SEISMIC PERFORMANCE OF STEEL FRAMES COUPLED WITH VISCO-ELASTIC BRACINGS

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ABSTRACT :

In the present paper the seismic behavior of semi-continuous steel frames coupled with visco-elastic dissipative bracings is analyzed. The advantages due the employment of the dissipative system for improving the seismic performance of semi-continuous steel frames in the case of both low and high intensity earthquake ground motions is pointed out. The design of the hybrid system is faced by solving a SDOF problem first, and then by considering the MDOF response related to the multi-storey coupled frame's behavior. On the basis of the proposed procedure the dissipative bracing for the seismic up-grading of a six-floor semi-continuous steel frame was designed. The seismic performance of such equipped system was then analyzed through incremental nonlinear dynamic analyses that allowed to check the accuracy of the design strategy as well as to point out the advantages given by the coupled system compared to those of the bare semi-continuous steel frame.

KEYWORDS: semi-continuous steel frames, visco-elastic dampers, nonlinear analysis.

1. INTRODUCTION

The current seismic codes, based on performance criteria, address the structural design towards solutions which effectively combine the safeguarding of human life with economical aspects. They are related, in fact, both to the preservation of functionality in the case of a seismic event characterized by a high probability to occur and to the limitation of the structural damage and the reduction of losses due to the downtime for the structures under more rare and heavier earthquakes. It corresponds to the checks at serviceability (SLS) and ultimate limit state (ULS) respectively.
Concerning the steel structures design, the achievement of these objectives is presently guaranteed through strategies essentially based on the plastic dissipation. In fact, the earthquake-resistant steel structures that are mostly employed nowadays correspond to bare frames or to traditional braced systems, which supply the energy demanded by the earthquakes through a plastic hysteretic behavior of some structural components.
The steel frames are characterized by a high flexural dissipative behavior. Through a suitable design, the plastic mechanisms are enforced to occur in the beams in proximity to the connections with the columns, or in the beam-to-column joints, in the case of continuous or semi-continuous partial restrained frames respectively. Such structures are generally characterized by a high lateral deformability that determines their design for earthquakes at SLS and by an extensive and generalized damage, even in important structural parts of the main gravity bearing system, at ULS in the case of strong earthquakes.
The coupling between steel semi-continuous frames and dissipative bracings lead to hybrid systems that combine effectively the typical advantages assured by the frames with those of the braced structures (Amadio et al. 2008). In such a system a viscoelastic dissipative bracing is arranged in parallel with a semi-rigid steel frame, which is characterized by a high redundancy, simple and cheap beam-to-column connections but also by a poor lateral stiffness. Also the viscoelastic brace shows a limited lateral stiffness but it dissipates the energy supplied by the earthquake through a viscoelastic behavior by means of the use of rubber dampers, so limited lateral displacements are experienced by the system even under strong earthquakes.
Such a structural solution can be used either for new structure or for seismic up-grading of existing frames that were not designed to satisfy all the criteria required by the modern design codes (e.g. frames designed according to

former design provisions or frames built in regions assumed to be not seismic at the time of the construction and now considered earthquake prone areas). Moreover it has been proved that, in the case of the seismic up-grade of exiting frames, the hybrid system assures seismic performance even superior than that guaranteed by the traditional bracings (Amadio et al. 2008). In fact, by adding the dissipative bracings a very limited plastic damage in the existing frames can be guaranteed under the seismic loads as well as no increase in base shear and, therefore, no increase in strength demand on foundations unlike in the case of the use of traditional bracing systems.
In the design of such advanced structural solutions the main goal that has to be pursued is that to only partially exploit the ductility resources of the semi-continuous frames, so as to limit the damage on both the structural and the non-structural components and to comply with the performance criteria established by the modern design codes. In particular, under the effect of frequent earthquakes at SLS, only the dissipative mechanism of the viscoelastic brace is employed to limit lateral displacements so as to preserve the frame inside the elastic domain and to protect the non-structural components. Instead, at ULS under heavier earthquakes, a coupling between the plastic dissipative behaviour of the frame and the viscous dissipative capacity of the viscoelastic bracing is exploited. In this way, the damage of the frame is dramatically limited even at ULS and the failure of the dampers, due to excessive lateral drifts, is avoided achieving an efficient overall seismic response.

