Analytical Response and Design of Buildings with Metallic Structural Fuses. I

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Abstract: Seismic design relies on inelastic deformations through hysteretic behavior. However, this translates into damage on structural elements, permanent system deformations following an earthquake, and possibly high cost for repairs. An alternative design approach, proposed in the past, is to concentrate damage on disposable and easy to repair structural elements (i.e., "structural fuses"), whereas the main structure is designed to remain elastic or with minor inelastic deformations. The implementation of the structural fuse concept into actual buildings would benefit from a systematic and simple design procedure. Such a general procedure is proposed here for designing new or retrofitted structures. The proposed structural fuse design procedure for multi-degree-of-freedom structures relies on results of a parametric study (presented in the paper), considering the behavior of nonlinear single degree of freedom systems subjected to synthetic ground motions. Nonlinear dynamic response is presented in dimensionless charts normalized with respect to key parameters. The proposed design procedure is illustrated as an example of application using Buckling-restrained braces as metallic structural fuses. This example is used in an experimental project (which is described in a companion paper) as a proof of concept to the developed design procedure.

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Introduction

Typically, in seismic design, code-specified loads are reduced by a response modification factor, R, which allows the structure to undergo inelastic deformations, whereas most of the seismic energy is dissipated through hysteretic behavior. Designs have always (implicitly or explicitly) relied on this reduction in the design forces. This methodology relies on the ability of specially detailed ductile structural elements to accommodate the inelastic deformations, without compromising the stability of the structure. However, inelastic behavior translates into some level of damage on these elements, permanent system deformations following an earthquake, and possibly high cost for repairs (sometimes, repairs are not viable, even though the structure has not collapsed, and the building must be demolished).

To achieve stringent seismic performance objectives for buildings, an alternative design approach is to concentrate damage on disposable and easy to repair structural elements (i.e., "structural fuses"), whereas the main structure is designed to remain elastic or with minor inelastic deformations. Following a damaging earthquake, only these special elements would need to be replaced (hence the "fuse" analogy), making repair works easier and more expedient. Further, in that instance, self-recentering of the structure would occur once the ductile fuse devices are removed, i.e., the elastic structure would return to its original undeformed position.

The structural fuse concept has not been consistently defined in the past. In some cases, "fuses" have been defined as elements with well-defined plastic yielding locations, but not truly replaceable as a fuse. For instance, Roeder and Popov (1977) called the segment of the beam yielding in shear in an eccentric braced frame a "ductile fuse" because of its energy dissipation capability. Although this system has a good seismic behavior, such links are not readily disposable elements (beams would need shoring, floor slabs might require repairs, etc.). Other researchers have used the term "structural fuse" in the same perspective for different types of structural systems (e.g., Aristizabal-Ochoa 1986; Basha and Goel 1996; Carter and Iwankiw 1998; Sugiyama 1998; Rezai et al. 2000; to name a few). In some other cases, structural fuses were defined as elements with well-defined plastic yielding locations and used more in the context of reducing (as opposed to eliminating) inelastic deformations of existing moment-resisting frames (also termed to be a "damage control" strategy) (Wada et al. 1992; Connor et al. 1997; Wada and Huang 1999; Wada et al. 2000; Huang et al. 2002). In applications consistent with the definition of interest here, fuses were used to achieve elastic response of frames that would otherwise develop limited inelastic deformations for high-rise buildings having large structural periods (i.e., $T > 4$ s) (e.g., Shimizu et al. 1998; Wada and Huang 1995), or for systems with friction brace devices intended to act as structural fuses (e.g., Filiatrault and Cherry 1989; Fu and Cherry 2000).

Generally, due to the large number of complex parameter interdependencies that exist in systems with structural fuses, the design procedures developed for these systems have relied on nonlinear time history analyses. In that perspective, a systematic and simplified design procedure to achieve and implement a structural fuse concept that would limit damage to disposable

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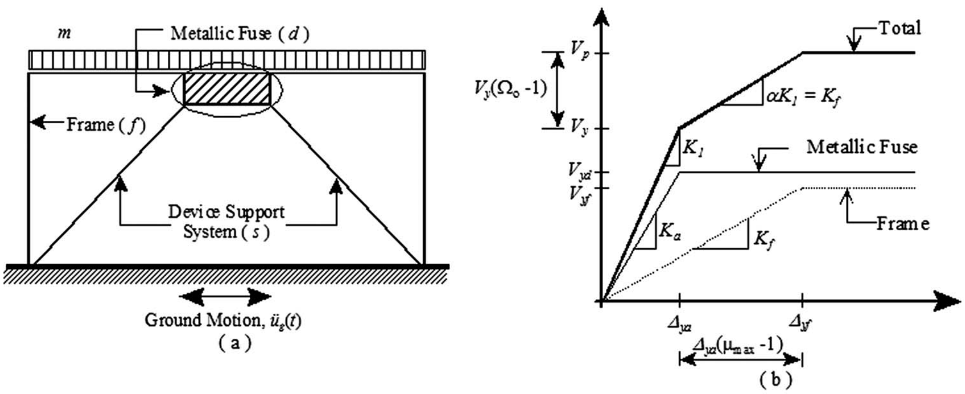

