Design methods for existing r.c. frames equipped with elasto-plastic or viscoelastic dissipative braces

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ABSTRACT
Dissipative braces have proven to be very efficient devices for new buildings and seismic retrofitting of existing structures. In this paper a design method for dissipative braces based on elastic-plastic or viscoelastic behaviour, inserted in reinforced concrete existing frames with limited ductility, is proposed. The design method takes into account the dissipative behaviour of both the two components (r.c. frame and dampers). With regard to elastic-plastic devices, buckling restrained braces (BRBs) are considered, whereas High Damping Rubber (HDR) based devices are considered as viscoelastic devices. The behaviour of HDR is quite complex and both stiffness and damping depend on the strain amplitude and strain rate. Equivalent linear models may however be used to simulate their behaviour at a fixed displacement amplitude and frequency, with an acceptable approximation level.

1 INTRODUCTION

Passive control systems have proven to be very efficient devices for new buildings and seismic retrofitting of existing structures. The usual classification of these devices is by elastic-plastic dampers, viscoelastic dampers, viscous fluid dampers and friction dampers (Soong and Dargush 1997) This paper focuses on the first and second kind of device. With regard to elastic-plastic devices, buckling restrained braces (BRBs) are considered. They usually consist of a steel core encased in a steel tube filled with concrete or grout. BRBs can constitute the whole diagonal brace of the bracing system or they can be placed in series with an over-strengthened brace that remains elastic. The introduction of BRB members permits to significantly increase the stiffness of the existing frame and its dissipation capacity, due to their large cyclic inelastic deformation capacity. Viscoelastic devices generally provide lower energy dissipation with respect to elastic-plastic dampers but may be preferable because they withstand a large number of cycles and no permanent deformation remains after a seismic event. Viscoelastic devices are usually obtained by using copolymers or glassy substances, though recently High Damping Rubber (HDR), already used in vibration or seismic isolators, has also been employed (Fuller et al. 2000, Bartera and Giacchetti 2004, Lee et al. 2005).The behaviour of HDR is quite complex and both stiffness and damping depend on the strain amplitude and strain rate to which they are subjected (Dall'Asta and Ragni 2006). It has however been demonstrated in the case of simple dynamical systems that linear viscoelastic models may be used in order to simulate their behaviour at a fixed displacement amplitude and frequency, with an acceptable approximation level (Dall'Asta and Ragni 2008). Several methods are proposed in technical literature (Soong and Dargush 1997, Kasai et al. 1998) for the analysis or design of structures equipped with elasto-plastic and linear viscoelatic dampers, which usually are based on a linear behaviour of the main frame. These methods may be extended to take into account the non linear behaviour of the frame in which they are used.

This paper proposes a design method for dissipating braces, based on elastic-plastic or linear viscoelastic behaviour, inserted in reinforced concrete frames with limited ductility, taking into account the nonlinear behaviour of the r.c. frame. In both cases, the method is based on the assumption of a deformation shape of the coupled system that, in the case of regular post-elastic frame behaviour, coincides with the first vibration mode of the original frame. The method may be however extended to frames with irregular post-elastic behaviour by choosing a deformation shape able to regularize the frame behaviour. Once the deformation shape is chosen, the design procedure is based on the classical non linear static analysis (push-over analysis) of the existing frame and on the concept of equivalent Single-Degree-Of-Freedom (SDOF) system. The evaluation of the coupled (frame and dissipative braces) equivalent SDOF system is achieved by using the capacity spectrum method (Freeman 1998, Faifar 1999) in order to satisfy a given limit state for a given level of seismic excitation. In this paper the case of regular frames is considered and the Collapse Limit State is taken into account, as suggested by several technical codes (EN1998-3, OPCM 3431, DM 2008). Finally a distribution criterion of dissipative braces along the frame height is furnished and nonlinear time history analyses are performed in order to validate the design procedures.