2. THE COUPLED SYSTEM

2.1. Analysis of the SDOF system

In order to effectively point out the behavior of the structural system made of a semi-continuous steel frame coupled with a dissipative bracing, its structural response can be studied through a simplified mechanical model which is the SDOF system, displayed in Figure 1 and equivalent to the equipped system (Soong et al. 1997).
The frame of mass m, elastic stiffness k_S and damping coefficient c_S is characterized by elastoplastic behavior and is arranged in parallel with the viscoelastic bracing. This structural component is made of a rubber damper of stiffness k_D and damping coefficient c_D positioned in series with the steel brace of elastic stiffness k_B.
Because generally $k_B >> k_D$ the mechanical characteristics of the bare brace do not influence the behavior of the coupled system. The design problem is focused basically on determining the parameters k_D e c_D that, according to the viscoelasticity theory, can be expressed as:

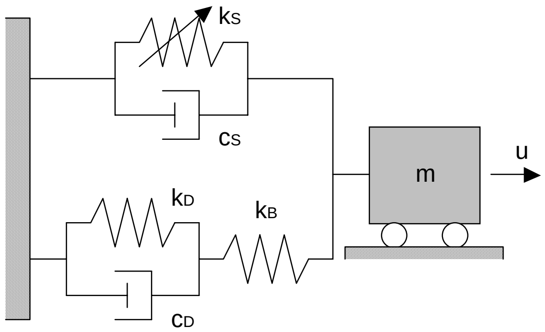


Figure 1. Equipped system mechanical model.

$$k_D = \frac{G'(\omega)A}{h} \quad (2.1), \qquad c_D = \frac{G''(\omega)A}{\omega h} = \eta_D \frac{k_D}{\omega} \quad (2.2)$$

where G', G" and η_D represent, respectively, the loss storage modulus, the shear storage modulus and the loss factor of the rubber which forms the dampers, whereas A and h are the area and the thickness of the rubber layers and ω represents the natural frequency of the device. The dynamic behaviour of the bare frame can be schematized by its natural frequency ω_S and the damping ratio ξ_S. In the case of coupled system, such properties (i.e. ξ_{SD} and ω_{SD}) can be achieved by adding the value for the viscoelastic bracing to those for the frame:

$$\xi_{SD} = \frac{(c_S + c_D)}{2m\omega_{SD}} \quad (2.3), \qquad \omega_{SD} = \sqrt{\frac{(k_S + k_D)}{m}} \quad (2.4)$$

After defining the type of rubber for the dampers and its loss factor η_D, it is possible to determine the characteristics of the dissipative system that allow the required level of overall damping to be reached. In fact, by substituting the eq. (2.2) and the eq. (2.4) into the eq. (2.3), a second order function of k_D is obtained whose roots are given by:

$$k_D = \frac{2\xi_S^2}{(\eta_D - 2\xi_{SD})^2}\left(1 + \frac{(\eta_D - 2\xi_{SD})}{\xi_S^2}\xi_{SD} \pm \sqrt{1 + \frac{(\eta_D - 2\xi_{SD})}{\xi_S^2}\eta_D}\right)k_S \tag{2.5}$$

The eq. (2.5) expresses the direct proportionality between the device stiffness and the frame stiffness through a factor that depends on the overall damping level ξ_{SD}, on the rubber characteristic η_D and on the frame damping ratio ξ_S. Between the two roots of eq. (2.5), only the solution which supplies the lower stiffness k_D is to be considered, as the other root would involve the use of a too large and physically unacceptable amount of rubber for the viscoelastic device. In order to evaluate the effects of the coupling between the viscoelastic device and the frame, some preliminary dynamic analyses were carried out through the SDOF system. An artificial earthquake ground motion (acc14) compatible with the elastic spectrum proposed by the Eurocode 8 (CEN 2003) for type A soil with a peak ground acceleration a_g = 0.35g was considered. A unit mass, a damping ratio ξ_S=5% and ductility factor μ characterized the SDOF model for the frame, whereas a loss factor η_D=0.65 was assumed for the rubber.

