Mixed and composite structures

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Seismic analysis of partially restrained composite frames coupled with dissipative bracings

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ABSTRACT: The paper investigates the dynamic behaviour of partially restrained steel-concrete composite frames coupled with viscoelastic dissipative bracings. A numerical model which accounts for both the resisting mechanisms of the joint and the viscoelastic contribution of the dissipative bracing is introduced and briefly discussed. The model is first validated against outcomes achieved at University of Ancona, Italy on a one-storey two-bay composite frame subjected to free vibrations. The model is then used to carry out a number of time-history analyses considering various types of earthquake ground motions, different values of peak ground acceleration and different mechanical characteristics of the dissipative device. It is pointed out that the viscoelastic bracing can be used as an effective dissipation system alternative to the plastic hysteresis, with the further benefit of preventing permanent structural damage.

1 INTRODUCTION

In order to improve the seismic performance of a structure, modern design approaches are founded on energy quantities. The structural capacity to resist seismic actions is estimated on the basis of the input energy, i.e. the total energy transferred by the earthquake to the structure, and on the basis of the dissipative capacity of the structure (Whittaker et al. 1991). The input energy can be adequately absorbed as described in the following:

- by reducing the energy transmitted to the structure using base isolation systems;
- by increasing the structural dissipative capacity so that a large part of the input energy can be absorbed;
- by combining the two aforementioned approaches.

The second approach is generally the most used for ordinary structures since it is the cheapest one. The increase of dissipative capacity can be obtained by hysteresis or by increasing the viscous damping. The energy dissipation by hysteresis can be achieved by allowing the structure to enter the post-elastic phase so that the large plastic strains induced by medium to high intensity earthquake ground motions can be absorbed without collapse. This approach which is based on the concept of "controlled ductility" cannot be accepted when the structural damage would result in important economical losses due to the cost of repairing and the impossibility to use the building. In these cases the use of dissipative systems is preferable since such systems can be easily removed and substituted if damaged by an earthquake. At the same time, the dissipative systems have to be designed so as to reduce the overall damage occurred on the structure under destructive earthquakes.

A dissipation system can be classified according to the dissipation mechanisms (Soong & Dargush 1997): in friction, elastic-plastic, viscous and viscoelastic system. The coupling of a dissipation device with a traditional bracing system leads to a hybrid system which is particularly effective in earthquake-prone regions because of the increase in stiffness and, at the same time, the possibility to dissipate energy by hysteresis.

For Partially Restrained (PR) steel or steel-concrete composite frames with partial strength connections, which are characterised by large flexibility and remarkable joint damageability, the coupling of the frame with energy dissipation bracings can lead to one or more of the following benefits:

– reduction of the interstorey drifts and increase in dissipative capacity of the structure under low intensity earthquake ground motions, in order to comply with the serviceability limit state;
– increase in dissipative capacity of the structure under strong intensity earthquake ground motions, in order to comply with the ultimate limit state;
– the energy dissipation only occurs in some elements, the energy dissipation systems, which can be easily substituted if strongly damaged so that the damage of other structural elements can be prevented and the damageability limit states may be satisfied.

The paper investigates the structural behaviour of a partially restrained frame with steel columns and steel-concrete composite beams coupled with dissipative bracing tested at University of Ancona, Italy (Dall'Asta et al. 2005). The used viscoelastic damper is made of natural rubber layers with addition of black carbon filler glued to two steel plates. Its energy dissipation takes place when vibration causes a relative movement between interior and exterior plates. The merits of the system are that it is always active, there is no threshold to begin acting and it is made of materials which can generally reabsorb residual strains. The first part of the paper introduces the numerical model, which is validated against results of experimental tests. The results of some time-history analyses carried out on the same type of frame are then presented and critically discussed.

2 NUMERICAL MODELLING OF THE SEISMIC RESISTANT SYSTEM WITH PR COMPOSITE FRAMES AND DISSIPATIVE BRACINGS

The structural response of a frame system coupled with a viscoelastic dissipative bracing may be computed using an equivalent model where the frame is linked in parallel with the bracing system (Antonucci et al. 2001). In the limit case of a frame with negligible stiffness compared to that of the bracing (e.g. pinned beam-to-column joints), the response of the equivalent model will only depend on the stiffness and dissipative capacity of the viscoelastic damper, which is markedly larger than that of the bare frame. If the dissipative bracing is connected to continuous or PR steel or steel-concrete composite frames, neither the stiffness increase nor the hysteretic dissipation of energy on the frame cannot be neglected. A sophisticated numerical model should then be employed to account for the cyclic inelastic behaviour of the beam-to-column joint and for the viscoelastic properties of the dissipative system.

