Energy dissipation devices in seismic up-grading of monumental buildings

A. Mandara
Second University of Naples, Department of Civil Engineering, Aversa, Italy

F.M. Mazzolani
University of Naples Federico II, Department of Structural Analysis and Design, Naples, Italy

ABSTRACT: The seismic behaviour of structural schemes typical of some monumental buildings, fitted with passive energy dissipation devices is examined in this paper. Damping devices are located at the connection between vertical walls and floor diaphragm, so as to provide an additional amount of dissipated energy, both viscous and hysteretic. A simplified model with concentrated masses is firstly investigated in the frequency domain, in order to put into evidence the main behavioural aspects. Time-history response of significant schemes under earthquake input is then evaluated. The results show that such a technique is quite effective in the reduction of inertia forces acting on all walls, in particular when plastic threshold devices are used together with viscous dampers, thus providing a significant increase of seismic protection. For this reason it can be considered as a reversible, simpler and, hence, cheaper alternative to other protection systems.

1 INTRODUCTION

As widely experienced in many applications carried out in the last years, an effective alternative to conventional strengthening operations in the seismic up-grading of existing buildings is represented by passive energy control and dissipation techniques. Main goals of such techniques are not only to increase the maximum earthquake magnitude tolerable by the structure, but also to limit the extent of strongly invasive consolidation interventions, which sometimes are not suitable to existing constructions, in particular when they possess monumental features. For this reason this innovative techniques are considered with great attention in the field of historical buildings, where a demand for more effective seismic protection solutions is deeply felt.

According to this approach, the level of structural reliability can be enhanced either by fitting the construction with adequate energy dissipation capability and/or by reducing the amount of earthquake energy transferred to the structure. As an alternative to the base isolation system, which reduces the input energy by means of a beneficial modification of vibration properties of the building but requires heavy interventions on the bearing structure (base cut, new foundation structure, etc.), when operating on historical constructions it can be more convenient to increase the level of structural safety against earthquake by means of suitable energy dissipation devices placed at key points of the construction, where relative displacements between members allow some energy to be dissipated by means of viscous and/or hysteretic effect. An appropriate installation site for such devices is usually at the wall-to-floor interface of masonry buildings, where relative motion can occur without risk of impairing the global structural integrity. The devices, usually of oleodynamic type, are located at each floor, including the roof level, sometimes together with a local isolation system made of elastomeric pads or even sliding supports.

Such a solution is frequently applied also to bridge structures in order to improve their behaviour under seismic actions, as well as to reduce the effect of both vehicle braking and thermal changes (Di Marzo et al. 2000). In recent years, because of its effectiveness at low cost, it has been also proposed for new structures (Mazzolani and Serino 1997), as well as for monumental

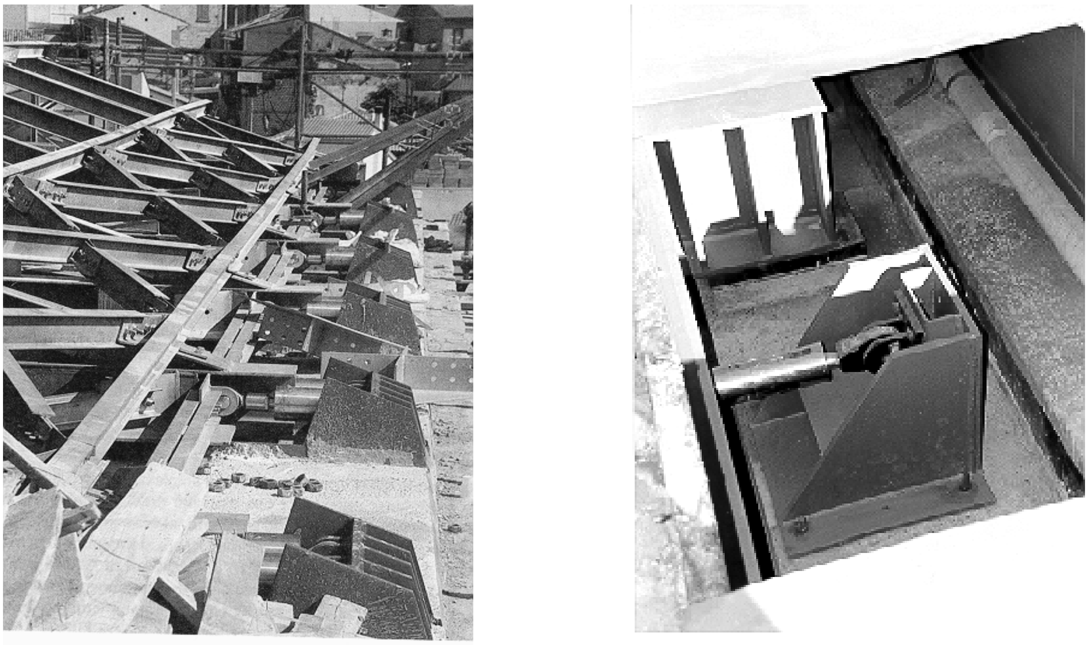


Figure 1: Oleodynamic devices installed on the roof of monumental buildings in Italy: the S. Giovanni Battista church in Carife (left) and the New Library of University Federico II in Naples (right).

buildings. To this purpose, it is worth noting that such an approach can represent the only solution when law dispositions do not permit to perform too much intrusive interventions on buildings. As a matter of fact, the interventions carried out in the S. Giovanni Battista Church in Carife-Italy (Mazzolani and Mandara 1994) and in the New Library of Federico II University (Mazzolani 2001) (Figure 1) represent the very first cases of seismic upgrading of historical constructions carried out in Italy by means of dissipative devices. A similar solution, making use of shape memory alloy devices for supplementing oleodynamic dampers, has been later on adopted for the seismic protection of the Basilica of S. Francesco in Assisi-Italy (Croci et al. 1999), severely damaged by the Umbro-Marchigiano earthquake in 1997.

