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Article Title: Development of nonlinear analytical model and seismic analyses of a steel frame with self-centering devices and viscoelastic dampers

Year of publication: 2011

Link to published article:
http://dx.doi.org/10.1016/j.compstruc.2010.08.013

Development of nonlinear analytical model and seismic analyses of a steel frame with self-centering devices and viscoelastic dampers

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ABSTRACT

This paper highlights the role of advanced structural analysis tools on the conception of high-performance earthquake-resistant structural systems. A new steel frame equipped with self-centering devices and viscoelastic dampers is described. A prototype building using this frame is designed and a detailed nonlinear analytical model for seismic analysis is developed. Seismic analyses results show the effectiveness of the proposed frame to enhance structural and non-structural performance by significantly reducing residual drifts and inelastic deformations, and by reducing drifts, total floor accelerations and total floor velocities. These results are the basis for further studies aiming to develop design methods and criteria for the proposed high-performance frame.

Keywords: Self-centering, viscoelastic dampers, steel MRF, seismic design, high-performance, damage-free

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1. Introduction

Conventional structural seismic-resistant systems, such as steel moment resisting frames (MRFs) [1] or concentric braced frames (CBFs) [2], are currently designed [3,4] to experience significant inelastic deformations and form a global plastic mechanism under moderate-to-strong earthquakes. Such a design philosophy [3,4], which results in inelastic deformations, has several advantages including economy and reduced forces developed in structural members and foundation due to inelastic softening. However, inelastic deformations result in damage, residual drifts and economical losses such as repair costs, costly downtime whilst the building is repaired and cannot be used or occupied, and, perhaps, building demolition due to the complications associated with repairing and straightening large residual drifts [5].

Modern resilient societies demand structural systems able to achieve high performance, i.e., no damage under small and moderate earthquakes, and, little damage which can be repaired without loss of building operation under strong earthquakes [6]. Performance-based seismic design is expected to focus on modern energy dissipation systems such as passive dampers and self-centering (SC) devices [6,7]. If carefully designed, these systems will slightly increase the initial building design cost and significantly reduce the great life-cycle cost related to earthquake damage [6].

Viscoelastic (VE) dampers provide supplemental stiffness and damping and when combined with flexible MRFs so that they would carry a large fraction of the lateral dynamic forces, they are becoming very effective in reducing peak structural response [8]. Dampers made of high-damping elastomer have been tested and found to exhibit a modest energy dissipation capacity but less sensitivity to frequency and temperature compared to conventional VE dampers [9]. It has been though found impossible to design elastomeric dampers and steel MRFs at practical sizes and cost for the building to remain elastic under strong earthquakes [10]. Both VE and elastomeric dampers transfer high forces on beams and columns of the MRF. These forces cannot be used directly in conventional capacity design rules since they are out of phase with the peak structural displacements. Recent studies by Karavasilis *et al*. [11-14] designed and tested steel MRFs with compressed elastomer dampers providing VE damping under small-to-moderate amplitudes of deformation, friction-based damping under high amplitudes of

deformation and a limit on the peak damper force. However, friction behavior results in permanent damper deformation and residual drifts in the structure following strong earthquakes.

Research efforts [15-17] developed new earthquake-resistant systems called self-centering (SC) systems with the potential to eliminate residual drifts and inelastic deformations under strong earthquakes. SC systems exhibit a softening flag-shaped force-drift behavior due to: (1) separations developed in structural interfaces (e.g., beam-to-column connections); (2) elastic pretensioning elements (e.g., high strength steel tendons); and (3) energy dissipation elements (EDs: friction-based, yielding or viscous) which are activated when separation in structural interfaces initiates. A recent work [18] proposed SC systems with a visco-plastic ED (similar to the damper tested and modeled in [11-14]) as a better alternative to SC systems with yielding or friction-based EDs. Shape memory alloys (SMA) materials also exhibit stable flag-shaped hysteresis and have been used to develop SC devices [19,20]. SC systems avoid structural damage since they eliminate inelastic deformations and residual drifts. However, they exhibit drift and acceleration demands comparable to those of conventional bilinear yielding systems of the same period and strength [21] and therefore, provide conventional performance in terms of non-structural damage.