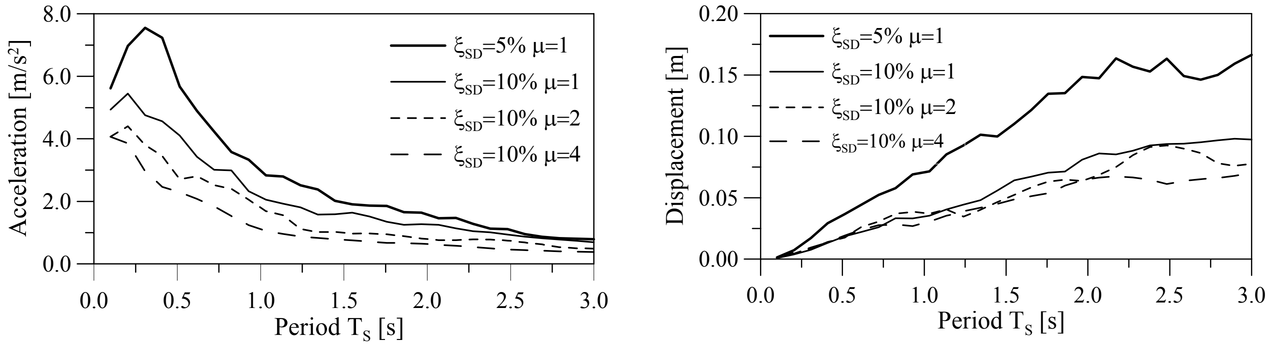


Figure 2. Acceleration and displacement spectra for the SDOF system.

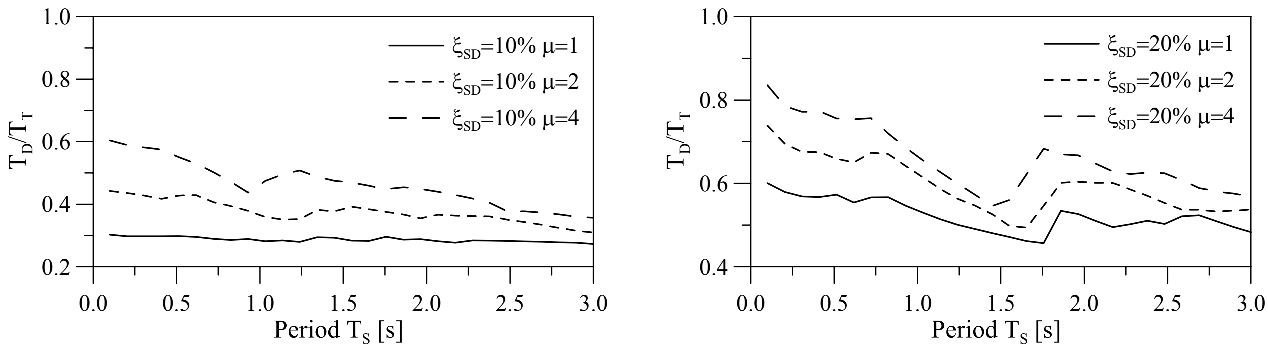


Figure 3. Spectra of the ratio T_D/T_T in the case of the acc14 earthquake.

In Figure 2 some numerical results are presented in terms of acceleration and displacement spectra. Many curves are reported. They have been achieved by considering a value ξ_{SD} = 10% for the overall damping ratio and different ductility factors (μ =1, μ =2 and μ =4) for the bare frame. It has to be pointed out how the spectral forms in Figure 2 are expressed in terms of the natural period of the bare frame and how the variation of such structural property due to the added dissipative bracing is implicitly taken into the equations employed to draw the spectral forms. By considering the response in terms of both acceleration and displacement, the advantages that can be achieved by combining quite low damping values (ξ_{SD} = 10%) with only a limited plasticisation of the frame (ductility factor required $\mu \cong 2$) can be pointed out.

In Figure 3 the ratio between the shear in the dampers T_D and the total shear T_T for the equipped system are shown. The figure points out how the dissipative bracing with low damping attracts only a limited amount of the total seismic force, unlike the traditional brace system that attract the whole seismic force. Nevertheless, the shear in the dampers rises by increasing the amount of rubber (ξ_{SD}=20%). The shown results evidence the effectiveness of the coupling between the ductility of the frame and the dissipative capacity of the viscoelastic device. A suitable combination of the two different resources allows to achieve good structural performance as far the reduced forces and displacements that invest the structure during the seismic event are concerned.