2 DISSIPATIVE BRACE MODELING

Usually dampers, both elastic-plastic or viscoelastic, have a limited length, consequently dissipating braces generally consist of coupled metallic braces and dampers and their properties depend on the characteristics of both components which are placed in series. In particular, in the case of BRBs, the stiffness (K_c) the yielding force (F_c) and the ductility (μ_c) of the dissipative brace depend on the properties of the BRB (its yielding force F_0 and its ductility μ_0) and on the elastic brace stiffness (K_b) according to the following relations

$$F_c = F_o \tag{1}$$

$$K_c = \frac{K_b K_o}{K_b + K_o} \tag{2}$$

$$\mu_c = \frac{K_0 + K_b \mu_0}{K_b + K_0} \tag{3}$$

The brace stiffness is proportional to its transverse section area A_b, according to $K_b = EA_b / L_b$, where E is the steel elastic modulus and L_b is the length of the metallic brace, given by the difference between the total diagonal length (L_c) and the device length (L_0). The design procedure of dissipating braces generally gives an assigned value of K_c and μ_c, for each brace. Once the BRB ductility μ_0 is known, these values may be obtained by adopting the following values of device and link brace stiffness (K_0 and K_b):

$$K_0 = K_c \frac{\mu_0 - 1}{\mu_c - 1} \tag{4}$$

$$K_b = \frac{K_0}{K_0 / K_c - 1} \tag{5}$$

A value of the ductility ratio μ_c / μ_0 close to 1 results in values of $K_0 / K_c \rightarrow 1$ and $K_b / K_c \rightarrow \infty$ which means that maximum ductility may be obtained for very high values of the metallic brace stiffness. It should be observed that adopting an excessive ductility ratio (μ_c very close to μ_c) leads to very onerous metallic brace dimensions. On the contrary a lower limit of the ratio, thus minimum dimensions of the metallic brace, depend on the buckling check of the all diagonal brace (BRB in series with the elastic brace). In dissipating braces based on HDR-devices, the HDR material is subjected to shear deformations and usually shows a nonlinear behaviour. A linear description may be adopted by defining a shear modulus $G_0(\omega,\gamma)$ and an equivalent damping factor $\xi_0(\omega,\gamma)$, which nonlinearly depend on the deformation amplitude γ and on the frequency input ω. In particular, HDR is characterized by a value of $G_0(\omega,\gamma)$ which is very sensitive to the strain amplitude and quite sensitive to the strain rate, while the variation of the damping factor $\xi_0(\omega,\gamma)$ is less significant. Once the strain amplitude $\bar{\gamma}$ and frequency $\bar{\omega}$ are fixed, a Kelvin-type model may be used for describing the HDR-based damper behaviour. The spring stiffness K_0 and the dashpot coefficient c_0 of the Kelvin-type model are given by the following expressions:

$$K_0 = \frac{G_0(\bar{\omega},\bar{\gamma}) A_0}{h_0} \tag{6}$$

$$c_0 = \frac{2\xi_o(\overline{\omega},\overline{\gamma})G_0(\overline{\omega},\overline{\gamma})A_0}{\overline{\omega}h_0} \quad (7)$$

where A_0 and h_0 are the rubber area and the rubber thickness of the device. The dissipating diagonal brace generally consists of coupled metallic brace and HDR-based damper. The global behaviour of the diagonal brace still represents a viscoelastic element which is described by means of the stiffness K_c and the damping factor ξ_c, for assigned frequency and strain amplitude. These two parameters may be expressed as a function of K_0, ξ_0 and the brace stiffness K_b, according to the following formulas:

$$K_c = \frac{K_b K_o\left(\left(1+4\xi_0^2\right)K_o + K_b\right)}{\left(K_b + K_o\right)^2 + \left(2\xi_0 K_o\right)^2} \quad (8)$$

$$\xi_c = \frac{\xi_0 K_b}{\left(1+4\xi_0^2\right)K_o + K_b} \quad (9)$$

Similarly to the case of BRBs, the design procedure of dissipating braces generally gives an assigned value of K_c and ξ_c, for each brace. Once the equivalent damping coefficient of the rubber ξ_o is known, these values may be obtained by adopting the following values of device and link brace stiffness (K_0 and K_b):

$$K_0 = \beta(\xi_0,\xi_c)K_c \quad (10)$$

$$K_b = \alpha(\xi_0,\xi_c)K_0 = \alpha(\xi_0,\xi_c)\beta(\xi_0,\xi_c)K_c \quad (11)$$

