Investigation of the Structural Fuse Concept

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ABSTRACT

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 structural fuse concept can be implemented in new or existing structures using various kinds of metallic passive energy dissipating (PED) elements. This paper describes how to use metallic dampers to implement the SF concept and improve the structural behavior of systems under seismic loads. Detailed design process is presented, as well as the modifications necessary to the process for retrofitting applications.

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INTRODUCTION

Typically, in seismic design, the loads resulting from an earthquake are reduced by a response modification factor, 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.

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 fuse"), while the main structure would be designed to remain elastic or with minor inelastic deformations. This approach has received some attention in the past (e.g., Roeder and Popov, 1977, Wada et al., 1992, Connor et al., 1997, Carter and Iwankiw, 1998, Shimizu et al., 1998, Wada and Huang, 1999, Rezai et al., 2000, to name a few).

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, Δ_a, is introduced as an upper bound limit to the charts, which produces ranges of admissible solutions, shown as shaded areas in the graphs. A generic retrofit case study is also presented to illustrate the benefits of adding metallic fuse elements to an existing frame. A comparative analysis is made between a bare frame (i.e., without metallic dampers), and the same frame retrofitted using metallic fuse elements, to improve the behavior of the existing structure

In the process of attempting to implement the SF concept into actual designs, it was observed that, due to the large number of complex parameters interdependencies, a systematic design procedure was necessary. Therefore, in this paper, such a general procedure is proposed for designing and retrofitting purposes.

ANALYTICAL MODEL OF A SDOF SYSTEM WITH METALLIC FUSES

A SDOF structure with metallic damper subjected to ground motion can be modeled as a lumped mass connected to the ground by elasto-plastic springs, and the inherent system viscous damping action represented by a linear dashpot. Figure 1 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, equal to the frame stiffness, K_f, plus the added structural fuse system stiffness, K_a.

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}. 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. Three important parameters used in this study are obtained from Figure 1, namely, the strain-hardening ratio, α, the maximum displacement ductility, μ_{max}, and the strength-ratio, η. The strain-hardening ratio, α, is the relationship between the frame stiffness and the total initial stiffness. 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. The strength-ratio, η, is the relationship between the yield strength, V_y, and the maximum ground force applied during the motion, $m \cdot PGA$, where m is the system mass, and PGA is the peak ground acceleration. In Figure 1, V_{yf} and V_{yd} are the shear capacity of the bare frame and the damping system, respectively; and V_y and V_p are the total system yield strength and shear capacity, respectively.

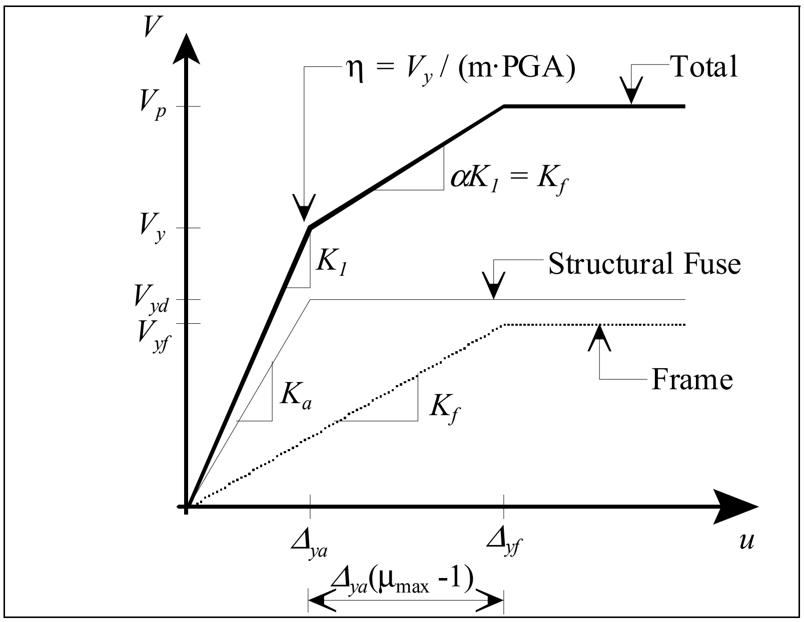

Figure 1. General Pushover Curve

PARAMETRIC FORMULATION

In linear dynamic analysis of SDOF systems, the spring force is considered proportional to the mass relative displacement. However, 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 (1983) proposed a normalized version of the nonlinear dynamic equation of motion adapted by this study, and expressed in terms of the above defined parameters (i.e., α, μ_{max}, and η) as:

$$\ddot{\mu}(t) + \frac{4\pi\xi}{T}\dot{\mu}(t) + \frac{4\pi^2}{T^2}\rho(t) = -\frac{4\pi^2}{T^2\eta}\left[\frac{\ddot{u}_g(t)}{PGA}\right] \quad (1)$$

where μ is the global ductility of the system, defined as the ratio of the maximum relative displacement, u_{max}, with respect to the yield displacement of the damping system, Δ_{ya} (i.e.,

