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DESIGN CRITERIA FOR THE USE OF SPECIAL DEVICES IN THE SEISMIC PROTECTION OF MASONRY STRUCTURES

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ABSTRACT

The combined application of steel strengthening structures and passive control techniques to masonry buildings is investigated in this paper, with particular emphasis for design problems. Techniques under consideration are based on the use of special viscous devices for achieving the best seismic performance. The use of additional steel bracing structures is considered 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 examined through the evaluation of a displacement-based damage parameter. The structural response is then evaluated in the light of performance requirements issued by FEMA. Provided the additional steel structure possesses an elastic stiffness not lower than that of the masonry structure to which it is connected, the proper choice of viscous damper properties allows to achieve a damage-free performance under design earthquake.

1. INTRODUCTION

Seismic design or upgrading of buildings according to pre-defined performance levels is nowadays a commonly adopted practice among engineers. As a matter of fact, most of existing codes refer to the "Performance Based Design" as a general framework for global design purposes, including seismic design or rehabilitation. The building performance is first of all to be related to the global safety for occupants during earthquake, even though aspects such as cost and feasibility of building restoration, reparation time, global economic and social impact on the larger community, also play a very important role in defining the overall performance requirements of a construction subjected to seismic actions (FEMA 356, 2000).

In case of existing masonry buildings to be seismically retrofitted, satisfying requirements for a given performance level can be relatively easy when additional strengthening elements and/or procedures can be applied to the structure, in such a way to enhance its overall resistance. Nevertheless, when higher performance levels are required, or when the applicability of conventional strengthening techniques is not straightforward, the use of passive protection systems may turn to be useful for protection purposes. As the application of such systems relies on energy dissipation devices (EDD) aimed at reducing the extent of member displacement, their effect is basically to increase the no-damage limit of the structure, that is the magnitude of earthquake actions which can be resisted without exceeding the elastic limit. In order to achieve the best performance, however, the passive system should be properly optimised by means of a suitable choice of its parameters (Mandara & Mazzolani, 2001a,b, Mazzolani & Mandara, 2002).

In this paper, an application of the "Performance Based Design" approach to masonry walls strengthened by means of additional steel frames and viscous devices is presented, according to the rules set out in FEMA 356 Seismic Rehabilitation Prestandard (2000). As each performance level is usually associated to a given extent of damage that would be tolerated by the building, a preliminary classification of possible damage levels has been defined in terms of maximum allowed in-plane displacement for the wall. For each of the defined performance levels, the corresponding properties of the viscous dissipative device have then been evaluated by means of a time-history analysis. The maximum performance improvement achievable by means of this protection systems has also been highlighted, by providing some hints for a design approach to the problem.

2. THE INVESTIGATED MODEL

Analysis dealt with a 6×5m masonry panel connected to a 36t heavy rigid diaphragm, according to the scheme of Figure 1a. This model is useful to represent buildings where strengthening bracings are inserted at or between end walls (Figure 2). The panel is connected to a steel braced frame, typically of EB type, by means of a linear viscous damper with constant c, so to give place to the equivalent plane scheme of Figure 1b. The system is 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} (Mandara & Laezza, 2002).

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 with a good degree of accuracy (Figure 3).

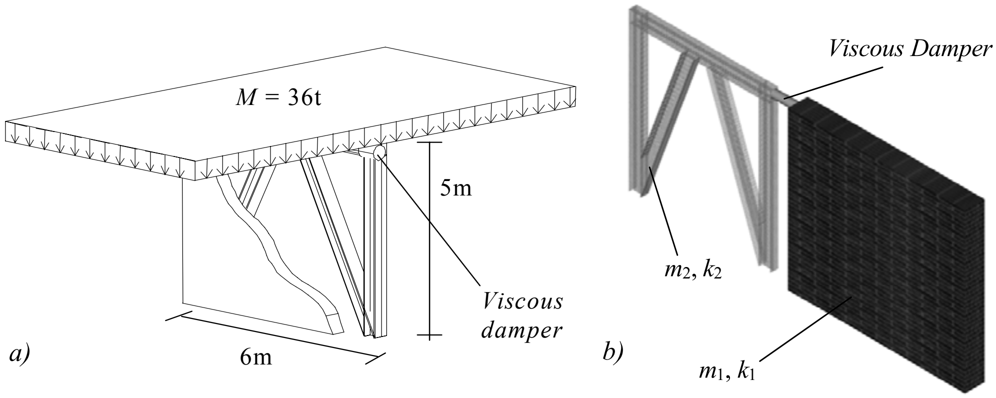


Figure 1. The model investigated in the analysis (a) and its plane schematization (b) .