where $\alpha(\xi_0,\xi_c)$ and $\beta(\xi_0,\xi_c)$ may be obtained by inverting Eqns. 3 and 4 and are:

$$\alpha(\xi_0,\xi_c) = \frac{(1+4\xi_0^2)\xi_c}{(\xi_0 - \xi_c)} \quad (12)$$

$$\beta(\xi_0,\xi_c) = \frac{(1+\alpha)^2 + 4\xi_0^2}{\alpha^2 + \alpha\left(1+4\xi_0^2\right)} \quad (13)$$

A value of the damping ratio ξ_c / ξ_0 close to 1 results in values of $K_0 / K_c \rightarrow 1$ and $K_b / K_c \rightarrow \infty$ which means that maximum damping may be obtained for very high values of the metallic brace stiffness. It should be observed that in many practical situations minimum dimensions of the metallic brace depend on the buckling check, therefore values of the dissipating bracing system damping ξ_c very close to the HDR damping ξ_0 are usually assumed in the design procedure.

3 DESIGN METHOD

In order to design the required dissipation system it is first necessary to reduce the behaviour of an Multi-Degree-Of-Freedom (MDOF) system, such as the r.c. frame or the coupled system, into an SDOF equivalent system. In this way the system capacity may be compared with the seismic demand in the acceleration-displacement plane and the required resistance and stiffness of the bracing system may be evaluated as a function of seismic intensity. For purposes of the displacement-based design approach this may be accomplished by assuming an objective displacement shape for the MDOF system, which may be considered as associated to the first vibration mode of the structure, in order to simplify the design procedure. If the original r.c. frame shows a regular behaviour along its height (no localization of displacement demand is presumed to occur in the inelastic field), it is reasonable to assume the deformed shape of the first vibration mode of the original r.c. frame as the objective displacement shape of the coupled system. As a consequence, the two resisting systems acting in parallel (r.c. frame and dissipative bracing system) are forced to deform mainly according to the assumed vibration mode, at least until the frame remains within the elastic field. Different shapes could be chosen if the frame behaves irregularly along its height and the aim is to change the frame behaviour. This paper considers the case of regular frames assuming an objective displacement shape coinciding with the first vibration mode of the r.c. frame. The inter-storey displacements of the first vibration mode of the frame, addressed as Δ^i (i=1..n), are defined with exception of a constant, and displacements at the different heights may be expressed as:

$$U^1 = \Delta^1 \; ; \; U^i = U^{i-1} + \Delta^i \;, \; (i=2...n) \quad (14)$$

where n is the total floor number. It is convenient to normalize the distributions of the inter-storey and storey displacements with respect to the ultimate floor displacement, which usually coincides with the control node in a pushover analysis:

$$\delta^i = \Delta^i / U^n \quad (15)$$

$$u^i = U^i / U^n \quad (16)$$

The equivalent SDOF mass m^* and the modal participating factor Γ associated to the system deforming according to the assumed shape are:

$$m^* = \sum_i m^i u^i \tag{17}$$

$$\Gamma = m^* \Big/ \sum_i m^i \left(u^i\right)^2 \tag{18}$$

The distribution of shear forces at each floor may be deduced from equilibrium:

$$V^n = \omega^2 m^n u^n;$$

$$V^i = V^{i+1} + \omega^2 m^i u^i \quad , \ (i = 1 \ldots n-1) \tag{19}$$

where ω is the circular frequency corresponding to the vibration mode. The stiffness distribution may be expressed as a function of the shear and inter-storey displacement at each level according to $K^i = V^i / \Delta^i$. In order to get rid of the dependency on frequency, the shear forces and stiffness of each level may be normalized with respect to the base shear and stiffness:

$$v^i = V^i / V^1 \tag{20}$$

$$k^i = K^i / K^1 \tag{21}$$

Once the objective shape and the relevant quantities described above are fixed, the design procedure may be started and the following steps defined: evaluation of the bare frame capacity by means of the equivalent SDOF system (*step1*), definition of the coupled SDOF equivalent system according to the seismic demand (*step2*), distribution of the base shear and base stiffness obtained along the MDOF system floors (*step3*), in the plane distribution of the bracing systems and design of each dissipative brace component (*step4*). Each step is described in detail below.

