SEISMIC UP-GRADING OF MASONRY STRUCTURES BY STEEL BRACES AND DISSIPATIVE DEVICES

MIGLIORAMENTO SISMICO DI STRUTTURE MURARIE MEDIANTE CONTROVENTI IN ACCIAIO E DISPOSITIVI DISSIPATORI

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ABSTRACT

The combined application of steel strengthening structures and passive control techniques to masonry buildings is pointed out in this paper, addressing specific problems of this constructional category. Such techniques are based on the use of special viscous devices for achieving the best seismic performance, whereas the use of additional steel bracing structures is conceived in order to provide additional strength and energy dissipation capability. A comprehensive numerical analysis is carried out, in which the time response of masonry panels strengthened with steel eccentric bracings is investigated. For the best performance to be achieved, the steel structure must possess an elastic stiffness not lower than that of the masonry structure to which it is connected. A simplified design criterion for the evaluation of optimal device properties is also proposed and discussed.

SOMMARIO

Nella memoria viene trattata l'applicazione combinata di strutture metalliche di rinforzo e di tecniche di controllo passivo a strutture in muratura, mettendo in rilievo i problemi tipici di tale tipologia costruttiva. Le tecniche in parola si basano sull'impiego di dispositivi speciali per ottimizzare la prestazione strutturale sotto sisma, mentre l'uso di strutture aggiuntive in acciaio viene concepito allo scopo di ottenere ulteriore resistenza e dissipazione di energia. Lo studio viene condotto attraverso un'ampia analisi della risposta temporale di pannelli murari rinforzati con controventi in acciaio di tipo eccentrico. Si rileva che la migliore prestazione viene ottenuta quando la struttura metallica aggiuntiva possiede una rigidezza elastica non inferiore a quella della struttura muraria a cui è collegata. Nel lavoro viene anche proposto e discusso un criterio semplificato per la valutazione delle proprietà ottimali dei dispositivi.

1. INTRODUCTION

Seismic rehabilitation of existing masonry structures is an important topic in the field of building engineering, which is why it recalls more and more an increasing interest among engineers and researchers. As an alternative to traditional methods using cement and mortar injections or reinforced concrete sandwich panels jointed to the masonry walls, a different approach consisting of connecting steel bracing to the wall by means of a viscous damper is investigated herein. This technique combines the advantages of steel frames (namely lightness, stiffness, ductility, ease of assemblage and reversibility) with those of energy dissipation devices (EDD), allowing a reduction of the seismic impact by means of an additional energy dissipation. The potential of the braced frame-viscous damper combined system for seismic protection of building is widely proved and recognized: the increase of stiffness, in fact, improves the behavior of the masonry at the serviceability limit state; the dissipation of energy, provided it occurs in a controlled and stable way, improves the structural response under seismic events of strong intensity, hence at the ultimate limit state. For this reason, the use of passive energy control and dissipation techniques may represent an effective alternative to conventional strengthening operations in the seismic up-grading of existing buildings, as experienced in many applications carried out in the last years, including buildings belonging to the historical or monumental field (Mazzolani & Mandara 1994, Croci et al. 2000, Indirli 2000). The results is that not only an increase of the maximum earthquake magnitude tolerable by the structure is achieved, but also a limitation of conventional strengthening interventions, which turns to be very useful when buildings possess monumental features.

From the seismic point of view, the most relevant problem with masonry buildings is that they require some additional provisions in order to control the out-of-plane vibration of unbraced walls. This may happen, for example, in long-bay buildings, where a good behaviour under seismic actions can be impaired by the inherent deformability of horizontal diaphragm structures and/or by the excess of inertia forces transferred between longitudinal and transverse walls (Mazzolani *et al.*, 2003). When the magnitude of actions applied to transverse walls is too great, then such walls need to be properly strengthened. To this purpose, additional bracing structures can be provided, possibly fitted with EDDs in order to optimize the dynamic structural behavior. Steel frames can be applied to either intermediate or end walls, according to schemes shown in Figure 1. Mandara & Laezza (2002) have shown that a strong reduction of seismic response can be achieved in this case, with a simultaneous reduction of the damage in the masonry walls. Also, both inelastic features and energy dissipation capabilities of masonry walls, as well as hysteretic capabilities of steel brace, are effectively exploited.

The application of dissipative devices requires the preliminary evaluation of both optimal position and best devices properties. Because of their inherent constructional features, masonry buildings do offer many chances for the installation of dissipative devices and/or structures. Devices, in fact, can be installed at the connection between walls and floors, as well as paralleled to existing walls or, in a more general sense, where relative motion can occur. As far as position is concerned, the most appropriated position is where the displacements are maximum, that is, as a rule, on the top of masonry panels. The appropriate choice of the best mechanical characteristic of both steel bracing and

energy dissipation devices can then be made on the basis of damage minimization in the masonry structure.