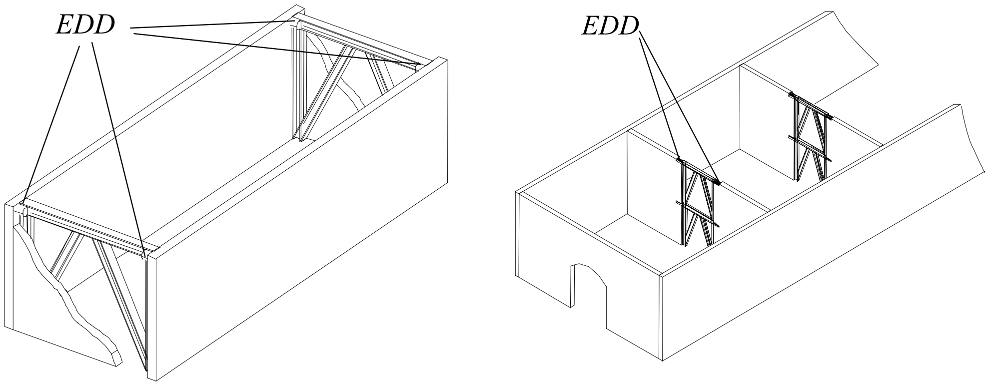
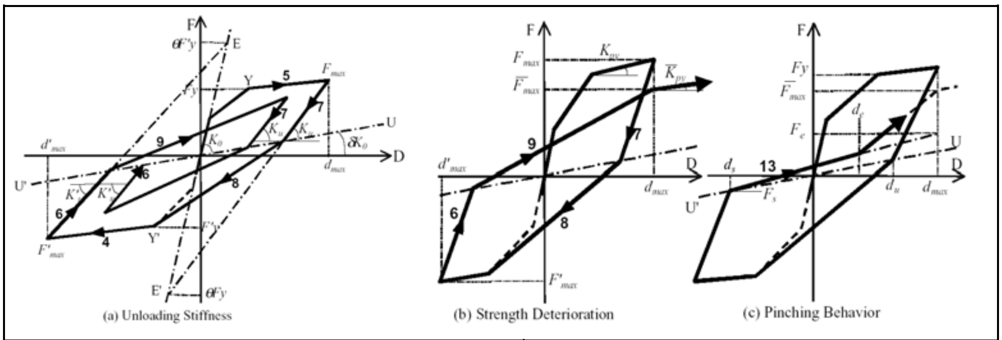
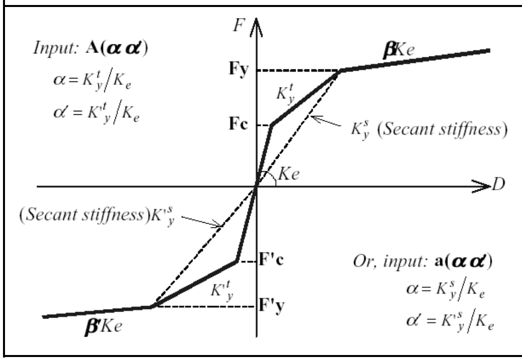


Figure 2. Possible locations of steel strengthening bracings and EDDs in masonry buildings





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

$$F_c = F'_c = \xi F_y\,;$$

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

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

Figure 3. The CANNY CA7 material model assumed for masonry (Li, 1996).

Eccentric bracing has been chosen as it provides a steady source of energy dissipation, with wide and stable hysteretic cycle. The plastic deformations of the link can be due either to shear (short links) or to bending (long links). The link can be designed in such a way to obtain the required horizontal stiffness and ultimate strength. 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 hysteretic response of such structures, as shown in Figure 4. Model BL2 has a bilinear skeleton curve. The stiffness degradation is independent of the sign of applied force and reloading follows through the unloading without stiffness change. Yielding may occur again before the displacement changes sign. For further details on both material models see also Mandara & Laezza, 2002 and Mazzolani *et al.*, 2003.