Referring to devices located at floor level, two basic approaches can be followed: the *Plastic Threshold Approach* (PTA) and the *Optimal Viscous Approach* (OVA) depending on which parameter is assumed for the design of devices. Based on PTA devices are conceived and sized in such a way to limit the magnitude of force transmitted across connected members to a maximum value, established according to the design resistance of structural elements involved. After the threshold has been reached, a hysteretic energy dissipation takes place, which allows additional protection to be provided without increasing the magnitude of the force acting on members. Below threshold the behaviour is virtually rigid, which ensures the maximum degree of redundancy to the structure under serviceability load conditions. According to OVA, the interaction between connected members is ruled by viscous properties of devices only, which are dimensioned in such a way to minimise the magnitude of the force acting on them independently of its value. Contrary to PTA, the connection between elements is never fully rigid, so that energy can be dissipated under moderate intensity earthquakes, too.

In the following, an example of application of both above approaches is presented referring to a practical case. A preliminary investigation of the dynamic response of a simplified scheme with concentrated masses is also performed, in order to highlight the basic behavioural aspects, as well as the influence of relevant parameters. Linear viscous devices are considered in this analysis, whose optimal features are determined in such a way to reduce the structural response in terms of displacement. In a second step, the time-history response under a specified earthquake input is evaluated for a number of one-storey box-like cases, aiming at emphasising the effectiveness of the proposed system when applied to typical schemes of masonry building. In this investigation, the use of plastic-threshold devices is also considered, arranged in series to the purely viscous ones, in order to optimise the global structural behaviour at ultimate limit state.

2 THE STRUCTURAL IDEALISATION

2.1 General aspects of structural behaviour

When applied to box-like structural schemes (Figure 2a), the above device configuration allows to control the forces transmitted across different wall orders. This typically occurs under an earthquake whose direction is parallel to some walls in the buildings. In such a case, the walls which are perpendicular to seismic waves, hereafter referred to as weak walls, offer a reduced out-of-plane stiffness and, for the same reason, they also have a poor resistance due to high values of out-of-plane bending and shear. A commonly adopted provision in this case is to create very stiff floor diaphragms rigidly connected to all vertical walls in such a way to obtain rigid end restraint conditions. Because of pinned end connection, this does improve the working conditions of walls but, nevertheless, due to diaphragm stiffness it may result in an excess of loading on the wall parallel to earthquake direction (strong walls), in particular when the length (L_2) of these walls is short compared with that (L_1) of weak walls. The transmission of load from weak to strong walls, in fact, is due to both the vertical connection at wall angles or crosses and to the stiffness of the diaphragm itself. In particular, when the building is very long in plant, as well as when longitudinal, weak walls are full of openings and ribbed or stiffened at discrete points, the connection with strong (transverse) walls fades out, so that wall orders can be considered as independent of each other (Figure 2b). Under this assumption, the amount of load transferred to strong walls can be minimised by inserting energy dissipation devices at the wall-to-floor interface, by exploiting the natural tendency of both wall orders to have different magnitudes of top displacement. The global behaviour, thus, profits of the introduction of elements characterised by high damping, ductility and energy dissipation capabilities, leading to a significant increase of the maximum level of the earthquake intensity corresponding to collapse.

2.2 The simplified elastic model

Based on the above considerations, if a one-storey building is assumed, it is possible to take out the simplified elastic two-DOFs model shown in Figure 2c. For the sake of simplicity, all masses are assumed concentrated at the top of respective walls, represented by elastic springs. Due to the rigid diaphragm, longitudinal walls (having global mass m_1 and vertical out-of-plane stiffness k_1) are connected to transverse walls (having global mass m_2 and vertical in-plane stiffness k_2) by means of the viscous damper (c), which is in turn paralleled by the spring k_{eq} representing the equivalent horizontal out-of-plane stiffness of longitudinal walls. When wall orders are independent of each other, k_{eq} tends to zero or to the stiffness value of the top r.c. beam, if present. Because of the stiff connection on the top of transverse walls, the mass of roof elements is assumed to be a part of m_2. Considering the model symmetry, the equations of motion are:

$$\begin{aligned} m_1\ddot{x}_1 + c(\dot{x}_1 - \dot{x}_2) + k_{eq}(x_1 - x_2) + k_1 x_1 = -m_1\ddot{x}_g \\ m_2\ddot{x}_2 + c(\dot{x}_2 - \dot{x}_1) + k_{eq}(x_2 - x_1) + k_2 x_2 = -m_2\ddot{x}_g \end{aligned} \tag{1}$$

where x_1 and x_2 are the displacement of m_1 and m_2 relative to ground, respectively, and x_g is the ground motion itself. It can be observed that, due to model symmetry, equations of motion are symmetric, too, with regard to both masses and springs.

By dividing Equation (1) by m_2 and introducing the following nondimensional parameters: $\kappa_1 = k_1 / k_{eq}$, $\kappa_2 = k_2 / k_{eq}$, $\nu = c / \sqrt{k_{eq} m_2}$, $\mu = m_1 / m_2$, Equations (1) are obtained in matrix form:

$$\begin{bmatrix} \mu & 0 \\ 0 & 1 \end{bmatrix} \begin{Bmatrix} \ddot{x}_1 \\ \ddot{x}_2 \end{Bmatrix} + 2\nu\omega_0 \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} \begin{Bmatrix} \dot{x}_1 \\ \dot{x}_2 \end{Bmatrix} + \omega_0^2 \begin{bmatrix} (\kappa_1 + 1) & -1 \\ -1 & (\kappa_2 + 1) \end{bmatrix} \begin{Bmatrix} x_1 \\ x_2 \end{Bmatrix} = -\begin{Bmatrix} \mu \\ 1 \end{Bmatrix} \ddot{x}_g \tag{2}$$

where the position $\omega_0^2 = k_{eq} / m_2$ has been made. The dynamic response is evaluated for the harmonic input $x_g(t) = x_g(\overline{\omega}) \cdot e^{i\overline{\omega}t}$, where $\overline{\omega}$ is the radial frequency of the applied ground motion.