This paper highlights the role of advanced structural analysis tools on the conception of high-performance earthquake-resistant structural systems. The concept of a new high-performance steel MRF equipped with self-centering viscoelastic damping devices (SCVDs) is described. SCVDs result from a novel and strategic in series combination of VE dampers and SC devices. A prototype building using the proposed MRF is designed and a nonlinear analytical model for seismic analysis is developed. Extensive seismic analyses results show the effectiveness of the proposed MRF to enhance structural and non-structural performance by significantly reducing residual drifts and inelastic deformations, and by reducing drifts, total floor accelerations and total floor velocities. These results are the basis for further studies aiming to develop design methods and criteria for the proposed high-performance MRF.

2. Concept and design of self-centering viscous damping devices (SCVD)

The SCVD results from a novel and strategic in series combination of a VE damper with a SC device. Fig. 1.a shows a simple mechanical analog of the SCVD where a generalized Maxwell (GM) model, representing a VE damper, is connected in series with a SC device consisting of a pretensioning elastic tendon and a friction-based ED. The design parameters of the SCVD are the initial pretension force in the tendon, F_t, the stiffness of the tendon, k_t, the force required to activate the friction-based ED, F_{ED}, and the properties of the VE damper, namely, the storage stiffness, k_d, and the loss factor, η. Both k_d and η can be easily obtained from displacement-controlled sinusoidal tests, i.e., k_d can be calculated as the ratio of the force at maximum displacement to the maximum displacement, while the loss factor is determined as follows:

$$\eta = \frac{ED}{2 \cdot \pi \cdot ES} \tag{1}$$

where *ED* is the energy dissipated per cycle of sinusoidal loading and *ES* is the maximum strain energy stored during a cycle of sinusoidal loading. *ED* can be determined by integrating the hysteresis loops and *ES* can be calculated from k_d and maximum displacement.

Fig. 1.b shows the hysteresis of the SCVD under earthquake loading. Under small amplitudes of deformation, *d*, there is no separation across the SC device due to the forces F_t and F_{ED}, and the SCVD behaves as a conventional VE damper offering VE (velocity-dependent) damping. When the force in the VE damper becomes larger than the sum of F_t and F_{ED}, separation (relative movement) is taking place in the SC device. This separation results in significant softening since the stiffness of the SC is now equal to the tendon stiffness assuming a zero stiffness of the ED during sliding. The SCVD now offers hysteretic (friction-based) damping. Upon unloading, the SCVD exhibits VE behavior until the ED will be activated again and eventually returns back to the initial position due to the re-centering force of the tendon. An optimum design of the SCVD device includes a force F_t equal to F_{ED} in order to maintain full re-centering capability and maximum hysteretic energy dissipation capacity, as well as a high value of η in order to amplify VE damping. SCVD strategically combines the advantages of the compressed elastomer damper designed and tested in [11-14] with the advantages of SC systems and offers: (1) VE damping under small-to-moderate amplitudes of deformation; (2)

hysteretic damping at large amplitudes of deformation; (3) limit on the peak device force; and (4) full re-centering capability. In addition, the peak force of the SCVD can be directly used in conventional seismic design capacity rules since it is velocity independent.

The SC device of the SCVD can be realized in different configurations. Fig. 2.a shows a possible implementation similar to that in [17] using: (1) two steel elements of the same length but with sections of different size so that the member with the smaller section can be inserted into the member with the larger section (the exterior element has an inverted U-shape, while the interior element has a hollow rectangular section); (2) end plates that are attached but not connected to the steel elements; and (3) pretensioning tendons anchored on the end plates so they can hold the plates and the steel members in place. Energy dissipation is provided with friction at the upper contact interface of the steel elements due to a normal force applied by tightening bolts. The relative displacement at the contact interface is enabled with a slot cut on the interior steel element.

