Seismic structural and non-structural performance evaluation of highly damped self-centering and conventional systems

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Abstract

This paper evaluates the seismic structural and non-structural performance of self-centering and conventional structural systems combined with supplemental viscous dampers. For this purpose, a parametric study on the seismic response of highly damped single-degree-of-freedom systems with self-centering flag-shaped or bilinear elastoplastic hysteresis is conducted. Statistical response results are used to evaluate and quantify the effects of supplemental viscous damping, strength ratio and period of vibration on seismic peak displacements, residual displacements and peak total accelerations. Among other findings, it is shown that decreasing the strength of nonlinear systems effectively decreases total accelerations, while added damping increases total accelerations and generally decreases residual displacements. Interestingly, this work shows that in some instances added damping may result in increased residual displacements of bilinear elastoplastic systems. Simple design cases demonstrate how these findings can be considered when designing highly damped structures to reduce structural and non-structural damage.

Keywords: Viscous dampers, Self-centering systems, Drift, Residual drift, Total acceleration, Structural damage, Non-structural damage

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1. Introduction

An important requirement of performance-based seismic design is the simultaneous control of structural and non-structural damage [1]. Structural damage measures are related to story drifts, residual drifts and inelastic deformations. Non-structural damage measures are related to story drifts, total floor accelerations and floor response spectra. Earthquake reconnaissance reports highlight that injuries, fatalities and economical losses related to failure of non-structural components far exceed those related to structural failures [2]. Explicit consideration of non-structural damage becomes vital in the design of critical facilities such as hospitals carrying acceleration-sensitive medical equipment which should remain functional in the aftermath of earthquakes [3].

Conventional seismic-resistant structural systems, such as steel moment resisting frames (MRFs) or concentrically braced frames (CBFs), are currently designed to experience significant inelastic deformations under the design seismic action [4]. Significant inelastic deformations result in damage and residual drifts, and hence, in economic losses such as repair costs, costly downtime during which the building is repaired and cannot be used or occupied, and, perhaps, building demolition due to the complications associated with straightening large residual drifts [5]. In addition, conventional seismic-resistant systems cannot provide harmonization of structural and non-structural damage since reduction of drifts or deformations and reduction of total floor accelerations are competing objectives, i.e., adding stiffness and strength to the structure decreases drifts and inelastic deformation demands but increases total accelerations [6].

Residual drift is an important index for deciding whether to repair a damaged structure versus to demolish it. McCormick et al. [7] reported that repairing damaged structures which had experienced residual story drifts greater than 0.5% after the Hyogoken-Nanbu earthquake was no financially viable. MacRae and Kawashima [8] studied residual displacements of inelastic single-degree-of-freedom (SDOF) systems and illustrated their significant dependence on the post-yield stiffness ratio. Christopoulos et al. [9] studied residual displacements of five SDOF systems using different hysteretic rules and showed that residual displacements decrease with an increasing post-yield stiffness ratio. An extensive study by Ruiz-Garcia and Miranda [10] showed that residual displacements are more sensitive to changes in local site conditions, earthquake magnitude, distance to the source range and hysteretic behavior than peak displacements. Pampanin et al. [11] studied the seismic response of multi-degree-of-freedom (MDOF) systems and highlighted a significant sensitivity of residual drifts to the hysteretic rule, post-yield stiffness ratio and global plastic mechanism. Recently, Pettinga et al. [12] examined the effect of stiffness, strength and mass eccentricity on residual displacements of one story buildings and suggested that a proper inclusion of orthogonal elements close to the building plan perimeter can result in reduced differences in permanent drifts across the building plan.

Rate-dependent passive dampers (viscous, viscoelastic, elastomeric; referred to herein as passive dampers) have been extensively used in seismic-resistant design and retrofit [13]. Lin and Chopra [14] studied highly damped elastic SDOF systems and showed that supplemental viscous damping is more effective in reducing displacements than total accelerations. Ramirez et. al. [15] studied inelastic SDOF systems for a wide range of periods of vibration and showed that added damping has no significant effect on the relation between peak elastic and peak inelastic displacements and also, confirmed the technical basis of FEMA 450 [16] to allow a 25% reduction in the minimum design base shear of damped buildings. Pavlou and Constantinou [17] showed that inelastic steel MRFs with passive dampers designed to achieve similar drifts with conventional MRFs experience lower total floor accelerations than conventional MRFs. Lee et al. [18] designed steel MRFs with elastomeric dampers and showed that design criteria that allow some inelastic behavior, but limit drift to 1.5% under the design earthquake lead to the most effective damper design. Vargas and Bruneau [19] studied the effect of supplemental viscous damping on the seismic response of inelastic SDOF structural systems with metallic dampers for three periods of vibration. Their results showed that viscous dampers increase total accelerations of systems whose original frame still behaves inelastically under strong earthquakes. A recent paper showed that retrofitting a building with viscous dampers improves both structural and non-structural fragilities [20]. Occhiuzzi analyzed different examples of frames with passive dampers found in literature and showed that values of the 1^{st} modal damping ratio higher than 20% seem to trade off a minor reduction of interstorey drifts with a significant increase of total floor accelerations [21]. Compressed elastomer dampers with viscoelastic behavior under small amplitudes of deformation and friction behavior under large amplitudes of deformation were designed and tested by Karavasilis et al. [22-23]. When combined with flexible steel MRFs of reduced strength, these dampers were found capable of significantly reducing drifts and inelastic deformations without increasing total floor accelerations.

Recent research developed self-centering (SC) steel MRFs with post-tensioned (PT) connections [24]. SC steel MRFs have the potential to eliminate inelastic deformations and residual drifts under strong earthquakes as the result of a softening force-drift behavior due to separations (gap openings) developed in beam-to-column connections; re-centering capability due to elastic pre-tensioning elements (e.g., high strength steel tendons) providing clamping forces to connect beam and columns; and energy dissipation capacity due to energy dissipation elements (EDs) which are activated when gaps open. The parallel combination of tendons and EDs results in self-centering flag-shaped hysteresis. SC steel MRFs experience drift and total accelerations similar to those of conventional steel MRFs of the same strength and stiffness, i.e., they have conventional seismic performance in terms of non-structural damage. A recent work developed self-centering energy-dissipative braces which eliminate residual drifts and provide story drifts lower and total floor accelerations similar to those achieved with buckling restrained braces (BRBs) [25]. Christopoulos et al. [26] showed that self-centering SDOF systems can match the response of elastoplastic SDOF systems in terms of ductility by using physically achievable energy dissipation and post-yielding stiffness. The same work found self-centering systems of high post-yield stiffness ratio to experience higher total accelerations than elastoplastic systems. Seo and Sause [27] showed that self-centering systems develop greater ductility demands than conventional systems when the lateral strength and post-yield stiffness ratio are the same. They also found that ductility demands can significantly decrease by increasing the energy dissipation capacity and the post-yield stiffness ratio of self-centering systems. Recently, Kam et al. [28] showed that a parallel combination of self-centering systems of sufficient hysteretic energy dissipation capacity with viscous dampers can achieve superior performance compared to other structural systems, especially when the peak viscous damper force is controlled by implementing a friction slipping element in series with the viscous damper.

Seismic design for harmonization of structural and non-structural damage has been the topic of few recent works. A new concept of weakening the main lateral load resisting system along with using passive dampers has been proposed [29] and validated with frames employing concrete rocking columns [30]. Recent works proposed design procedures for optimal location and capacities of added passive dampers and weakening structures based on optimal control theory [31 and references therein].

The literature survey shows that more work is needed to evaluate the structural and non-structural performance of highly damped conventional and self-centering structural systems. In particular, the increase in total accelerations of conventional yielding and self-centering systems due to added damping [19, 28-29] should be quantified. A detailed evaluation of the effect of added damping on residual displacements of conventional yielding systems is missing. The decrease in total accelerations due to strength reductions should be evaluated [29-30]. Moreover, a comparison of the response of highly damped conventional and self-centering systems is needed.