The steady-state response for x_1 and x_2 is harmonic as well with the same frequency $\overline{\omega}$ and is represented by the functions:

$$x_1(t) = x_1(\overline{\omega}) \cdot e^{i\overline{\omega}t}\,; \quad x_2(t) = x_2(\overline{\omega}) \cdot e^{i\overline{\omega}t} \tag{3}$$

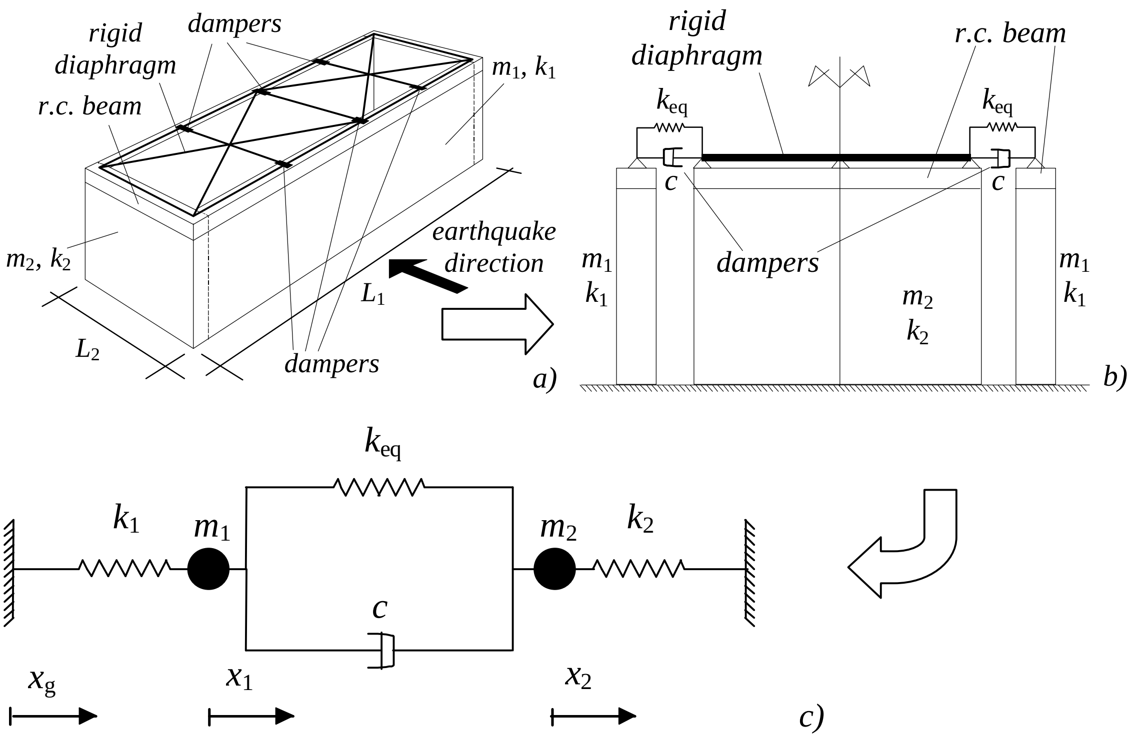


Figure 2 : The structural model and the corresponding two-DOFs idealisation.

By substituting Equations (3) into (2) one may obtain:

$$\begin{Bmatrix} x_1(\varpi) \\ x_2(\varpi) \end{Bmatrix} = \beta^2 \begin{bmatrix} -\beta^2\mu + (\kappa_1+1) + 2\upsilon\beta i & -1-2\nu\beta i \\ -1-2\nu\beta i & -\beta^2 + (\kappa_2+1) + 2\upsilon\beta i \end{bmatrix}^{-1} \begin{Bmatrix} \mu \\ 1 \end{Bmatrix} x_g(\varpi) \tag{4}$$

where $\beta = \varpi / \omega_0$. Apart from k_1 and k_2, Equations (4) provide the global shear forces transmitted to weak and strong walls, respectively. Thus, they are highly representative of the global structural behaviour under dynamic excitation. By inverting the equation system (4), the following transfer functions for x_1 and x_2 are obtained:

$$\frac{x_1(\varpi)}{x_g(\varpi)} = \beta^2 \left[\frac{1+\mu\left(1+\kappa_2-\beta^2\right)+2\nu\beta i(1+\mu)}{\kappa_1+\kappa_2+\kappa_1\kappa_2-\beta^2(1+\kappa_1)-\mu\left(\beta^2+\beta^2\kappa_2-\beta^4\right)+2\nu\beta i\left(\kappa_1+\kappa_2-\mu\beta^2-\beta^2\right)} \right] \tag{5}$$

$$\frac{x_2(\varpi)}{x_g(\varpi)} = \beta^2 \left[\frac{1+\kappa_1+\mu\left(1-\beta^2\right)+2\nu\beta i(1+\mu)}{\kappa_1+\kappa_2+\kappa_1\kappa_2-\beta^2(1+\kappa_1)-\mu\left(\beta^2+\beta^2\kappa_2-\beta^4\right)+2\nu\beta i\left(\kappa_1+\kappa_2-\mu\beta^2-\beta^2\right)} \right] \tag{6}$$

Equations (5) and (6) are plotted in Figure 3 for given values of ν, κ_1, κ_2 and μ. For an undamped system ($\nu = 0$), it is possible to observe the typical two-peak resonant trend corresponding to the non dimensional free circular frequencies $\beta_1 = \omega_1/\omega_0$ and $\beta_2 = \omega_2/\omega_0$ of the two-DOFs system. For $\nu = \infty$, a rigid connection between m_1 and m_2 occurs and, therefore, a third peak is present at $\beta_3 = \omega_3/\omega_0$, where the free vibration frequency ω_3 of the system is given by:

$$\omega_3^2 = (k_1+k_2)/(m_1+m_2) = \omega_0^2(\kappa_1+\kappa_2)/(\mu+1) \tag{7}$$

It is evident from Figure 3 that an optimal value ν_{opt} for the damping ratio ν does exist for both x_1 and x_2, leading to a minimum of the steady-state response under harmonic input. At ν_{opt} the behaviour is practically of a non-resonant type and the global response is very smooth over a large frequency range. The optimal value of ν can easily evaluated as the one which minimises the amplification factors $A_1 = |x_1/x_g|$ and $A_2 = |x_2/x_g|$ of transfer functions (5) and (6). It is important to observe that, when the condition $m_1/k_1 = m_2/k_2$ occurs, that is when $\mu = \kappa_1/\kappa_2$, then the subsystems (m_1, k_1) and (m_2, k_2) have the same natural vibration frequency. This results in dampers having no effect on the dynamic behaviour, as no relative displacement would occur under any base input.