Alternatively, Fig. 2.b shows another possible implementation [19] using two steel elements and two sets of SMA wires anchored in such a way so that relative displacement between the steel elements will produce tension in one of the two sets of SMA wires. A slot cut on the bottom steel element permits the relative displacement between the two steel elements while additional energy dissipation is provided with friction at the interface.

3. High-performance steel MRF with SCVDs

3.1 Prototype building

Fig. 3.a shows the plan view of the 5-story, 3-bay by 3-bay prototype office building used for the study. The building has two 3-bay identical perimeter steel MRFs (one at each side) to resist lateral forces in the N-S direction. The design study focuses on one perimeter MRF. This MRF is designed either as a conventional MRF or as MRF with SCVDs in order to compare their seismic response. The SCVDs are supported by braces and connected to the bottom flange of the beam of the steel MRF as shown in Fig. 3.b.

The yield stress of structural steel is assumed to be equal to 275 MPa. The dead

and live gravity loads considered in the design are selected according to Eurocode 1 [22]. The design seismic action, referred to herein as design basis earthquake (DBE), has a return period equal to 475 years and is expressed by the Type 1 elastic response spectrum of the Eurocode 8 (EC8) [3] with a peak ground acceleration equal to 0.3g and ground type B (average shear wave velocity between 360 and 800 m/s.). The program SAP2000 [23] is utilized for designing the steel MRFs according to Eurocode 3 [24] and EC8 [3].

The 2D SAP2000 model used for design is based on the centerline dimensions of the MRFs without accounting for the finite panel zone dimensions. A "lean-on" column is included in the SAP2000 model to account for the P-Δ effects of the vertical loads acting on the gravity columns in the tributary plan area (half of the total plan area) assigned to the perimeter MRF. This column is pinned at its base, continuous over the height of the building, and carries the vertical loads acting on the gravity columns. The cross-section area and flexural stiffness of the lean-on column is based on the summation of the areas and flexural stiffnesses of the gravity columns. The column base and beam-to-column connections of the MRFs are assumed to be fully rigid. A rigid diaphragm constraint is imposed at the nodes of each floor level.

3.2 Design of conventional steel MRF

The perimeter MRF of the building is designed as a conventional MRF using the modal response spectrum analysis procedure of EC8. The MRF satisfies Ductility Class High [3] by using compact Class 1 cross-sections [24] and hence, the behavior (or "strength reduction") factor is equal to $q=5\bullet\alpha_u/\alpha_1=6.5$, where α_u/α_1 is the overstrength factor with a recommended value of 1.3 for multi-bay multi-story steel MRF. The displacement behavior (or "displacement amplification") factor is equal to q, i.e., EC8 adopts the equal-displacement rule to estimate peak inelastic drifts. These drifts are then used to check second order (P-Δ) effects. Additionally, EC8 imposes a serviceability limit on the peak story drift, θ_{max}, under the frequently occurred earthquake (FOE) with a return period equal to 95 years. The FOE has intensity equal to 40% (reduction factor v=0.4 [3]) of the intensity of the DBE and the associated θ_{max} is equal to 0.75% assuming ductile non-structural components. The MRF satisfies the strong column-weak beam capacity design rule of EC8 [3], the beam-to-column connections are designed to be fully rigid, and the panel zones are strengthened with doubler plates to avoid yielding [3]. A

strength-based design with q=6.5 under the DBE was first performed. However, beams and columns from strength-based design had to be increased to satisfy the serviceability drift requirements under the FOE. The final sections were found iteratively, i.e., by decreasing the value of q, designing the MRF and then checking drifts under the FOE.

3.3 Design of high-performance steel MRF with SCVDs

The response of steel MRFs with SCVDs is complex due to the different sources of energy dissipation (VE and hysteretic damping) and cannot be predicted using results and conclusions applicable to SC systems [21] or systems with VE dampers [8]. However, a simplified conservative design approach that neglects the supplemental damping provided by the VE material and makes use of modal response spectrum analysis has been adopted for this preliminary investigation.