This paper aims to address the aforementioned research needs as well as to independently verify the findings of earlier investigations. For this purpose, a parametric study on the seismic response of highly damped single-degree-of-freedom (SDOF) systems with self-centering flag-shaped or bilinear elastoplastic hysteresis was conducted. Statistical response results were used to evaluate the effects of supplemental viscous damping, strength ratio and period of vibration on seismic peak displacements, residual displacements and peak total accelerations. Simple design cases demonstrate how the aforementioned effects can be considered when designing highly damped structures to reduce structural and non-structural damage.

It is emphasized that the results and conclusions presented in this paper are based on the response of SDOF systems and cannot be directly extended to MDOF buildings. It has been shown that the distributions of peak story drifts, peak residual story drifts and peak total floor accelerations along the building height depend on the fundamental period of vibration, number of stories and level of inelastic deformation [32- 33].

2. Methodology

2.1 Simplified nonlinear structural systems with viscous dampers

Fluid viscous dampers dissipate energy by forcing incompressible fluids to flow through orifices and provide a damping force output, f_D , equal to

$$f_D = c_d |\dot{u}_d|^{\alpha} \operatorname{sgn}(\dot{u}_d) \tag{1}$$

where c_d is the damping constant; α is the velocity exponent that usually takes values between 0.15 and 1.0 for seismic applications and characterizes damper nonlinearity; $\dot{u}_d$ is the velocity across the damper; and sgn is the signum function [13].

Dampers are placed between successive floors of a building by using supporting braces which are designed to be stiff enough so that story drift produces damper deformation rather than brace deformation [22-23]. Lin and Chopra [14] showed that brace flexibility has negligible effect on the peak responses of elastic systems for practical applications where braces are designed to have stiffness more than 5 times larger than the story stiffness (e.g., design cases of steel MRFs with dampers in [22-23]). In addition, the same work showed that damper nonlinearity has negligible effect on peak responses of elastic systems. Based on these observations, this work adopts the simplest case of linear viscous dampers (i.e., α=1) supported by rigid braces to evaluate the effect of supplemental viscous damping on structural response.

The governing equation of motion of a nonlinear SDOF system equipped with supplemental linear viscous dampers (supported by rigid braces) under earthquake loading is

$$m\ddot{u} + (c + c_d)\dot{u} + f_R = -m\ddot{u}_g \tag{2}$$

where m is the mass of the system; c is the inherent damping coefficient; f_R is the nonlinear restoring force of the system; $u, \dot{u}$ and $\ddot{u}$ are the displacement, velocity and acceleration of the system; and $\ddot{u}_g$ is the ground acceleration. The nonlinear restoring force f_R depends on the hysteretic rule of the structural system. Eq. (2) can be also written in the form

$$a_t(t) = -\frac{(c + c_d)\dot{u}(t) + f_R(t)}{m} \tag{3}$$

where $a_t = \ddot{u} + \ddot{u}_g$ is the total acceleration of the system.

In the case of elastic structures, added damping c_d decreases displacements, velocities and total accelerations [6, 14]. Experience has shown that it is impossible to avoid yielding in steel frames equipped with dampers under strong earthquakes (i.e., seismic intensities equal or higher than the design earthquake) for a reasonable size and cost of added dampers and steel structural members [13, 18, 22-23]. Inelastic systems with zero post yielding stiffness impose a limit on the restoring force f_R; equal to their yield strength f_y. Eq. (3) shows that a_t of inelastic structures decreases by decreasing f_y yet increases by increasing supplemental damping c_d. For mildly inelastic structures, this increase may not be large since the peaks of the damper force would still be out of phase with the peaks of the restoring force f_R (note that the peaks of the damper force are always out of phase with the peaks of the restoring force for elastic systems).

2.2 SDOF system parameters

This study used SDOF systems with periods of vibration, T, equal to 39 discrete values ranging from 0.1 to 1.0 s. with a step of 0.05 s, and ranging from 1.0 to 3.0 s. with a step of 0.1 s. This period range covers the fundamental periods of vibration of steel frames with different heights and lateral load resisting systems (MRFs and CBFs).

The inherent viscous damping ratio was set equal to 5%. The added viscous damping ratio ξ_d was considered equal to 10, 20 and 30% and hence, the total viscous damping ratio ξ_t is equal to 15%, 25% and 35%, respectively. These damping values cover the majority of damping ratios used in design cases of frames with viscous dampers encountered in literature [21].

Two hysteretic behaviors are considered, namely the bilinear elastoplastic (BEP) and the self-centering (SC) flag-shaped hysteresis (Fig. 1). The BEP hysteresis aims to describe the approximate global hysteretic behavior of steel MRFs with fully rigid connections or steel CBFs using BRBs. BRBs exhibit a stable BEP hysteretic behavior. Steel MRFs exhibit stiffness and strength deterioration under large cyclic inelastic drift demands. However, steel MRFs with dampers are designed to experience drifts significantly lower than those associated with possible stiffness and strength deterioration in the plastic hinge regions. As shown in Fig. 1 (left), the bilinear rule can be fully characterized by the yield strength f_y, the elastic stiffness k_e and the post-yield stiffness ration *p*.

The SC hysteresis aims to resemble the approximate global hysteretic behavior of post-tensioned steel MRFs [24] or steel CBFs [25]. These systems are known to exhibit flag-shaped hysteretic behavior without stiffness or strength deterioration under large drifts. As shown in Fig. 1 (right), the SC hysteresis can be fully characterized by the yield strength f_y, the elastic stiffness k_e, the post-yield stiffness ratio *p* and the relative hysteretic energy dissipation ratio, β_E, provided by added yielding or friction-based EDs [24-25]. The β_E can range from 0 to 50% for systems that maintain self-centering capability. The extreme cases of β_E=0.0 and β_E=1.0 represent the bilinear elastic and the bilinear elastoplastic hysteretic rules, respectively.

The yield strength, f_y, of the SDOF systems for a given ground motion, *gm*, was determined by

$$f_y = \frac{mS_a(T, \xi = 5\%, gm)}{R_\mu} \quad (4)$$

where $S_a(T,\xi=5\%, gm)$ is the spectral pseudo-acceleration of the ground motion, *gm*, for 5% damping (referred to herein as spectral acceleration) and R_μ is the ratio of the required elastic strength for 5% damping to the yield strength (referred to herein as strength ratio). The SDOF system force-displacement behavior can be established through an approximate idealization of the global base shear force-drift behavior of the building [4, 34] and hence, the strength ratio R_μ reflects the ductility-based portion of the response modification factor *R* (or *q* [4]) used in seismic codes [34]. A detailed evaluation of methods using SDOF systems (and the strength ratio R_μ) to estimate seismic demands in building structures can be found in FEMA440 document [35]. R_μ was considered equal to 2, 4, 6 and 8 in order to cover a wide range of strengths of nonlinear SDOF systems representing the global base shear force-drift behavior of steel frames.

The yield strength f_y of the highly damped SDOF systems is determined according to Eq. (4), i.e., from the 5% damped response spectrum and not from the response spectrum with additional damping. Therefore, the f_y of a highly damped SDOF system for a given R_μ and ground motion *gm* is the same regardless of the additional damping ratio. In that way, the effect of different values of the added damping ratio on the response of a system with specific strength and period of vibration can be isolated and studied.

The f_y can be also written as

$$f_y = \frac{mS_a(T, \xi_t, gm)}{R_{\mu,d}} = \frac{mS_a(T, \xi = 5\%, gm)}{B \cdot R_{\mu,d}} \tag{5}$$

where $B = S_a(T, \xi = 5\%, gm) / S_a(T, \xi = \xi_t, gm)$ is the damping reduction factor [11] and $R_{\mu,d}$ is the ratio of the required elastic strength to the yield strength with reference to the highly damped spectrum of the ground motion *gm*.