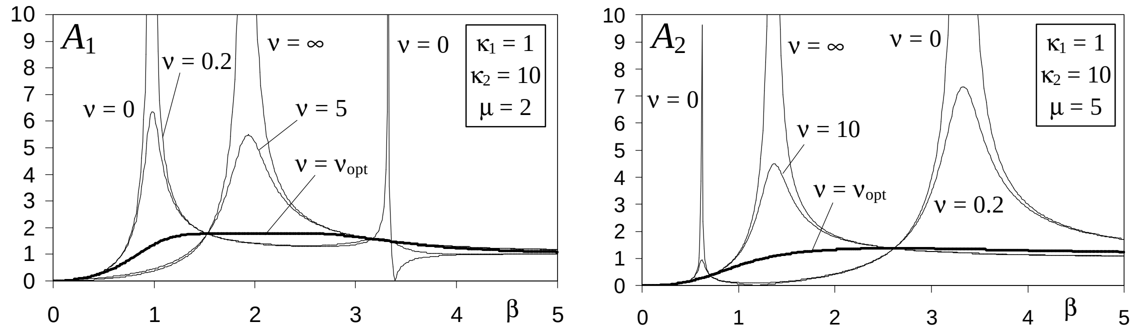


Figure 3 : Transfer function of x_1 and x_2 for several values of ν, κ_1, κ_2 and μ.

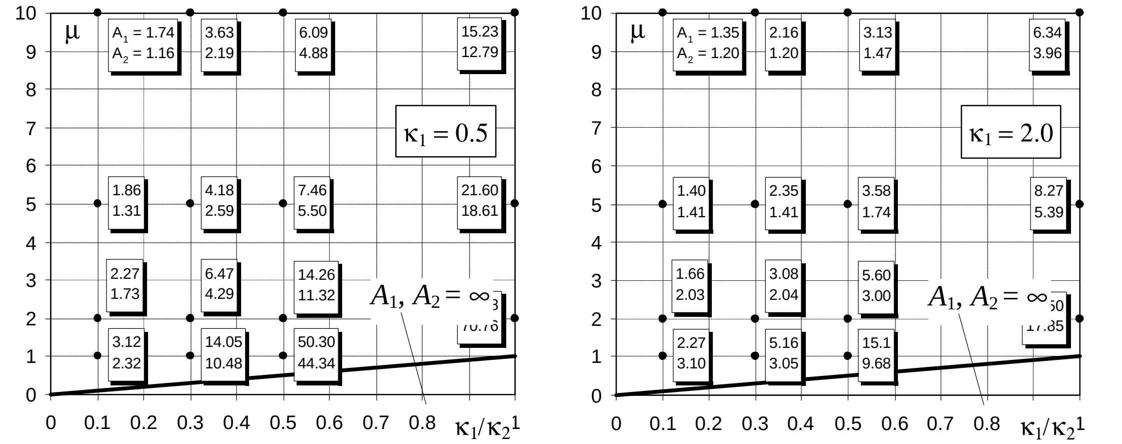


Figure 4 : Values of amplification factor A_1 and A_2 for the two-DOFs system when $\nu = \nu_{opt}$.

Figure 4 shows the values of A_1 and A_2 in the κ_1/κ_2 - μ plane for $\kappa_1 = 0.5$ and $\kappa_1 = 2.0$. The line $\mu = \kappa_1/\kappa_2$ is also plotted, corresponding to an undamped response ($A = \infty$) due to the complete ineffectiveness of damping devices. The corresponding values of ν_{opt} are shown in Figure 5 as a function of μ. Looking at Figure 4, it is possible to observe that the action of devices increases as long as μ , κ_1 and κ_2/κ_1 increase. As an example, for $\kappa_1 = 2.0$, $\kappa_1/\kappa_2 = 0.1$ and $\mu = 10$, the corresponding amplification values for $\nu = \nu_{opt}$ are $A_1 = 1.35$ and $A_2 = 1.20$, meaning values only slightly higher than unity, that is the value corresponding to $\beta = \infty$. This consideration highlights the appropriateness of the proposed techniques in case of long-bay buildings, where both μ and κ_1 ratios are usually much greater than 1 and κ_1/κ_2 is generally much lower than 0.1. It can be easily recognised that such schemes are very common in churches, royal palaces and other monumental constructions, due to lack of intermediate stiffening elements between end walls.

3 TIME-HISTORY PARAMETRIC ANALYSIS

3.1 General

The above outlined procedure allows to put into evidence the basic aspects of the structural behaviour, as well as the mutual relationship among involved variables. Nevertheless, it can not be directly used for design purposes or, generally speaking, in all cases when a quantitative sizing of dampers in necessary. This is a logical consequence of the inherent minimalism of the model, which can not take into account all the aspects of the very complex behaviour of the actual structure. A really accurate analysis, in fact, should give consideration to many aspects, both geometrical and mechanical, including, for example, the actual tri-dimensional structural configuration, the effect of higher order vibration modes, the non linear behaviour of masonry, the effect of global structural damping, the cyclic cumulate damage under seismic action, and so on. This is to say that, in general, a fully non linear FEM analysis would be necessary for a thorough understanding of the behaviour of such structures under seismic input. As well known, such an approach is very expensive in terms of computation time and is hardly within the range of common PCs. On the other hand, even if a fully non linear analysis could be affordable, the accurate evaluation of actual material properties would be in any case very difficult, in particular when facing historical buildings. For these reasons, a simplified FEM analysis has been performed in this context, the simplification basically consisting in assuming an indefinitely linear elastic behaviour for masonry walls. It is worth being emphasised that keeping masonry perfectly elastic does not lessen the meaning of the investigation if the evaluation of the structural response in presence of viscous and/or hysteretic devices is sought. On the contrary, it allows tohighlight the beneficial effect of dampers on the global performance of the structure, mainly consisting of a reduction of displacements under seismic action and, hence, of an increase of the magnitude of actions which can be absorbed in elastic range.