The SAP2000 model used for design includes the members of the steel MRF (modeling details described in Section 3.1), the supporting braces (pinned connected) and a horizontal spring representing the storage stiffness of the VE damper k_d. The SC device is assumed to be rigid before separation initiates. The q factor is equal to 6.5 to enable a direct comparison with the conventional steel MRF and defines the force level at which separation in the SC initiates. The design is performed iteratively by selecting values of k_d at each story so that they produce a uniform distribution of drift demands based on modal response spectrum analysis. Braces are sized to be stiff enough so that the story drift produces SCVD deformation rather than brace deformation. For all stories, a ratio of total brace horizontal stiffness per story to damper stiffness equal to 10 is adopted. Beams and columns are designed according to capacity design rules so that they do not yield under the DBE. They also satisfy the strong column-weak beam capacity design rule of EC8 [3]. Beam-to-column connections are designed to be fully rigid and panel zones are strengthened with doubler plates [3]. The strength-based design of the MRF with SCVDs under the DBE was found to satisfy the serviceability limit on θ_{max} under the FOE.

The VE damper area, A_d, at each story is determined by $A_d=(k_d \cdot t_d)/G'(\omega_1, temp)$, where t_d is the thickness of the VE damper, $G'(\omega_1, temp)$ is the storage shear modulus of the VE material, ω_1 is the first-mode cyclic frequency of the MRF with SCVDs and *temp* is the design ambient temperature (considered equal τo 24 ^{o}C). The VE material used in this study is the ISD-110 material studied by Fan [25].

The SCVD device is designed with the configuration shown in Fig. 2.a. The activation force in the SC device at each story (i.e., F_t+F_{ED}) is designed equal to the force in the spring of the SAP2000 model. F_{ED} is designed equal to F_t. The tendons are cables made of composite polymers tested by Christopoulos et al. [17] and have cyclic modulus E_t equal to 93 GPa and elongation capacity close to 2.5%. With the F_t known, the area of the tendons A_t at each story is determined by A_t=F_t/σ_o, where σ_o is the initial pretension stress in the tendon material. The stiffness k_t of the tendon is determined by $(A_t \cdot E_t)/L_t$. The length of the device is designed equal to 5 m by considering the tendon elongation capacity and the expected drifts under the maximum considered earthquake (MCE). The MCE has return period equal to 2500 years and intensity equal to 150% of the intensity of the DBE.

3.4 Design details

Table 1 compares properties of the conventional MRF and the MRF with SCVDs. The table lists the column cross-sections, beam cross-sections, steel weight, fundamental period of vibration (T_1), and equal-displacement rule estimates of θ_{max} under the FOE and the DBE. Table 1 shows that significant reductions in steel weight and higher performance in terms of θ_{max} can be achieved by using SCVDs. It is emphasized that θ_{max} for the MRF with SCVDs are expected to be lower than those listed in Table 1 due to the supplemental damping of the VE material which was not considered in the design process (Section 3.3). Table 2 provides the properties of the SCVD designs at each story. The storage shear modulus $G'(\omega_1$=4.16 rad/s, *temp*=24 ^oC) of the VE material is equal to 1086 kPa [25]. Table 2 shows that both components (VE material and tendons) of the SCVDs can be designed to have practical sizes.

Fig. 4 compares the base shear coefficient (V/W) – roof drift (θ_r) curves of the two frames obtained from nonlinear cyclic static analysis using analytical models described in the next section. V is the base shear force and W is the seismic weight. The analysis was performed at two cycles with roof drift amplitudes equal to 1.5% and 2.5% of the total building height. Fig. 4 shows that the MRF with SCVDs has less strength, smaller yield displacement and less energy dissipation capacity than the conventional MRF. For the MRF with SCVDs, softening begins at θ_r equal to 0.5% due to separations in the SC.

Beams and columns remain elastic for θ_r lower or equal to 1.5%; indicating that the MRF with SCVDs sustains no damage and exhibit full re-centering capability (minimal residual drifts) under the DBE (see θ_{max} estimates in Table 1). For θ_r higher than 1.5%, the MRF with SCVDs sustains inelastic deformations and possible residual drifts which are though significantly smaller than those sustained by the MRF.