$\mu = u_{max} / \Delta_{ya}$), T is the elastic period of the structure, and $\rho(t)$ is the ratio between the force in the inelastic spring and the yield strength of the system.

For a specific ground acceleration, $\ddot{u}_g(t)$, the equation of motion can be solved throughout nonlinear dynamic analyses, in terms of the selected parameters, assuming a damping ratio, ξ, of 5% in this study. The system response can be expressed not only in terms of the global ductility, μ, but also in terms of the frame ductility, μ_f, which is the ratio of the maximum relative displacement, u_{max}, with respect to the frame yielding, Δ_{yf} (i.e., $\mu_f = u_{max} / \Delta_{yf}$).

NONLINEAR DYNAMIC RESPONSE

A design response spectrum was constructed based on the National Earthquake Hazard Reduction Program Recommended Provisions (NEHRP 2000) 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. (1999), a set of three spectra-compatible synthetic ground motions were generated to match the NEHRP 2000 target design spectrum.

Nonlinear time history analyses were conducted using the Structural Analysis Program, SAP 2000, (Computers and Structures, Inc. 2000). Analyses were performed for a range of systems 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 elastic period, T = 0.1 s, 0.25 s, 0.50 s, 1.0 s, 1.5 s, 2.0 s. The combination of these parameters resulted in 288 analyses for each ground motion generated (i.e., a total of 864 nonlinear time history analyses), where the response of the system is expressed in terms of the frame ductility, μ_f, as a function of the above system parameters.

Many alternatives for plotting results in either two or tri-dimensional charts were evaluated. However, for the purpose of parametric analysis, two dimensional charts were found to be more appropriate, since a matrix of plots can be formed for the whole set of parameters. Figure 2 shows the matrix of results corresponding to the nonlinear analyses conducted in terms of frame ductility, μ_f, as a function of the elastic period, T. Each 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 2 represent elastic behavior of the frame (which is the objective of the structural fuse concept). Allowable story drift, Δ_a, has also been introduced in terms of period limit, T_L, in order to control relative displacements between consecutive floors. Maintaining the lateral displacement under tolerable levels, instability problems due to secondary effects (frequently called P-Δ effects), as well as damage to some nonstructural components can be prevented.

For illustration purposes here, the NEHRP 2000 provisions recommended story drift limit of 2% is used, which translates into a limit period of about 0.5 s. This selected story drift limit along with the maximum allowable ductility (i.e., $\mu_f \leq 1.0$) define the range of acceptable solutions that satisfy the structural fuse concept as shaded areas on figures such as Figure 2.

Note that for large strength-ratio and period values (i.e., $\eta > 0.6$ and $T > 1.0$ 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).