As being stated, in order to evaluate the effectiveness of the proposed technique when applied to practical cases, a set of dynamic analyses in the time domain has been performed by means of the computation code CSI-SAP 2000NL Version 7.1. This code allows to introduce dashpot elements (NLLinks) characterised by non linear damping or hysteretic properties and able to behave as energy dissipation devices. Because of its long duration and wide frequency content, the E-W Calitri recording of the italian Campano-Lucano earthquake (1980) scaled to PGA = 0.156g has been assumed as seismic input. The corresponding elastic response spectrum is plotted in Figure 6a for several values of the global structural damping ratio ν_0. This spectrum is characterised by a wide flat amplification region, so as to be a rather general seismic input for the evaluation of the optimal damping values of devices. The maximum values of the amplification factor fall in the period range from 0.2s to 1.0s, that is in the frequency range typical of the constructions under consideration, as shown in the following.

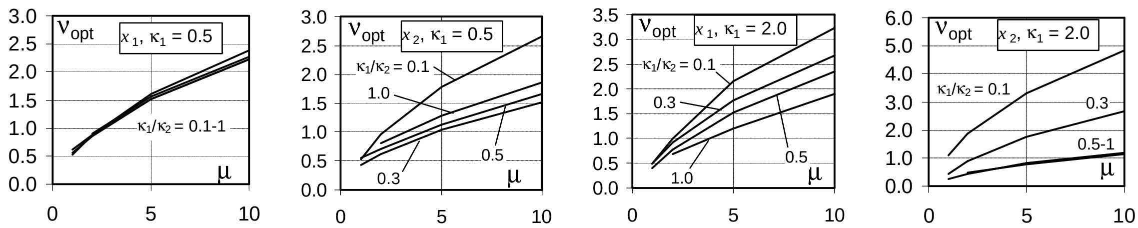
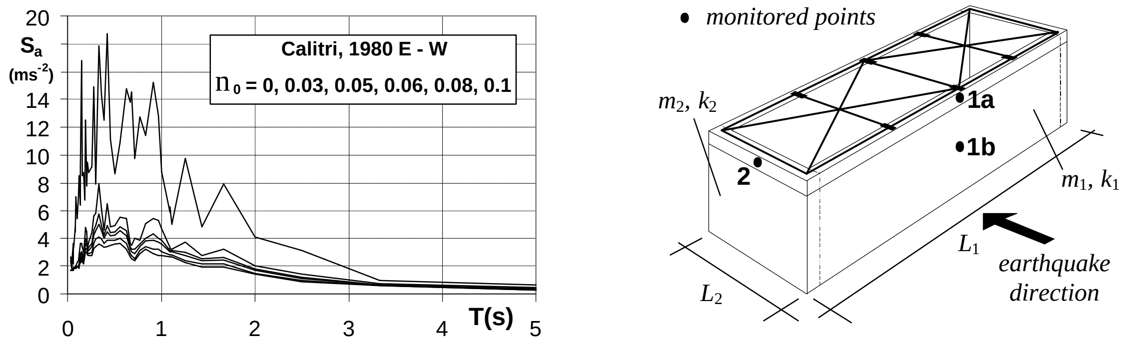


Figure 5 : Values of ν_{opt} for the two-DOFs system as a function of μ and κ_1/κ_2.

The geometry range considered for the time history analysis covers L_1/L_2 ratios equal to 2, 6 and 10, so as to consider practically all possible cases that can occur in practice. In particular, the transverse walls are 10m long and 12m high. Linear viscous devices, each of them having damping constant *c*, have been located, 4m spaced, between the top of both longitudinal walls and the rigid diaphragm (see Figure 2). In order to limit the number of active variables, the mass of roof structure, including the diaphragm, has been neglected. This assumption is quite acceptable when light-weight steel lattice structures are used for the top level, in which case the roof mass is generally less than 10% of the mass of long walls (Mazzolani and Mandara, 1994).

A constant wall thickness of 0.8m has been assumed for all walls in the construction, which is why four-node shell elements have been used throughout the structural model. Masonry having unit weight γ = 20kN/m^3 and elastic modulus E = 2kN/mm^2 has been considered. As already said, the behaviour of masonry wall has been assumed perfectly elastic. Nonetheless, an equivalent damping ratio ν_0 = 0.03 has been considered in order to take account of global structural damping properties. The influence of higher damping ratios (0.06 and 0.08) has been also evaluated in some cases. Furthermore, both cases of fully connected and unconnected wall angles have been studied. As underlined before, the case of unconnected walls is representative of the situation when longitudinal walls may have openings alongside the angle with transverse walls, or when they are stiffened at discrete points by counterforts, ribs or males. Also, such a configuration is the one which presumably occurs when a certain damage has been caused by seismic quakes. Masonry crossings, in fact, are usually very weak in historical buildings, unless additional strengthening operations of wall connections have been carried out



a) b)

Figure 6 : The elastic response spectrum of Calitri E – W recording (a) and the points in the structure monitored in the time history analysis (b).

Table 1 : Fundamental vibration periods for analysed cases.

L_1/L_2	2 Unconnected	2 Connected	6 Unconnected	6 Connected	10 Unconnected	10 Connected
T_1(s)	1.0754	0.4272	1.0925	0.9568	1.0664	1.0267

As a final step of the analysis, the effect of four plastic threshold devices placed on the top of each transverse wall has been investigated. The function of such devices is to limit the magnitude of forces applied to these walls, providing at the same time an extra amount of dissipated energy.