4. Nonlinear dynamic analyses

4.1 Analytical model development

2D nonlinear analytical models of the conventional MRF and the MRF with SCVDs were developed for nonlinear dynamic analysis using OpenSEES [26]. A distributed plasticity force-based beam column was used to model beams and columns. An elastic beam column element was used to model the lean-on column that accounts for the P-Δ effects of the vertical loads on the interior gravity columns of the prototype building. An elastic truss element was used to model the braces. The panel zones of the beam-column joints were modelled as proposed by Herrera *et al*. [27]. The SC devices were modelled using a zero-length element exhibiting flag-shaped hysteresis. This zero-length element has a high initial stiffness representing the rigidity of the SC device before activation of the ED. This element is suitable for the SC device shown in Fig. 2.a. However, a different self-centering flag-shaped model is needed for the SMA wires of the SC device shown in Fig. 2b [19].

The shear stress (τ) – shear strain (γ) behavior of the VE material was modelled using the GM model shown in Fig. 5. Under harmonic loading of cyclic frequency ω, the GM model provides storage shear modulus equal to

$$G'(\omega) = G_0 + \sum_{m=1}^{n} \frac{(\omega\beta_m)^2}{1+(\omega\beta_m)^2} \cdot G_m \tag{2}$$

and loss factor equal to

$$\eta(\omega) = \frac{(\omega\beta_0)G_0 + \sum_{m=1}^{n} \frac{(\omega\beta_m)}{1+(\omega\beta_m)^2} \cdot G_m}{G_0 + \sum_{m=1}^{n} \frac{(\omega\beta_m)^2}{1+(\omega\beta_m)^2} \cdot G_m} \tag{3}$$

These equations can be used to calibrate the GM model against experimentally obtained values of G' and η for different cyclic frequencies. Fan [25] calibrated the GM model of Fig. 5 with m=4 for the ISD-110 VE material and the resulting values of the parameters are given in Table 3. Fig. 6 compares the experimentally obtained values of G' and η at 24 ^{o}C [24] with those obtained by using the parameters of Table 3 in Equations (2) and (3). Acceptable agreement is observed. The GM model was represented in OpenSEES as a combination of linear springs and dashpots. The stiffness of the springs of the GM model was determined by multiplying G_0 and G_m with the ratio A_d/t_d of each story to transform τ-γ behavior to force ($F=\tau \cdot A_d$) - deformation ($d= \gamma \cdot t_d$) behavior.

The Newmark method with constant acceleration was used to integrate the equations of motion. The Newton method with tangent stiffness was used for the solution algorithm. A Rayleigh damping matrix was used to model inherent 2% critical damping at the first two modes of vibration. A diaphragm constraint was imposed on the nodes of each floor level. A nonlinear load control static analysis under the gravity loads was first performed and then, the nonlinear dynamic earthquake analysis was executed. Each dynamic analysis was extended beyond the actual earthquake time to allow for damped free vibration decay and correct residual drift calculation.

4.2 Earthquake ground motions

An ensemble of 20 earthquake ground motions recorded on ground type B were used in 2D nonlinear dynamic analyses to evaluate the performance of the conventional MRF and the performance of the MRF with SCVDs. None of the ground motions exhibit near-fault forward-directivity effects. The ground motions were scaled to the DBE level using the scaling procedure of Somerville [28]. Table 4 provides the scale factors and information on the 20 ground motions. Fig. 7 compares the DBE elastic response spectrum of EC8 with the mean (μ) and mean plus/minus one standard deviation ($\mu+\sigma$) spectra of the DBE ground motions. The amplitudes of the DBE ground motions were further scaled by 0.4 and 1.5 to represent FOE and MCE ground motions, respectively.

5. Seismic response results

The seismic performance of the MRFs is quantified in terms of the θ_{max}; the peak residual story drift, $\theta_{r\text{-}max}$; the peak total floor acceleration, a_{max}; and the peak total floor velocity, v_{max}. θ_{max} and $\theta_{r\text{-}max}$ quantify structural damage and damage of drift-sensitive non-structural components, while a_{max} and v_{max} quantify damage of rigidly attached equipment and non-rigidly attached block-type objects, respectively [29].