Step1

The capacity of the bare structure may be evaluated by nonlinear static analysis under a set of forces equal to the inertia forces ($F^i = \omega^2 m^i u^i$) of the first vibration mode. The pushover curve may be obtained by plotting the frame base shear as a function of the displacement of the control node, which is usually the top floor displacement. The frame base shear V_f^1 and the ultimate displacement s_u, corresponding to the failure of the frame section which first reaches its ultimate rotation capacity (if other fragile mechanisms are not met) may be determined from this analysis. The yielding displacement s_y of an elastic-perfectly plastic system equivalent to the frame may be obtained according to the code provisions (EN1998-3, OPCM 3431, DM 2008). The ductility of the elastic-perfectly plastic system equivalent to the frame is given by $\mu_f = s_u / s_y$. Finally, if necessary, the nonlinear system may be substituted by an equivalent linear system having the following properties:

$$K_f = V_f^1 / s_u \tag{22}$$

$$\xi_f = 0.05 + 0.565 \cdot \left(\frac{\mu_f - 1}{\pi \mu_f} \right) \tag{23}$$

The value of the equivalent viscous damping ξ_f is derived using the formulas to support the direct displacement-based design (Priestley et al. 2007). To this coefficient a value which is generally assumed equal to 0.05 must be added in order to take into account the dissipation due to non structural elements.

<u>Step2</u>

In the case of dissipative braces based on BRBs, the dissipative bracing system may also be represented by an equivalent elastic-perfectly plastic system with base shear V_d^1 and ductility μ_d . The coupled system is still an elastic-perfectly plastic system whose total base shear is $V^1 = V_f^1 + V_d^1$ and whose ductility μ is deduced by applying the areas equivalence criterion

$$\mu = \frac{\mu_d \mu_f \left(V_f^1 + V_d^1\right)}{V_f^1 \mu_d + V_d^1 \mu_f} \tag{24}$$

The maximum acceleration that the equivalent SDOF system is able to withstand is $a^* = V^1 / m^* \Gamma$, whereas the maximum displacement is $s_u^* = s_u / \Gamma$, where m^* and Γ remain unchanged despite the introduction of the braces. A useful representation may be obtained by plotting the capacity and demand curves in the acceleration-displacement plane. In this case the seismic demand is given by the inelastic spectra defined in (Fajfar 1999). The value of the base shear V_d^1 may be varied until the capacity curve intersects the demand curve at the ultimate frame displacement. The associated stiffness of the dissipating bracing system at the first floor is $K_d^1 = \mu_d V_d^1 / s_u \delta^1$. In the case of HDR-based dissipative braces, the dissipative bracing system may be represented by an equivalent SDOF viscoelastic system with stiffness K_d and equivalent damping ξ_d. The coupled systems is still a viscoelastic system whose total stiffness K is equal to the sum of the single stiffness and whose damping factor ξ is deduced by applying energy equivalence criteria (Soong and Dargush 1997):

$$K = K_f + K_d \tag{25}$$

$$\xi = \frac{\xi_f K_f + \xi_d K_d}{K_d + K_f} \tag{26}$$

In this case the maximum acceleration that the equivalent SDOF system is able to withstand is $a^* = K s_u / m^* \Gamma = (K / m^*) s_u^*$, whereas the maximum displacement is $s_u^* = s_u / \Gamma$, where m^* and Γ remain unchanged despite the introduction of the braces. Also in this case, the capacity of the system must be compared with the seismic demand and this can be made by plotting the capacity and demand curves in the acceleration-displacement plane. The value of the bracing system stiffness K_d may be varied until the capacity curve intersects the demand curve at the ultimate frame displacement. The base shear required by the dissipating bracing system is $V_d^1 = K_d s_u$, while its stiffness at the first floor is $K_d^1 = V_d^1 / s_u \delta^1$.