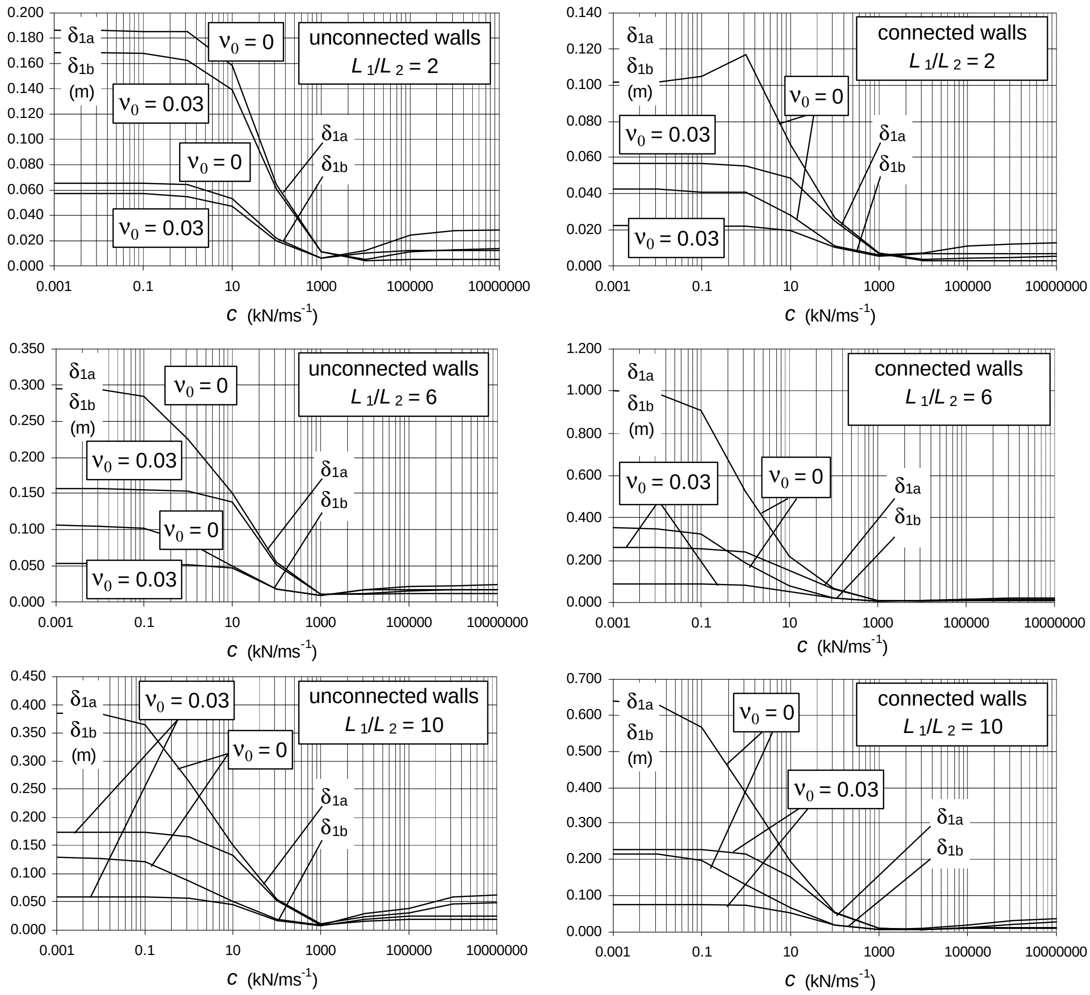


Figure 7 : Displacement at the top (δ_{1a}) and at mid-height (δ_{1b}) of longitudinal walls (Points 1a and 1b of Figure 6b).

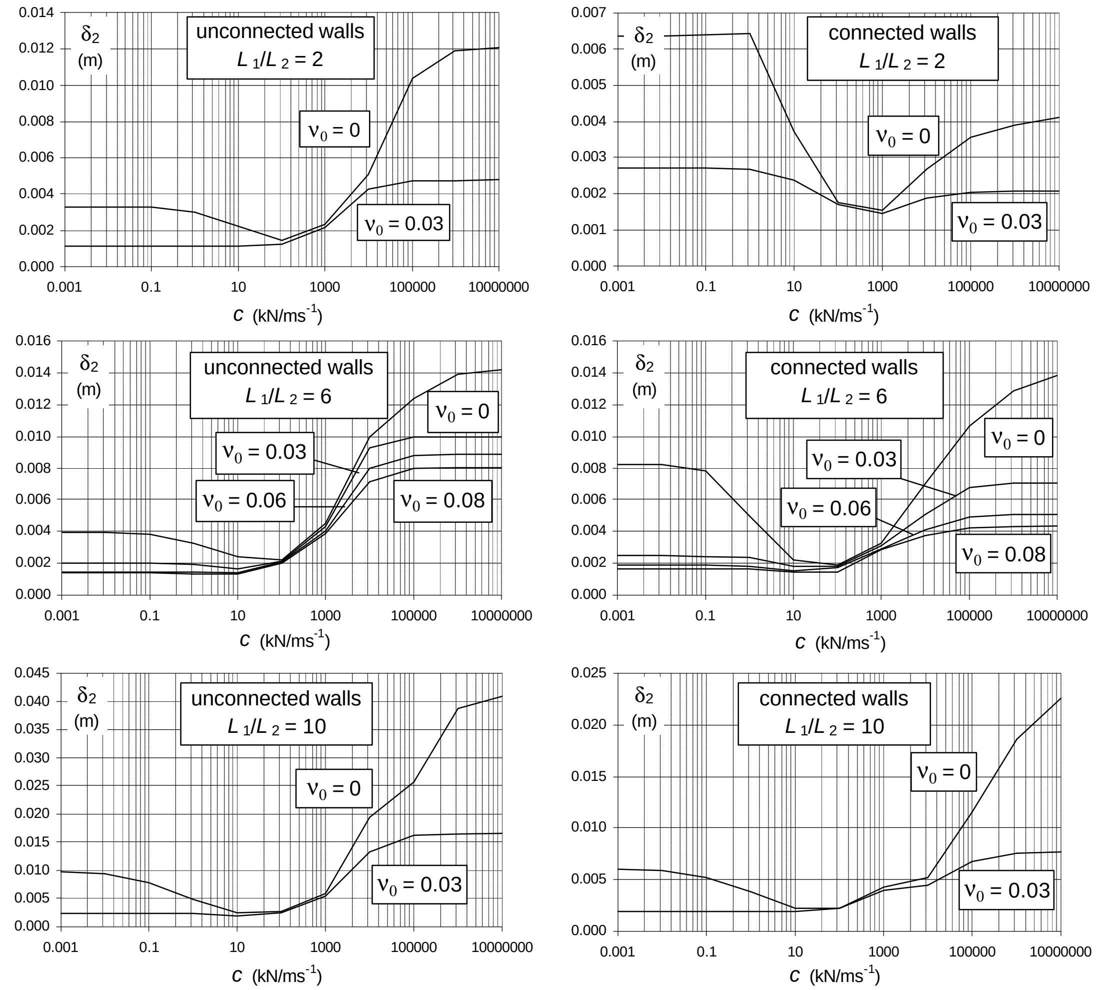


Figure 8 : Displacement at the top of transverse walls (Point 2 of Figure 6b).