Fig. 1. (a) Sample model of a SDOF system with metallic fuses; (b) general pushover curve

structural elements for any general structure, without the need for complex analyses, can be helpful. Such a general procedure is proposed here for designing and retrofitting purposes.

In this paper, the structural fuses are passive energy dissipation (PED) devices, (a.k.a. hysteretic dampers) designed to yield (and hence absorb energy) well before the frame yields. The proposed structural fuse design procedure for multi-degree-of-freedom (MDOF) structures relies on results of a parametric study (presented here), 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 in the design process. Although, there are no limits to the type and characteristics of MDOF systems for which the procedure rooted in nonlinear dynamic analysis of SDOF systems is valid, the proposed design process has been illustrated for three different types of PED devices working as metallic structural fuses in Vargas and Bruneau (2006a), namely: buckling-restrained brace (BRB), triangular added damping and stiffness systems, and shear panel systems. Only the former is fully presented in this paper.

Parametric Formulation

Fig. 1 shows a general pushover curve and the model for a SDOF structure, in which frame and metallic fuses are represented by elastoplastic springs acting in parallel. The total curve is trilinear with the initial stiffness, K_1, calculated by adding the stiffness of the frame and the structural fuses, K_f and K_a, respectively. Once the structural fuses reach their 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 defining parameters used in this study are obtained from Fig. 1: the stiffness ratio, α, and the maximum displacement ductility, μ_{max}.

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

$$\alpha = \frac{1}{1 + (K_a/K_f)} \tag{1}$$

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 structural fuses, Δ_{ya}. In other words, μ_{max} is the maximum displacement ductility that the metallic fuses experience before the frame undergoes inelastic deformations. This parameter can be written as

$$\mu_{max} = \frac{\Delta_{yf}}{\Delta_{ya}} \tag{2}$$

with μ_{max} being greater than one.

In Fig. 1, V_{yf} and V_{yd} are the base shear capacity of the bare frame and the structural fuses, respectively; and V_y and V_p are the total system yield strength and base shear capacity, respectively. Further, note that in Fig. 1, the calculation of the postyielding stiffness, αK_1, defines a relationship between α and μ_{max}, equal to

$$\Omega_o = \alpha(\mu_{max} - 1) + 1 \tag{3}$$

where Ω_o=overstrength factor, defined as

$$\Omega_o = \frac{V_p}{V_y} \tag{4}$$

Pushover curves for different values of α and μ_{max} are presented in Fig. 2, 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 Fig. 1. As a result, Fig. 2 also shows the structural fuses and frame capacities as percentages of the total base shear capacity. The frame contribution to the total base shear capacity increases with both α and μ_{max}, whereas the structural fuses contribution decreases with increases in α and μ_{max} values. Note that the overstrength factor, Ω_o, is proportional to the frame contribution to the total base shear capacity. Note that this model does not include deterioration of stiffness and strength, because in experimental studies metallic fuses have shown stable hysteretic loops at sustained large levels of displacements (Tsai et al. 1993; Vargas and Bruneau 2006b).

For a nonlinear SDOF with hysteretic behavior, Mahin and Lin (1983) proposed a normalized version of the nonlinear dynamic equation of motion adapted as shown in the following:

$$\ddot{\mu}(t) + \frac{4\pi\xi}{T}\dot{\mu}(t) + \frac{4\pi^2}{T^2}\rho(t) = -\frac{4\pi^2}{T^2\eta}\left[\frac{\ddot{u}_g(t)}{\ddot{u}_{g\,\max}}\right] \tag{5}$$

where $\mu(t)$=system response in terms of displacement ductility, ξ=damping ratio; T=elastic period of the structure; $\rho(t)$=ratio between the force in the inelastic spring and the yield strength of the system; $\ddot{u}_g(t)$=ground acceleration; and η=strength-ratio determined as the relationship between the yield strength and the