Step3

In both cases, the shear and stiffness that must be provided by the dissipating bracing system at each level may be determined according to Eqns. (20,21) and are:

$$V_d^i = V_d^1 v^i \tag{27}$$

$$K_d^i = K_d^1 k_i \tag{28}$$

Step4

Once the bracing system number and position are fixed, the design stiffness values of a single dissipation diagonal brace (K_c) may easily be determined according to geometrical considerations. Other geometric relationships may be introduced to calculate the ultimate displacements required for the diagonal brace s_{cu}, from the ultimate inter-storey displacements $\Delta_u^i = s_u \cdot \delta^i$. Finally, the geometry of the devices may be defined. In particular, in the case of BRBs, by assuming a ductility of a single diagonal brace μ_c equal to the ductility provided by the bracing system μ_d, the stiffness of the elastic link brace (K_b) and the stiffness of the BRB (K_0) may be defined by applying Eqns. (4,5). The area (A_0) and the length (L_0) of the internal BRB core may be define once the material is chosen (yield stress f_{yc} and elasticity modulus E_c). The area of the metallic brace is given by $A_b = K_b \cdot L_b / E$.

In the case of HDR-based dissipative braces, by assuming a damping coefficient of a single diagonal brace ξ_c equal to the damping coefficient provided by the bracing system ξ_d, the stiffness of the elastic link brace (K_b) and the stiffness of the HDR-based device (K_0) may be defined by applying Eqns. (10,11). The thickness of the HDR-based device may be directly obtained from the expression $h_0 = s_{cu} / \bar{\gamma}$, for the considered value of design maximum shear strain $\bar{\gamma}$, while the device area A_0 follows from Eqn. (6), since the fundamental vibration frequency of the upgraded structure ($\bar{\omega} = \sqrt{m^* / K}$) is known and consequently the rubber shear modulus $G_0(\bar{\omega}, \bar{\gamma})$ is defined. Once the length of the damper and consequently the length of the metallic brace are fixed, the metallic brace area may be defined. Finally, the dashpot coefficients of the single dissipating diagonal brace (c_c) may also be determined, according to

$$c_c = \frac{2\xi_c(\bar{\omega}, \bar{\gamma}) K_c(\bar{\omega}, \bar{\gamma})}{\bar{\omega}} \tag{29}$$

and applied in structural models to perform time history analyses.

4 APPLICATION

In order to illustrate the design procedure a simple two dimensional r.c. frame is considered. The frame is typical of many structures designed and built during the 80s without any particular seismic detailing in Italy. The frame consists of 6 spans and 4 stories. Columns have a 35×35 cm^2 square section at the base and a 30×30 cm^2

square section at the other levels. Beams are 30×50 cm^2 at each floor. The frame geometry and some reinforcement detailing are shown by Figure 1.