Fig. 8 compares the roof drift time histories of the conventional MRF and the MRF with SCVDs under the HSP ground motion scaled to the DBE (Table 4). Near the end of the time history the MRF with SCVDs oscillates around the origin, indicating negligible residual drift, while the conventional MRF experiences appreciable residual drift due to inelastic deformations.

Fig. 9 shows μ and $\mu+\sigma$ values of θ_{max} in both frames under the earthquake ground motions of Table 4 scaled to the FOE, DBE and MCE. Both frames show uniform height-wise distributions of θ_{max}. The MRF with SCVDs exhibits significantly higher, higher and slightly higher performance than the conventional MRF under the FOE, DBE and MCE, respectively. The associated decreases in θ_{max} are 65%, 24% and 8%. For the MRF with SCVDs, θ_{max} is 0.32% (μ) and 0.42% ($\mu+\sigma$) under the FOE, 1.35% (μ) and 1.80% ($\mu+\sigma$) under the DBE, and 2.30% (μ) and 2.90% ($\mu+\sigma$) under the MCE. The corresponding median values are 0.31%, 1.30% and 2.30%. The above mean (μ) and median values of θ_{max} indicate performance levels of immediate occupancy (IO), close to IO and life safety (LS) under the FOE, DBE and MCE, respectively [30].

Fig. 10 shows μ and $\mu+\sigma$ values of $\theta_{r\text{-}max}$. Under the FOE, both frames experience anticipated zero residual drifts; indicating IO [30]. Under the DBE and MCE, the MRF with SCVDs shows a significantly higher performance than the conventional steel MRF. The associated decreases in $\theta_{r\text{-}max}$ are approximately 80% and 70%. For the MRF with SCVDs, $\theta_{r\text{-}max}$ is 0.05% (μ) and 0.15% ($\mu+\sigma$) under the DBE, and 0.2% (μ) and 0.36% ($\mu+\sigma$) under the MCE. The corresponding median values are 0.02% and 0.19%. The above mean (μ) and median values of $\theta_{r\text{-}max}$ indicate performance levels of IO under the FOE and close to IO under the DBE and MCE [30].

Fig. 11 shows μ and $\mu+\sigma$ values of a_{max}. The MRF with SCVDs shows significantly higher, higher and slightly higher performance than the conventional MRF under the FOE, DBE and MCE, respectively. The associated decreases in a_{max} are

approximately 49%, 21% and 9%; indicating less damage in rigidly attached non-structural components. For the MRF with SCVDs, a_{max} is 1.93 (μ) and 2.76 ($\mu+\sigma$) m/s.^{2} under the FOE, 5.29 (μ) and 7.29 ($\mu+\sigma$) m/s.^{2} under the DBE, and 7.7 (μ) and 10.37 ($\mu+\sigma$) m/s.^{2} under the MCE. The corresponding median values are 1.63, 4.82 and 7.28 m/s.^{2}

Fig. 12 shows μ and $\mu+\sigma$ values of v_{max}. The MRF with SCVDs shows significantly higher performance under the FOE and higher performance under the DBE and MCE than the conventional MRF. The associated decreases in v_{max} are approximately 53%, 29% and 22%; indicating less damage in non-rigidly attached block-type objects. For the MRF with SCVDs, v_{max} is 0.26 (μ) and 0.34 ($\mu+\sigma$) m/s. under the FOE, 0.71 (μ) and 0.92 ($\mu+\sigma$) m/s under the DBE, and 1.0 (μ) and 1.35 ($\mu+\sigma$) m/s. under the MCE. The corresponding median values are 0.24, 0.65 and 0.97 m/s.

The response of the MRF with SCVDs depends on the value of q which defines the force level at which SCVDs are transformed from VE dampers to SC devices. A decrease in q increases the force at which activation in the SC device initiates, and therefore, increases the velocity-dependent energy dissipation capacity of SCVDs. Such a design results in lower values of θ_{max} than those presented above for the DBE and MCE but also increases the peak force developed in SCVDs and as a result, increases the sizes of beams and columns which are designed according to capacity design rules. Increasing member sizes to achieve higher performance seems acceptable since the MRF with SCVDs (designed with q=6.5) is 30% lighter in steel weight than the conventional MRF. More work is needed in order to show how the seismic performance and weight of the MRF with SCVDs are affected by the value of q.

