In order to study the dynamic response of the system existing frame plus external passive system it is useful to separate the displacements associated with the masses, and thus involving inertial forces, from the displacements describing the internal degrees of freedom, related to stiffness and damping forces only. Accordingly, the total displacement vector $\mathbf{u}(t)$ can be split into the active components collected in the vector $\mathbf{x}(t) \in R^m$ and the other components $\mathbf{y}(t) \in R^n$ ($l = m + n$). The matrices describing the linear operators and the distribution vector can be consequently partitioned as follows

$$\begin{vmatrix} \mathbf{M}_{xx} & 0 \\ 0 & 0 \end{vmatrix} \begin{vmatrix} \ddot{\mathbf{x}} \\ \ddot{\mathbf{y}} \end{vmatrix} + \begin{vmatrix} \mathbf{C}_{xx} & \mathbf{C}_{xy} \\ \mathbf{C}_{yx} & \mathbf{C}_{yy} \end{vmatrix} \begin{vmatrix} \dot{\mathbf{x}} \\ \dot{\mathbf{y}} \end{vmatrix} + \begin{vmatrix} \mathbf{K}_{xx} & \mathbf{K}_{xy} \\ \mathbf{K}_{yx} & \mathbf{K}_{yy} \end{vmatrix} \begin{vmatrix} \mathbf{x} \\ \mathbf{y} \end{vmatrix} = \begin{vmatrix} \mathbf{M}_{xx} & 0 \\ 0 & 0 \end{vmatrix} \begin{vmatrix} \mathbf{p}_x \\ 0 \end{vmatrix} a_g \tag{2}$$

As usual, only the masses related to the horizontal floor displacements are considered in order to reduce the dimension of the dynamic problem and to simplify the interpretation of the results.

Both the two arrangements induce non-classical damping because the distribution of damper results in a damping matrix which is not proportional to the global mass matrix, nor to the stiffness one. The first arrangement corresponds to a highly non-classically damped system, whereas the second one is characterized by a damping distribution similar to the mass distribution and it is closer to a classically-damped system. For the solution of the dynamic problem, a state-space approach is convenient because it gives the opportunity to perform the complex modal analysis of the coupled system, leading to the knowledge of the modal properties in presence of non-classical damping. For this purpose, the vector $\mathbf{v}(t) = \dot{\mathbf{x}}(t)$ and the state vector $\mathbf{z}(t) = |\mathbf{x}(t) \quad \mathbf{v}(t) \quad \mathbf{y}(t)|^T$ collecting the displacements and the velocities of the active displacements and the displacements of the internal nodes, are introduced. Eqn. (1) can be reduced to a first-order state space form $\dot{\mathbf{z}}(t) = \mathbf{A}\mathbf{z}(t) + \tilde{\mathbf{p}} a_g(t)$, where:

$$\mathbf{A} = \begin{vmatrix} \mathbf{0} & \mathbf{I} & \mathbf{0} \\ -\mathbf{M}_{xx}^{-1}\left(\mathbf{K}_{xx} - \mathbf{C}_{xy}\mathbf{C}_{yy}^{-1}\mathbf{K}_{yx}\right) & -\mathbf{M}_{xx}^{-1}\left(\mathbf{C}_{xx} - \mathbf{C}_{xy}\mathbf{C}_{yy}^{-1}\mathbf{C}_{yx}\right) & -\mathbf{M}_{xx}^{-1}\left(\mathbf{K}_{xy} - \mathbf{C}_{xy}\mathbf{C}_{yy}^{-1}\mathbf{K}_{yy}\right) \\ -\mathbf{C}_{yy}^{-1}\mathbf{K}_{yx} & -\mathbf{C}_{yy}^{-1}\mathbf{C}_{yx} & -\mathbf{C}_{yy}^{-1}\mathbf{K}_{yy} \end{vmatrix} \quad \tilde{\mathbf{p}} = \begin{vmatrix} 0 \\ \mathbf{M}_{xx}^{-1}\mathbf{p} \\ 0 \end{vmatrix} \tag{3}$$

The free vibration problem can be solved by assuming a solution of the form $\mathbf{z}(t) = \boldsymbol{\varphi} e^{\lambda t}$, where $\lambda, \boldsymbol{\varphi}$ are a eigenvalue-eigenvector pair of $\mathbf{A}$. Knowing the modal properties, the problem solution can be obtained as a linear combination of the single mode contributions. Let $\mathbf{\Lambda}$ be the diagonal matrix containing the complex eigenvalues and $\mathbf{\Phi} = |\boldsymbol{\varphi}_1, \boldsymbol{\varphi}_2, \ldots, \boldsymbol{\varphi}_{2m+n}|$ the complex eigenmatrix containing the eigenvectors, such that the orthogonality property $\mathbf{\Lambda} = \mathbf{\Phi}^{-1}\mathbf{A}\mathbf{\Phi}$ holds. Finally the seismic response is determined via modal decomposition method.