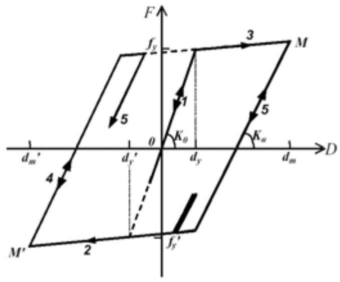


Figure 4. The CANNY BL2 material model assumed for steel brace (Li 1996).

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

A – Low-strength masonry

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	*Wall thickness* (mm)
0.02	0.5	22	132	0.70	0.7	1.5	9.20	14.83	22.24	600

B – Mid-strength masonry

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	*Wall thickness* (mm)
0.04	1.5	55	330	0.80	0.8	2.0	7.83	10.28	20.55	500

C – High-strength masonry

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	*Wall thickness* (mm)
0.1	2.5	110	660	0.85	0.8	2.5	7.08	8.64	20.75	400

3. THE PARAMETRIC ANALYSIS

Three masonry types have been considered, whose properties are listed in Table 1. To each of them, three earthquake recordings have been applied, namely El Centro (1940), Taiwan (1999) and Calitri (1980), scaled at a PGA equal to 0.40g, which corresponds to the maximum conventional design value set by the Italian seismic code for the equivalent static analysis of masonry buildings. Results of the analysis, summarized in Figures 5 to 12, are presented as a function of the device viscous constant *c*, so as to put into evidence its influence on the global structural behavior. The inelastic response of the masonry panel is represented by means of the following displacement-based damage parameter *DDI* (Deformation Damage Index):

$$DDI = \frac{\delta - \delta_{\min}}{\delta_u - \delta_{\min}}$$

where δ is the in-plane displacement of the wall, $\delta_{\min}$ is the minimum in-plane wall displacement value across the entire earthquake time-history and δ_u is the ultimate in-plane displacement of the wall. Note that, because of its inherent definition *DDI* can also results greater than unity. Values of *DDI* > 1 mean that the ultimate displacement has been exceeded and, therefore, collapse has been attained.

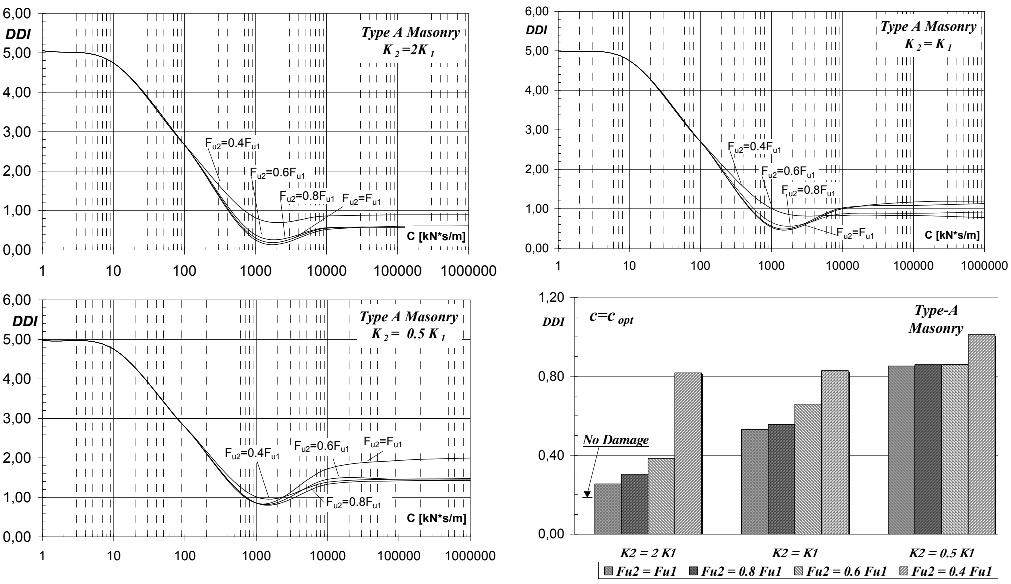


Figure 5. Type-A masonry: values of DDI as a function of viscous constant *c* (Calitri earthquake).