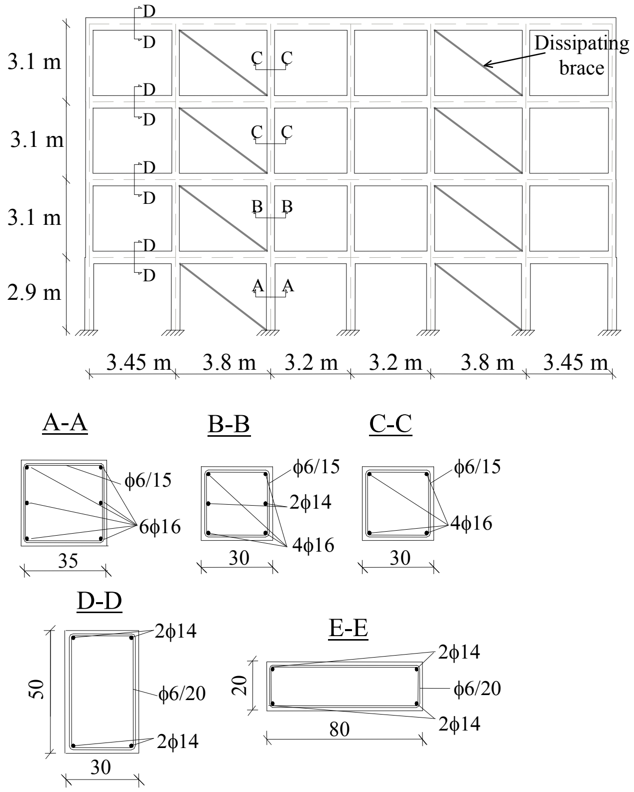


Figure 1. Frame and reinforcement details

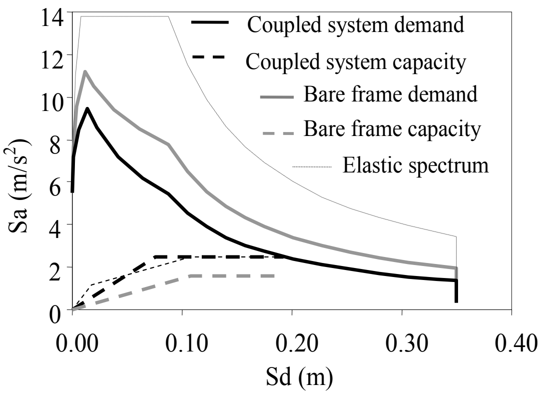
The seismic combination of dead and live loads results in a uniformly distributed load equal to 35 kN/m at the top floor and 26 kN/m at the other floors, while seismic masses are 95 kNs^2/m at the top floor and 72 kNs^2/m at the other floors. Structural analyses were carried out with the support of the finite element software SAP2000. Beams and columns were modelled as beam elements with reduced stiffness to take into account concrete cracking while the non linear behaviour was lumped at the beam ends by means of two plastic hinges. The moment-rotation curves were calculated according to (EN1998-3). Modal analysis of the bare frame showed a value of the first mode period equal to 1.44 s and a participating mass ratio above 79%. The results of modal analysis are summarized by Table 1. The nonlinear static analysis of the bare frame under the inertia forces of the first vibration mode demonstrated regular and quite ductile behaviour with plastic hinges reaching their ultimate capacity at the beams of the first storey. The push-over curve of the bare frame is characterized by a value of the maximum base shear $V_f = 397$ kN and an ultimate displacement $s_u = 0.239$ m. The ductility corresponding to the bilinear system is $\mu_f = 1.775$. The properties of the equivalent elasto-plastic SDOF system are: $\Gamma = 1.2455$, $m^* = 199.4$ kNs^2/m, $s_u^* = 0.192$ m, $a_y^* = 1.6$ m/s^2 and $\mu^* = 1.775$, the viscous damping of the equivalent linear system is $\xi_f = 0.128$. By assuming a pseudo acceleration spectrum given by OPCM 3431 code with a soil class C (soil factor S=1.25) and a design acceleration equal to a_g = 0.30g, the frame is not able to withstand the seismic action at the collapse limit state, as shown by Figure 2a, where inelastic spectra are used or by Figure 2b where elastic spectra are used

Table 1. Frame modal analysis results

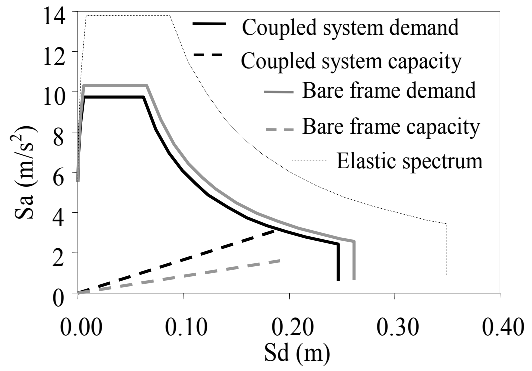
floor	*mass* (kNs^2/m)	U^i (m)	Δ^i (m)	u^i	δ^i
4	95.4	0.07910	0.0160	1.000	0.202
3	72	0.06310	0.0246	0.798	0.311
2	72	0.03850	0.0258	0.487	0.326
1	72	0.01270	0.0127	0.161	0.161

floor	V^i (kN)	K^i (kN/m)	v^i	k^i
4	95	420	0.380	0.478
3	153	491	0.396	0.766
2	188	576	0.464	0.942
1	199	1242	1.000	1.000