Eqs. (4) and (5) show that $R_{\mu,d} = R_\mu/B$. The simplified B factor, recommended by FEMA [16], takes values equal to 1.35 for ξ_t equal to 15%, 1.65 for ξ_t equal to 25% and 1.95 for ξ_t equal to 35%, provided that the system has a period of vibration within the constant velocity spectral region [15]. Therefore, the $R_{\mu,d}$ values associated with R_μ=8 are approximately equal to 5.92, 4.56 and 4.08, and the $R_{\mu,d}$ values associated with R_μ=2 are approximately equal to 1.48, 1.14 and 1.02, for ξ_t equal to 15%, 25% and 35%, respectively. Different R_μ ratios represent either a given structure under different seismic hazard levels or different structures under a given seismic hazard level. Hence, systems with $R_{\mu,d}$ equal to 5.92=8(R_{μ})/1.35(B) represent highly damped structures expected to experience significant damage (global ductility μ=$R_{\mu,d}$=5.92 based on the equal displacement rule), while systems with $R_{\mu,d}$ equal to 1.02=2(R_{μ})/1.95(B) represent highly damped structures likely to respond elastically under an unscaled ground motion represented by its spectral acceleration value S_a used in Eq. (4). Based on the designs of steel MRFs with dampers presented in [22-23], $R_{\mu,d}$ ratios close to 2.5 represent design cases where dampers are used to achieve conventional performance (i.e., story drifts equal to 2% under the design earthquake), while $R_{\mu,d}$ ratios lower than 1.7 represent design cases where dampers are used to achieve higher performance (i.e., story drifts lower than 1.5% under the design earthquake).

The SC system requires an additional parameter to specify its hysteretic energy dissipation capacity; the relative hysteretic energy dissipation ratio β_{E}.. Although the full range of β_E values (i.e., 0 to 100%) is possible [36], practical β_E values are within the range of 25 to 50% in order to maintain self-centering capability [24-25]. β_E values equal to 25% and 50% were considered in this investigation.

A post-yield stiffness ratio p equal to 2% was assumed for both SC and BEP systems in order to effectively compare the response of these systems. The p value of SC systems typically ranges from 5 to 10% [24, 25]. It is emphasized that p can significantly affect the response of SC and BEP systems (discussion in Section 1). However, the p value is not expected to change the effect of added damping on the response of SC and BEP systems. The properties of the SDOF systems examined are summarized in Table 1.

2.3 Ground motions

A set of 22 recorded far-field ground motion pairs (total of 44 recordings) developed by the Applied Technology Council (ATC) Project 63 [37] were used for nonlinear dynamic history analyses. These ground motions were recorded during 8 California earthquakes and 6 earthquakes from five other countries. The magnitudes of these earthquakes are within a range of 6.5 to 7.6. All ground motions were recorded on stiff soil and do not exhibit pulse-type near-fault characteristics. Fig. 2 plots the 5% acceleration response spectra of the ground motions along with their geometric mean spectrum. The ground motions were not scaled, but instead the strength f_y of the system was scaled to produce specific values of the R_μ ratio according to Eq. (4).

2.4 Response quantities

The main response quantities of interest are (1) the maximum (peak) displacement u_m; (2) the residual displacement u_r; and (3) the maximum (peak) total acceleration $a_{t,m}$. The peak displacement and total acceleration are obtained as the maximum of the absolute values of their time histories while residual displacement is obtained as the absolute of the last value of the displacement time history.

The Newmark average acceleration method along with Newton-Raphson iterations [6] was used to integrate the nonlinear equation of motion. The integration time step was selected equal to 0.0005 s. since it has been shown that a particularly small integration time step is needed to accurately predict accelerations [38, 39]. Smaller time steps led to practically same response results. Each dynamic analysis was executed well beyond the actual earthquake time to allow for damped free vibration decay and correct residual displacement calculation.

This work first investigates the effect of the strength ratio R_μ on the peak response of 5% damped systems. Then, the effect of supplemental damping on the peak response quantity, x, is evaluated by presenting the ratio $x(T,R_\mu,\xi=\xi_t)/x(T,R_\mu,\xi=5\%)$. Finally, a direct comparison of the peak response of highly damped BEP and SC systems is also provided.

Structural response shows significant scatter due to ground motion variability and hence, a statistical evaluation is used to identify trends. Assuming that the response to a set of ground motions follows the lognormal distribution, the geometric mean (or referred to herein as the median) of the response quantity x (or the ratio of x) is used to represent the central tendency of the response.

3. Nonlinear seismic response results

3.1 Effect of strength ratio on 5% damped systems

The peak total acceleration, $a_{t,m}$, and peak displacement, u_m, of 5% damped BEP and SC systems are calculated and normalized with respect to the corresponding peak responses of the elastic system (R_μ=1) having the same period of vibration. The residual displacement of BEP systems is also calculated and normalized with respect to the peak displacement of the elastic system. The median of the normalized responses to the ground motion set is calculated and shown in Figs. 3 and 4.

Fig. 3(a) shows that the normalized $a_{t,m}$ of BEP systems can be effectively decreased by increasing R_μ. This effect is more significant for systems with low R_μ values. For example, increasing R_μ of systems with T=1 s. from 2 to 4 (50% decrease in strength for a given ground motion) results in 45% decrease in $a_{t,m}$, while increasing R_μ

from 4 to 8 (50% decrease in strength for a given ground motion) results in 39% decrease in $a_{t,m}$. For a given R_μ and for T>0.3 s., the normalized $a_{t,m}$ is approximately constant and period independent. For very short period systems, $a_{t,m}$ increases as period decreases and eventually approaches the peak ground acceleration as T tends to zero [6].

Fig. 3(b) shows that the normalized u_m of BEP systems tends toward infinity as T decreases and toward unity for T longer than 0.5 s. regardless of the R_μ value. The results confirm the well-known equal displacement rule (i.e., $u_m(R_\mu)=u_m(R_\mu=1)$) for long period systems as well as the strong dependence of u_m on the strength ratio for short period systems [6].

Fig. 3(c) shows that the normalized u_r of BEP systems is approximately constant regardless of the R_μ value in the long period region and increases in the short period region. These results are consistent with the findings by previous researchers for systems with nonzero positive post-yielding stiffness [10].

Fig. 4 displays the influence of the strength ratio on the median of the normalized peak response of 5% damped SC systems with β_E=25% or 50%. Figs. 4 (a) and (c) show that the normalized $a_{t,m}$ of SC systems with β_E=25% exhibit almost identical trends with those of SC systems with β_E=50%. A close examination of Fig. 4(b) reveals that the normalized u_m of SC systems with β_E=25% and $R_\mu \geq 4$ is above unity over the entire period region. This indicates that the equal-displacement rule would be unconservative for these systems. A comparison of Figs. 4(b), 4(d) and 3(b) indicates that SC systems yield larger u_m than those of BEP systems, a finding which is consistent with the results of previous works [26-27]. The u_r of self-centering systems was not studied since SC systems oscillate around the origin and result in zero residual displacement.

3.2 Highly damped bilinear elastoplastic systems

In order to investigate the influence of supplemental viscous damping on the peak response of BEP systems, the peak responses of highly damped BEP systems are normalized with respect to the corresponding peak responses of 5% damped BEP systems having the same period of vibration and R_μ factor. The median of the normalized responses to the ground motion set is calculated and presented in the following figures.

Fig. 5 shows the normalized $a_{t,m}$ of highly damped BEP systems for different R_μ and ξ_t values. Fig. 5(a) shows that the normalized $a_{t,m}$ of BEP systems with R_μ=2 is almost unity over the entire period region regardless of the ξ_t value. An increase in ξ_t tends to slightly decrease the normalized $a_{t,m}$ for T <1.0 s., while this trend is reversed for T>1.0 s. The results indicate that added damping has no influence on $a_{t,m}$ of systems with low strength ratio (R_μ=2). This can be explained since highly damped systems with R_μ=2 remain mildly inelastic (i.e., they have approximate $R_{\mu,d}$ values equal to 1.48, 1.14 and 1.02 for ξ_t = 15%, 25% and 35%, respectively; refer to Section 2.2) and therefore, the peaks of the damping force f_D are generally out of phase with the peaks of the restoring force f_R (see Eq. (3)). Figs. 5 (b)-(d) show that the normalized $a_{t,m}$ of BEP systems with R_μ≥4 increases as ξ_t increases, with the exception of very short period systems. This increase becomes more pronounced for systems with longer period and larger R_μ. For example, by increasing ξ_t from 5% to 25%, the $a_{t,m}$ increases by 60% for R_μ=4 and

T=1.0 s.; 90% for R_{μ}=4 and T=3.0 s.; 95% for R_{μ}=8 and T=1.0 s.; and, 150% for R_{μ}=8 and T=3.0 s. These increases are not surprising since the behavior of highly damped systems with $R_{\mu}\geq 4$ is fully inelastic and the phase difference between the peaks of the damping force f_D and the peaks of the restoring f_R is small (see Eq.(3)). The $a_{t,m}$ of very short period systems approaches the peak ground acceleration as T tends to zero and is not influenced by added damping.