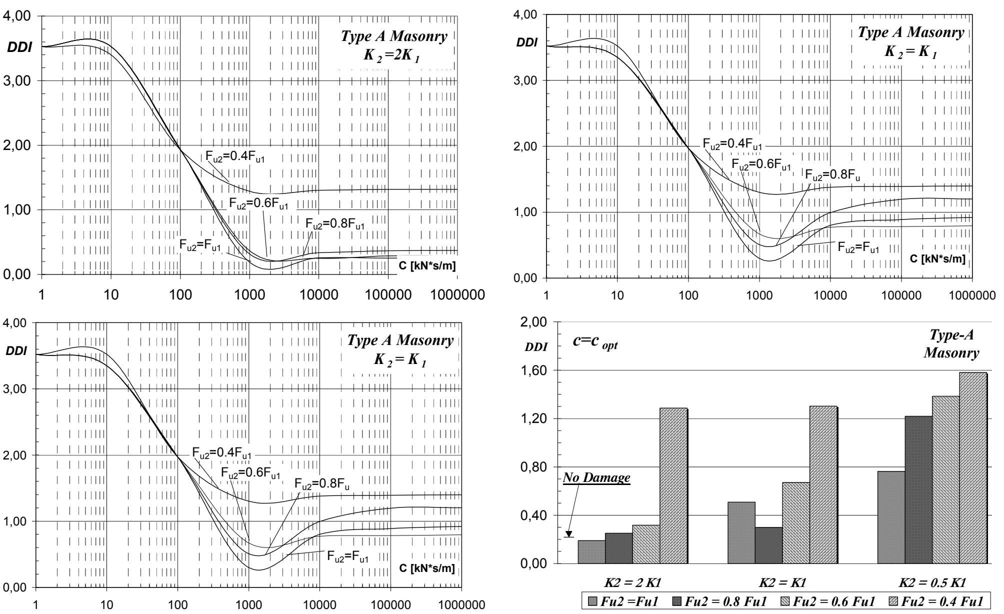


Figure 6. Type-A masonry: values of DDI as a function of viscous constant *c* (El Centro earthquake).

Figures 5 to 12 show the effectiveness of combining dampers and additional steel bracing on the global behavior of the structure. It is also possible to see as a minimum of the structural response in terms of *DDI* can be found for a well defined value c_{opt} of the device viscous constant. General guidance for the evaluation of the optimal viscous constant c_{opt} is given in Mazzolani *et al.* (2003).

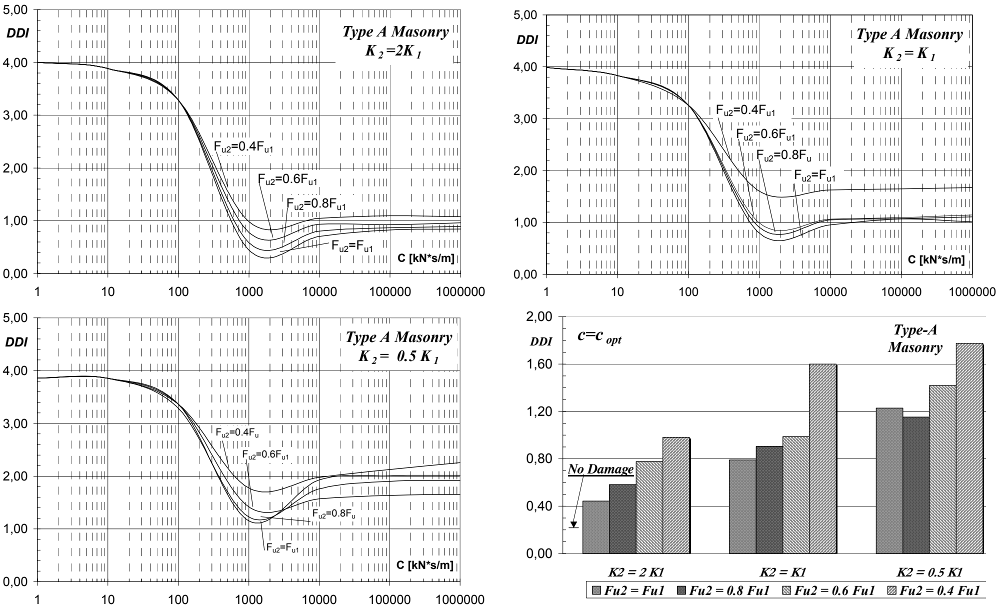


Figure 7. Type-A masonry: values of DDI as a function of viscous constant *c* (Taiwan earthquake).