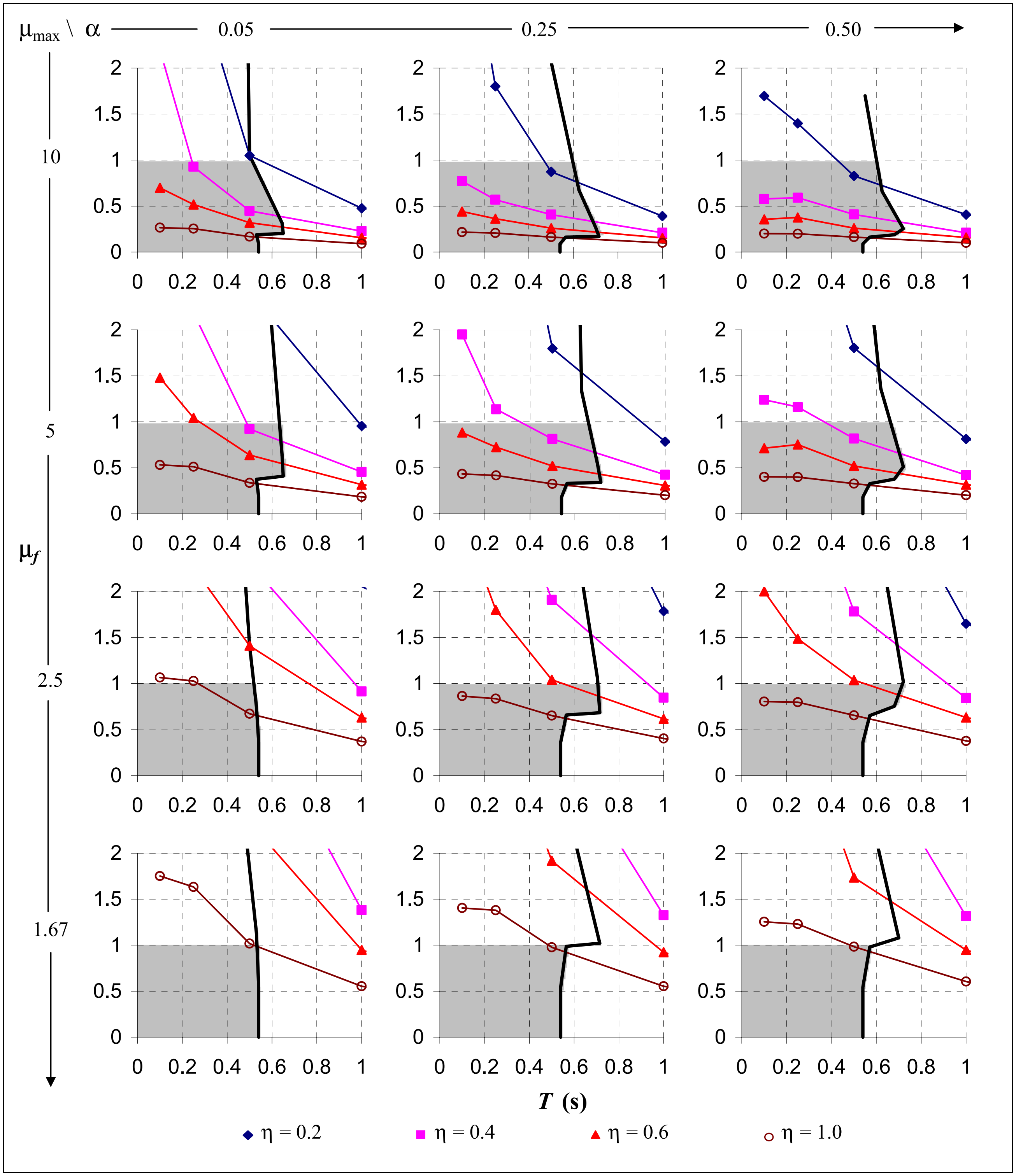

Figure 2. Regions of Admissible Solutions in terms of Frame Ductility, μ_f, and Story Drift of 2%

GENERIC RETROFIT CASE STUDY

In this section a case study comparison is made between seismic response of a SDOF without metallic dampers called the bare frame (BF) and the same SDOF system retrofitted with a structural fuse (SF). The same format used to present results for the SDOF system with structural fuses is used to show ductility demand of the BF system as a function of other characteristic parameters. The BF system is modeled as an elasto-plastic SDOF, i.e., with strain-hardening ratio and maximum displacement ductility taken as $\alpha = 1$, and $\mu_{max} = \infty$, respectively.

For the purpose of this case study, a BF with m = 0.044 kN·s^2/mm, K_f = 1.75 kN/mm, and V_{yf} = 127.4 kN (i.e., T = 1.0 s) is arbitrarily selected as a system that does not meet the drift requirements, and that would behave inelastically without seismic retrofit under an earthquake with peak ground acceleration of 0.58 g. That existing frame is then retrofitted by the addition of a structural fuse, with K_a = 5.25 kN/mm, and V_{yd} = 76.4 kN (i.e., $\alpha = 0.25$, $\mu_{max} = 5$, T = 0.5 s, and $\eta = 0.4$).

Figure 3 shows the response of both systems. The arrow in this figure shows how the behavior of the retrofitted system has been "moved" into the area of admissible solutions. The period is reduced to one half of the original value (T = 0.5s), and the frame ductility reduces from 1.9 to 0.8 (i.e., frame response remains elastic). Note the reduction of the strength-ratio of the systems (from 0.5 to 0.4). This is caused partly by the fact that for the chosen parameters for the case study the SF has a yield strength lower than that of the corresponding BF (i.e., $V_y < V_{yf}$).