6. Summary and conclusions

The concept of a new high-performance steel moment resisting frame (MRF) equipped with self-centering viscoelastic damping devices (SCVDs) is described. SCVDs result from a novel and strategic in series combination of viscoelastic dampers and self-centering devices. SCVDs offer: (1) viscoelastic damping at small amplitudes of deformation; (2) friction-based damping at large amplitudes of deformation; (3) limit on the peak device force; and (4) full re-centering capability. In addition, the peak force of

the SCVD can be directly used in conventional capacity seismic design rules since it is velocity independent.

A prototype building using the proposed MRF is designed and a detailed nonlinear analytical model for seismic analysis is developed. Seismic analyses results show the effectiveness of the proposed MRF to enhance structural and non-structural performance by significantly reducing residual drifts and plastic deformations, and by reducing drifts, total floor accelerations and total floor velocities.

In terms of the peak story drifts and residual drifts, the performance of the proposed steel MRF is found close to immediate occupancy under the design earthquake action (return period equal to 475 years) of Eurocode 8. Under the same seismic action, the peak total floor accelerations and velocities are reduced approximately 20% and 30% compared to a conventional steel MRF. Moreover, the proposed MRF results in significant reductions in steel weight (i.e., 30%) compared to a conventional MRF.

The promising results presented herein are the basis for further studies aiming to develop design procedures and criteria for the proposed high-performance steel MRF. Design equations able to predict peak story drifts by considering the effect of both viscoelastic (velocity-dependent) damping and hysteretic (friction-based or yielding-based) damping are needed. Performance-based design of the proposed system can be optimized by identifying the optimum value of the behavior factor q which defines the force level at which SCVDs are transformed from viscoelastic dampers to self-centering devices and which produce the desired performance under different earthquake intensities. Emphasis should be also given to the collapse capacity of the proposed system by considering P-Δ effects under seismic intensities even higher than the MCE, and by investigating the ultimate response of the SCVDs. Finally, an experimental research program is needed to validate design procedures/criteria and help move the proposed high-performance MRF into practice.

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Frame	Columns		Beams		Braces		Steel Weight (kN)	T_1 (sec)	θ_{max} (%)	
	Story	Section	Story	Section	Story	A_b (m^2)			FOE	DBE
Conventional MRF	1	HEB400	1	IPE450	-	-	180	1.70	0.72	1.75
	2	HEB400	2	IPE450						
	3	HEB400	3	IPE400						
	4	HEB360	4	IPE400						
	5	HEB360	5	IPE360						
MRF with SCVDs	1	HEB280	1	IPE270	1	8.28e-4	124	1.51	0.60	1.50
	2	HEB280	2	IPE270	2	6.54e-4				
	3	HEB280	3	IPE270	3	6.54e-4				
	4	HEB240	4	IPE240	4	3.90e-4				
	5	HEB240	5	IPE240	5	3.02e-4				

Table 1: Properties of conventional MRF and MRF with SCVDs

Story	k_d (kN/m)	t_d (m)	A_d (m^2)	$P_t + P_{ED}$ (kN)	A_t (m^2)	k_t (kN/m)
1	29691	0.04	1.09	371	2.08e-4	3870
2	34015	0.04	1.25	350	1.51e-4	2810
3	30437	0.04	1.12	313	1.35e-4	2510
4	22449	0.04	0.83	231	9.96e-5	1850
5	17103	0.04	0.63	176	7.59e-5	1410

Table 2: Properties of SCVDs

G_0 (kPa)	G_1 (kPa)	G_2 (kPa)	G_3 (kPa)	G_4 (kPa)
191.4	2713.7	1356.3	11018.4	409.5
β_o	β_1	β_2	β_3	β_4
0.1225	0.040	0.1356	0.0061	1.2394