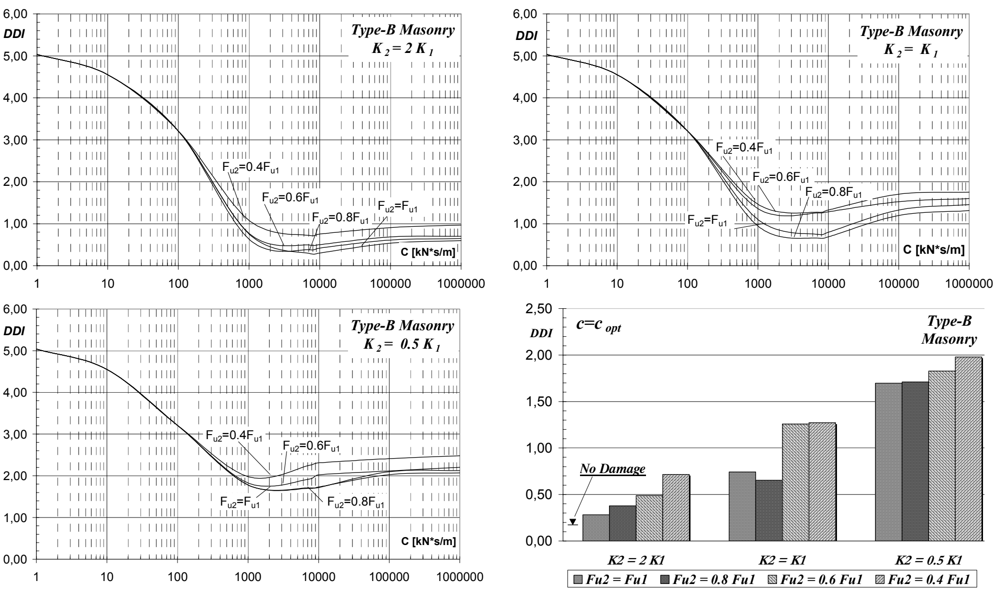


Figure 8. Type-B masonry: values of DDI as a function of viscous constant *c* (Calitri earthquake).

As a rule, the use of viscous devices strongly reduces the structural response also in terms of applied force on the wall. 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. As shown in Mandara & Laezza (2002), the best effectiveness of this provision is achieved when the steel bracing stiffness is higher than the panel in-plane stiffness, typically when $k_2 \geq (1 \div 2)k_1$. Nevertheless, 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, as long as $F_{u2}/F_{u1} > 0.5$.