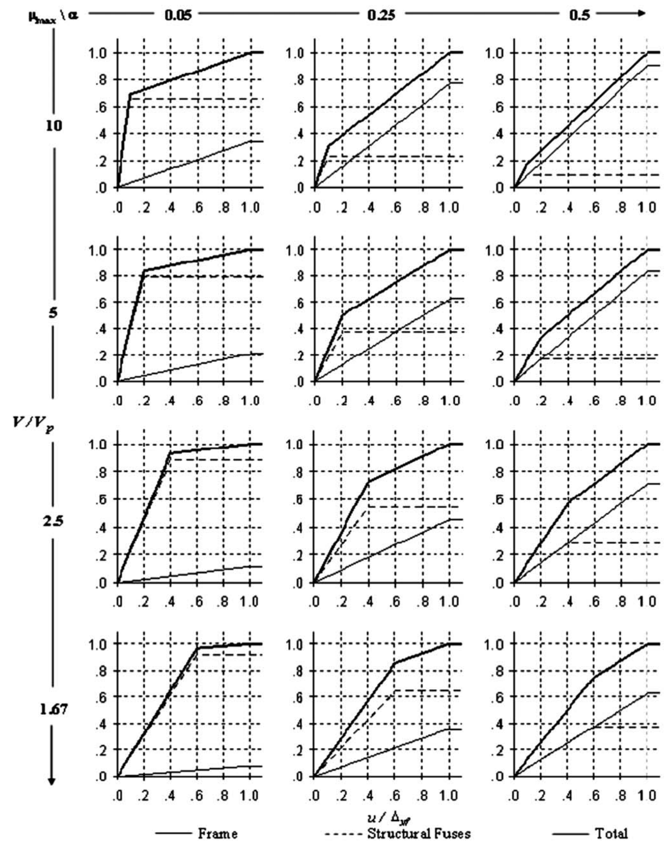

Fig. 2. Pushover curves for the studied cases

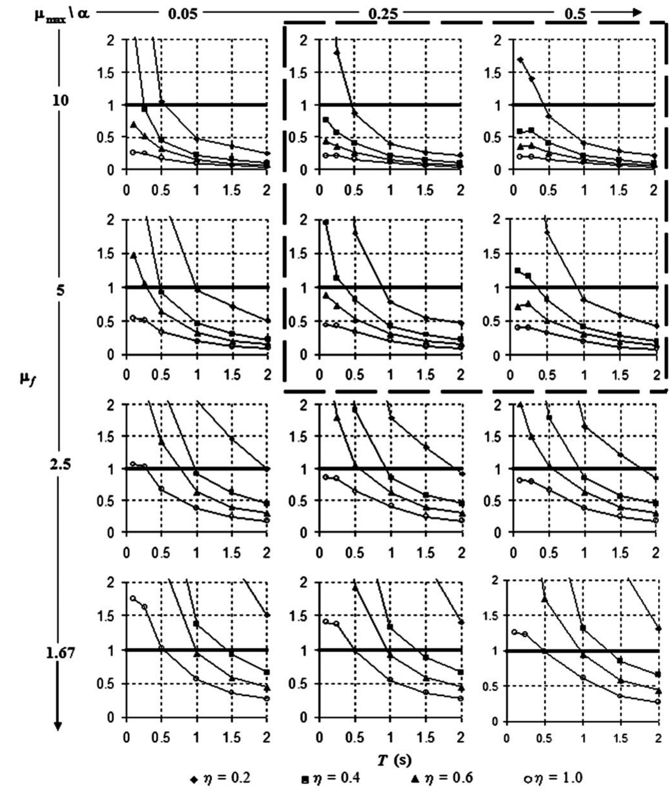

Fig. 3. Response in terms of frame ductility (box shows preferred recommended design solutions)

maximum ground force applied during the motion, defined as

$$\eta = \frac{V_y}{m\ddot{u}_{g\,\max}} \tag{6}$$

where $\ddot{u}_{g\,\max}$=peak ground acceleration. For a specific ground motion, $\ddot{u}_g(t)$, Eq. (5) can be solved in terms of the previous parameters, assuming a damping ratio, ξ, of 0.05 in this study.

Nonlinear Dynamic Response

In this study, one of the SAC model buildings was selected as the prototype for the experiment. Recall that SAC was a joint effort between the Structural Engineers Association of California, the Applied Technology Council, and California Universities for Research in Earthquake Engineering, established to address performance problems of steel moment–frame connections found after the 1994 Northridge earthquake (FEMA 2000). The selected SAC project consists of a three-story steel building located on stiff soil [soil type B as per FEMA 450 (2003)].