A BRB based dissipation system is firstly designed so that the frame is able to withstand the seismic demand. By choosing a design ductility of the diagonal brace , μ_d, equal to 11, the base shear the dissipative braces must provide was found to be V_d^1= 220 kN, while the base stiffness is K_d^1= 63025 kN/m (Figure 2a). Two braces are provided at each storey and placed in two symmetric spans (Figure 1). The application of the design method leads to values of stiffness of each diagonal brace at different stories (K_c^1) which are summarized by Table 2. The values of the ultimate displacement (s_{cu}^i) and the maximum axial force N_c^i the braces must sustain are also reported. The modal analysis of the coupled system showed a value of the first mode period equal to 0.77s and a first vibration mode coinciding with the fist mode of the bare r.c. frame, as expected according to the design procedure adopted. Finally, the geometry of the elastic braces and BRBs has been defined and they are shown by Table 3, a ductility $\mu_0 = 15$ is assumed for the BRBs. Thickness and diameters of metallic braces are chosen in order to satisfy buckling checks.



(a)



(b)

Figure 2. Capacity vs Demand in ADRS plane: inelastic case (a), elastic case (b)

Table 2. Distribution of dissipative brace properties along the height (BRB)

floor	V_d^i (kN)	K_d^i (kN/m)	K_c^i (kN/m)	s_{cu}^i (m)	N_c^i (kN)
4	105	23945	19940	0.0375	68
3	169	24950	20777	0.0576	109
2	207	29224	24353	0.0604	134
1	220	63065	49898	0.0305	138

Table 3. BRB device and elastic brace properties along the height

floor	K_0^i (kN/m)	A_0^i (mm²)	L_0^i (m)
4	27916	161	1.21
3	29088	259	1.87
2	34095	318	1.96
1	69857	329	0.99

floor	L_b^i (m)	K_b^i (kN/m)	A_b^i (mm²)	D_b^i (mm)	t_b^i (mm)
4	3.69	69790	1226	133	3
3	3.03	72721	1051	114	3
2	2.94	85237	1194	129	3
1	3.79	174641	3152	150	7

For HDR-based dissipative braces, a design rubber strain equal to $\bar{\gamma} = 1.5$ was chosen and a damping coefficient $\xi_d = 0.17$ was assumed for the dissipative diagonal braces, which is very close to the rubber equivalent damping coefficient for the design strain considered. The base shear the dissipative braces must provide was found to be V_d^1 = 394.5 kN , which corresponds to a base stiffness K_d^1 =1650.5 kN/m. Consequently the damping coefficient of the coupled system is about ξ =0.15, the base shear is V^1=791.72 kN and the base stiffness is K^1=3311.7 kN/m (Figure 2b). Also in this case two braces are provided at each storey and placed in two symmetric spans (Figure 1). The application of the design method leads to values of stiffness of each diagonal brace at different stories (K_c^i) which are summarized by Table 4. The values of the ultimate displacement (s_{cu}^i), the dashpot coefficients (c_c^i) and the maximum axial force N_c^i are also reported. The modal analysis of the coupled system showed a value of the first mode period equal to 1.19 s and a first vibration mode coinciding with the fist mode of the bare r.c. frame, as expected according to the design procedure adopted. The geometry of the elastic braces and HDR-based devices can finally be defined. The devices are supped to be constituted by two coaxial pipes with an interposed rubber layer. Since the circular frequency of the upgraded structure is $\overline{\omega} = 5.7$ s^{-1} and the maximum strain is $\bar{\gamma} = 1.5$, the HDR damping factor is about $\xi_0(\overline{\omega}, \bar{\gamma}) = 0.18$ while the elastic modulus is about $G_0(\overline{\omega}, \bar{\gamma}) = 0.7$ N/mm². The geometry of the elastic braces and HDR-based devices are shown by Table 5. Buckling checks were carried out with positive results. Dissipative braces have in this case larger dimensions because HDR-based devices are less dissipative with respect to BRBs.