Fig. 6 displays the normalized u_m of highly damped BEP systems for different R_{μ} and ξ_t values. Fig. 6 reveals that the normalized u_m is always less than unity, decreases with increasing damping, and is relatively constant for almost the entire period region. The u_m reductions due to added damping are more pronounced for short period systems. A comparison between Figs. 6 (a) and (d) reveals that for a given ξ_t, the median of the normalized u_m of systems with low R_{μ} has similar values with the normalized u_m of systems with high R_{μ}. This indicates that the effect of added damping to reduce u_m is independent of the strength ratio of the system and further suggests that the damping modification factor derived from linear elastic systems (e.g., B factor in FEMA [16]) can be also used for estimating the peak displacement response of highly damped inelastic systems.

Fig.7 shows the normalized u_r of BEP systems for different R_{μ} and ξ_t values. Fig. 7(a) shows that, except for short period systems, BEP systems with R_{μ}=2 and $\xi_t \geq 25\%$ have zero normalized u_r (i.e., zero residual displacement) since they remain nearly elastic (i.e., the approximate $R_{\mu,d}$ factors of these systems are less than 1.1). Figs. 7 (b)-(d) show that, in general, the normalized u_r decreases with increasing ξ_t. The difference in the normalized u_r for different ξ_t becomes less obvious for systems with large R_μ values. Therefore, adding damping is less effective in reducing residual displacements of systems with high strength ratio. Interestingly, this work uncovers that added damping may increase u_r for particular systems (e.g., the normalized u_r is larger than unity for R_μ=4, T=1.8 s. and ξ_t=15%).

3.3 Highly damped self-centering systems

Fig. 8 displays the median of the $a_{t,m}$ of highly damped SC systems normalized with respect to the $a_{t,m}$ of 5% damped SC systems for different R_μ and ξ_t values. In comparison with Fig. 5, the trend of the normalized $a_{t,m}$ for highly damped SC systems is very similar to that observed in highly damped BEP systems. The similarity in the trends is also observed between SC systems with different β_E values. As a result, conclusions similar to those for BEP systems can be made for SC systems.

Fig. 9 shows the median of the u_m of highly damped SC systems normalized with respect to the u_m of the 5% damped SC system for different R_μ and ξ_t values. Compared to the highly damped BEP systems shown in Fig. 6, similar trends in the relationship between u_m and added damping are observed. As a result, conclusions similar to those for BEP systems can be made for SC systems. Additionally, it is shown that β_E has little influence on the relation between added damping and u_m.

3.4 Comparison of highly damped bilinear elastoplastic and self-centering systems

Figs. 6 to 9 show that added damping has similar effects on the peak responses of BEP systems and SC systems of the same period and strength ratio. However, a better insight into the seismic behavior of these systems can be gained by a direct comparison of their peak responses. The median of the ratios of the peak responses of highly damped SC systems to the corresponding responses of highly damped BEP systems having the same T, ξ_t, and R_μ has been calculated and presented in the following figures.

Fig. 10 displays the median of the ratio of u_m of highly damped SC systems with β_E =50% to the u_m of highly damped BEP systems for different R_μ and ξ_t values. The ratio is typically larger than unity for $R_\mu \geq 4$ and $T<1.0$ s., and, approaches unity as T increases. Therefore, BEP and SC systems have comparable displacements for $T>1.0$ s. Increasing damping slightly decreases the ratio of u_m between SC and BEP systems. This indicates that added damping is slightly more effective in reducing the peak displacements of SC systems rather than the peak displacements of BEP systems. Fig. 10(a) shows that SC and BEP systems experience identical displacements for all values of damping when R_μ=2. Highly damped systems with R_μ=2 remain mildly inelastic and therefore, the hysteretic behavior (SC or BEP) has no effect on peak displacement response.

Fig. 11 displays the median of the ratio of $a_{t,m}$ of highly damped SC systems with β_E =50% to the $a_{t,m}$ of highly damped BEP systems for different R_μ and ξ_t values. The ratio is larger than unity in the short period region and slightly larger than unity in the long period region, indicating that $a_{t,m}$ of highly damped SC systems are larger than that of highly damped BEP systems. Increasing damping decreases the ratio of $a_{t,m}$ between SC and BEP systems. This indicates that added damping affects more the peak total accelerations of SC systems rather than those of BEP systems. Fig. 11(a) shows that SC and BEP systems experience identical peak total accelerations for all values of damping when R_μ=2.

4. Design for high-seismic structural and non-structural performance

This section demonstrates how the results presented in Section 3 can be considered when designing highly damped structures to achieve structural and non-structural damage reductions. The discussion is based on the equivalent nonlinear SDOF representations of four 2-story conventional steel MRFs presented in Karavasilis et al. [22-23].

Table 2 provides information about the four steel MRFs. MRF100 is a conventional steel MRF that satisfies the strength and drift criteria of the IBC 2003 [34]. MRF75, MRF50 and MRF25 are MRFs designed for base shears equal to 0.75V_d, 0.50V_d and 0.25V_d, where V_d is the design base shear of the MRF100. Table 2 includes the fundamental period of vibration T_1, the base shear coefficient V/W (V is the base shear strength from pushover analysis and W is the seismic weight), the design spectral acceleration S_a at period T_1, the strength ratio R_μ, the peak displacement u_m, the residual displacement u_r and the peak total acceleration $a_{t,m}$. The spectral acceleration, S_a, was obtained from the 5% damped design response spectrum with parameters S_{DS}=1.0g, S_{D1}=0.6g, T_0=0.12 s. and T_s=0.6 s. [34]. The R_μ factor was determined as

$$R_\mu = \frac{mS_a}{V} = \frac{S_a}{g \cdot \frac{V}{W}} \tag{7}$$

The peak displacement was calculated on the basis of the equal displacement rule, i.e., $u_m=(T/2\pi)^2S_a$. With the R_μ and u_m known, the u_r was obtained from Fig. 3(c), while $a_{t,m}$ was obtained from Fig. 3(a) based on the fairly accurate assumption that the peak total acceleration for R_μ=1 is equal to the spectral acceleration for 5% damping, i.e., $a_{t,m}(R_\mu=1,\xi=5\%)\cong S_a(\xi=5\%)$ [6].

Linear viscous dampers are installed in the MRFs to achieve 25% total damping ratio at the fundamental period of vibration. With the total damping ratio known, the peak total acceleration, peak displacement and residual displacement of the highly damped frames (denoted as DMRF100, 75, 50 and 25) are determined from Figs. 5, 6, and 7, respectively. Table 3 provides the peak displacement, residual displacement and peak total acceleration of the highly damped MRFs normalized with the corresponding response quantities of the conventional MRF100 without dampers.

DMRF100 provides the highest displacement reduction, and also eliminates residual displacement. However, DMRF100 experiences peak total accelerations similar to those of the MRF100. DMRF25 provides the highest total acceleration reduction but experiences displacements similar to those of MRF100. DMRF75 and DMRF50 are design cases with the potential to achieve a simultaneous reduction of structural (drift, residual drift) and non-structural (drift, total acceleration) damage. These design cases illustrate that structural and non-structural performances of buildings with rate-dependent dampers significantly depend on the mechanical properties (strength ratio and period) of the initial frame design. In addition, the results presented in Figs. 3-11 along with the simple design cases of this section shed more light on important outcomes of previous works evaluating the structural and non-structural seismic performance of highly damped frames with different strengths, periods of vibration and total damping ratios (e.g., [17, 20-23]).

