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SEISMIC RESPONSE OF SINGLE DEGREE OF FREEDOM STRUCTURAL FUSE SYSTEMS

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SUMMARY

Passive energy dissipation (PED) devices have been implemented to enhance structural performance by reducing seismically induced structural damage. In this paper metallic dampers are defined to be structural fuses (SF) when they are designed such that all damage is concentrated on the PED devices, allowing the primary structure to remain elastic. Following a damaging earthquake, only the dampers would need to be replaced, making repair works easier and more expedient. Furthermore, SF introduce self-centering capabilities to the structure in that, once the ductile fuse devices have been removed, the elastic structure would return to its original position. A comprehensive parametric study is conducted leading to the formulation of the SF concept, and allowing to identify the possible combinations of key parameters essential to ensure adequate seismic performance for SF systems. Nonlinear time history analyses are conducted for several combinations of parameters, in order to cover the range of feasible designs. The effects of earthquake duration and strain-hardening on response of short and long period systems are also considered as part of this process.

INTRODUCTION

Typically, in seismic design, the loads resulting from an earthquake are reduced by a response modification factor, R, which allows the structure to undergo inelastic deformations, while most of the energy is dissipated through hysteretic behavior. Designs have always (implicitly or explicitly) relied on this reduction in the design forces. However, this methodology relies on the ability of the structural elements to accommodate inelastic deformations, without compromising the stability of the structure. Furthermore, inelastic behavior translates into some level of damage on these elements. This damage leads to permanent system deformations following an earthquake, leading to high cost for repair works, in the cases when repairs are possible. In fact, it is frequently the case following earthquakes, that damage is so large that repairs are not viable, even though the structure has not collapsed, and the building must be demolished.

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To achieve stringent seismic performance objective for buildings, an alternative design approach is desirable. In that perspective, it would be attractive to concentrate damage on disposable and easy to repair structural elements (i.e., "structural fuses"), while the main structure would be designed to remain elastic or with minor inelastic deformations. Following a damaging earthquake, only the dampers would need to be replaced (hence the "fuse" analogy), making repair works easier and more expedient, without the need to shore the building in the process. Furthermore, in that instance, self-recentering capabilities of the structure would be possible in that, once the ductile fuse devices are removed, the elastic structure returns to its original position.

In this paper, the structural fuses are passive energy dissipation (PED) devices, (a.k.a. metallic dampers) designed such that all damage is concentrated on the PED devices. The structural fuse concept is described in this study in a parametric formulation, considering the behavior of nonlinear single degree of freedom (SDOF) systems subjected to synthetic ground motions. Nonlinear dynamic response is presented in dimensionless charts normalized with respect to key parameters. Allowable story drift is introduced as an upper bound limit to the charts, which produces ranges of admissible solutions, shown as shaded areas in the graphs. Earthquake duration and strain-hardening ratio effects are also analyzed.

ANALYTICAL MODEL OF A SDOF SYSTEM WITH STRUCTURAL FUSES

Figure 1 depicts a single-story one-bay structure subjected to ground motion, whose frame, device support system, and metallic damper are modeled as a lumped mass connected to the ground by elasto-plastic springs, and the inherent system viscous damping action is represented by a linear dashpot (Figure 1b). The three-spring model can be simplified, as well, to an equivalent one-spring model (Figure 1c) with lateral stiffness, K_1, equal to:

$$K_1 = K_f + K_a \tag{1}$$

where K_f and K_a are the lateral stiffness of the frame, and added damping system, respectively. The damping system consists of the device support system and damper itself, whose equivalent added stiffness, K_a, becomes:

$$K_a = \frac{K_s K_d}{K_s + K_d} \tag{2}$$

where K_s and K_d are the lateral stiffness of the device support system (which may be optional, depending on whether the device requires to be attached to a support system), and the damper, respectively. It is worthwhile to mention that for device support system much stiffer than dampers, the deformation of the device support system could be ignored, without significant loss of accuracy, and Equation (1) simplifies to:

$$K_1 = K_f + K_d \tag{3}$$

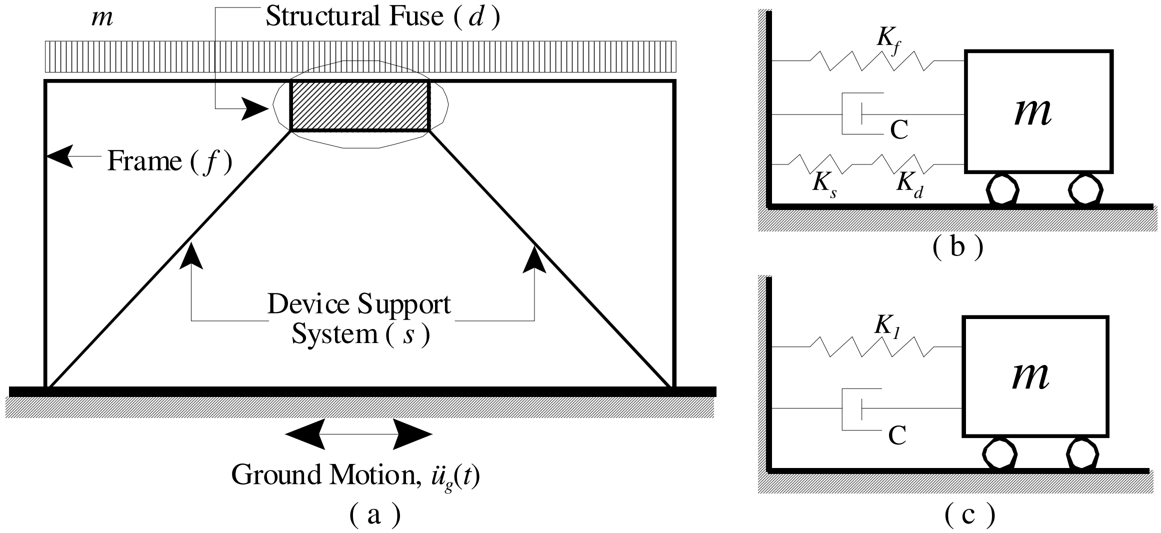

Figure 1. Model of a SDOF system with structural fuse; (a) One-bay single-story structure, (b) Equivalent three-spring system, (c) Equivalent one-spring system

Accordingly, the increased stiffness caused by the inclusion of metallic dampers reduces the period of the primary structure (bare frame), changing it from:

$$T_f = 2\pi \left(\frac{m}{K_f} \right)^{1/2} \tag{4}$$

to:

$$T = 2\pi \left(\frac{m}{K_f + K_a} \right)^{1/2} \tag{5}$$

The structural fuse concept requires that yield deformation of the damping system, Δ_{ya}, be less than the yield deformation corresponding to the bare frame, Δ_{yf}. Considering the deformation of the device support system, the yield deformation of the added damping system is equal to:

$$\Delta_{ya} = \Delta_{yd} \left(1 + \frac{K_d}{K_s} \right) \tag{6}$$

where Δ_{yd} is the damper yield deformation. Figure 2 shows a general pushover curve for a SDOF system with two elasto-plastic springs in parallel. The total curve is tri-linear with the initial stiffness, K_1, calculated using Equations (1) and (2). Once the damping system reaches its yield deformation, Δ_{ya}, the increment on the lateral force is resisted only by the bare frame, being the second slope of the total curve equal to the frame stiffness, K_f. Two important parameters used in this study are obtained from Figure 2: the strain-hardening ratio, α, and the maximum displacement ductility, μ_{max}.

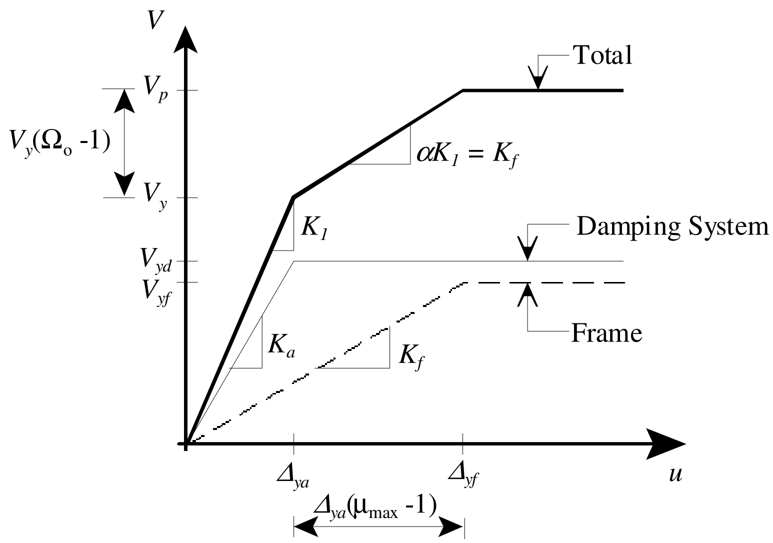

Figure 2. General Pushover Curve

The strain-hardening ratio, α, is the relationship between the frame stiffness and the total initial stiffness, which can be calculated as:

$$\alpha = \frac{1}{1 + \frac{K_a}{K_f}} \tag{7}$$

with α being a dimensionless parameter less than one.