A design response spectrum was constructed based on the National Earthquake Hazard Reduction Program (NEHRP) Recommended Provisions (FEMA 2003) for the West Coast and site soil-type class B. Accordingly, the design spectral accelerations corresponding to the earthquake with 10% of probability of being exceeded in 50 years are S_{DS}=1.30 g, and S_{DI}=0.58 g (i.e., $\ddot{u}_{g\,\max}$=0.40S_{DS}=0.52 g). In order to avoid the uncertainties in estimating the actual ground motion at the site (which is not an objective of this study) a set of synthetic ground motions were used. Using the Target Acceleration Spectra Compatible Time Histories (TARSCTHS) code, by Papageorgiou et al. (1999), three spectra-compatible synthetic ground motions were generated to match the NEHRP 2003 target elastic design spectrum for 5% of critical damping.

Nonlinear time history analyses were conducted using the structural analysis program, SAP 2000 (Computers and Structures Inc. 2000). Analyses were performed for the range of systems described in Fig. 2, using the following parameters: α=0.05, 0.25, and 0.50; $\mu_{\max}$=10, 5, 2.5, and 1.67; η=0.2, 0.4, 0.6, and 1.0; and T=0.1, 0.25, 0.50, 1.0, 1.5, and 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). Some arbitrarily chosen cases were verified using a suite of seven artificially generated earthquakes [as commonly done in accepted practice (FEMA 2003)], and the results were found to be similar to those obtained using three synthetic ground motions.

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

$$\mu_f = \frac{u_{\max}}{\Delta_{yf}} \tag{7}$$

$$\mu = \frac{u_{\max}}{\Delta_{ya}} \tag{8}$$

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

Many alternatives for plotting results in either two- or three-dimensional charts were evaluated. However, for the purpose of parametric analysis, two dimensional charts were found to be more appropriate, as a matrix of plots can be constructed for the

Table 1. Minimum η Values to Satisfy the Structural Fuse Concept

	T (s)					
μ_{max}	0.1	0.25	0.5	1	1.5	$\geqslant 2$
			$\alpha = 0.05$			
1.67	N/A	N/A	1.00	0.60	0.40	0.35
2.5	N/A	1.00	0.80	0.40	0.30	0.20
5.0	0.80	0.60	0.40	0.20	0.15	0.10
10	0.50	0.40	0.20	0.10	0.08	0.05
			$\alpha = 0.25$			
1.67	N/A	N/A	1.00	0.60	0.40	0.35
2.5	1.00	1.00	0.60	0.40	0.30	0.20
5.0	0.60	0.50	0.40	0.20	0.15	0.10
10	0.40	0.30	0.20	0.10	0.08	0.05
			$\alpha = 0.50$			
1.67	N/A	N/A	1.00	0.60	0.40	0.35
2.5	1.00	1.00	0.60	0.40	0.30	0.20
5.0	0.50	0.50	0.40	0.20	0.15	0.10
10	0.30	0.30	0.20	0.10	0.08	0.05

set of parameters considered. Fig. 3 shows the matrix of results corresponding to the 864 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, whereas each curve represents a constant strength-ratio, η. All the points having $\mu_f < 1$ in Fig. 3 represent elastic behavior of the frame (which is the objective of the structural fuse concept). Although the charts shown in Fig. 3 can be used directly to read ductility demands as a function of other defined parameters, there may be instances where closed form solutions are desirable, as for use in computer programs or in the formulation of design algorithms. Such equations can be formulated through regression analyses and are presented in Vargas and Bruneau (2006a).

Allowable Story Drift

In some instances story drift (maximum relative displacement between consecutive floors) may need to be controlled, e.g., to prevent excessive $P-\Delta$ effects, excessive inelastic strains and low cycle fatigue of metallic fuses, or damage to nonstructural elements, such as partitions, ceilings, enclosures, and windows and door frames, that are sensitive to lateral deformations.

Consequently, story drift shall be kept less than a selected limit to maintain the building lateral displacement under a tolerable level. In the case of MDOF systems the maximum inelastic displacement for a given structure may be considered approximately equal to the maximum displacement that would be obtained if the structure behaved elastically. The allowable drift can then be converted into a corresponding period limit, T_L, by

$$T_L = \frac{4\pi^2 \Delta_{ar}}{\Gamma_1 \phi_{r1} S_{D1}} \tag{9}$$

where Δ_{ar} = allowable displacement of the roof, taken as a percentage of the building height (usually between 0.5 and 2%); ϕ_{r1} = first mode component of the roof displacement; and Γ_1 = modal participation factor of the first mode, calculated as