5. Summary and conclusions

The purpose of this paper was to address various research needs relevant to the structural and non-structural seismic behavior of conventional and self-centering systems equipped with viscous dampers. For this purpose, a parametric study on the seismic response of highly damped single-degree-of-freedom systems with self-centering (SC) flag-shaped or bilinear elastoplastic (BEP) hysteresis was conducted. Statistical response results were used to evaluate the effects of supplemental viscous damping , strength ratio R_μ defined with reference to the 5% damped spectrum, and, period of vibration on seismic peak displacements, residual displacements and peak total accelerations.

The effect of the strength ratio on the peak responses of 5% damped SC and BEP systems was investigated. The results confirmed findings of previous works relevant to peak displacements and residual displacements. In addition, the following conclusions were drawn:

1. Peak total accelerations can be effectively reduced by increasing the strength ratio (decreasing the strength for a given ground motion) of yielding systems. This effect is practically independent of the period of vibration except for very short period systems.
2. Peak total acceleration reduction due to strength ratio increase is more significant for systems with initial low values of the strength ratio. For example, increasing R_μ of systems with T=1 s. from 2 to 4 and from 4 to 8 (50% decrease in strength for a given ground motion) results in 45% and 39% decrease in total acceleration, respectively.

The effect of supplemental viscous damping on the peak response of SC and BEP systems was evaluated. The results confirmed findings of previous works relevant to peak displacements. In addition, the following conclusions were drawn:

3. Adding damping to systems with low strength ratio (R_μ=2) does not influence peak total accelerations.
4. The peak total acceleration of systems with $R_\mu \geq 4$ increases when added damping increases.
5. The increase in total acceleration due to added damping becomes more pronounced for systems with a longer period and larger R_μ values. For example, by increasing the viscous damping ratio from 5% to 25%, peak total acceleration increases by 60% for R_μ=4 and T=1.0 s.; 90% for R_μ=4 and T=3.0 s.; 95% for R_μ=8 and T=1.0 s.; and, 150% for R_μ=8 and T=3.0 s.
6. The relative hysteretic energy dissipation ratio has no influence on the relation between added damping and peak displacement of SC systems.
7. In general, added damping decreases residual displacements of BEP systems. This effect is less pronounced for systems with high strength ratio values.
8. In some instances, added damping may result in increased residual displacements.

A direct comparison between the peak responses of highly damped BEP and highly damped SC systems was also conducted and the following conclusions were drawn:

9. The two systems experience identical peak total accelerations for all values of damping when R_μ=2.
10. Added damping affects more the peak total accelerations of SC systems rather than those of BEP systems.

11. Added damping is slightly more effective in reducing the peak displacements of SC systems rather than the peak displacements of BEP systems.

Simple design cases of a conventional 2-story steel MRF and 2-story steel MRFs equipped with linear viscous dampers providing a total damping ratio equal to 25% at the fundamental period of vibration were presented. The results of the parametric study on single-degree-of-freedom systems were used to evaluate the peak responses of the steel MRFs and the following conclusions were drawn:

12. The structural and non-structural performance of buildings with rate-dependent dampers significantly depends on the mechanical properties (strength ratio and period of vibration) of the initial frame design.
13. Highly damped steel MRFs with strength within the range of 50 to 75% of the strength of conventional steel MRFs are able to simultaneously reduce peak drifts, residual drifts and peak total accelerations.

Interpretation of the aforementioned conclusions needs to be made on the basis of the assumptions for the structural models and ground motions used in this paper.

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TABLES

Table 1: SDOF system parameters and selected values

T (s.)	R_μ	ξ(%)	ξ_t(%)	p (%)	β_E (%)*
0.1 to 3.0	2, 4, 6, 8	5	15, 25, 35	2	25, 50

*Only for self-centering systems

Table 2: Properties and global response (equivalent SDOF) of steel MRFs designed in [16]

Frame	T_1 (s.)	V/W	S_a (g)	R_μ	u_m (m)	u_r (m)	$a_{t,m}$ (g)
MRF100	1.08	0.27	0.56	2.06	0.16	0.04	0.33
MRF75	1.26	0.20	0.48	2.38	0.19	0.05	0.25
MRF50	1.48	0.14	0.41	2.90	0.22	0.06	0.17
MRF25	1.83	0.09	0.33	3.64	0.27	0.07	0.11

Table 3: Properties and global response (equivalent SDOF) of highly damped steel MRFs having 25% damping ratio at the fundamental period of vibration

Frame	$u_m/u_{m,\mathrm{MRF100}}$	$u_r/u_{r,\mathrm{MRF100}}$	$a_{t,m}/a_{t,m,\mathrm{MRF100}}$
DMRF100	0.61	0.00	1.00
DMRF75	0.71	0.12	0.82
DMRF50	0.84	0.34	0.65
DMRF25	1.03	0.68	0.47

FIGURES

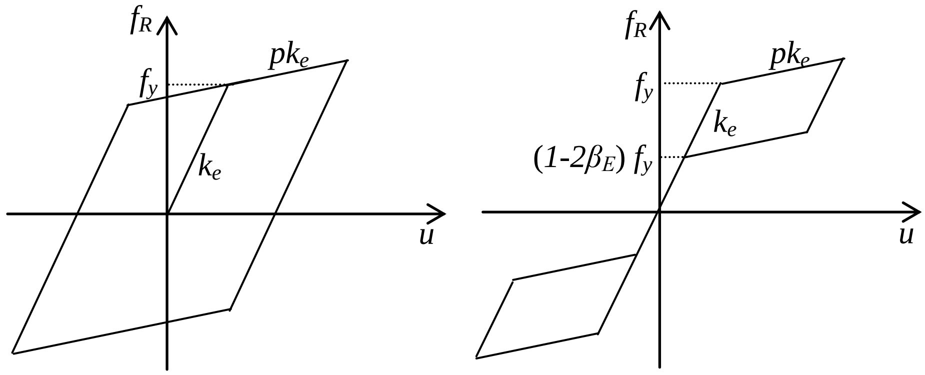

Figure 1: Hysteretic behavior of bilinear elastoplastic, BEP, (left) and self-centering, SC, flag-shaped (right) systems

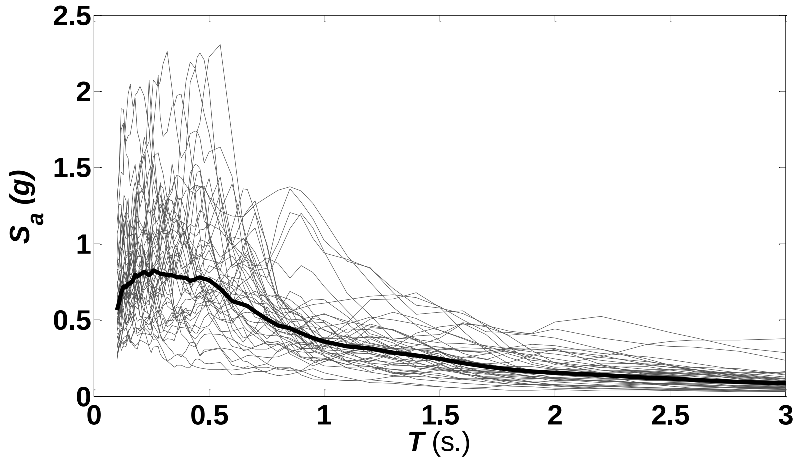

Figure 2: Individual 5% damped acceleration response spectra (light lines) and geometric mean response spectrum (heavy line) of the ground motions considered in this study