Table 4. Distribution of dissipative braces properties along the height (HDR)

floor	V_d^i (kN)	K_d^i (kN/m)	K_c^i (kN/m)	s_{cu}^i (m)	c_c^i (kNs/m)	N_c^i (kN)
4	189	3903	3250.2	0.0375	273	122
3	302	4067	3386.7	0.0576	285	195
2	372	4767	3969.6	0.0604	334	240
1	394	10280	8133.3	0.0305	684	248

Table 5. HDR device and elastic brace properties along the height

floor	K_0^i (kN/m)	h_0^i (m)	A_0^i (m^2)	L_0^i (m)	D_0^i (m)
4	3375.23	0.025	0.120	0.35	0.110
3	3516.95	0.038	0.193	0.50	0.123
2	4122.28	0.040	0.237	0.60	0.126
1	8446.10	0.020	0.245	0.50	0.156

floor	L_b^i (m)	K_b^i (kN/m)	A_b^i (mm^2)	D_b^i (mm)	t_b^i (mm)
4	4.55	76926	1668	134	3.8
3	4.40	80156	1681	161	3.2
2	4.30	93952	1925	166	3.6
1	4.28	192498	3923.4	176	6.8

In order to validate and compare the two design procedures, nonlinear time history analyses were performed under seven artificially generated ground motions that match the seismic spectrum according to the criteria given by OPCM 3431. The results obtained have been averaged. Maximum roof displacements of the upgraded structure under design peak ground acceleration (at collapse limit state) and maximum base shears obtained in the two cases analyzed are compared in Table 6. Additionally, Figure 3 reports the maximum axial displacements experienced by the diagonal braces at each floor comparing these with the design displacements. Here again, in both cases, the values obtained are very close to the predictions.

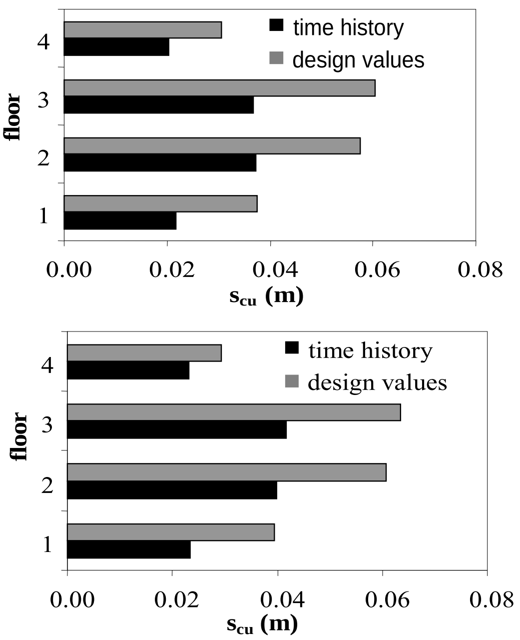


Figure 3. Diagonal brace displacements

It should be observed that the introduction of the dissipating braces also resulted in a major diffusion of inelastic demand which also affects the higher storeys of the frame. Ultimate rotation capacity of the plastic hinges was not reached. However the introduction of the dissipating systems results in a deviation of column axial forces from the values previously supported. Consequently, after the bracing system introduction, the strength and rotation capacity of the columns involved in the bracing system must be checked. Finally, Table 7 reports the inter-story drifts, the maximum ductility experienced by BRBs and the rubber strain levels at the damage limitation state. The damage limitation requirements are satisfied by both the systems, (an inter-story limitation equal to 0.005h was assumed), even if in the case of BRBs significant permanent deformations occur at the damage limitation state.

Table 6. Results of on linear dynamic analyses

HDR				BRB			
Analysis		Design		Analysis		Design	
s_u (m)	V_u (kN)	s_u (m)	V_u (kN)	s_u (m)	V_u (kN)	s_u (m)	V_u (kN)
0.16	812	0.24	792	0.14	695	0.24	617

Table 7. Results at damage limitation state

	BRB		HDR	
floor	Δ_sld (%)	μ	Δ_sld (%)	γ_sld (%)
4	0.255	2.52	0.271	25.6
3	0.381	2.44	0.411	25.3
2	0.416	2.55	0.435	26.1
1	0.205	2.39	0.216	24.1

5 ACKNOWLEDGMENTS

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