All the above recalled aspects underline the necessity to use low-mass and hi-stiffness (or dissipative) structures as strengthening systems in masonry buildings. Such categories address steel structures as the most appropriate for obtaining the required bracing and/or dissipative effect on the masonry structure. Steel structures, in fact, offer lightness, stiffness and energy dissipation capabilities, and allow ease of erection and reversibility (Mazzolani & Mandara 2002a). Some studies carried out by the authors (Mandara & Mazzolani 2001a,b,c, Mazzolani & Mandara 2002b) have also shown that the combined use of bracing steel structures with viscous devices can yield a significant improvement of seismic performance of masonry buildings, in terms of both displacement and damage reduction. In this paper, the application of such techniques to a generic masonry panel is pointed out, with special emphasis to the problem of determining the optimal features of both devices and bracing system. Results show that a proper choice and sizing of both EDDs and bracing structure can lead to a global response optimization. This problem has been thoroughly investigated by Mandara & Laezza (2002), leading to some general conclusions on the most convenient range of brace-to-wall stiffness and strength ratios to be adopted in practice.
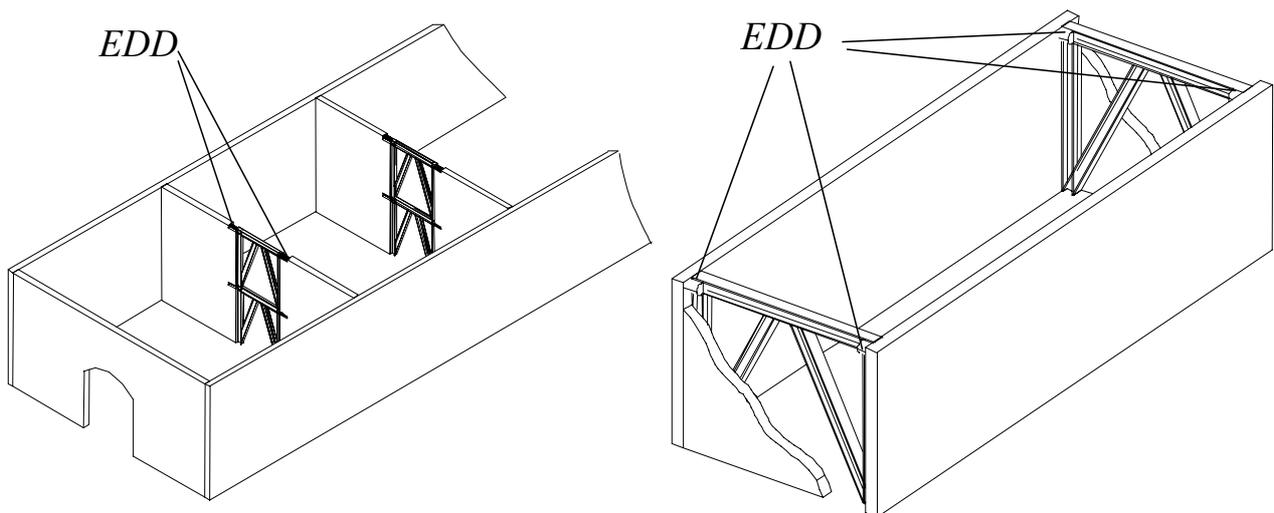
Figure 1 – Possible configurations of steel bracing elements and energy dissipation devices (EDD) in masonry buildings.

2. THE INVESTIGATED MODEL

2.1 General

The analysed scheme consists of a 6×5m masonry panel connected to a 36t heavy rigid diaphragm, according to the scheme of Figure 2a. The panel is paralleled with a steel braced frame, typically of EB type. A linear viscous damper with constant c is placed in series between the wall and the bracing, so that the equivalent plane scheme of Figure 2b is obtained. From the static point of view, the system can be characterized through the mass m_2/m_1 and stiffness k_2/k_1 ratios and, when the elastic limit is exceeded in either masonry wall or steel brace, by the resistance ratio F_{u2}/F_{u1}.
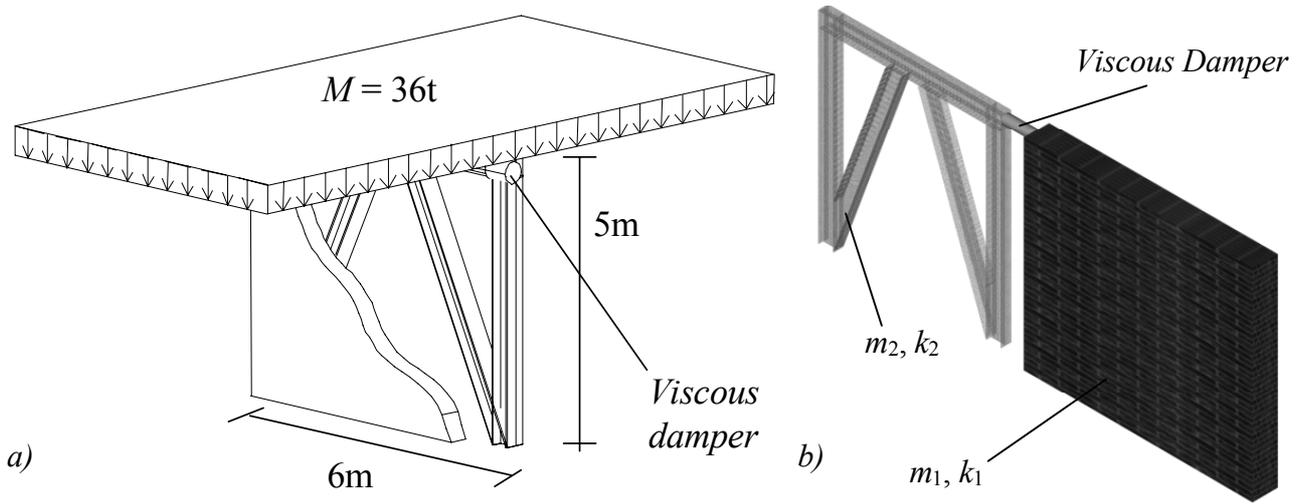
Figure 2. The structural model investigated (a) and its plane schematisation (b).