It can be useful to observe that the limit case of infinitely stiff contrasting structure leads to a single-degree of freedom system with constrained linear deformation in the former case, and to a multi-degree of freedom system with mass proportional damping (classically damped) in the latter case.

3. Case study and seismic hazard

The Van Nuys building is a 7-storey 3 bay-by-8 bay cast-in-place r.c. moment-resisting frame building, with non ductile column detailing, designed in 1965 in compliance to the lateral force requirements of 1964 Los Angeles City Building Code. The structural system consists of perimeter moment frames and interior slab-column frames, as shown by the planar view and the transverse section (N-S direction) of Fig. 2.

The seismic hazard has been described by employing a design value of PGA, related to a given Limit State (Life Safety) having a probability of exceedance of 10% in 50 years, therefore a return period of 475 years. The ground motions variability, instead, is represented by a set of 60 records provided by Somerville et al. used in the SAC project, whose characteristics are reported in literature [18]. Among these records two suites of ten time-history, for the site of Los Angeles, area in which the benchmark structure is located, have been chosen.

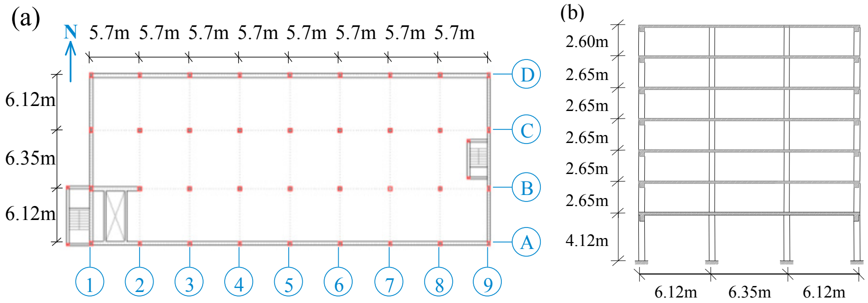

Fig. 2. Van Nuys Building: (a) planar view and (b) transverse section

The dynamic system is described by considering only the motion along the transverse N-S direction. The floors are assumed to be rigid in the horizontal plane and the masses are concentrated at the storey levels so that the vector of active degrees of freedom $\mathbf{x}$ collects the seven floors motions only, for both the two analyzed configurations. The vector $\mathbf{y}$,instead, has dimension two in the case of the rocking system, since it collects the vertical displacements of the dampers located at the tower base, and dimension seven in the case of the other arrangement, since it collects the horizontal displacements of the dampers at the seven floors of the external braced frame. In both the two upgrading configurations analyzed, the external structures are the same, in terms of members sizes and geometrical dimensions. Moreover, their contribution in terms of stiffness, evaluated by imposing a unit horizontal displacement at their top and by evaluating the corresponding base reaction, is equal to the one of the building.

The dampers have been designed by considering a target value of the added damping ξ_{add} of 30%, evaluated through the expression proposed in the ASCE Standard [19]:

$$\xi_{add} = \frac{1}{4\pi E_f} \sum_{j=1}^{N} E_j \qquad (4)$$

where E_j is the work done by j-th device in one complete vibration cycle at the fundamental frequency of the coupled system, and E_f is the relevant maximum strain energy. The target value assumed in the both the upgrading configurations is ξ_{add} =30%.

4. Modal properties and seismic response comparison

This section provides first an insight into the modal properties of the different arrangements analyzed, in terms of vibration periods and damping ratio of the first mode. Successively, it reports and compares the seismic responses of the bare Van Nuys building (*As is*), and of the two alternative upgraded configurations, i.e., the one employing the dissipative rocking bracing (*RB*) system, and the one corresponding to dampers located horizontally at the storey levels, between the frame and the fixed base contrasting structure (*FB*) The response of the different systems is evaluated by employing the modal decomposition method. The values of the EDPs reported below (displacements, interstorey drifts, absolute accelerations and shear actions) are the mean of the maximum values obtained by considering twenty ground motion records time-histories.

The two retrofit configurations are characterized, respectively, by 7 complex modes and two overdamped ones in the *RB* configuration and 7 complex modes and 7 overdamped ones in the case of the *FB* arrangement. The first vibration period of the bare frame is equal to T_1=1.204 s, that of the system with rocking bracings is 1.037 s (14% less) and that of the case of the *FB* system is 1.047 s (13% less). The two external arrangements provide also similar values of added damping to the first mode, that is ξ_{add}=0.342 (*RB*) and ξ_{add} =0.329 (*FB*). Fig. 3 (a) and Fig. 3 (b) reports the floor displacements and the interstorey drifts (IDRs) for the three configurations analyzed (*As is, FB* and *RB*). It is observed that the reduction of the frame displacements is nearly 52% both in the *FB* case and in the *RB* configuration. Fig. 3 (a) depicts the displacements distribution (x_i) along the height of the building, for i=1, 2, .., 7, while Fig. 3 (b) reports the distribution of the IDRs (Θ_i). It is noteworthy that the coupling with the rocking bracing leads to a linearization of the frame displacements and to a more constant distribution of IDRs among the elevations, whose maximum value is equal to 0.0089; the coupling with a *FB* system, instead, does not affect the shape of both displacements and IDRs distribution. Nevertheless, the *FB* configuration yields a significant reduction of the IDRs, which are however higher than those corresponding to the rocking configuration.