3.2 Discussion of results

In order to have a complete view of the structural behaviour, the displacement at three points of the walls has been monitored in the analysis, according to the scheme of Figure 6b. Furthermore, the global energy dissipated by dampers has been also recorded. The fundamental vibration periods for analysed cases are given in Table 1. Note that the relevant period for L_1/L_2 = 6 and connected walls (T_1 = 0.9568s) corresponds to a peak of the response spectrum, which involves very high values of displacement at all monitored points. Such values are even greater than in case of unconnected walls. Both Figure 7 and 8 show that a minimum can be obtained for displacement at all monitored points. In particular, according to OVA, a rather small range of *c* values can be found for all examined cases, outside of which the magnitude of displacements strongly increase in both wall orders. This is even more evident when considering the influence of global structural modal damping (ν_0), leading in general to a reduction of displacements for all values of *c*, except for $c = c_{opt}$, in which case the structural displacement is independent of ν_0. The magnitude of displacement reduction is very significant, especially when ν_0 = 0. For ν_0 = 0.03, which represents a realistic damping ratio for actual masonry structures, the values of displacement reduction ratio at wall-top are shown in Table 2. Such ratio is referred to the maximum displacement values ($\delta_{1a,max}$, $\delta_{1b,max}$, $\delta_{2,max}$) achieved in case of pinned connection between the rigid diaphragm and the longitudinal walls. As previously supposed, the obtained values confirm that the effectiveness of this provision increases as long as the L_1/L_2 ratio increases. Of course, the greatest reduction is obtained for transverse walls, whose displcement values are in average knocked-down to 0.123 ÷ 0.240 of the corresponding values obtained with pinned diaphragm-to-wall connection. Furthermore, looking at Figure 9, where the global energy *E* dissipated in viscous dampers is plotted, it can be observed that the value of c_{opt} for transverse walls is generally very close to the one corresponding to the maximum value of dissipated energy, confirming that the beneficial effect of this technique is strictly dependent on the energy dissipation capabilities of devices. As far as longitudinal walls are concerned, the effect of dampers is less evident, but quite significant as well, in particular on the displacement at the wall mid-height in case of unconnected walls.

When the use of additional plastic threshold devices on the top of transverse walls is considered, the minimum values of displacement achieved with viscous dampers only do not change (Figure 10). In the spirit of PTA, the effect provided by additional devices is basically to reduce the response of transverse walls for values of c higher than c_{opt}, in which case an optimal value of the plastic threshold F_y of each damper can be found, which leads to a minimum of displacement values. In the case referred to in Figure 10(unconnected walls, $L_1/L_2 = 6$, $\nu_0 = 0$), $F_{y,opt} \cong 200$ kN. Below such value of F_y displacements on the top of longitudinal walls increase significantly, even when $c = c_{opt}$ (Figure 10, right), meaning that a lower protection is provided to the structure.

Table 2 : Values of displacement reduction ratio at wall-top for $\nu_0 = 0.03$.

L_1/L_2 $\nu_0 = 0.03$	2 Unconnected	2 Connected	6 Unconnected	6 Connected	10 Unconnected	10 Connected
$\delta_{1a}/\delta_{1a,max}$	0.889	0.968	0.802	0.771	0.536	0.562
$\delta_{1b}/\delta_{1b,max}$	0.486	0.839	0.488	0.504	0.338	0.544
$\delta_2/\delta_{2,max}$	0.240	0.708	0.169	0.251	0.123	0.246