2.2. Analysis of the MDOF system

The extension of the results obtained for the SDOF model to the case of MDOF systems, requires the introduction of more simplified assumptions concerning the vibration modes of the equipped structure (Soong et al. 1997). The analysed structural systems correspond to a steel frame with semi-rigid joints, characterized by three degree of freedoms for each node (two translations and one rotation). Therefore the stiffness and damping matrices indispensable for determining the solution of the dynamic problem assume very large dimensions even in the case of a limited number of storeys, so the employment of static condensation method is required. Moreover, for the design purposes, it is sufficient to consider only the contribution of the first vibration mode of the overall MDOF system response, as it generally prevails over the higher modes. If the system is linear and the damping matrix is a Rayleigh damping matrix, the modal superposition can be used to compute the characteristic parameters of the MDOF system equipped with the viscoelastic bracing, that are given by:

$$\omega_{SD} = \sqrt{\frac{\sum_{j=1}^{N}\sum_{i=1}^{N}\left(k_{S_{ij}} + k_{D_{ij}}\right)\varphi_{SD_i}^{(1)}\varphi_{SD_j}^{(1)}}{\sum_{i=1}^{N} m_i \varphi_{SD_i}^{(1)2}}} \qquad (2.6),$$

$$\xi_{SD} = \frac{1}{2\omega_{SD}} \frac{\sum_{j=1}^{N}\sum_{i=1}^{N}\left(c_{S_{ij}} + c_{D_{ij}}\right)\varphi_{SD_i}^{(1)}\varphi_{SD_j}^{(1)}}{\sum_{i=1}^{N} m_i \varphi_{SD_i}^{(1)2}} \qquad (2.7)$$

where $\varphi^{(1)}_{SDi}$ is the i^{th} component of the equipped system's first mode, while k_{Sij}, k_{Dij}, c_{Sij}, c_{Dij} are the general terms of the stiffness and damping matrices of translation for the bare frame and for the dissipative bracing respectively. The main design goal for the viscoelastic dissipative device concerns the determination of the coefficients k_{Dij} and c_{Dij} for the eqs. (2.6) and (2.7) which represent, according to the assumptions made above, the stiffness and the damping coefficient that the viscoelastic device introduces into the system.

The approach employed neglects the modal forms variation, because it depends on the equipped system stiffness matrix that is an unknown value. Through this approach, the variation of the natural period is considered only and the eqs. (2.4) and (2.5), computed in the elastic domain in the case of the SDOF system, are employed. Therefore the expression for the overall damping ratio can be defined as:

$$\xi_{SD} = \xi_S \frac{\omega_S}{\omega_{SD}} + \frac{1}{2\omega_{SD}} \frac{\sum_{j=1}^{N}\sum_{i=1}^{N} c_{D_{ij}} \varphi_{S_i}^{(1)}\varphi_{S_j}^{(1)}}{\sum_{i=1}^{N} m_i \varphi_{S_i}^{(1)2}} \qquad (2.8)$$

For a practical use of the presented design method, it is then necessary to define the two matrixes K_D and C_D that characterize the dissipative system and to simplify the eq. (2.8). Moreover, the employed matrixes have to be able to effectively define the actual contribution for the overall equipped system behaviour, according to the results achieved in the case of the SDOF model. Among the admissible solutions, it has been checked that a tri-diagonal matrix representation, as for the shear-type frame, complies with such a request adequately. According to this assumption the damping ratio can be computed through the equation:

$$\xi_{SD} = \xi_S \frac{\omega_S}{\omega_{SD}} + \frac{1}{2\omega_{SD}} \frac{\sum_{i=1}^{N} c_{D_i} \varphi_{rS_i}^{(1)2}}{\sum_{i=1}^{N} m_i \varphi_{S_i}^{(1)2}} \qquad (2.9)$$