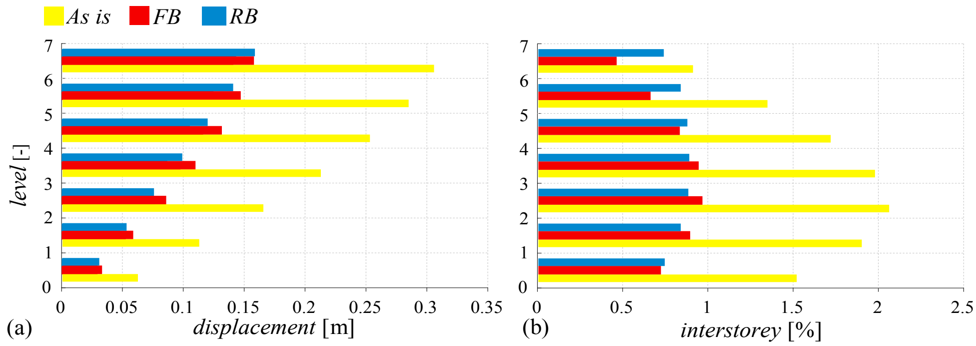

Fig. 3.Comparison of displacements (a) and interstorey drifts (b) distributions

Fig. 4(a) and (b) report the distribution along the height of respectively the shear action of the frame and of the external structures, for all the analyzed configurations.

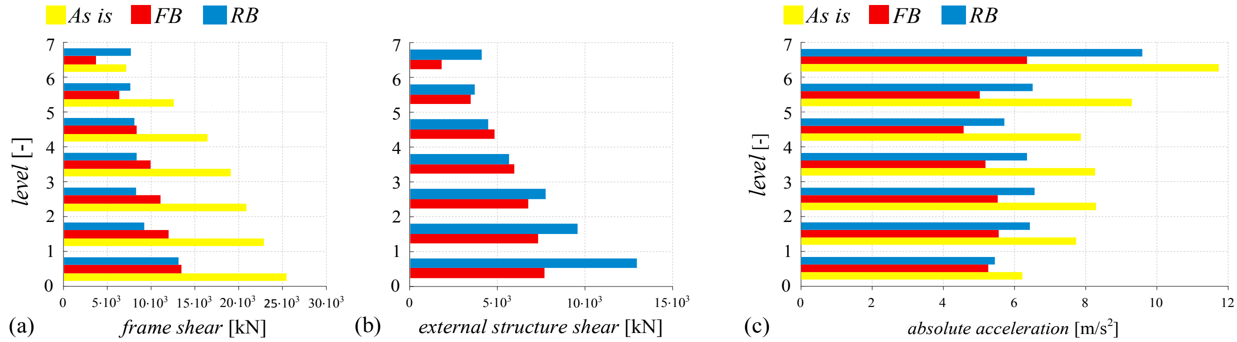

Fig. 4. Shear actions resisted by the frame (a) and by the external structures (b), absolute acceleration distribution along the height (c)

As already discussed for the displacements, the addition of the two external dissipative systems results in a reduction of the global shear demand on the frame, with respect to the bare frame. The addition of rocking bracings, leading to a displacement shape linearization, results also in higher shear actions on the frame at some levels. The relative reduction of the maximum base shear acting on the frame, with respect to the bare building, is nearly 47% for the *FB* case and 49% for the *rocking* one. Fig. 4 (c) shows the values of the floor absolute accelerations observed at the various levels of the building for the configurations investigated. It is observed that both the retrofit configurations induce a reduction of the maximum absolute acceleration values with respect to bare frame (*VN*). A better result is achieved with the *FB* system, leading to a reduction of 46%, while the rocking bracings (*RB*) provides a reduction of 18%. This result is of particular importance for the performance evaluation of acceleration-sensitive non structural components, and may impair the benefits of the retrofit with the rocking system.

Conclusions

In this paper, two alternative arrangements of external passive retrofitting systems are presented, each one characterized by a different kinematic behavior. A problem formulation concerning the dynamic of the coupled system formed by an existing frame and an external damping system is presented in general terms.

The results of the analysis of the seismic response of a benchmark case study in two different upgrading configurations (i.e. the coupling with an external dissipative rocking system and the case where dampers are placed horizontally at the storey levels, providing the same amount of added damping) show that the addition of the external dissipative rocking system provides the best distribution of interstorey drifts, related to a linearization of displacements distribution, but also higher shear actions on the frame at some levels. The result in term of absolute accelerations reduction is satisfactory especially for the FB system, while the rocking bracings provide a lower reduction, especially if compared to the results achieved in terms of displacements.

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