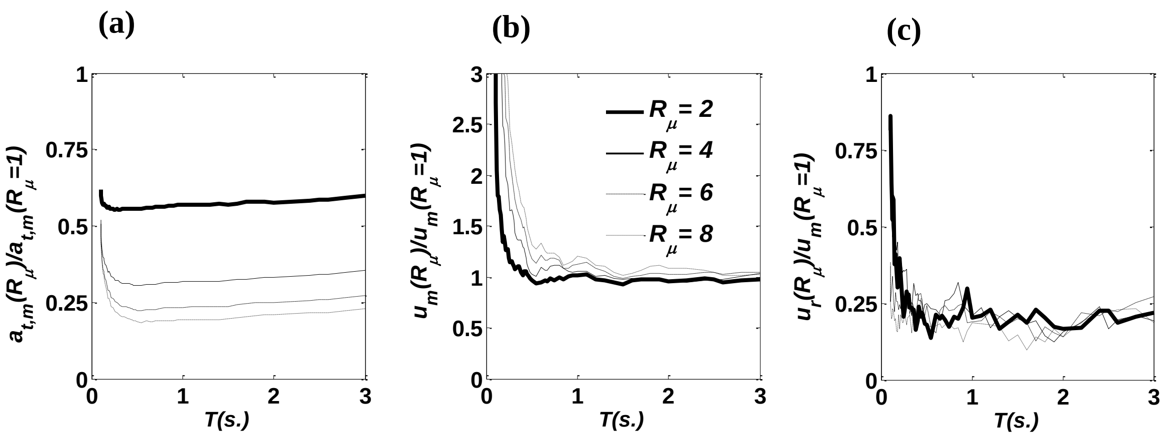

Figure 3: Influence of strength ratio R_μ on: (a) peak total acceleration $a_{t,m}$; (b) peak displacement u_m; and (c) residual displacement u_r of 5% damped bilinear elastoplastic systems.

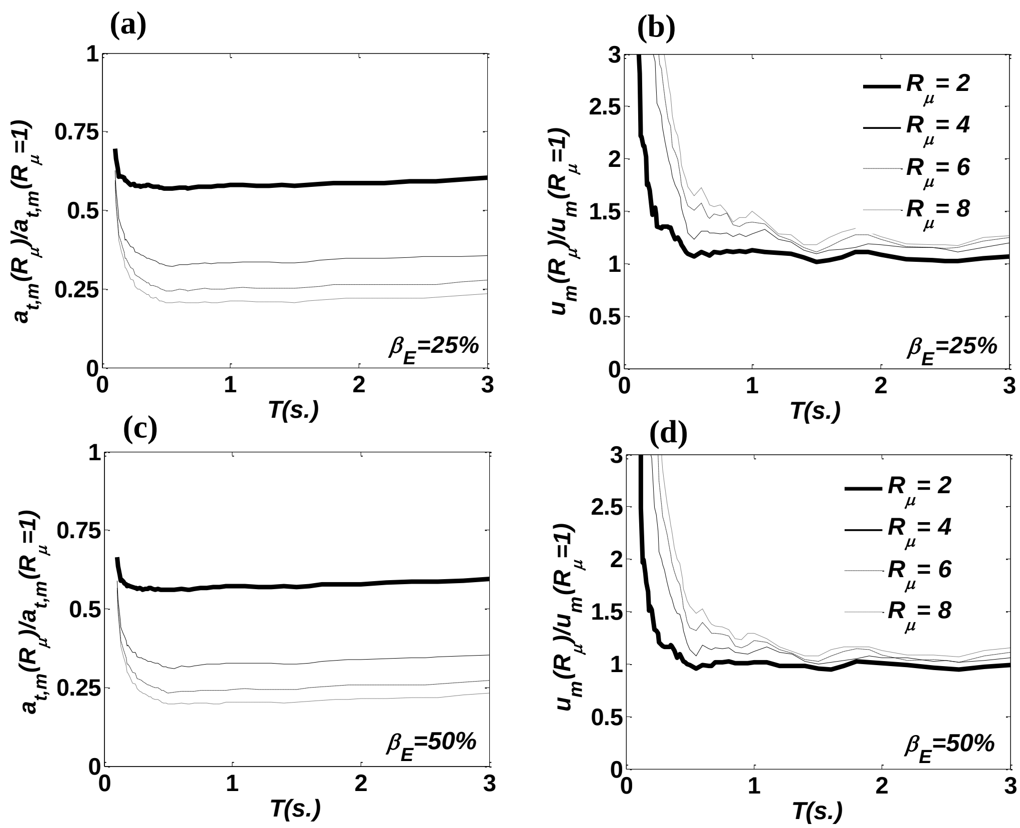

Figure 4: Influence of strength ratio R_μ on peak total acceleration $a_{t,m}$ and peak displacement u_m of 5% damped self-centering systems with relative hysteretic energy dissipation ratio β_E equal to 25% and 50%.

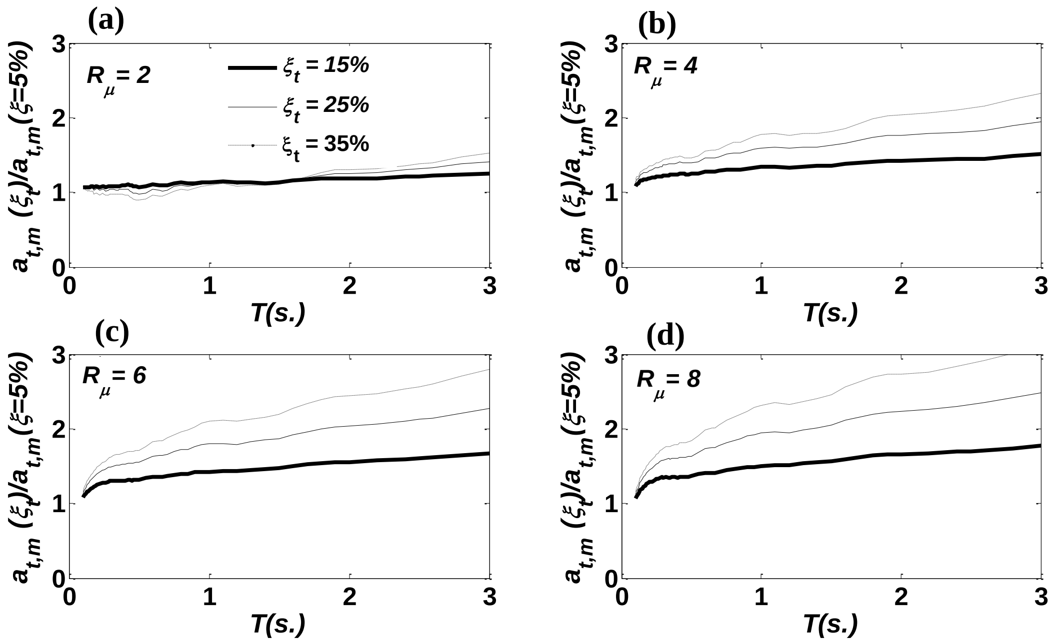

Figure 5: Influence of total viscous damping ratio ξ_t on peak total acceleration $a_{t,m}$ of bilinear elastoplastic systems with different strength ratios R_μ

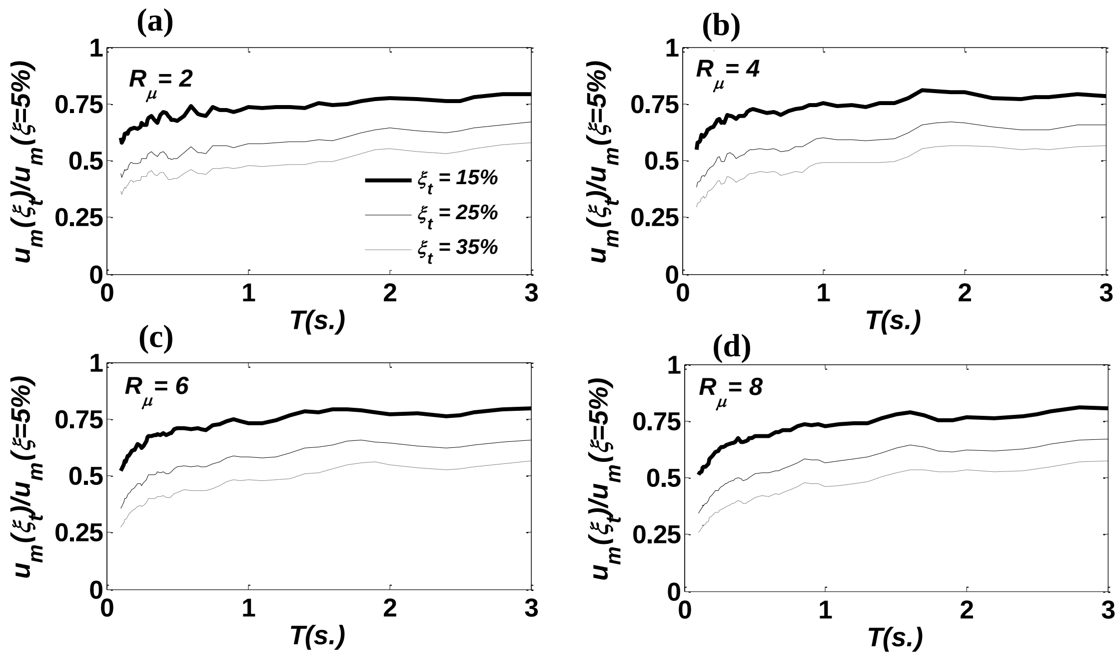

Figure 6: Influence of total viscous damping ratio ξ_t on peak displacement u_m of bilinear elastoplastic systems with different strength ratios R_μ