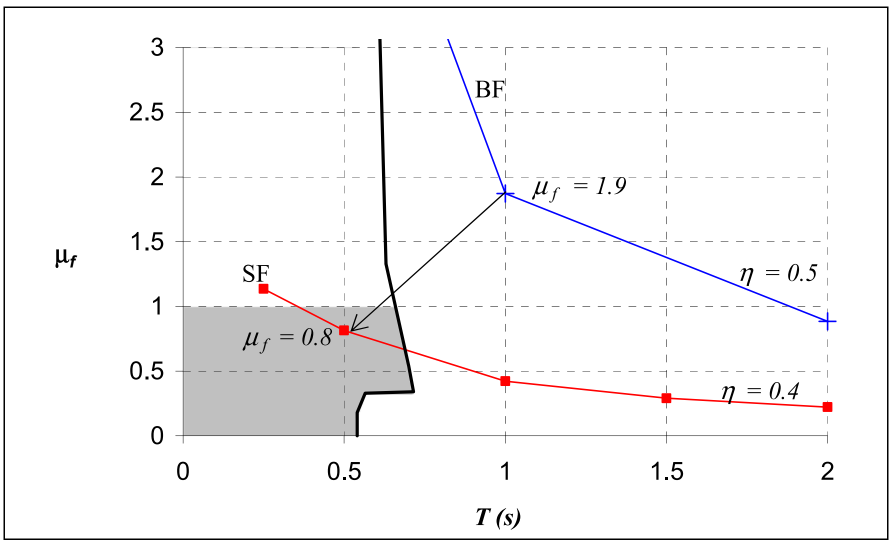

Figure 3. Bare Frame (BF) and Structural Fuse (SF) Response

Figure 4 shows the difference in energy dissipation between the BF and SF systems. Initially, in the BF, the energy is absorbed by viscous damping action while the frame is still elastic. Once the yield point is reached (at 4.7 s) the increment in input energy is dissipated mainly by hysteretic behavior of the frame. The inclusion of a structural fuse eliminates any frame hysteretic energy in the SF case (i.e., BF remains elastic), by introducing hysteretic action exclusively in the fuses, while the energy absorbed by viscous damping is not significantly affected. While in this example, the inclusion of a structural fuse causes an important increase in the input energy, this increase is totally absorbed by the fuse action, as shown in Figure 4.

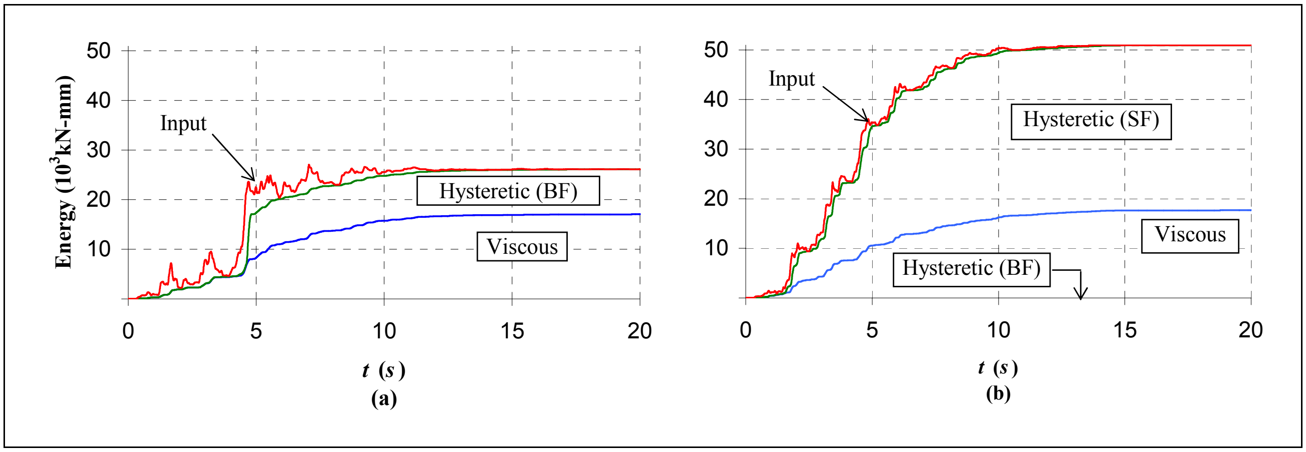

Figure 4. Energy Dissipated; (a) Bare Frame (BF), (b) Structural Fuse (SF)

DESIGN FOR A SPECIFIED SET OF PARAMETERS

The structural fuse concept can be satisfied by many combinations of parameters that define the structural system and its seismic response. However, some of these combinations may not be efficient (or even correspond to physical systems of realistic or practical sizes and dimensions). One possible measure of structural efficiency can be defined by the selection of the lightest possible steel structure that behaves in a desired way. To have an efficient (and realistic) design, it is useful to have some guidance on how (and in which order) to select the values for the key parameters that define satisfactory fuse systems.