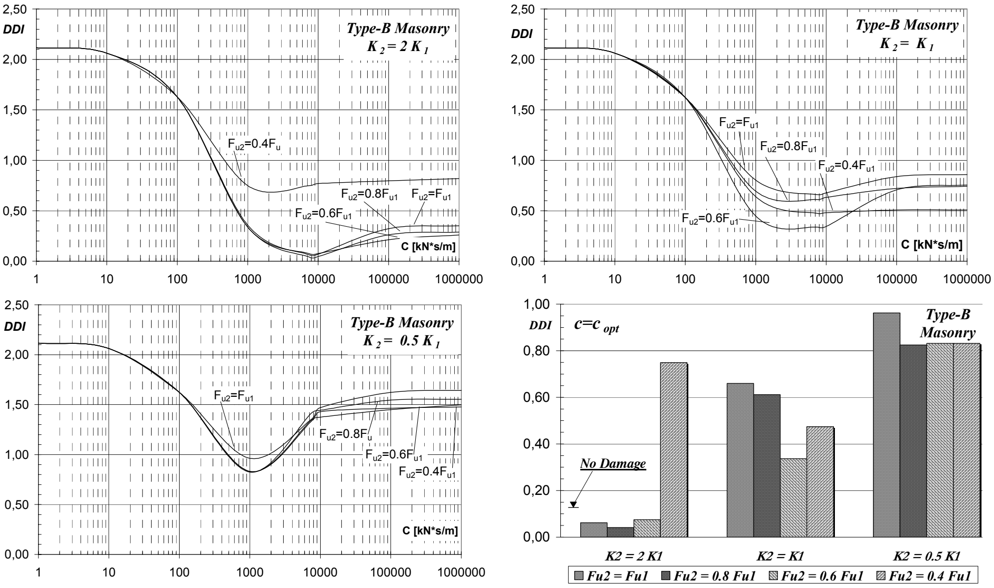


Figure 9. Type-B masonry: values of DDI as a function of viscous constant *c* (El Centro earthquake).

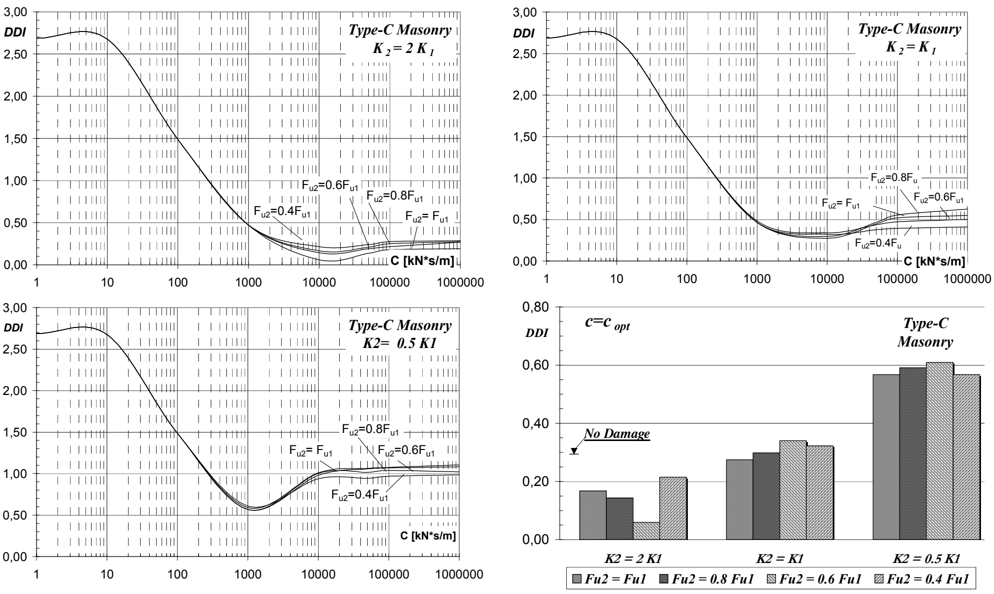


Figure 10. Type-C masonry: values of DDI as a function of viscous constant *c* (Calitri earthquake).

As an additional amount of dissipated energy by plastic hysteresis is achieved, this effect is similar to the one obtained by the use of additional plastic threshold devices in series with viscous dampers (Mandara & Mazzolani, 2001a,b, Mazzolani *et al.*, 2003). Similarly to that case, the main favorable effect of adding plastic threshold hysteretic elements is that the range of optimal values of the viscous constant c is remarkably enlarged.