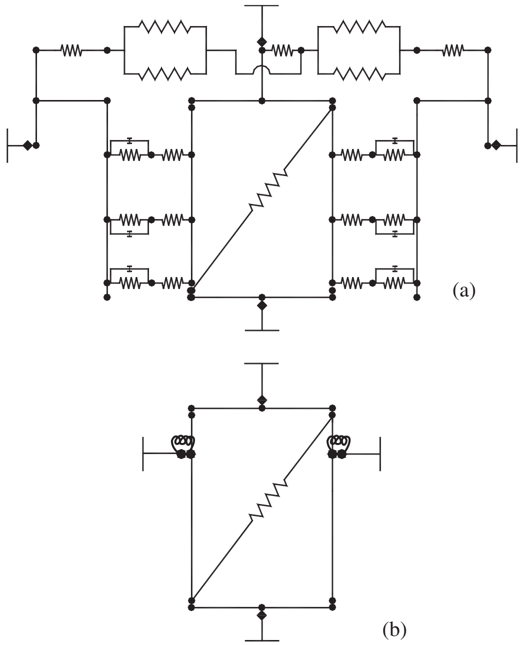

Figure 1. Mechanical models: mod1 (a), mod2 (b).

2.1 *Modelling of the PR frame*

The PR frame with steel-concrete composite beams and beam-to-column joints may be schematized by a finite element model (Abaqus – HKS 1998). The joint is modelled using a mechanic model defined as "spring models" or "component models", which is based on the use of both rigid and flexible elements appropriately linked to each other (Tschemmernegg et al. 1998, Faella et al. 2000, Amadio et al. 2004, Rassati et al. 2004). Two different models, herein after referred to as mod1 and mod2 (Fig. 1), have been adopted. The mod1 model corresponds to the most refined schematisation since every resistant part of the joint (column web panel, steel beam-to-column connection, reinforced concrete slab) is schematised using a set of trilinear axial springs with kinematik hardening. The mod2 model is instead a simplified model with one non-linear axial spring representing the column web panel and a non-linear rotational spring schematizing the steel beam-to-column connection together with the influence of the reinforced concrete slab. The mechanical properties of the latter spring have been obtained by assembling the properties of the mod1 springs under a monotonic symmetric rotation of the joint.

2.2 *Modelling of the viscoelastic dissipation device.*

In dynamic analyses the viscoelastic dissipative bracing may be schematised using the modal strain energy method (Shen et al. 1995). The simplified mechanical model representing this system is made of an elastic spring with stiffness K_b, which schematises the bare bracing system, linked in series with a Kelvin element made of a spring with elastic stiffness K_d connected in parallel with a viscous damper characterised by a loss factor η_d. The damping ratio ξ_i of the global frame-dissipative bracing system for the ith vibration mode is given by:

$$\xi_i = \xi_f + \frac{\left(\eta_d - 2\xi_f\right)}{2} \cdot \frac{\phi_i^T K_{bd} \phi_i}{m_i \omega_i^2} \tag{1}$$

where $\xi_f =$ damping ratio of the bare frame; $K_{bd} =$ the equivalent stiffness of the dissipative bracing; $m_i =$ the mass which vibrates with the ith mode; $\omega_i =$ the circular frequency of the ith vibration mode.

The stiffness K_d and the loss factor η_d for the viscoelastic device can be easily computed using Eq. (1) when the structure is made of frames equipped with dissipative bracings characterized, in dynamic conditions, by only a predominant vibration mode. The quantities in Eq. (1), which characterize the dynamic behaviour of the bare frame (ξ_f, ω_f) and of the equipped system (ξ, ω), can be measured on the basis of the free vibrations of the two systems. Hence the quantities K_d and η_d can be computed using the following equations drawn for single-degree-of-freedom systems when $K_b \gg K_d$:

$$K_d = m \cdot \left(\omega^2 - \omega_f^2\right); \quad \eta_d = \frac{2m\omega^2}{K_d} \cdot \left(\xi - \xi_f\right) \tag{2}$$

where $m =$ the entire mass; $\omega =$ the circular frequency of the whole equipped system; $\omega_f =$ the circular frequency of the bare frame; $\xi =$ the damping ratio of the equipped system.