The non-linear dynamic code CANNY has been used for the time-history analysis of the above system (Li 1996). The CA7 model (*Canny Sophisticated Trilinear Model*) has been chosen in order to accurately interpret the fully non-linear behaviour of the masonry panel under cyclic actions. Seven parameters are used in such model to take into account the unloading stiffness (δ, θ), the strength deterioration (λ_e, λ_u, λ_3) and the pinching behavior (ε, λ_s) of the masonry (Figure 3), as described hereafter. Model parameters have been calibrated on the basis of the properties assumed for the masonry, as shown in the following. In order to consider the effect of the vertical load on the shear resistance of the masonry, the panel has been modeled through five horizontal layers, each of which has been characterized with a proper value of ultimate shear resistance, as a function of the acting compressive stress σ_0. The meaning of magnitudes in the CA7 model is illustrated in Figure 4.

2.2 The CA7 model for masonry panel

Stiffness degradation - The unloading curve following a new peak displacement is directed to a target point E (or E') depending on the parameter θ, as shown in Figure 3a. The instantaneous stiffness of unloading branches is given by:

$$K_U = \frac{\theta F_y + F_m}{\theta F_y / K_0 + d_m}$$ (1)

The unloading branch from an outside loop ends at an sloped axis UU', whose gradient axis is equal to δK_0, where K_0 is stiffness at the previous loop.

Strength deterioration - The model can represent the strength deterioration by directing the reloading curve towards a reduced strength level $\overline{F}_{max}$ at the same displacement corresponding to the previous peak strength F_{max}, as shown in Figure 3b. $\overline{F}_{max}$ is evaluated considering the attained ductility and the dissipated hysteresis energy according the following relationship:

$$\overline{F}_{max} = F_{max} \left(1 - \lambda_e \frac{E_h}{F_y d_{max} + F'_y d'_{max}} - \lambda_u \left(1 - \frac{1}{\mu} \right) \right)$$ (2)

(a) Unloading Stiffness   (b) Strength Deterioration   (c) Pinching Behavior
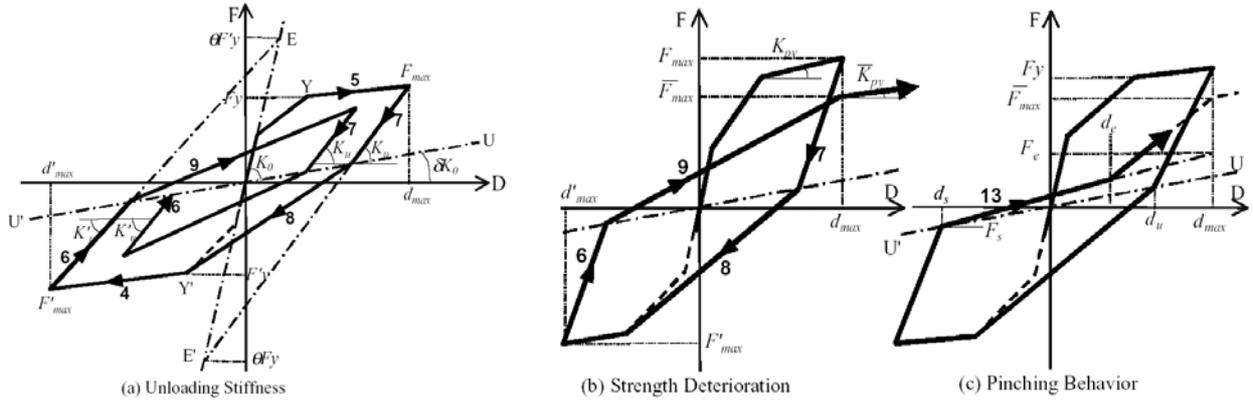
Figure 3. The CANNY material model assumed in the analysis for masonry (CA7) (Li 1996).

The softening effect is represented by a third parameter λ_3 that lowers the post-yield envelope:

$$\overline{K}_{py} = K_{py}\left(1 - \lambda_3\left(1 - \frac{1}{\mu}\right)\right) \quad with \quad \overline{K}_{py} \geq 0.01 K_{py} \tag{3}$$

<u>Pinching behavior</u> – The pinching behavior simulates the opening and closing of cracks. Target point (F_e, d_e) controls the slip branches, as shown in Figure 3c.

$$\begin{cases} d_e = \varepsilon d_u \\ F_e = \lambda_s\left(\overline{F}_{max} - \delta \cdot K_0 \cdot d_{max}\right) + \delta \cdot K_0 \cdot d_{max} \end{cases} \tag{4}$$

Input: **A(α α')**

$\alpha = K_y^t/K_e$

$\alpha' = K'^t_y/K_e$

K_y^s (Secant stiffness)