Table 3: GM model parameters [29]: G in kPa and τ in s. (NOT t all beta)

Earthquake	Station	Component	Magnitude (M_w)	Distance (km)	Scale factor		
					FOE	DBE	MCE
Imperial Valley 1979	Cerro Prieto	H-CPE237	6.53	15.19	0.82	2.05	3.08
Loma Prieta 1989	Hollister - S & P	HSP000	6.93	27.67	0.29	0.72	1.08
Loma Prieta 1989	Woodside	WDS000	6.93	33.87	1.40	3.49	5.24
Loma Prieta 1989	WAHO	WAH090	6.93	17.47	0.48	1.20	1.80
Manjil 1990	Abbar	ABBAR--T	7.37	12.56	0.28	0.70	1.05
Cape Mendocino 1992	Fortuna - Fortuna Blvd	FOR000	7.01	15.97	0.99	2.47	3.71
Cape Mendocino 1992	Rio Del Overpass - FF	RIO360	7.01	14.33	0.50	1.25	1.88
Landers 1992	Desert – Hot Springs	LD-DSP000	7.30	21.78	0.95	2.37	3.56
Northridge 1994	LA - W 15th St	W15090	6.69	25.60	1.14	2.86	4.29
Northridge 1994	Moorpark - Fire Sta	MRP180	6.69	16.92	0.78	1.94	2.91
Northridge 1994	N Hollywood - Cw	CWC270	6.69	7.89	0.53	1.33	2.00
Northridge 1994	Santa Susana Ground	5108-360	6.69	1.69	0.78	1.95	2.93
Northridge 1994	LA - Brentwood VA	0638-285	6.69	12.92	0.85	2.12	3.18
Northridge 1994	LA - Wadsworth VA	5082-235	6.69	14.55	0.62	1.54	2.31
Kobe 1995	Nishi-Akashi	NIS090	6.90	7.08	0.48	1.19	1.79
Kobe 1995	Abeno	ABN090	6.90	24.85	1.00	2.49	3.74
ChiChi 1999	TCU105	TCU105-E	7.62	17.18	0.96	2.39	3.59
ChiChi 1999	CHY029	CHY029-N	7.62	10.97	0.53	1.32	1.98
ChiChi 1999	CHY029	CHY041-N	7.62	19.83	0.56	1.40	2.10
Hector 1999	Hector	HEC090	7.13	10.35	0.42	1.04	1.56

Table 4: Properties of the ground motions used for nonlinear dynamic analyses

Figure Captions

Figure 1: (a) Mechanical analog of the proposed SCVD; (b) SCVD hysteresis under earthquake loading.

Figure 2: Possible configurations of the SCVD: (a) pretensioning tendons and friction-based energy dissipation [17]; (b) SMA wires and friction-based energy dissipation [19].

Figure 3: Prototype building structure: (a) plan view and (b) perimeter MRF with SCVDs and supporting braces.

Figure 4: Comparison of the base shear coefficient – roof drift responses from nonlinear cyclic static analysis.

Figure 5: GM model for VE material.

Figure 6: Comparison of experimental and analytical values of the mechanical properties of the VE material at 24 ^{o}C (a) G' and (b) η.

Figure 7: Comparison of the DBE EC8 spectrum with the spectra of the DBE ground motions used for nonlinear dynamic analyses.

Figure 8: Comparison of the roof drift time histories from nonlinear dynamic analysis under the HSP ground motion scaled to the DBE.

Figure 9: Statistics of peak story drifts from nonlinear dynamic analyses under the set of 20 ground motions: (a) FOE; (b) DBE; and (c) MCE.

Figure 10: Statistics of peak residual story drifts from nonlinear dynamic analyses under the set of 20 ground motions: (a) FOE; (b) DBE; and (c) MCE.

Figure 11: Statistics of peak total floor accelerations from nonlinear dynamic analyses under the set of 20 ground motions: (a) FOE; (b) DBE; and (c) MCE.

Figure 12: Statistics of peak total floor velocities from nonlinear dynamic analyses under the set of 20 ground motions: (a) FOE; (b) DBE; and (c) MCE.