3 VALIDATION OF THE MODEL: NUMERICAL – EXPERIMENTAL COMPARISONS

The finite element numerical model has been validated against the experimental results carried out by Dall'Asta et al. (2005). The analysed model is made of two one-storey two-bay steel-concrete composite moment resistant frames coupled with two dissipative bracings (Fig. 2). The single damper manufactured by T.A.R.R.C. is made of superposition of two rubber layers. The dampers were locked at the bottom flange of the steel profile and at the bracing system (Chevron-type bracing) so as to be subjected to shear forces under horizontal displacements of the frame. Geometrical and mechanical characteristics of the experimental set-up are displayed in Figures 2–3 and reported in Dall'Asta et al. (2005).

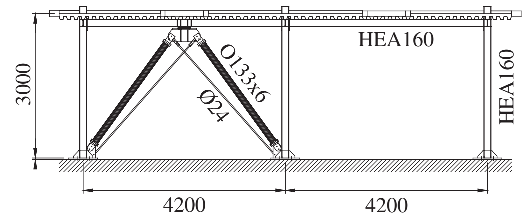

Figure 2. Experimental set-up: braced frame.

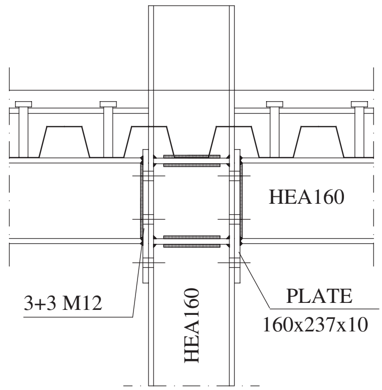

Figure 3. Experimental set-up: beam-to-column connection.

The structural system was subjected to some snap-back tests in order to investigate its dynamic behaviour in elastic phase. The tests were performed on the system without bracings first and on the equipped system later on. The numerical results have been carried out using the Abaqus FE code and both schematisations described in the previous paragraph with the mod1 and mod2 models for the beam-to-column connections. The viscoelastic device has been schematised using the "dashpot" element, which represents an elastic spring linked in parallel with a viscous damper, provided for by Abaqus. The characteristic parameters of the dashpot, K_d and η_d, have been evaluated through Eq. (2) by using the experimental data measured for the loss factor and proper period of both bare frame and equipped system (Dall'Asta et al. 2005). Figures 4–5 display some numerical-experimental comparisons for the dynamic response of the bare frame (Fig. 4) and equipped system (Fig. 5 with $K_d = 4653$ kN/m and $\eta_d = 0.653$) in free vibrations when an interstorey drift of 4 mm is applied. Significant differences between the mod1 and mod2 models arise only in the case of bare frame, where the former leads to the best results while the latter overestimates the natural period.

The numerical curves carried out using both mod1 and mod2 models are nearly coincident for the equipped system. In this case the dynamic response of the system is mainly affected by the stiffness and damping properties of the dissipative device. Numerical and experimental curves are very close and this demonstrates the accuracy of the numerical modelling proposed. Performed analyses have shown that the dynamic behaviour in elastic phase of the equipped system is affected by the stiffness of the frame. For the case under study, the beam-to-column connections with stiffened column web panel exhibit good stiffness and the response of the system is close to that of a frame with rigid joints where only the deformability of the web panel is considered. The numerical response carried out by assuming pinned joints is instead markedly different from that corresponding to the mod1 model and from the numerical curve obtained by neglecting the connection deformability.

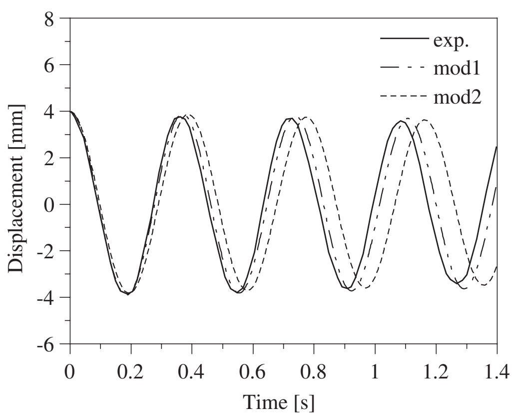

Figure 4. Bare frame: free vibration test at 4 mm.