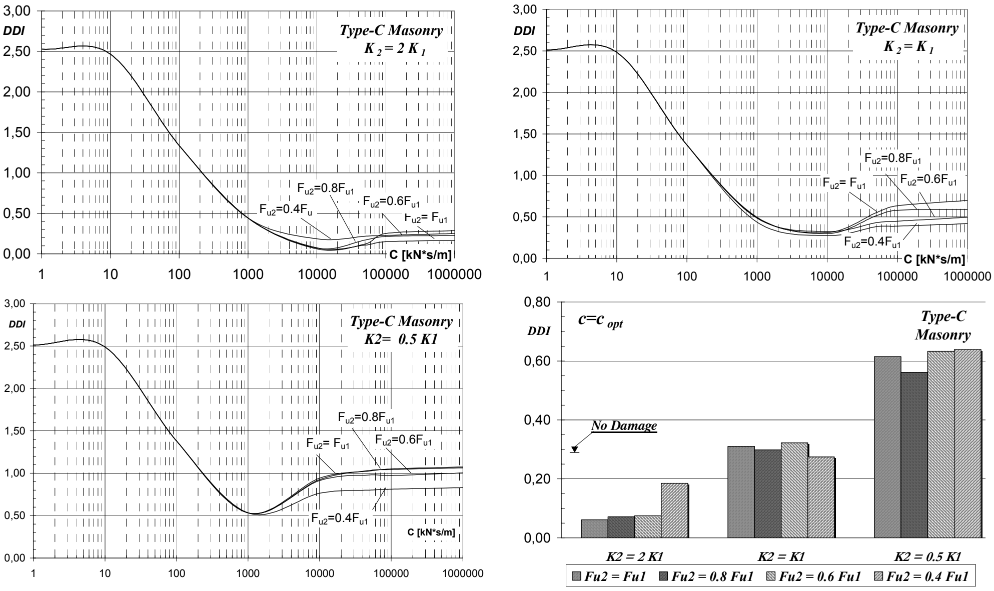


Figure 11. Type-C masonry: values of DDI as a function of viscous constant c (El Centro earthquake).

4. PERFORMANCE LEVELS

As customary in performance analyses, performance levels are defined in terms of corresponding damage levels. Assuming the FEMA 356 Seismic Rehabilitation Prestandard (2000) as a guideline for the choice of performance levels, the limits and corresponding ranges set out in Table 2 have been considered. Damage characterisation relevant to each of defined levels is detailed in the FEMA code as a function of the maximum in-plane displacement which can be tolerated by an unreinforced masonry wall. Distinction is also made in the code between primary (load-bearing) and secondary (infill) walls.

Taking into account FEMA performance limits, and also considering the behavioural models assumed for materials, the following conventional limit values of top-wall in-plane displacement have been assigned to the model: Immediate Occupancy (IO) $\Delta_{eff} \leq 1$ cm; Life Safety (LS) $\Delta_{eff} \leq 2$ cm; Collapse Prevention (CP) $\Delta_{eff} \leq 3$ cm. Note that such limits are independent of material and, also, are slightly higher than the corresponding limit displacement δ_{0c}, δ_{0y} and δ_u, set in Table 1, in particular as far as the CP performance level is concerned. This was due to the opportunity to account for some residual post-cracking ductility available in the panel during earthquake motion. Performance levels have been related to the device viscous constant c, in order to give useful indication for design and sizing of dampers. Figure 13 shows such dependence in nondimensional form for the cases $K_2/K_1 = 1$ and $K_2/K_1 = 2$. Curves referring to each of above defined performance levels, evaluated on the basis of the maximum demand in terms of viscous constant c for considered earthquakes, are plotted, also including curves corresponding to the optimal value c_{opt}. Stiffness ratio E_1/E_{ref} is normalized to the elastic modulus of Type-C masonry (E_{ref} = 660 N/mm^2). The constant c, instead, is nondimensionalised with respect to actual mass m_1 and stiffness K_1 of masonry structure under design. Plots in Figure 13 also report the influence of F_{u2}/F_{u1} ratio, showing its relatively reduced impact on the value of viscous constant necessary to achieve a given performance level. It can be observed in both diagrams that adopting $c = c_{\mathrm{opt}}$ always involves a performance level able to guarantee the immediate occupancy (IO) of the structure, in particular when K_2/K_1 = 2. Curves in Figure 13 can be helpfully used in practice when a design PGA = 0.4 g is assumed.