where c_{Di} is the damping coefficient for the dissipative bracing at the i^{th} floor, whereas $\varphi_{rSi}^{(1)}$ corresponds to the inter-story drift of the i^{th} floor for the first mode. After choosing a distribution for c_{Di} along the height of the structure, suitable values for both the damping and the corresponding stiffness (eq. 2.2) can be determined so as to model the effect of the dissipative system for each floor. Moreover the eq. (2.9) enables the unknown design values for the viscoelastic device to be determined as functions of the semi-rigid frame parameters, of the rubber loss factor, of the equivalent equipped SDOF system's natural period and of the overall damping level that has to be achieved. The choice of this last parameter has to be made on the basis of the design objectives and it represents the fundamental phase of the whole proposed approach. The design criteria for the viscoelastic device are essentially based on the choice of the overall damping level that has to be achieved through the employment of the dissipative bracing. According to the Performance Based Seismic Design philosophy, the structural performance has to be defined, for each security level, in term of maximum lateral drifts and of the damage due to the seismic actions and allowed for structural and non-structural components. The maximum lateral displacement, which can be expressed either as the top floor displacement or as the maximum floor drift, and the required ductility can be defined on the basis of the spectral forms shown in Figure 2. The damping ratio, that has to be achieved by means of the use of the dissipative bracing, can be then straightforward determined through the displacement spectra, only by knowing the natural period of the semi-rigid frame. Furthermore the bare frame can be preliminarily designed by using the standard rules for steel structures without considering any dissipative brace contribution. Then, the most suitable curve for the whole equipped system can be chosen. After, fixing the desired damping level, the coefficients c_{Di} can be calculated through the eqs. (2.4), (2.5) and (2.9). A constant distribution for the coefficient c_D can be employed for the typical applications. So this coefficient can be extracted form the summation of the eq. (2.9) and the stiffness k_D can be determined through the eq. (2.2):

$$c_D = 2\frac{\sum_{i=1}^{N} m_i \varphi_{S_i}^{(1)2}}{\sum_{i=1}^{N} \varphi_{rS_i}^{(1)2}}\left(\omega_{SD}\xi_{SD} - \omega_S\xi_S\right) \qquad (2.10),$$

$$k_D = 2\omega_{SD}\frac{\sum_{i=1}^{N} m_i \varphi_{S_i}^{(1)2}}{\eta_D \sum_{i=1}^{N} \varphi_{rS_i}^{(1)2}}\left(\omega_{SD}\xi_{SD} - \omega_S\xi_S\right) \qquad (2.11)$$

In the last phase of the proposed design method the rubber layers for each device at each floor are dimensioned. The calculation of the overall rubber height h has to be carried out by using the maximum slip allowed for the viscoelastic material, whereas the area A of the dissipative device can be determined through the floor stiffness k_D according to the eq. (2.1). Therefore the expressions for h and A are given by:

$$h = u_{r,max}/\gamma_D \qquad (2.12),$$

$$A = \left(k_D/G'\right)h \qquad (2.13)$$

where γ_D represents the maximum design slip for the viscoelastic material and $u_{r,max}$ is the maximum inter-storey drift allowable in the design.
Such a procedure is very simple to apply both for the seismic up-grade of existing constructions and for the design of new structures. In the former case the characteristics of the existing bare frame are known, while, in the latter case, the choice of the desired damping level has to be done also by considering the design criteria employed for the semi-rigid steel frame.

3. NUMERICAL ANALYSES

In order to check the effectiveness of the design procedure for the viscoelastic devices and to point out the benefits assured by coupling a semi-continuous steel frame with a dissipative bracing the nonlinear seismic response of a real coupled structure was investigated. In Figure 4 the elevation and some structural details of the analysed system are shown. The structure 21.50 m height has six floors. The floor height is h_0=4.0 m for the ground floor and h_1=3.5 m for the upper floors, whereas the beam span is L=6.0 m.