For a target set of α, μ_{max}, and η values chosen to provide a satisfactory system response with the structural fuse concept, the procedure listed below shows how satisfactory designs can be obtained for a frame with given geometry, for given structural mass and yield strength of beams and columns, and for given seismic conditions.

Step 1. Define the allowable drift limit, Δ_a, as the upper bound lateral displacement (generally established as a percentage of the story height, H).

Step 2. Determine the elastic period limit, T_L, corresponding to the drift limit from the target design spectrum.

Step 3. Given the elastic period limit, T_L, a set of target parameters (i.e., α, μ_{max}, and η) may be selected from Figure 2 to satisfy the structural fuse concept.

Step 4. Given the mass, m, and the peak ground acceleration, PGA, calculate the required yield base shear, V_y, as:

$$V_y = \eta m PGA \tag{2}$$

Step 5. Calculate the target stiffness for the structure, K_1, the frame, K_f, and the damping system, K_a, respectively, as:

$$K_1 = \frac{4\pi^2}{T_L^{\ 2}} m \tag{3}$$

$$K_f = \alpha K_1 \tag{4}$$

$$K_a = (1-\alpha) K_1 \tag{5}$$

Step 6. Determine the yield displacements for the damping system, Δ_{ya}, and the frame, Δ_{yf}, respectively, as:

$$\Delta_{ya} = \frac{V_y}{K_1} \tag{6}$$

$$\Delta_{yf} = \mu_{\max} \Delta_{ya} \tag{7}$$

Step 7. Calculate the base shear capacity for the frame, V_{yf}, and the damping system, V_{yd}, respectively, as:

$$V_{yf} = K_f \Delta_{yf} \tag{8}$$

$$V_{yd} = K_a \Delta_{ya} \tag{9}$$

Step 8. Design the bare frame and structural fuse elements to match as close as possible the stiffness and the base shear capacity requirements defined by Equations (4), (5), (8), and (9). For some specific metallic dampers (e.g., Triangular Added Damping and Stiffness, T-ADAS and Shear Panels, SP), many combinations of sizes and properties are possible, and judgement must be exercised in designing these elements.

Step 9. Recalculate T, α, μ_{max}, and η parameters from the actual properties obtained in Step 8.

Step 10. Evaluate system response either by performing time history analysis, or indirectly by reading the charts.

Step 11. Verify that the system response is still satisfactory. If the structural fuse concept is not satisfied (i.e., $\mu_f > 1$, or lateral displacements greater than allowable story drift are obtained), new frame and damper properties may be chosen to improve the system seismic behavior, and the procedure is repeated from Step 9.

Step 12. Verify that the new parameters calculated in Step 9 are sufficiently close to the target parameters selected at the beginning of the process. If not, new frame and damper properties should be selected to match as close as possible the target parameters, and the procedure is repeated from Step 9. Alternatively, in a worse case scenario, it may be necessary to change the frame geometry, and might even be possible to change the system mass, although project constraints may make this difficult.

This general procedure can be used to design SDOF systems using metallic structural fuses. However, to retrofit an existing structure, the above procedure must be modified, because in addition to other constraints, the bare frame properties are generally fixed. It found from the results that, in most cases, α has an insignificant influence on the set of η and μ_{max} that can be chosen to satisfy the structural fuse concept. Therefore, in the retrofit case, η and μ_{max} may be selected regardless of α value, because α can no longer be freely selected; it must be calculated as follows, provided that the frame stiffness, K_f, and base shear capacity, V_{yf}, are known:

$$\alpha = \frac{V_{yf}}{\eta \mu_{\max} PGA} \tag{10}$$

where α shall not be greater than $(T_L^2\ K_f)$ / $(4\pi^2\ m)$ to satisfy the drift limit. Accordingly, the total stiffness, K_1, and the elastic period, T, may be calculated, respectively, as:

$$K_1 = \frac{\eta \mu_{\max} m PGA}{\Delta_{yf}} \tag{11}$$

$$T = 2\pi \sqrt{\frac{\Delta_{yf}}{\eta \mu_{\max} PGA}} \tag{12}$$

where K_1 shall be greater than $(4\pi^2 m) / (T_L^2)$, and T shall not be greater than T_L to satisfy the drift limit. Figure 5 shows the above procedure in a flowchart.