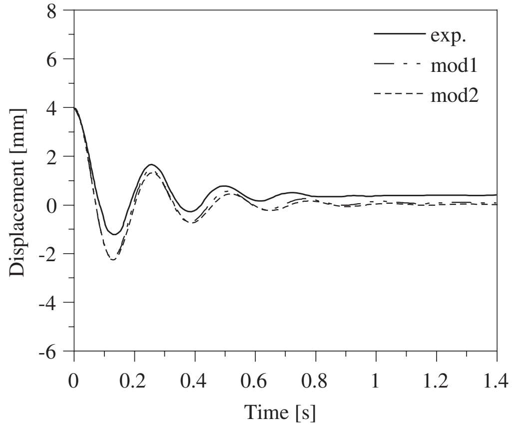

Figure 5. Equipped frame: free vibration test at 4 mm.

4 TIME-HISTORY ANALYSES

Some time-history analyses have been carried out in order to investigate the dynamic behaviour of the frames equipped with dissipative bracings. The numerical model introduced in the previous paragraph have been subjected to two different recorded earthquake ground motions (El Centro 1940 and Friuli, Tolmezzo 1976) and one artificial shaking compatible with the elastic spectra proposed by the Eurocode 8 for ground type A (CEN 1996). Three peak ground accelerations have been considered for each of the ground motions: $a_g = 0.15$ g, $a_g = 0.30$ g and $a_g = 0.40$ g (the last representing the design acceleration of the bare frame). Different viscoelastic dissipative systems: equipped frame 1 to 4, corresponding to devices with 1 to 4 rubber layers respectively, have also been also considered. The stiffness K_d and the loss factor η_d of the devices are proportional to the values computed for the equipped frame previously introduced in the numerical-experimental comparisons.

Figures 6–8 display the trend in time of the energy dissipated by the frames under the El Centro 1940 earthquake ground motion scaled on a peak ground acceleration $a_g = 0.40$ g. The benefit in terms of total input energy for the equipped frames is evident. The equipped and bare frames dissipate energy according to different mechanisms. The dissipated energy is mainly viscous for the former frames. If the mechanical characteristics of the dissipative device are adequate the equipped frame remains in elastic phase even under a peak ground acceleration $a_g = 0.40$ g as depicted in Figure 8 for the equipped frame 2. If more rubber layers are introduced (equipped frames 3 and 4) only a slight reduction of the input energy is observed and the systems remain in the elastic phase like the equipped frame 2. Instead the bare frame enters the plastic phase even for $a_g = 0.15$ g and dissipates

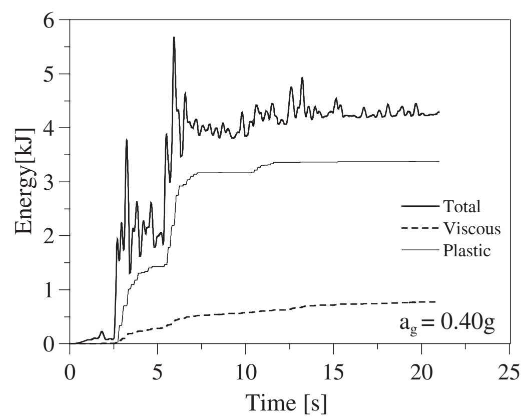

Figure 6. Bare frame: time history of dissipated energy.

mainly hysteretic plastic energy. The plasticization occurs in the column web panel of the beam-to-column connection first, and can involve also the connection between column and foundation.

The trend in time of the top floor displacements referred to both the bare frame and the equipped frame 2 under the earthquake ground motion recorded in Friuli in 1976 are reported in Figures 9–10 at $a_g = 0.15$ g and $a_g = 0.40$ g respectively. The figures show how the introduction of the dissipative bracing into the moment resisting frame leads to a considerable reduction of the maximum displacement at every level of the peak ground acceleration a_g. In the case of bare frame the column plasticization occurs for $a_g = 0.40$ g and leads to a permanent displacement of about 20 mm experienced by the structure (Fig. 10). Conversely, the equipped frame exhibits small displacements even for the high intensity shaking.

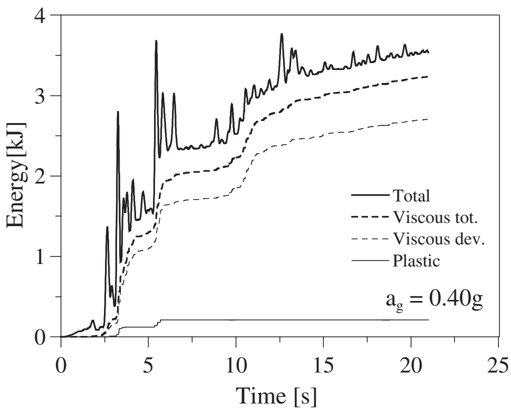

Figure 7. Equipped frame 1: time history of dissipated energy.