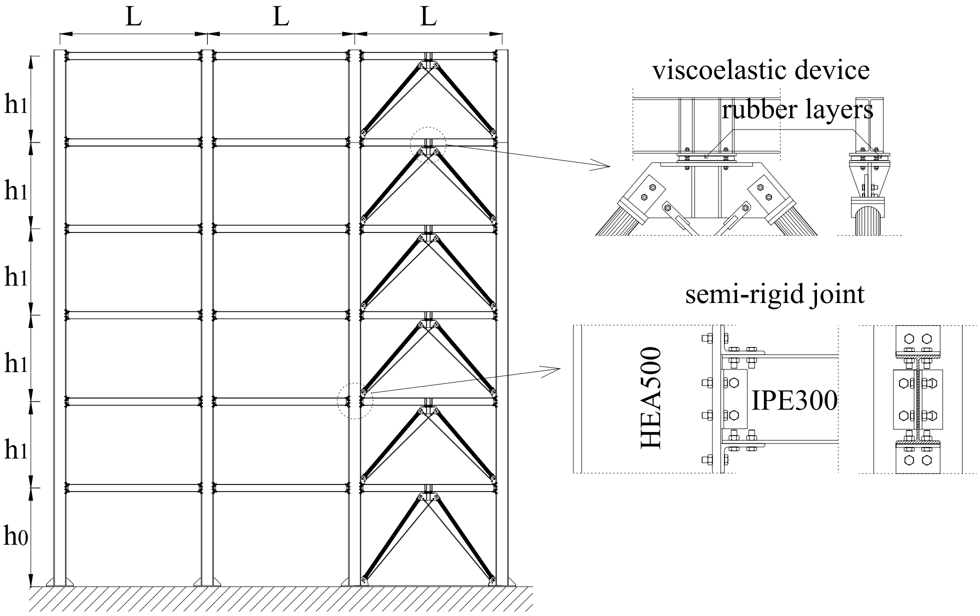


Figure 4. Elevation and structural details of the six-floor frame equipped with the dissipative bracings.

The preliminary structural design has been carried out by considering only the bare semi-continuous steel frame. In particular the rules supplied by the Eurocode 8 with a peak ground acceleration a_g=0.35g, a type A soil, a behaviour factor q=6 and floor loads referred to a width of 6 m, have been assumed. In this first design phase, the most critical condition for the frame was the checks at SLS for the lateral drift control. In order to comply with a limit drift of 0.75%, columns with a steel profile HEB500 have been employed, whereas beams with a steel profile IPE300 have been determined to support the vertical loads. The semi-rigid beam-to-column joints correspond to web and flange bolted angles connections. In particular two steel angles 80x80x10 for the web and one steel angle 150x100x12 for the two flanges of each beam have been considered. The used viscoelastic device is made of two steel flanges with an inner natural rubber layer. It is linked through bolts to the lower flange of the beam on top and to the joint plate for the two diagonal brace members at the bottom. The diagonal elements correspond to two steel tubes coupled with two steel ties that assure the parallelism of the device to the beam (Fig. 4). The analyses have been carried out by considering three different overall damping levels ξ_{SD}: 10%, 15% and 20%. A conventional structural damping of 5% for the bare frame has been assumed as well. A medium stiff rubber has been used for the device, which guarantees a loss factor η_D=0.65 and a stiffness modulus G'=1.00 MPa. The numerical analyses were carried out by using the Abaqus FE code (Abaqus 2001) schematizing the beam-to-column joint response through an advanced component modelling as it strongly influences the whole system seismic behaviour (Amadio et al. 2008). The viscoelastic device was designed by considering the fundamental period and the corresponding modes for bare frame and the rubber height was defined considering two limit values for the inter-storey drift: 1% and 2% and a limit slip for the viscoelastic material γ=1.5. In Table 1 the results achieved according to the proposed design approach are presented.

Table 1. Design values for the equipped six-floor frame.

ξ_{SD}	T_{SD} [s]	c_D [Ns/mm^2]	k_D [N/mm^2]	h [mm] Drift 1%	A [cm]	h [mm] Drift 2%	A [cm]
10%	1.68	307	1764	25	21 x 21	50	30 x 30
15%	1.50	690	4448	25	33 x 33	50	47 x 47
20%	1.29	1212	9080	25	48 x 48	50	67 x 67

In order to highlight the seismic response of the equipped system, the results achieved through dynamic nonlinear analyses under the artificial earthquake ground motion acc14 compatible with the elastic spectrum proposed by the Eurocode 8 for type 1 soil are shown. In Figure 5 the seismic responses in term of time-history of the inter-storey drift at the 6th floor obtained for the bare frame and for the equipped system are compared. The presented results point out how by considering an overall damping ξ_{SD} = 10% favourable reductions of maximum displacements can be obtained. Moreover it has to be remarked how the viscous damping effect, provided by the dissipative bracing, enables the higher frequencies to be eliminated and it allows the structure to vibrate according to the first mode, as it has been assumed in the design of the system.