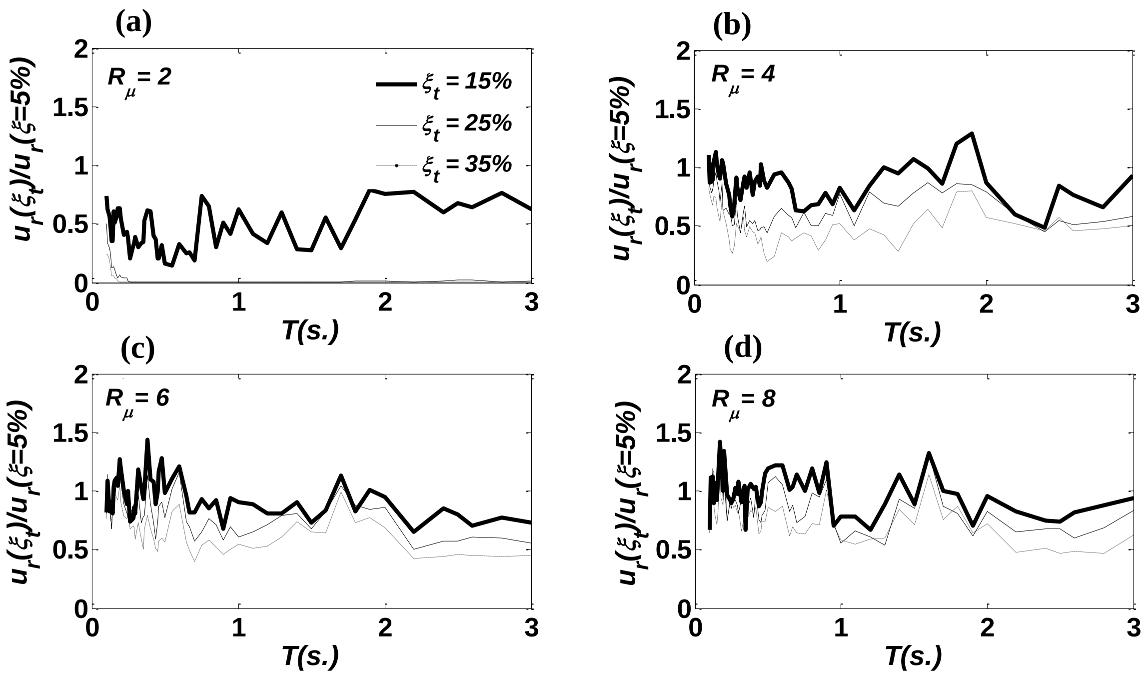

Figure 7: Influence of total viscous damping ratio ξ_t on residual displacement u_r of bilinear elastoplastic systems with different strength ratios R_μ

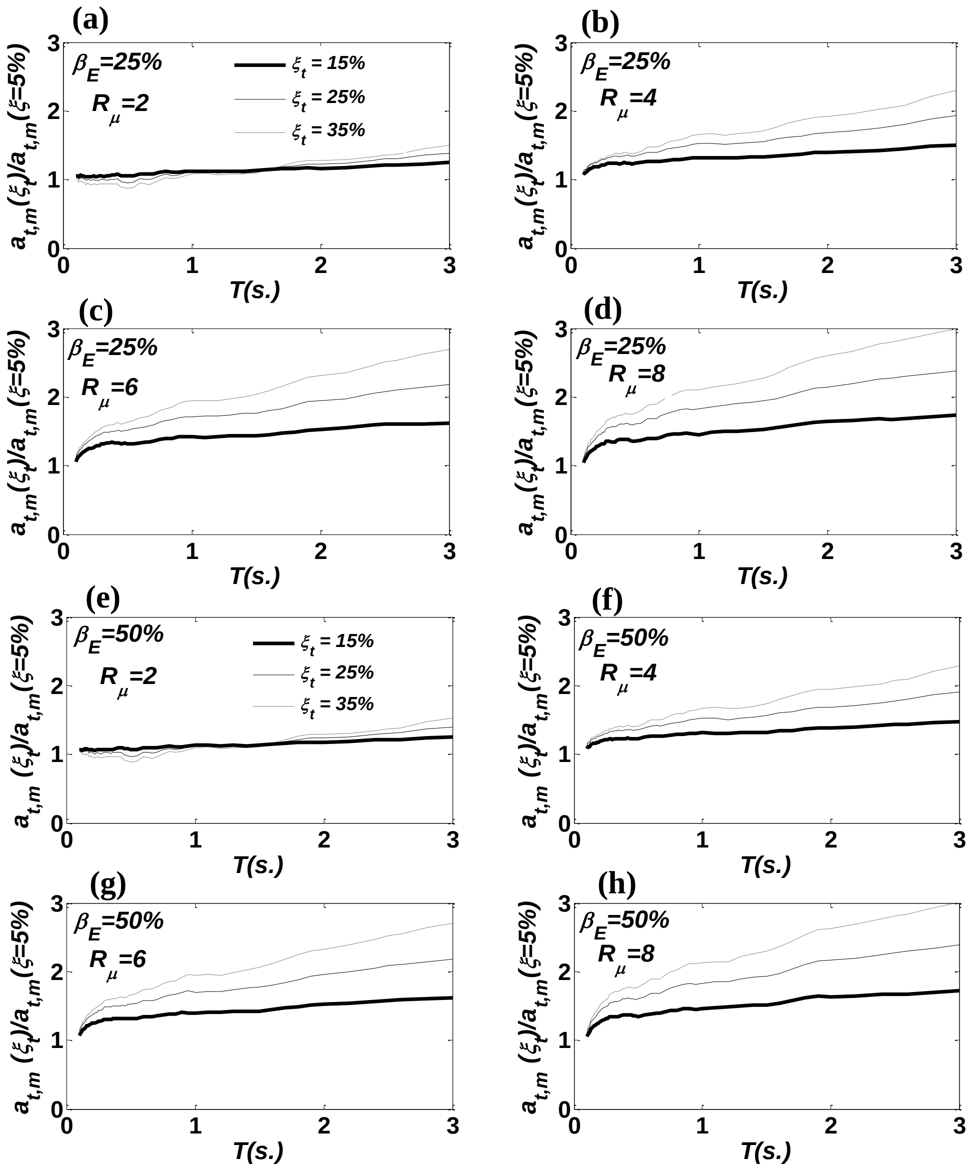

Figure 8: Influence of total viscous damping ratio ξ_t on peak total acceleration $a_{t,m}$ of self-centering systems with relative hysteretic energy dissipation ratio β_E equal to 25% or 50% and different strength ratios R_μ