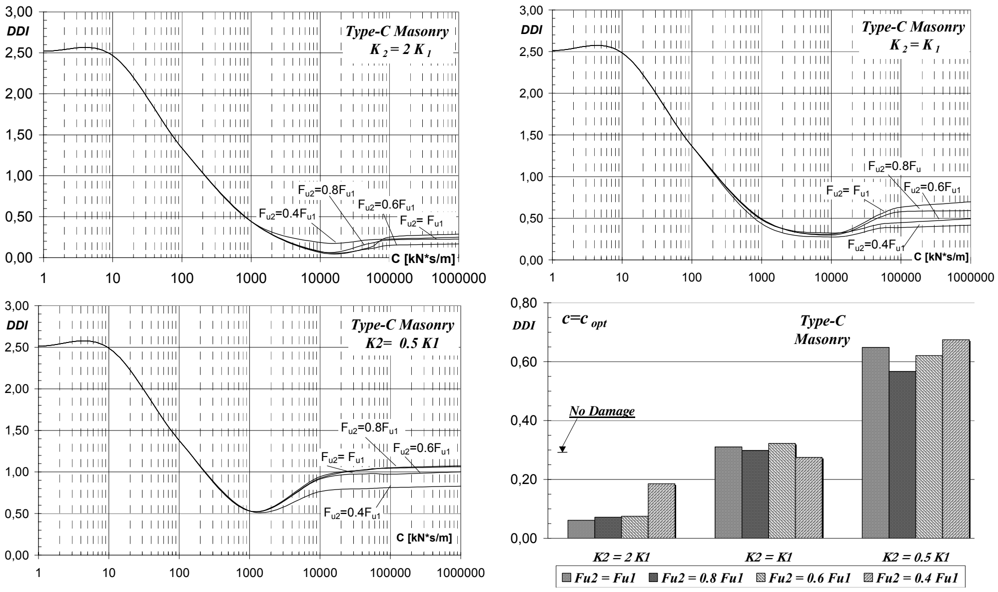


Figure 12. Type-C masonry: values of DDI as a function of viscous constant c (Taiwan earthquake).

Table 2. Acceptance criteria of top displacement for primary masonry walls according to FEMA 356.

	Performance level		
	Immediate occupancy (**IO**) (%)	Life safety (**LS**) (%)	Collapse prevention (**CP**) (%)
Bed-joint sliding	0.1	0.3	0.4
Rocking	0.1	$0.3h_{\mathrm{eff}}/L$ (*)	$0.4h_{\mathrm{eff}}/L$ (*)

(*) h_{eff} and L are the wall effective height and length, respectively.

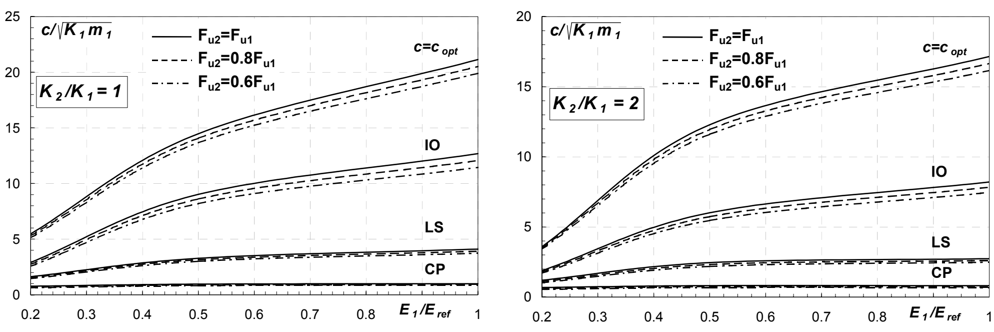


Figure 13. Nondimensional viscous constant for a given performance level versus E_1/E_{ref} (E_{ref} = 660N/mm^2).

5. CONCLUSIONS

The analysis described herein aimed at investigating the seismic behaviour of masonry structures strengthened by means of integrative steel structures and passive control techniques. The considerable effect of energy dissipation due to dampers on the magnitude of displacements in the structure under dynamic input has been emphasised. 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 possibility of adjust both steel bracing structure and energy dissipation system has been highlighted. In the end, results of the analysis in terms of structural damage have been related to conventional performance levels set out in the FEMA 356 recommendations. This led to link both optimal damper properties and bracing ultimate strength to the performance levels achievable by the structure. Obtained results, presented in a nondimensional designer-friendly form, show that, provided the elastic stiffness of the steel bracing is not lower than that of the masonry panel, also damping constant values lower than c_{opt} can lead, under certain circumstances, to achieve a performance level compatible with the immediate occupancy (IO) of the structure. The highest impact on the seismic performance of the building, attainable when $c = c_{opt}$, yields a performance far above requirements for the prompt use of the construction.

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