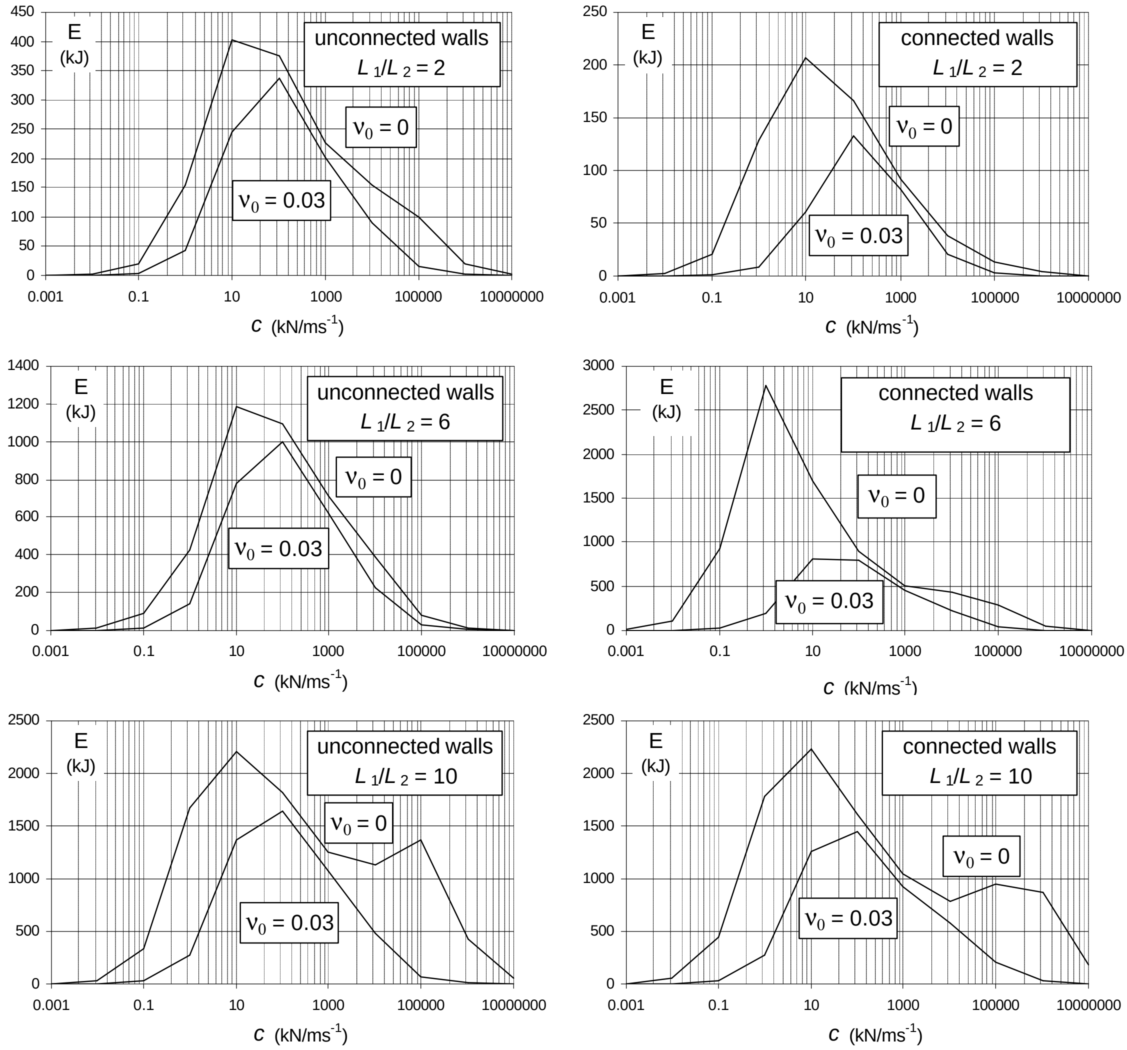


Figure 9 : Energy dissipated in viscous devices for analysed cases.

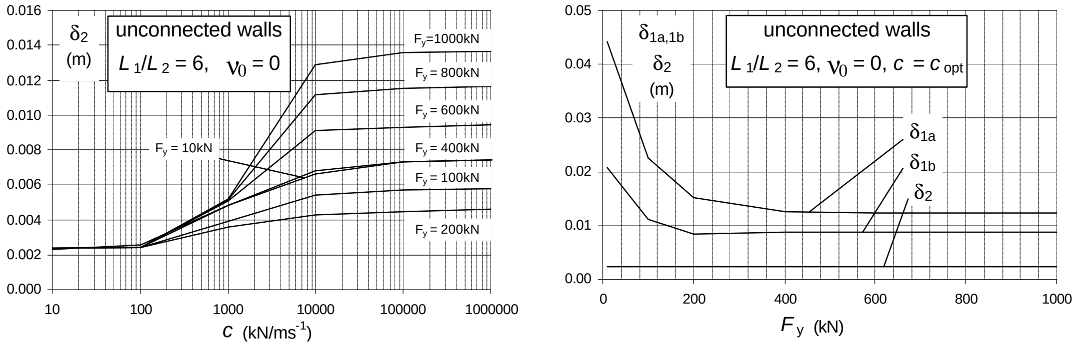


Figure 10 : Effect of additional plastic threshold devices on wall displacements.

4 CONCLUSIVE REMARKS

The above analysis has shown that the effect of energy dissipation due to dampers is basically to reduce the extent of displacements in the structure under dynamic input. In simple words, if a limit value is assumed for such displacements, e.g. corresponding to the upper limit of members elastic range, this means that the application of dissipative devices produces an increase of the maximum earthquake intensity which can be resisted by the structure without exceeding the elastic domain. Of course, this does not necessarily imply that code requirements about seismic retrofit are fully satisfied for the case under consideration. Nevertheless, in a more general view, if the minimisation of actions transmitted to the structure is assumed as a possible design criterion for dampers - according to either OVA or PTA outlined above - a rigorous seismic retrofitting can be achieved as well by increasing the structural resistance at elastic limit state by means of conventional strengthening operations. As the effect of dampers is to reduce significantly the magnitude of actions on the structure, the extent of these additional interventions can be rather limited - for instance simple prestressed ties into vertical walls - and this is very important when historical constructions are faced. If the achievement of full seismic retrofit is not mandatory, as usually accepted for monuments, then the sole application of energy dissipation devices can produce an effective, cheap and reversible upgrading of the structural safety under seismic actions.

ACKNOWLEDGEMENTS

The above activity has been developed within the *“Progetto Finalizzato Beni Culturali”*, issued by Italian C.N.R.. The helpful contribution of Eng. G. Acchiappati in the development of numerical analyses during the preparation of his diploma thesis is gratefully acknowledged.

REFERENCES

Croci G., Bonci A., Viskovic A. The use of shape memory alloys devices in the Basilica of St Francis in Assisi. In Proceedings of the *Final Workshop of ISTECH Project*, Ispra (Italy), 2000.

Di Marzo D., Mandara A., Serino G. Earthquake Protection of Buildings and Bridges with Viscous Energy Dissipation Devices. In: F.M. Mazzolani & R. Tremblay (eds), *Behaviour of Steel Structures in Seismic Areas STESSA 2000*. Balkema, Amsterdam, 2000.

Mazzolani F.M., Mandara A. Seismic upgrading of churches by means of dissipative devices. In F.M. Mazzolani & V. Gioncu (eds), *Behav. of Steel Struct. in Seismic Areas STESSA ’94*. E & FN SPON, London, 1994.

Mazzolani F.M., Serino G. Viscous energy dissipation devices for steel structures: modelling, analysis and application. In F.M. Mazzolani & H. Akiyama (eds), *Behaviour of Steel Structures in Seismic Areas STESSA ’97*. Ed. 10/17, Salerno (Italy), 1997.

Mazzolani F.M., Mandara A. Advanced metal systems in structural rehabilitation of monumental constructions. In Proc. of the The Int. Conf. on Struct. Eng., Mech. and Computation, Cape Town, 2000.

Mazzolani F.M. Passive control technologies for seismic resistant buildings in Europe. In Progress in Structural Engineering and Materials, John Wiley & Sons, London (Under press, 2001).