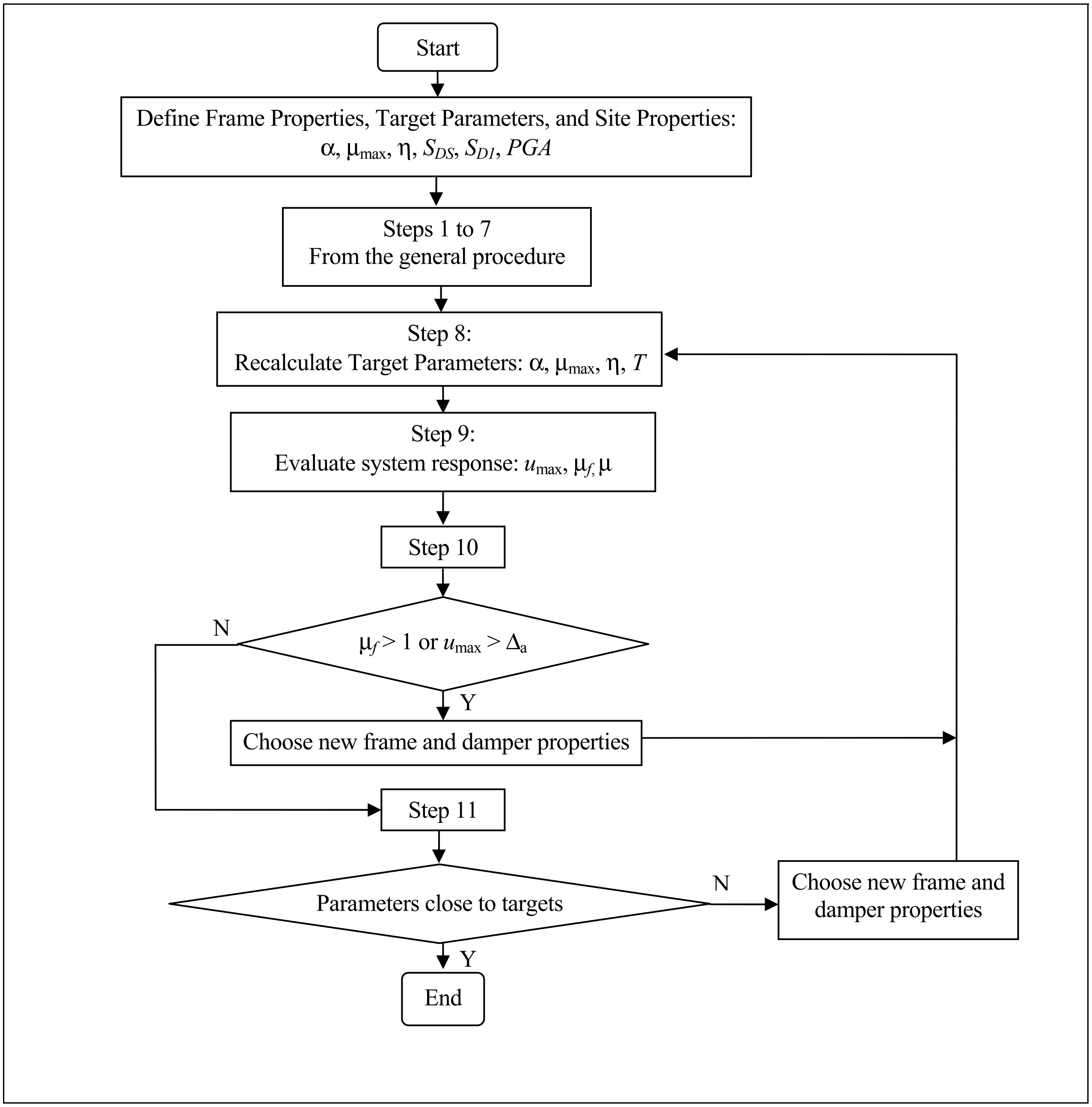

Figure 5. Procedure to Design Systems satisfying the Structural Fuse Concept

CONCLUSIONS

The structural fuse concept has been investigated 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 2, as a design tool, this can be represented graphically with shaded areas delimiting the range of admissible solutions. It was found that systems having $\mu_{max} \geq 5$ offer a broader choice of acceptable designs over a greater range of η values.

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