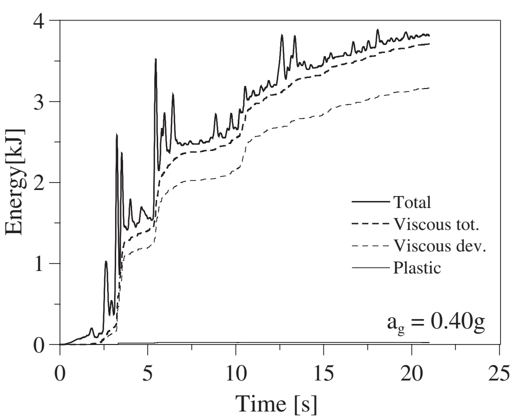

Figure 8. Equipped frame 2: time history of dissipated energy.

In the Figures 11–12 the maximum top floor displacement and the maximum base shear computed for the different frames under the earthquake ground motion recorded at El Centro in 1940 are displayed. The number of the rubber layers which characterize the dissipative device of the frames is reported in abscissa. The figures show that the use of dissipative devices with one or two rubber layers represents the best solution. Thanks to the enhanced damping ratio of the systems, the reduction of both the maximum displacement and the maximum base shear with respect to the values computed for the bare frame is notable and the plastic damage of the structures is limited (Figs. 7–8). The improvement of the mechanical properties of the viscoelastic device, introducing more rubber layers

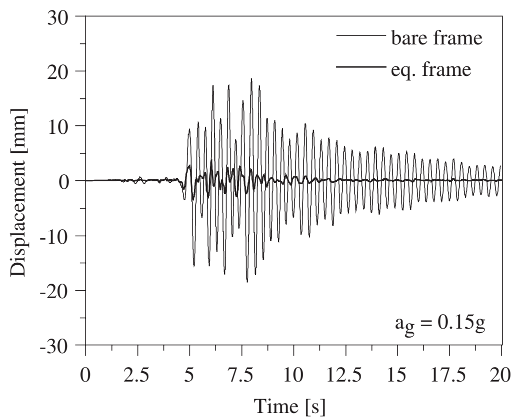

Figure 9. Time-history of top floor displacements under the earthquake ground motion recorded in Friuli, 1976 at $a_g = 0.15$ g.

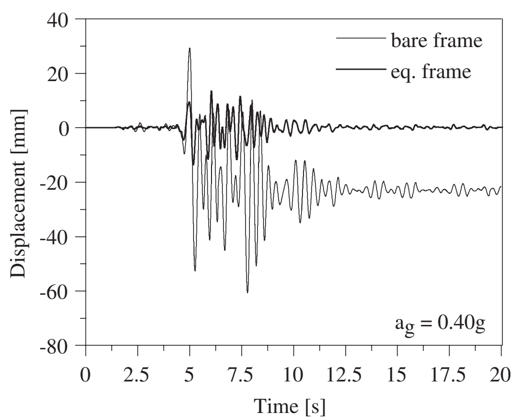

Figure 10. Time-history of top floor displacements under the earthquake ground motion recorded in Friuli, 1976 at $a_g = 0.40$ g.

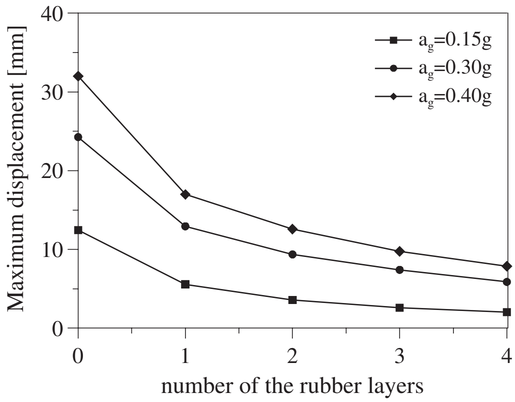

Figure 11. Maximum displacement versus the number of the rubber layers of the dissipative device.