(Secant stiffness)K'^s_y

Or, input: **a(α α')**

$\alpha = K_y^s/K_e$

$\alpha' = K'^s_y/K_e$

$$F_y = F'_y = A \cdot \tau_u = b \cdot s \cdot \tau_k\sqrt{1 + \frac{\sigma_0}{1.5\tau_k}} \quad ; F_c = F'_c = \xi F_y$$

$$K_e = \frac{G \cdot A}{1.2h}\frac{1}{1 + \frac{1}{1.2}\frac{G}{E}\left(\frac{h}{b}\right)^2} \quad ; \qquad K^t_y = \alpha \cdot K_e$$

$$\delta_{0c} = \frac{F_c}{K_e} \quad ; \qquad \delta_{0y} = \delta_{0c} + \frac{F_y - F_c}{K^t_y} ; \quad \delta_u = \mu \cdot \delta_{0y}$$

where $E/G = 6$ and $G = 1100 \, \tau_k$.
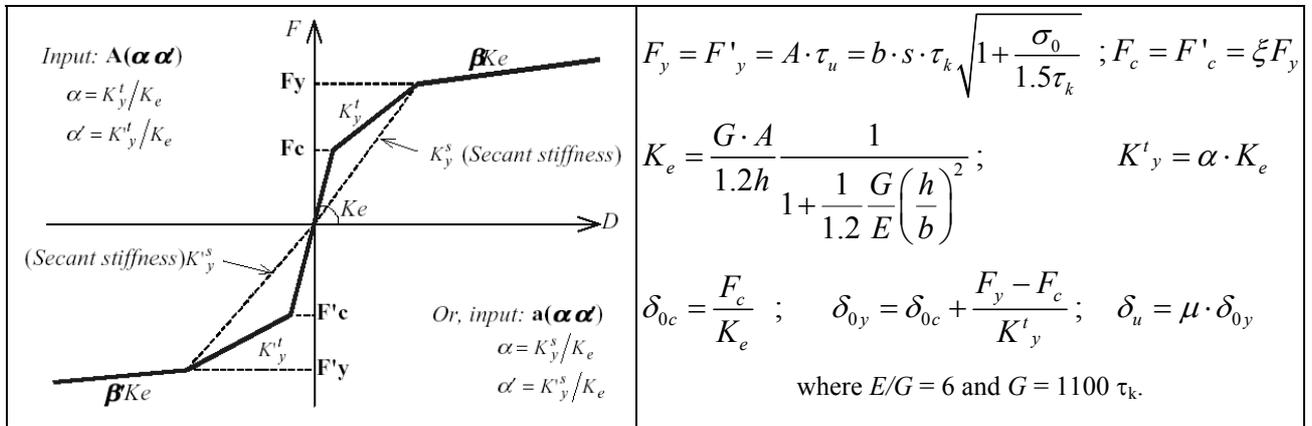
Figure 4 – The skeleton curve of CA7 model with explanation of symbols and magnitudes introduced.

2.5 The BL2 model for steel bracing

Eccentric bracing has been chosen for the analysis because it represents a good compromise between the stiffness of CBF systems and the ductility of MRF systems. The analysis, design and performance of eccentrically braced steel frames have been extensively studied by many researchers, confirming that well-designed links provide a steady source of energy dissipation, with wide and stable hysteresis cycle. The plastic deformations of the link can be due either to shear (short link) or to bending (long links), or to both of them. The link can be designed in such a way to obtain the required horizontal stiffness and ultimate strength. From the point of view of energy dissipation, short links offer the best performance.

According to Engelhardt and Popov (1989a,b), the link is considered short if its length e is shorter than e^*, given by:

$$e^* = 2\frac{M_u}{V_u} = 2\frac{1.2 \cdot M_p}{1.5 \cdot V_p} = 1.6\frac{M_p}{V_p} \tag{5}$$

in which due account has been made of material hardening in the evaluation of ultimate shear V_u and moment M_u of the link on the basis of corresponding first yielding values V_p and M_p.

The global behaviour of the brace has been modeled by means of Canny BL2 model, with an elastic-plastic relationship which closely reproduces the stable hysteresis response of such structures, as shown in Figure 5.
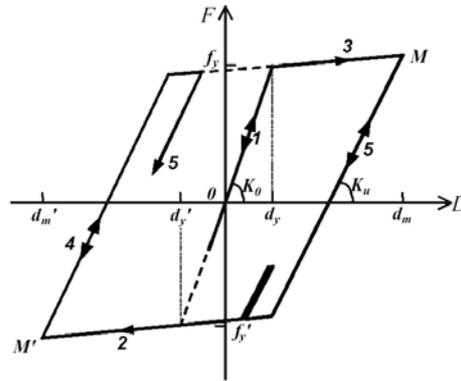
Figure 5 - The CANNY material model assumed in the analysis for steel brace (BL2) (Li 1996).

Model BL2 has a bilinear skeleton curve. The stiffness degradation is independent of the sign of applied force and reloading curve follows the unloading branch without stiffness change. Yielding may occur again before the displacement changes sign. The model is characterized by the only parameter γ, which rules the slope of the unloading curve:

$$K_u = K_0\left(\frac{d_y - d'_y}{d_m - d'_m}\right)^{\gamma} \tag{6}$$

In the case under consideration, according to the results of numerous tests referred to in literature, no stiffness degradation has been assumed for the model and hence $\gamma = 0$.