The maximum displacement ductility, μ_{max}, is the ratio of the frame yield displacement, Δ_{yf}, with respect to the yield displacement of the damping system, Δ_{ya}. In other words, μ_{max} is the maximum displacement ductility that the structure experiences before the frame undergoes inelastic deformations. This parameter can be written as:

$$\mu_{\mathbf{max}} = \frac{\Delta_{yf}}{\Delta_{yd}\left(1 + \frac{K_d}{K_s}\right)} \tag{8}$$

with μ_{max} being greater than one. In Figure 2, V_{yf} and V_{yd} are the base shear capacity of the bare frame and the damping system, respectively; and V_y and V_p are the total system yield strength and base shear capacity, respectively.

Pushover curves for different values of α and μ_{max} are presented in Figure 3, with horizontal and vertical axes respectively normalized with respect to the yield displacement of the frame, Δ_{yf}, and the system total base shear capacity, V_p, as shown in Figure 2. As a result, Figure 3 also shows the damping system and frame capacities as percentages of the total base shear capacity.

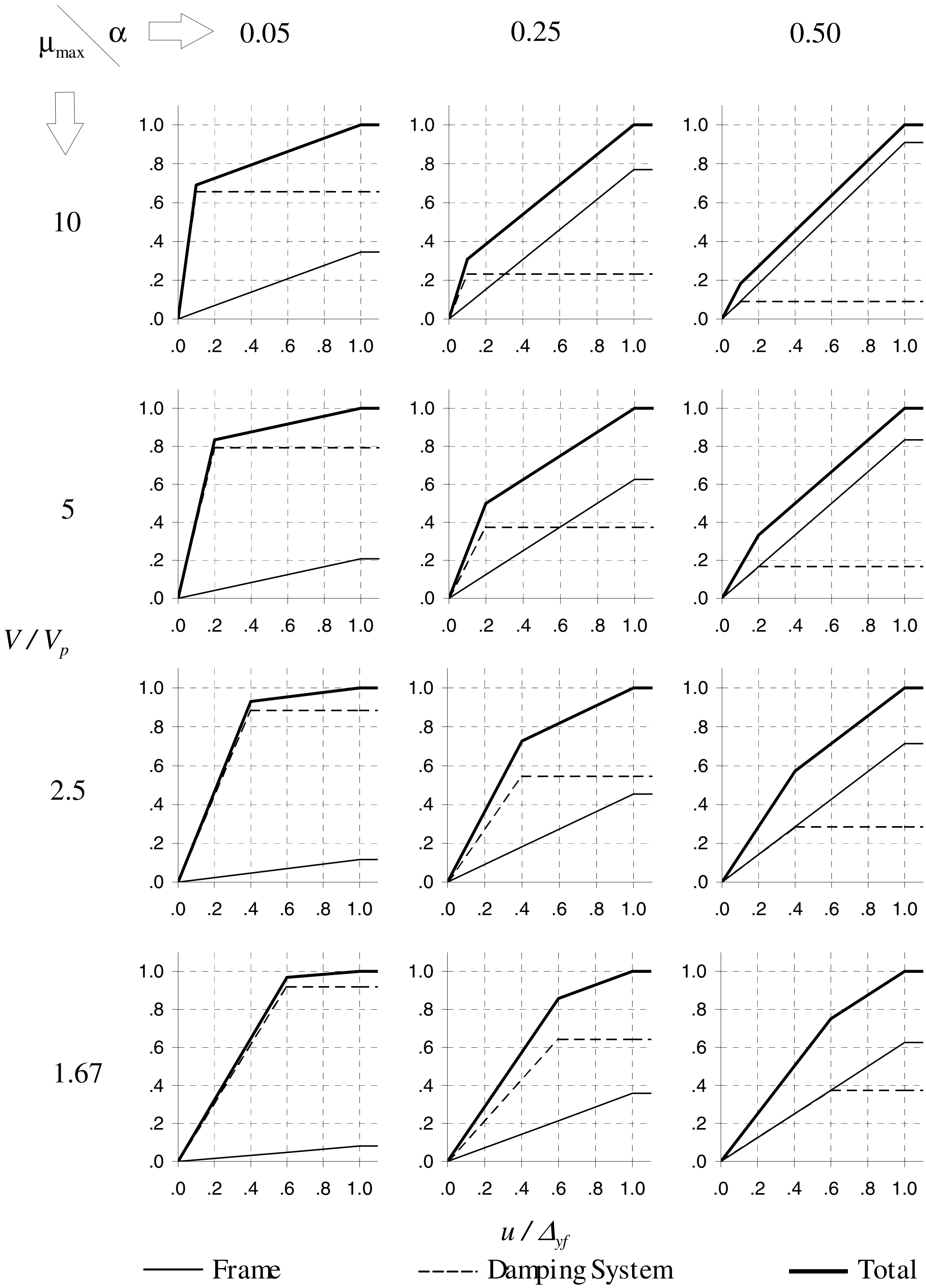

Figure 3. Pushover Curves for the Studied Systems, Normalized by V_p, Δ_{yf}

PARAMETRIC FORMULATION

In linear dynamic analysis of SDOF systems, the equation of motion is commonly written as:

$$m\ddot{u}(t) + c\dot{u}(t) + ku(t) = -m\ddot{u}_g(t) \tag{9}$$

where m, c, k, are the mass, damping coefficient, and linear spring stiffness of the system, respectively, and $\ddot{u}_g(t)$ is the ground acceleration. Solving Equation (9) gives the system response, expressed in terms of the relative displacement, $u(t)$, relative velocity, $\dot{u}(t)$, and relative acceleration, $\ddot{u}(t)$.

For a nonlinear SDOF with hysteretic behavior, once the yield point is exceeded, the spring force is no longer proportional to the relative displacement. Mahin and Lin [1] proposed a normalized version of the nonlinear dynamic equation of motion adapted as shown below. Considering the force in the inelastic spring as time-dependent, $R(t)$, and substituting $R(t)$ for $ku(t)$ into Equation (9), gives:

$$m\ddot{u}(t) + c\dot{u}(t) + R(t) = -m\ddot{u}_g(t) \tag{10}$$

Introducing the natural circular frequency, $\omega = \sqrt{k/m}$, and damping ratio, $\xi = c/(2m\omega)$, Equation (10) can be written as:

$$\ddot{u}(t) + 2\xi\omega\,\dot{u}(t) + \frac{R(t)}{m} = -\ddot{u}_g(t) \tag{11}$$

Equation (11) can be transformed to express the system response in terms of displacement ductility, $\mu(t)$, of the added damping system, which is defined as:

$$\mu(t) = \frac{u(t)}{\Delta_{ya}} \tag{12}$$

where Δ_{ya} is the yield displacement of the damping system, calculated using Equation (6). Differentiating Equation (12) with respect to time, yields:

$$\dot{\mu}(t) = \frac{\dot{u}(t)}{\Delta_{ya}} \tag{13}$$

and:

$$\ddot{\mu}(t) = \frac{\ddot{u}(t)}{\Delta_{ya}} \tag{14}$$

Substituting Equations (12), (13), and (14) into (11) gives the normalized equation of motion used in this study:

$$\ddot{\mu}(t) + \frac{4\pi\xi}{T}\dot{\mu}(t) + \frac{4\pi^2}{T^2}\rho(t) = -\frac{4\pi^2}{T^2\eta}\left[\frac{\ddot{u}_g(t)}{\ddot{u}_{g\max}}\right] \tag{15}$$

where T is the elastic period of the structure, defined by Equation (5), and $\rho(t)$ is the ratio between the force in the inelastic spring and the yield strength of the system, calculated as:

$$\rho(t) = \frac{R(t)}{V_y} \tag{16}$$

and η is the strength-ratio determined as the relationship between the yield strength and the maximum ground force applied during the motion, defined as:

$$\eta = \frac{V_y}{m\ddot{u}_{g\max}} \tag{17}$$

where $\ddot{u}_{g\max}$ is the peak ground acceleration.

For a specific ground motion, $\ddot{u}_g(t)$, Equation (15) can be solved in terms of the above parameters, assuming a damping ratio, ξ, of 0.05 in this study. Note that the impact of the strain-hardening ratio, α, (Equation (7)) on inelastic response is accounted for by the term defined in Equation (16).