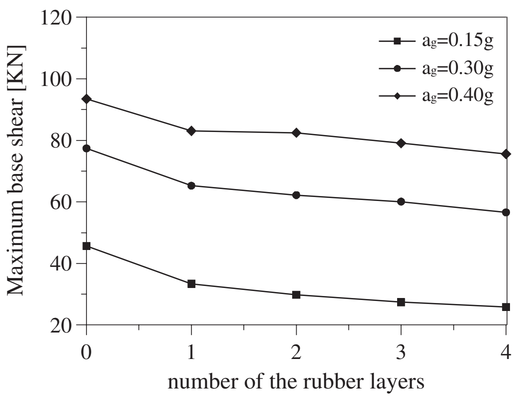

Figure 12. Maximum base shear versus the number of the rubber layers of the dissipative device.

Table 1. Maximum top floor displacement computed for bare and equipped frame with semi-rigid beam-to-column connection.

		bare frame		equipped frame 2	
Earthquake		mod1 [mm]	mod2 [mm]	mod1 [mm]	mod2 [mm]
EL Centro	0.40 g	32.0	33.1	12.4	12.2
	0.30 g	24.5	26.5	9.3	9.1
	0.15 g	12.6	17.1	3.5	3.4
Friuli	0.40 g	60.7	59.8	13.7	14.4
	0.30 g	34.0	36.8	10.2	10.6
	0.15 g	18.7	17.6	3.7	3.7
Artificial	0.40 g	43.4	34.5	12.7	12.7
	0.30 g	30.1	27.6	9.6	9.9
	0.15 g	16.5	17.7	3.3	3.5

Table 2. Maximum top floor displacement computed for equipped frame with rigid and pinned beam-to-column connection.

Earthquake		rigid joint [mm]	pinned joint [mm]
EL Centro	0.40 g	11.5	15.7
	0.30 g	8.6	11.2
	0.15 g	3.2	4.1
Friuli	0.40 g	12.7	24.9
	0.30 g	9.5	18.6
	0.15 g	3.4	5.4
Artificial	0.40 g	12.0	20.4
	0.30 g	9.6	15.3
	0.15 g	3.2	4.6

(three or four), allows the structures to remain in elastic phase. However, no significant increase in the structural performances can be noted. The same behaviour has been obtained under the other two earthquake ground motions.

In order to evaluate the influence of the beam-to-column connection properties, on the seismic behaviour of both the bare and equipped frame, numerical comparisons using different schematizations for the joints have been carried out. Tables 1 and 2 report the values of the maximum top floor displacement computed as result of the time-history analyses in the cases of bare and equipped frame 2 with semi-rigid (Fig. 3), rigid and pinned connections respectively. The results obtained using the mod1 and mod2 models, for the semi-rigid connections are compared to check whether the more effective and less time consuming computational schematization, the mod2 model, is adequate to predict the actual behaviour of the beam-to-column connection.

Table 1 show that the use of the mod 2 (simplified) model leads to reasonably accurate evaluation of the solution in terms of displacements (an accurate evaluation of base shear has also been obtained). It is important to point out that the contribution of the inherent stiffness of the bare frame is crucial in order to reduce the maximum displacements of the equipped frame. Table 2, in fact, highlights that the displacements computed in the case of pinned beam-to-column connections would be much larger than those of the actual PR frame, thus the correct behaviour of the dissipative bracing may be prevented by the possible failure of the devices under large displacement. Furthermore, for the equipped frame, the response of the adopted semi-rigid joint (table 1) is very close to that of the rigid joint (table 2).

5 CONCLUSIONS

The paper presents the main outcomes of the numerical analyses carried out on some PR steel-concrete composite frame systems coupled with viscoelastic dissipative bracings. Numerical-experimental comparisons demonstrate the accuracy of the model where the properties of the viscoleastic device have been computed using experimental data. Some time-history analyses carried out under different earthquake ground motions point out that considerable reduction of both maximum displacements and structural damage can be achieved using the dissipative bracings. Such devices, in fact, augment the dissipated viscous energy and reduce both the plastic hysteretic energy and the total input energy. The contribution in terms of stiffness given by the PR composite frame into the equipped system is relevant especially under high intensity earthquake ground motions, where a reduction in the maximum horizontal displacement is needed in order to prevent the failure of the dissipation device. The comparison between the outcomes of the dynamic analyses carried out by modelling the composite beam-to-column connection through a component model and a more simplified model shows that the latter one is suitable to predict the most significant parameters such as maximum base shear and top floor displacement with reasonable accuracy.

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