3. DESCRIPTION OF THE ANALYSIS

Material properties under consideration are listed in Table 1. Three earthquake recordings have been considered, namely El Centro (1940), Taiwan (1999) and Calitri (1980), all of them scaled at a PGA equal to 0.40g. This value corresponds to the maximum conventional design value set by Italian seismic code for the equivalent static analysis of masonry buildings. Further details on the research are reported in Mandara & Laezza (2002). Results of the analysis, referring to Calitri earthquake recording only, are summarized in Figures 6 and 7. Results are presented as a function of the viscous constant c, so as to put into evidence its influence on the global structural behaviour.

The inelastic response of the panel is represented by means of the following damage parameters:

$$DDI = \frac{\delta - \delta_{min}}{\delta_u - \delta_{min}} \qquad \text{Deformation Damage index;}$$

$$SDI = \frac{F - F_{min}}{F_u - F_{min}} \qquad \text{Shear Damage Index,}$$

with the following meaning of the symbols:

δ in-plane displacement of the wall;

δ_{min} minimum in-plane wall displacement value across the entire earthquake time-history;

δ_u ultimate in-plane displacement of the wall;

F in-plane shear force in the wall;

F_{min} minimum in-plane shear force in the wall across the entire earthquake time-history;

F_u ultimate in-plane shear force of the wall.

Because of its inherent definition DDI can also result greater than unity, whereas SDI $\in$ [0, 1]. When maximum values of δ and F across the entire earthquake time-history are considered, such indexes give a global representation of wall damage in terms of displacement and in-plane shear, respectively.

Table 1 - Properties assumed in the analysis for the masonry panel.