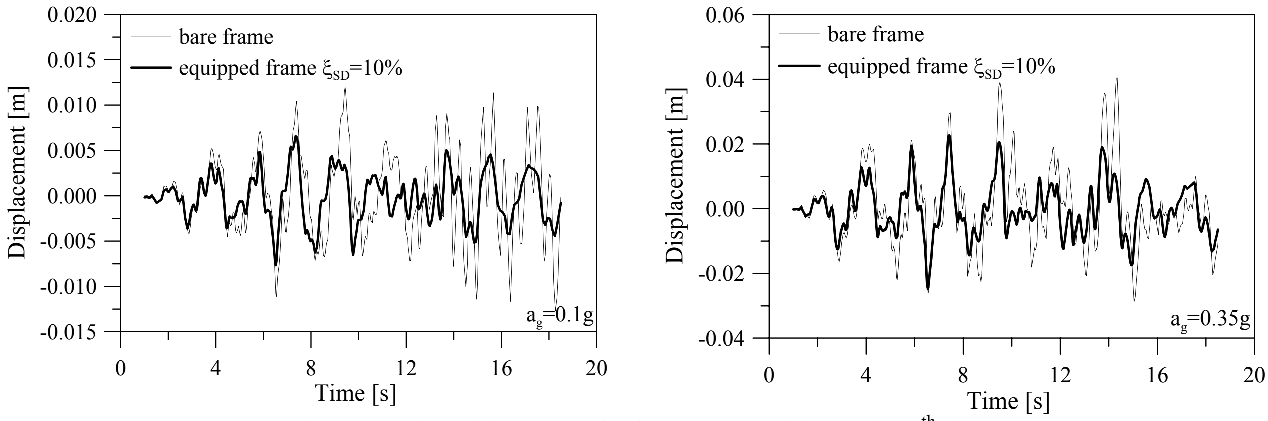
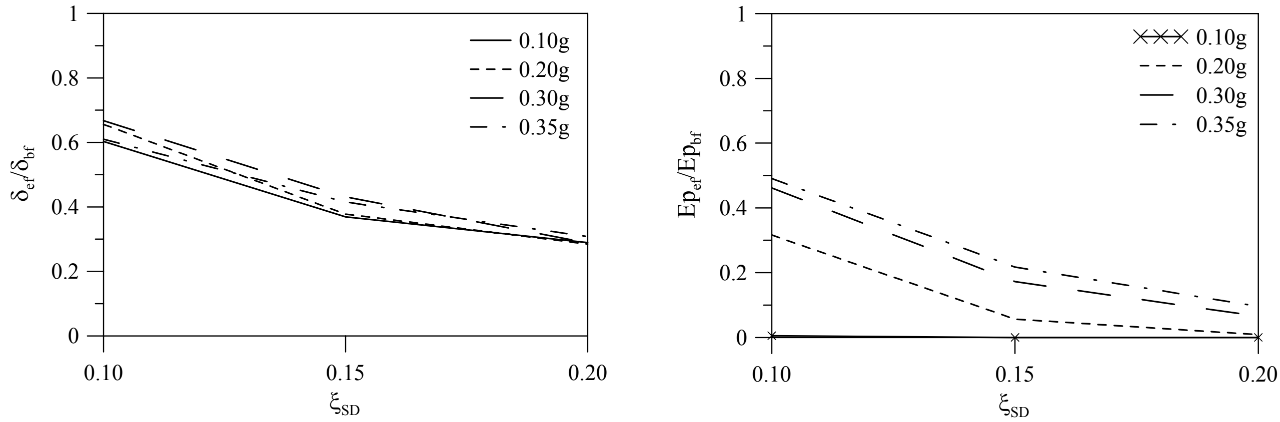
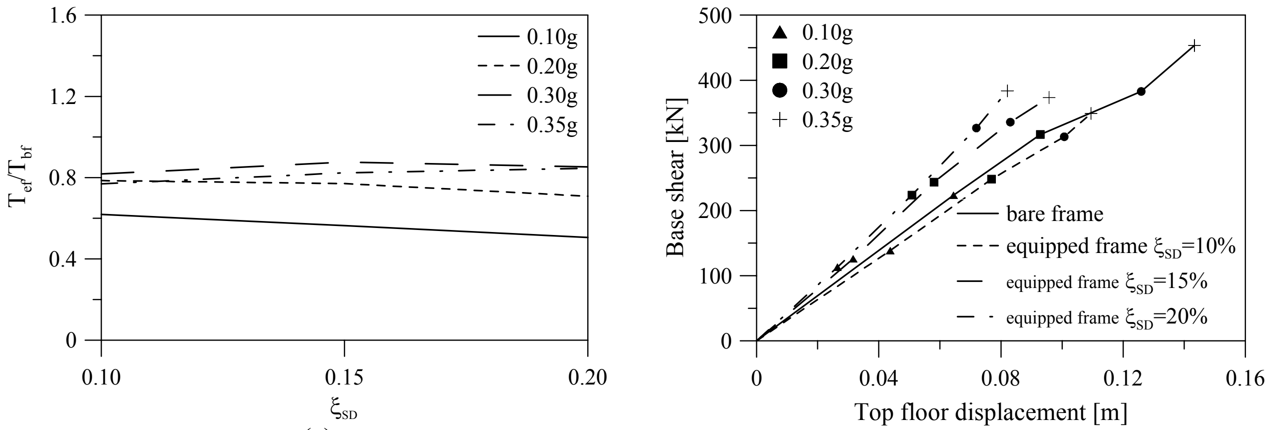