NONLINEAR DYNAMIC RESPONSE

A design response spectrum was constructed based on the National Earthquake Hazard Reduction Program Recommended Provisions (NEHRP 2000 [2]) for Sherman Oaks, California, and site soil-type class B. This site was chosen because it corresponds to the location of the Demonstration Hospital used by the Multidisciplinary Center for Earthquake Engineering Research (MCEER) in some of its projects. Accordingly, the design spectral accelerations for this site are S_{DS} = 1.3 g, and S_{D1} = 0.58 g. Using the Target Acceleration Spectra Compatible Time Histories (TARSCTHS) code, by Papageorgiou et al. [3], three spectra-compatible synthetic ground motions were generated to match the NEHRP 2000 target design spectrum. All synthetic strong motion records generated were 15 seconds in duration. The effect of longer duration records is further investigated using synthetic ground motion of 60 seconds in duration.

Nonlinear time history analyses were conducted using the Structural Analysis Program, SAP 2000, (Computers and Structures, Inc. [4]). Analyses were performed for the range of systems described in Figure 3, using the following parameters: α = 0.05, 0.25, 0.50; μ_{max} = 10, 5, 2.5, 1.67; η = 0.2, 0.4, 0.6, 1.0; and T = 0.1 s, 0.25 s, 0.50 s, 1.0 s. The combination of these parameters resulted in 192 analyses for each ground motion generated (i.e., a total of 768 nonlinear time history analyses).

The response of the system is expressed in terms of the frame ductility, μ_f, and the global ductility, μ, defined as follows:

$$\mu_f = \frac{u_{\max}}{\Delta_{yf}} \tag{18}$$

$$\mu = \frac{u_{\max}}{\Delta_{ya}} \tag{19}$$

where $u_{\max}$ is the maximum absolute displacement of the system, taken as the average of the maximum absolute responses caused by each of the applied ground motion.

Figure 4 shows the matrix of results corresponding to the 768 nonlinear analyses conducted in terms of average frame ductility, μ_f, as a function of the elastic period, T. Every plot corresponds to a fixed set of α and μ_{max} values, while each curve represents a constant strength-ratio, η. All the points having $\mu_f < 1$ in Figure 4 represent elastic behavior of the frame (which is the objective of the structural fuse concept).