High-strength masonry	Wall thickness (mm)	τ_k (N/mm^2)	σ_k (N/mm^2)	G (N/mm^2)	E (N/mm^2)	ξ	α	μ	δ_{0c}	δ_{0y}	δ_u
	400	0.1	2.5	110	660	0.85	0.8	2.5	7.08	8.64	20.75
Low-strength masonry	Wall thickness (mm)	τ_k (N/mm^2)	σ_k (N/mm^2)	G (N/mm^2)	E (N/mm^2)	ξ	α	μ	δ_{0c}	δ_{0y}	δ_u
	500	0.04	1.5	55	330	0.80	0.8	2.0	7.83	10.28	20.55
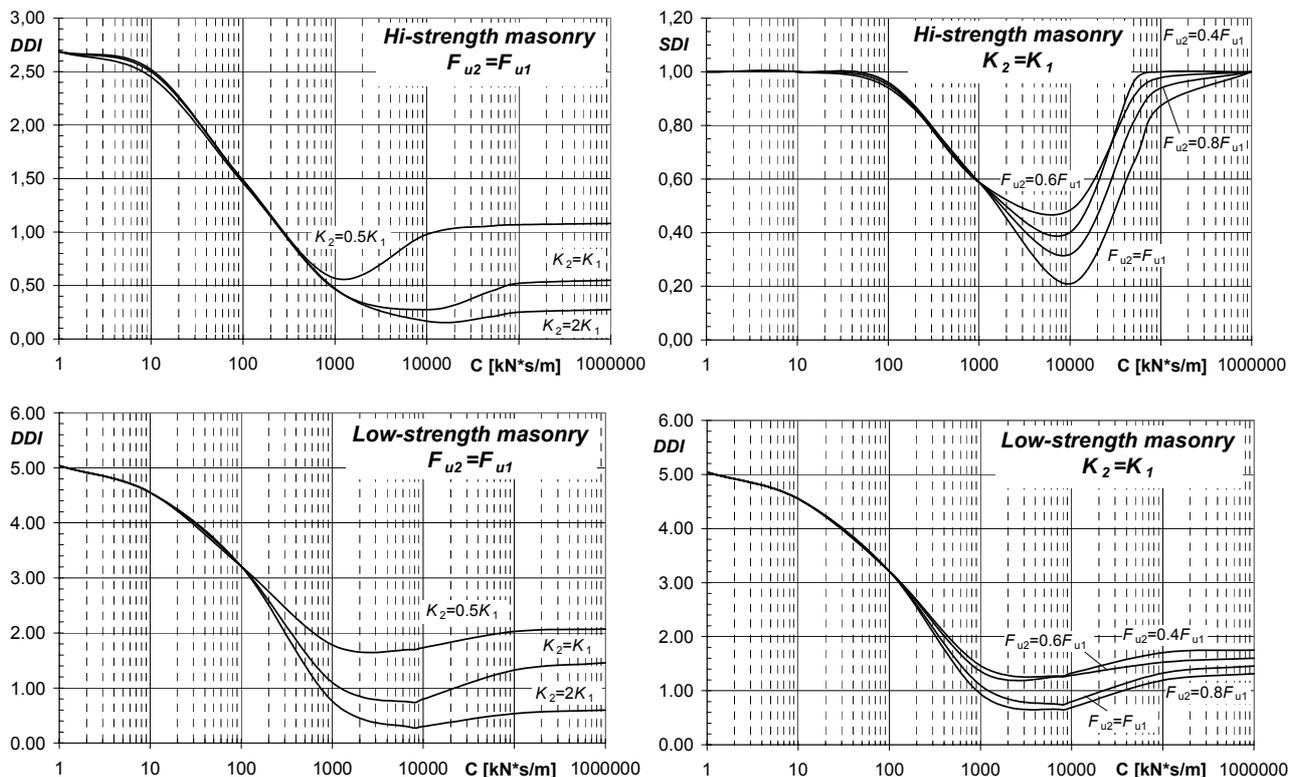
Figure 6 - Values of *DDI* and *SDI* as a function of the device viscous constant *c*.
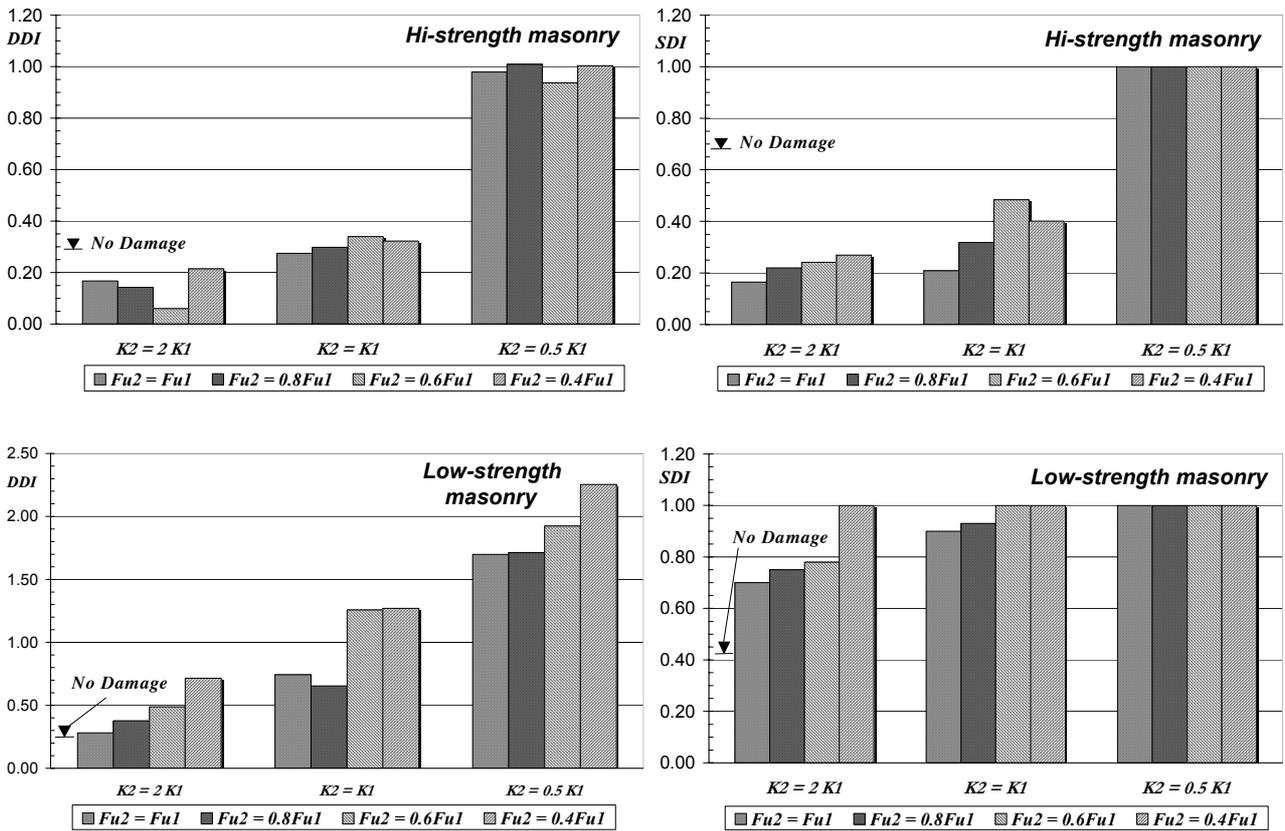
Figure 7 - Values of *DDI* and *SDI* for $c = c_{opt}$ compared with the relevant damage threshold.
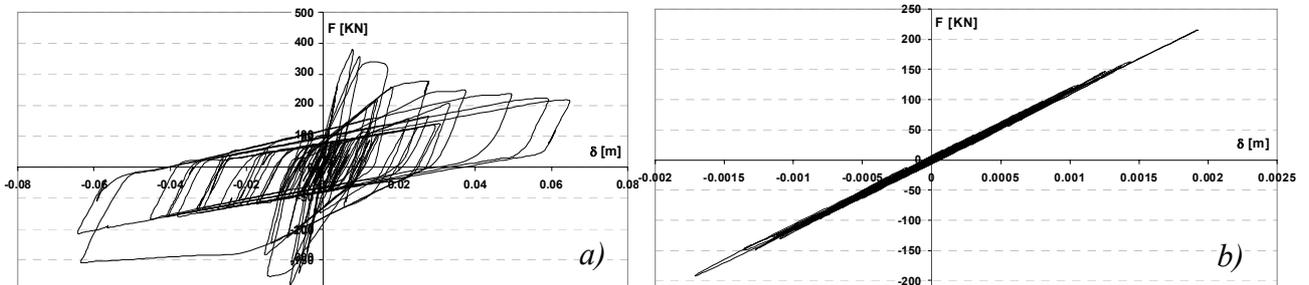
Figure 8 - Force-displacements hysteresis cycles under seismic action for hi-strength masonry panel without bracing (*a*) and with bracing and $c = c_{opt}$ (*b*).