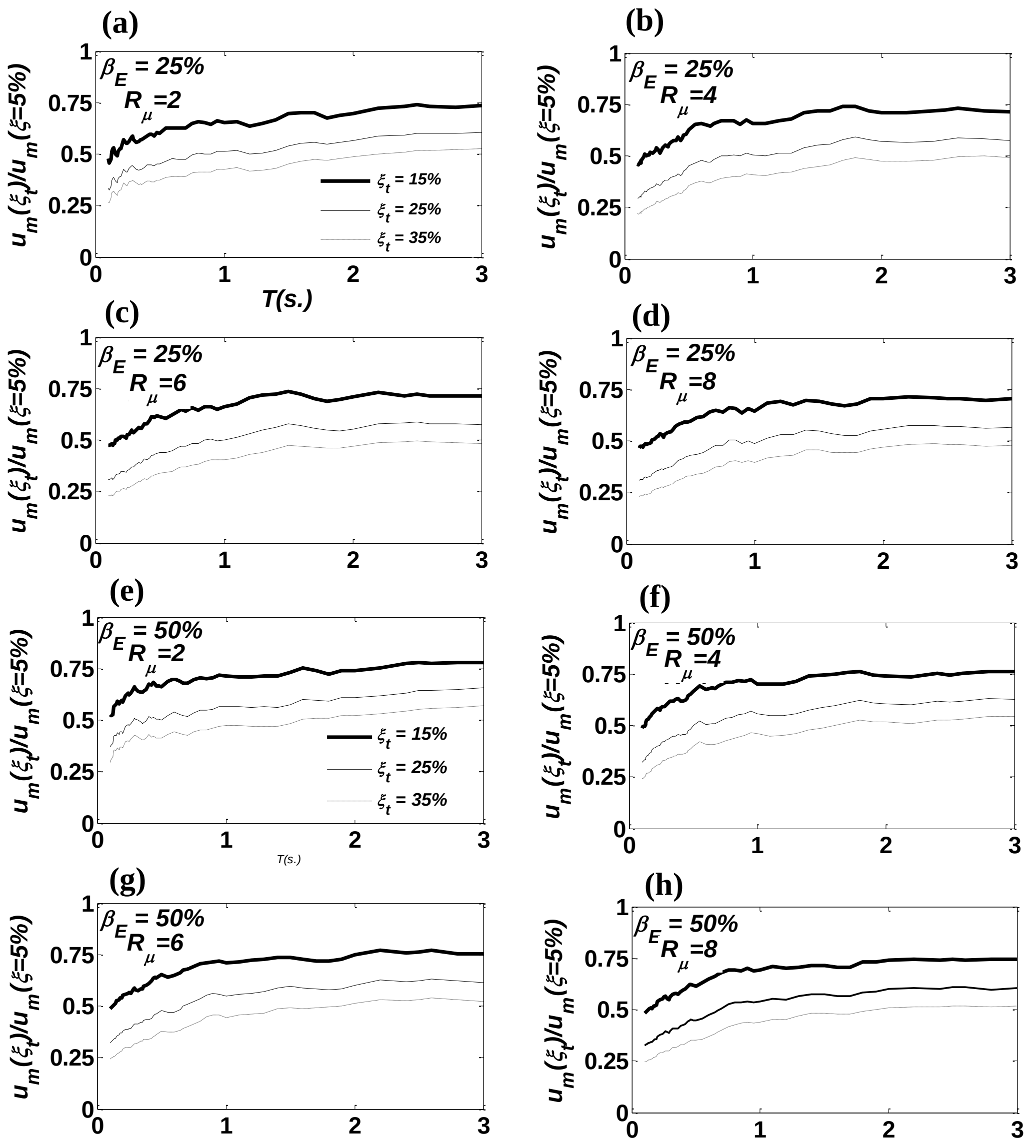

Figure 9: Influence of total viscous damping ratio ξ_t on peak displacement u_m of self-centering systems with relative hysteretic energy dissipation ratio β_E equal to 25% or 50% and different strength ratios R_μ

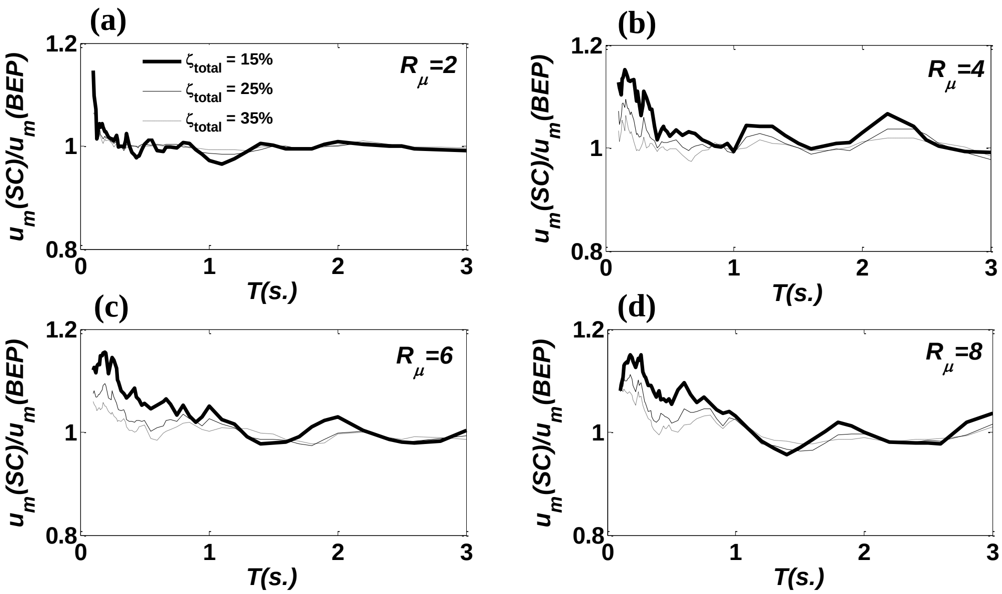

Figure 10: Influence of total viscous damping ratio ξ_t on the ratio of peak displacements of self-centering systems with β_E=50% to peak displacements of bilinear elastoplastic systems for different strength ratios R_μ.

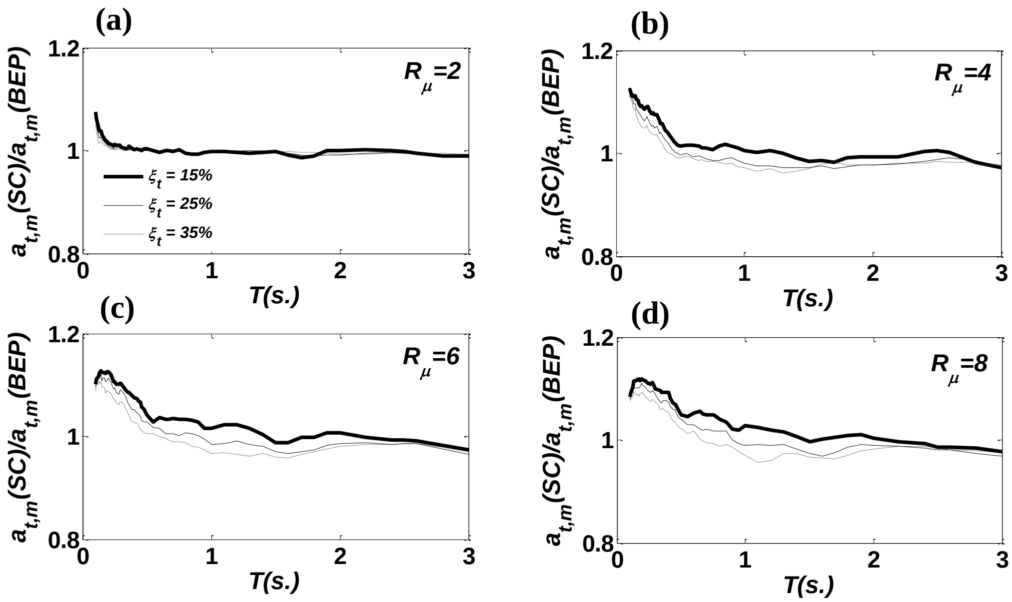

Figure 11: Influence of total viscous damping ratio ξ_t on the ratio of peak total accelerations of self-centering systems with β_E=50% to peak total accelerations of bilinear elastoplastic systems for different strength ratios R_μ.