Figure 5. Time-history of the inter-storey drift at the 6th floor.



(a) (b)

Figure 6. Trend of the ratio δ_{ef}/δ_{bf} (a) and of the ratio Ep_{ef}/Ep_{bf} (b).



(a) (b)

Figure 7. Trend of the ratio T_{ef}/T_{bf} (a), IDA curves (b).

In Figure 6a the ratio between the floor displacements δ_{ef} of the frames equipped with viscoelastic bracings and those of the corresponding bare frames δ_{bf} are diagrammed by varying the damping ratio ξ_{SD} and the peak ground acceleration. The ratio between the dissipated plastic energies Ep_{ef} and Ep_{bf} that refer to the equipped and bare frames respectively are shown in Figure 6b. The trend of the latter ratio points out the effectiveness of the dissipative bracing to prevent the damage by increasing the overall damping level assured by the device. Moreover it can be highlighted how in the case of a_g=0.10g the frame does not contribute to dissipate the energy supplied by the earthquake. In fact, in such a case, even the use of a low overall damping ξ_{SD}=10% guarantees the frame to remain in the elastic field. Concerning the floor accelerations and the floor shears, the viscous damping allows an effective reduction of the maximum values, even though the structural stiffness increases because of the contribution due to the device. In Figure 7a the ratio between the maximum base shear T_{ef} experienced by equipped frames and that relating to the bare frame T_{bf} are also presented. In the case of equipped frame the shear is reduced up to 50%, respect to the values for the bare system, this is due to the damping supplied by the device. In Figure 7b the curves achieved through incremental dynamic analyses (IDA) are shown. They represent the trend of the base shear against the top-floor displacement in the case of the acc14 earthquake. The Performance Points for the frames are also highlighted in the curves, according to the peak ground acceleration variation. The comparisons between these dynamic responses point out the benefits guaranteed by the use of the viscoelastic dissipative devices coupled with the semi-rigid frame. The target displacement for every peak acceleration value decreases proportionally to the added damping provided by the viscoelastic device. Whereas the base shear increment, due to the higher lateral stiffness of the equipped system, remains around acceptable values. The trend of IDA curves for the bare frame point out a relevant influence of higher modes that determine its dynamic response. The benefits assured by the added damping are notable: the top-floor displacements are reduced and the structure vibrates according to the first mode and it eliminates the peaks of shear due to the higher modes. For these reasons the Performance Points achieved for each seismic acceleration move towards shear values lower than those of the bare frame.

4. FINAL REMARKS

In the paper a comprehensive approach for the design of equipped systems made of semi-continuous steel frames with dissipative bracings has been presented. According to such design procedure the damping devices for a six-floor frame have been determined. Then the equipped frame has been studied through non-linear dynamic analyses. The improved structural performance, achieved by using the dissipative bracing, has been pointed out in the case of both frequent earthquakes at SLS and heavier earthquakes at ULS. In the first case the semi-rigid frame remains in the elastic field and limited displacements is assured mainly by the added damping. While the effect of the bare frame ductility combined with the device's viscous damping strongly influences the structural performance at ULS. In fact, in the case of strong earthquakes, the hysteretic dissipative capacity of the semi-rigid frame limits the seismic actions (in particular the base shear) and the viscous damping provided by the dampers limits the excessive displacement due to the joint plasticisation.

The proposed approach allows for assuring a relevant flexibility in the design strategies according to the desired protection level. The optimal solution can be defined through an economical balance that considers structural and non-structural aspects, i.e the save of material, the costs of construction and maintenance of the dissipative device, costs for repairing the damage elements of the steel frame etc. Moreover such an approach can be particularly effective in the case of seismic upgrade of existing constructions.

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