From the analysis of Figures 6 and 7 it is possible to evaluate the beneficial effect of combined dampers and additional steel bracing on the global behaviour of the structure. For $c = c_{opt}$ the structural response is generally quite reduced as compared to the case of fixed connection between masonry panel and steel frames. Figure 8, illustrating one of the cyclic hysteresis diagrams of the hi-strength masonry panel, shows a visual example of the response improvement involved by added damped bracings. For values of the device viscous constant within the optimal range, the response reduction in terms of both shear force and displacement is very noticeable and may lead, particularly in the case of hi-strength masonry, to the complete absence of damage in the panel for the seismic input considered. From Figures 6 and 7 it is also clear that the best effectiveness of this provision is achieved when the steel bracing stiffness is higher than the panel in-plane stiffness. In general adopting $k_2 \geq 2k_1$ would be the best choice, but also $k_2 = k_1$ yields good results in terms of

response reduction. In some cases, e.g. very stiff masonry panels, this would require in turn very stiff braces, which is why this provision is mostly useful in long buildings without top and transverse diaphragms. In this case the stiffness ratio to be assumed is the one between the steel brace and the longitudinal wall in the out-of-plane direction (see for example Figure 1).

Notwithstanding, the ultimate shear force of steel bracing system can be also lower than that of the panel. Results, in fact, are not so strongly dependent on the F_{u2}/F_{u1} ratio. Hence, keeping $F_{u2} = 0.6 \div 0.8 F_{u1}$ would ensure results quite similar to those achieved when $F_{u2} = F_{u1}$, but with an additional amount of dissipated energy. This effect is similar to that pointed out in Mandara & Mazzolani, 2001b and Mazzolani $et\ al.$, 2003, about the use of additional plastic threshold devices on the building transverse walls. Likewise, the main favourable effect of adding elements with plastic threshold and stable hysteresis behaviour is that the range of optimal values of the viscous constant c is significantly enlarged.
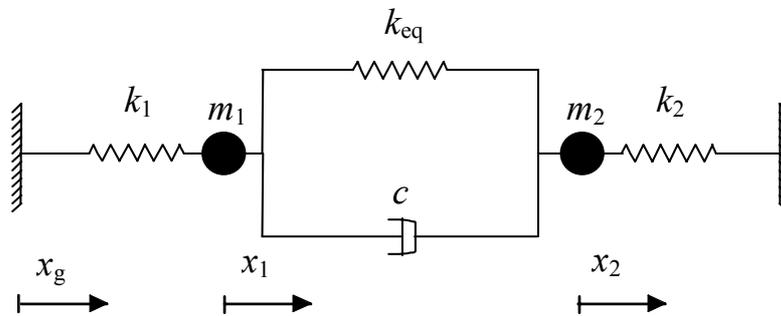
Figure 9 - Simplified 2-DOF model considered for the definition of the design method.

4. SIMPLIFIED DESIGN PROCEDURE

Systems like those dealt with before can be interpreted by means of the simplified 2DOF-model illustrated in Figure 9 (Mandara & Mazzolani 2001b, Mazzolani $et\ al.$, 2003). From the mechanical point of view, the system relies on the existence, within the same structure, of two subsystems characterized by strongly different dynamic properties. The insertion of EDDs between such parts and, hence, the exploitation of differential motion, allows the required structural damping and energy dissipation. Subsystems k_1, m_1 and k_2, m_2 may represent, for example, longitudinal and transverse walls (Figure 1), or the masonry panel and the steel bracing (Figure 2), respectively. As emphasized in Mandara & Mazzolani (2001b), the system is effective as long as $k_1/m_1 \neq k_2/m_2$. This model can be profitably used as a simplified design tool for a first estimation of the optimal value c_{opt} to be given to the device viscous constant. As the optimal range of this magnitude is quite large, even a relatively rough estimation of its value will be acceptable for design purposes.

The equations of motion for the system illustrated in Figure 9, written in matrix form, are:

$$\begin{bmatrix} m_1 & 0 \\ 0 & m_2 \end{bmatrix} \cdot \begin{Bmatrix} \ddot{X}_1 \\ \ddot{X}_2 \end{Bmatrix} + \begin{bmatrix} c & -c \\ -c & c \end{bmatrix} \cdot \begin{Bmatrix} \dot{X}_1 \\ \dot{X}_2 \end{Bmatrix} + \begin{bmatrix} K_1 + K_{eq} & -K_{eq} \\ -K_{eq} & K_2 + K_{eq} \end{bmatrix} \cdot \begin{Bmatrix} X_1 \\ X_2 \end{Bmatrix} = -\begin{Bmatrix} m_1 \\ m_2 \end{Bmatrix} \cdot \ddot{X}g \tag{7}$$