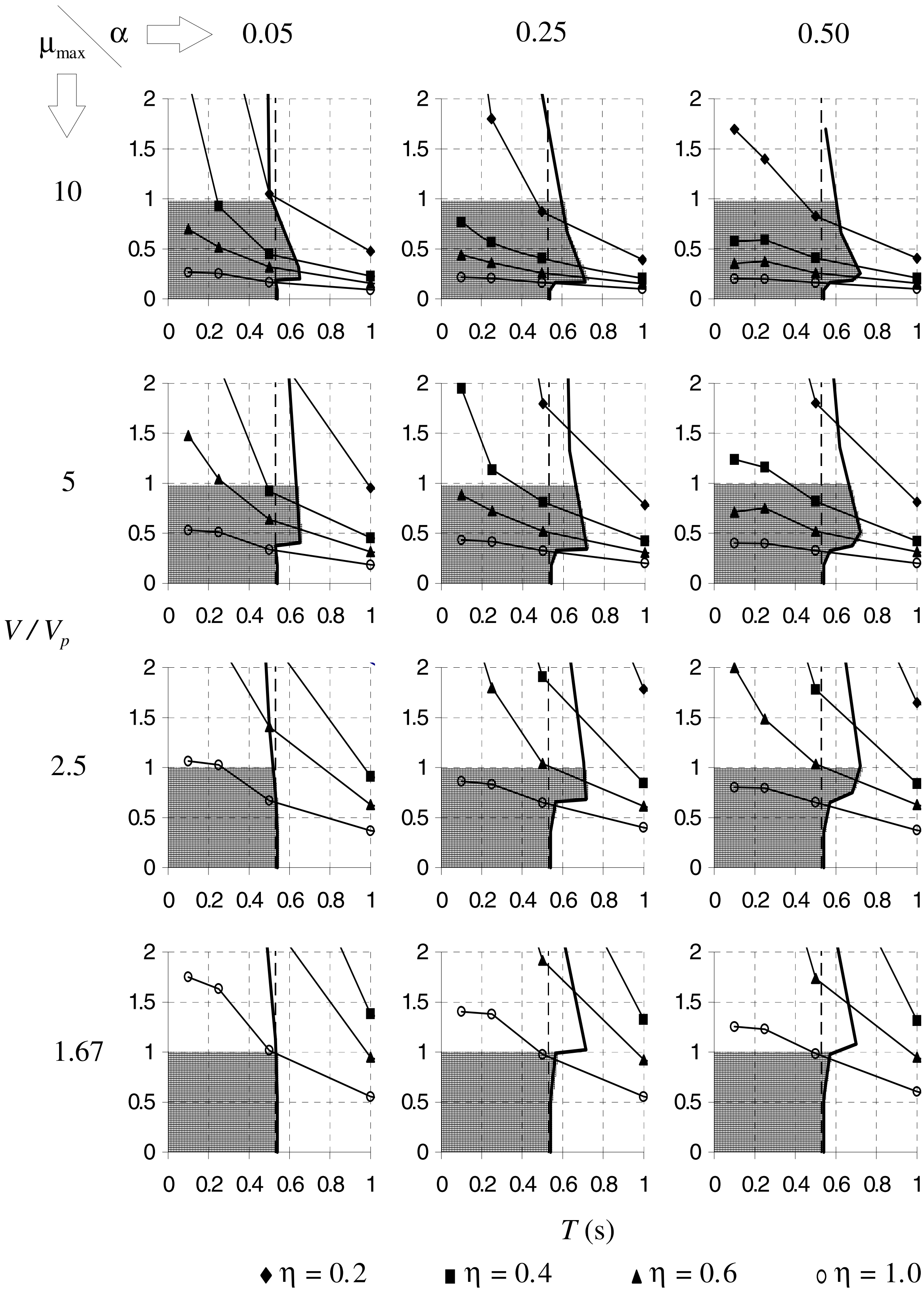

Figure 4. Regions of Admissible Solutions in terms of Frame Ductility (μ_f), and Story Drift of 2%

Figure 4 also shows the regions of acceptable solutions that satisfy the structural fuse concept defined by the following boundaries:

- Maximum ductility: To keep the primary structure elastic, the frame ductility shall be less or equal to one (i.e., $\mu_f \leq 1.0$). Accordingly, the global ductility shall be less than the maximum displacement ductility (i.e., $\mu \leq \mu_{max}$).
- Allowable drift limit ($u_{max} / H \leq \Delta_a$): To maintain the lateral displacement under a tolerable level, story drift shall be kept less than the selected limit, Δ_a, as a function of the story height, H. A drift limit of 2% of the story height has been used, which corresponds to a period of 0.53 s (shown as the dashed line).

Note that for large strength-ratio and period values (i.e., $\eta \geq 0.6$ and $T \geq 1.0$ s) the structure tends to behave elastically, which means that metallic dampers only provide additional stiffness with no energy dissipation. Elastic behavior of the metallic dampers contradicts the objective of using PED devices, other than the benefit of reducing the lateral displacements to below certain limits (something that could be done just as well with conventional structural elements).

Also, a study on the effects of earthquake duration on the above results for SDOF systems, using 15 s and 60 s long synthetic accelerograms revealed slight differences in the above results. A maximum difference of 20% was obtained in some cases for the average ductility values, variations are not significant considering the random characteristics of earthquake excitations. However, even though earthquake duration does not appreciably affect the maximum ductility response, it does increase the number of hysteresis cycles developed during the motion, causing an important increase in the amount of energy dissipated. In some circumstances, this larger number of inelastic cycles could have an impact on the fatigue life of the structural fuses, but this is unlikely for well designed ductile devices, and consideration of this effect is beyond the scope of this study.

CONCLUSIONS

The structural fuse concept has been introduced in this paper and validated through a parametric study of the seismic response of SDOF systems. It has been found that the range of admissible solutions that satisfy the structural fuse concept can be parametrically defined, including (as an option) the story drift limit expressed as an elastic period limit. As shown in Figure 4, as a design tool, this can be represented graphically with shaded areas delimiting the range of admissible solutions. Systems having $\mu_{max} \geq 5$ offer a broader choice of acceptable designs over a greater range of η values. Even though ductility demand, μ_f and μ, does not vary significantly with α (except for small values, i.e., $\alpha = 0.05$), the hysteretic energy substantially increases with decreases in α values. In other words, substantially different amount of hysteretic energy can be dissipated by system having identical ductility demands.

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