By defining the following non-dimensional parameters:

$$\omega_0^2 = \frac{K_2}{m_2}; \quad k = \frac{K_1}{K_2}; \quad \alpha = \frac{K_{eq}}{K_2}; \quad v = \frac{c}{\sqrt{K_2 m_2}}; \quad \mu = \frac{m_1}{m_2};$$

and considering a harmonic input:

$$Xg(t) = Xg(\overline{\omega}) \cdot e^{i\overline{\omega}t} \tag{8}$$

the transfer functions for x_1 and x_2 can be found:

$$\frac{x_1(\overline{\omega})}{x_g(\overline{\omega})} = \frac{\left[\mu\beta^2 - \mu\beta^4 + \alpha\beta^2(\mu+1)\right] + j\left[v\beta^3(\mu+1)\right]}{\left[\mu\beta^4 - \mu\beta^2 + (\kappa+\alpha) - (\kappa+\alpha)\beta^2 + \alpha(\kappa - \mu\beta^2)\right] + j\left[v(\beta - \beta^3 + \kappa\beta - \mu\beta^3)\right]} \tag{9}$$

$$\frac{x_2(\overline{\omega})}{x_g(\overline{\omega})} = \frac{\left[\kappa\beta^2 - \mu\beta^4 + \alpha\beta^2(\mu+1)\right] + j\left[v\beta^3(\mu+1)\right]}{\left[\mu\beta^4 - \mu\beta^2 + \alpha(1-\beta^2) + (1+\alpha)(\kappa - \mu\beta^2)\right] + j\left[v(\beta - \beta^3 + \kappa\beta - \mu\beta^3)\right]} \tag{10}$$

where the position $\beta = \overline{\omega}/\omega_0$ has been made. From the transfer functions the optimal value v_{opt} can be found as a function of $\kappa = k_1/k_2$ and $\mu = m_1/m_2$ for both elements 1 and 2. Some values of v_{opt} are plotted in the curves of Figure 10 for $\alpha = 0$ and several values of κ and μ.
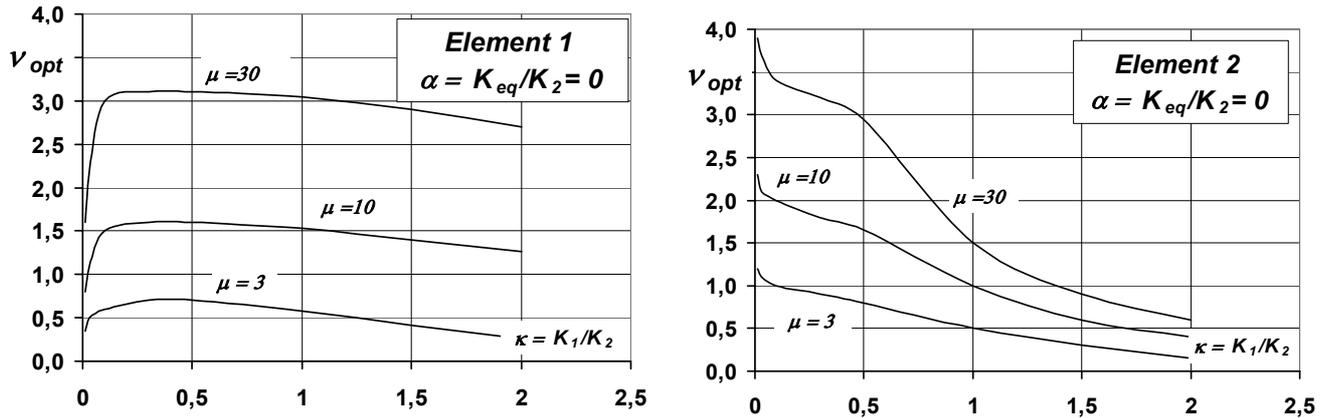
Figure 10 - Values of v_{opt} for Element 1 (masonry panel) and Element 2 (steel bracing) obtained from transfer functions.

5. CONCLUSIONS

The research work reported in this paper aimed at investigating the possibility to conveniently use strengthening steel structures and passive control techniques in the seismic up-grading of masonry buildings. As widely demonstrated herein and elsewhere, the effect of energy dissipation due to dampers is basically to reduce the extent of displacements in the structure under dynamic input. When earthquake would cause the structure to exceed the elastic range, such reduction of displacements involves in turn a reduction of damage in the masonry walls. On the other hand, this also means that the application of dissipative devices produces an increase of the maximum earthquake intensity which can be resisted by the structure without damage. This has been clearly shown in the inelastic dynamic analysis described in the paper, where the combined use of steel bracing elements

and optimised viscous devices has led to obtain a strong reduction of the global structural response under seismic action. In most cases, and particularly in case of masonry with good mechanical properties, the beneficial effect of this provision is such to prevent the structure from severe damage.

Also, the analysis has shown that an important role is played by the steel bracing system. Steel structures, in fact, in particular when are implemented in the form of EBF, do possess both stiffness and energy dissipation capabilities. In this case, both stiffness and ultimate strength can be easily adjusted by sizing the bracing shear link accordingly. Results have shown that, in order to get the best effectiveness on the global structural response, additional bracing must have a stiffness equal to or greater than that of the masonry panel which it is connected to. This would lead, in some cases, to the use of concentrically braced frames. Notwithstanding, the brace can have a lower (by a factor of 0.6÷0.8) ultimate shear resistance than the masonry panel.

In conclusion, the use of integrative steel structures fitted with energy dissipation devices can yield a significant improvement of the seismic behaviour of masonry buildings. Such improvement is generally much greater than that achievable by means of conventional stiffening and/or strengthening techniques and is generally obtained with a limited addition of masses and structural elements. In addition, depending on the building constructional features, both additional bracing structures and energy dissipation devices can be arranged in a great number of ways, all of them leading to a significant reduction of the response under seismic actions. The fine adjustment of both steel bracing structure and energy dissipation system, in the end, will allow to get the highest impact on the seismic performance of the building.

ACKNOWLEDGEMENTS

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