$$\Gamma_1 = \frac{\boldsymbol{\phi}_1^T \mathbf{M} \tilde{\mathbf{1}}}{\boldsymbol{\phi}_1^T \mathbf{M} \boldsymbol{\phi}_1} \tag{10}$$

where $\mathbf{M}$ = known mass matrix; $\boldsymbol{\phi}_1$ = vector corresponding to the first mode shape; and $\tilde{\mathbf{1}}$ = vector of unit values, which represents a rigid body motion of the system due to horizontal ground excitation. Note that, to determine the modal participation factor, Γ_1, a mode shape, ϕ_1, should be assumed. Many approaches have been proposed to select appropriate mode shapes, and obtain "reasonable" estimation of system dynamic characteristics (Clough and Penzien 1993). In this study, a linear mode shape is assumed, as the results showed it is sufficiently accurate to determine the system dynamic properties (Vargas and Bruneau 2006a).

In summary, the structural fuse concept is fully satisfied when the frame remains elastic (i.e., $\mu_f \leqslant 1.0$), and the building is designed to have a period shorter than the limit period associated with the story drift limit (i.e., $T \leqslant T_L$). Minimum η values that satisfy the structural fuse concept are presented in Table 1, which was built based on the results shown in Fig. 3 in order to make more expedient the design process.

Note that for combined 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 fuses only provide additional stiffness with no energy dissipation. Elastic behavior of the metallic fuses 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).

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.

The procedure listed here shows how satisfactory designs can be obtained for a frame with given geometry, for given structural mass and yield stress of beams and columns, and for given seismic conditions.

- Step 1. Define the allowable drift limit 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. A minimum η value may be selected from Table 1 for a given set of target parameters α and $\mu_{\max}$, and recognizing that the actual period should be shorter than the elastic period limit, T_L. Therefore, a too small value should not be assigned to α. Based on results and observations made in this research, it has been found that $\alpha \geqslant 0.25$ provide adequate results for most cases. Selecting such an α value also helps to ensure that beams and columns have enough capacity to transfer yielding forces from metallic fuses (capacity design principle), and that the frame elements are not too flexible in comparison to the structural fuse system. It is also recommended that $\mu_{\max}$ should be chosen large enough to maximize the metallic fuses energy dissipation capacity and to prevent inelastic deformations on the frame. In this perspective, values of $\mu_{\max} \geqslant 5$ were found to be appropriate for most cases. Note that this region of viable designs with $\alpha \geqslant 0.25$ and $\mu_{\max} \geqslant 5$ has been boxed in Fig. 3, along with the cutoff target of $\mu_f \leqslant 1$ for the elastic frame.
- Step 4. Given the mass, m, and the peak ground acceleration, $\ddot{u}_{g\,\max}$, calculate the required yield base shear, V_y, and base shear capacity, V_p, as

$$V_y = \eta m \ddot{u}_{g\,\max} \tag{11}$$

$$V_p = \Omega_o V_y \tag{12}$$

- Step 5. Calculate the base shear capacity for the frame, V_{yf}, and the structural fuses, V_{yd}, respectively, as

$$V_{yf} = \alpha \mu_{\max} V_y \tag{13}$$

$$V_{yd} = (1 - \alpha) V_y \tag{14}$$

In this study, these shears are vertically distributed through the height of the building, using a vertical distribution function proportional to the assumed mode shape, ϕ_1.

- Step 6. Design frame members and metallic fuses for V_{yf} and V_{yd}, respectively. Follow capacity design principles to protect beams and columns against undesirable failure mechanisms.
- Step 7. Determine the actual parameters (i.e., α, $\mu_{\max}$, and η) for the designed system from a static pushover analysis, conducted using a load pattern proportional to ϕ_1.
- Step 8. Solve the dynamic eigenvalue problem, and obtain the fundamental period of vibration of the structure, T.
- Step 9. Evaluate system response either by performing time history analysis, or indirectly by reading the charts (Fig. 3), or using approximate closed form solutions (Vargas and Bruneau 2006a).
- Step 10. Verify that the system response is still satisfactory. If the structural fuse concept is not satisfied, increase frame and fuses stiffness and strength (i.e., greater K_f, V_{yf}, K_d, and V_{yd}) to improve the system seismic behavior, and repeat the procedure from Step 7, until a satisfactory response is achieved. For example, if the story drift limit is not satisfied, the system should be stiffened (i.e., greater K_f and K_d). On the other hand, if the frame undergoes inelastic deformations (i.e., $\mu_f > 1$), the system should be strengthened (i.e., greater V_{yf} and V_{yd}).

This general procedure can be used to design MDOF systems using metallic structural fuses. However, to retrofit an existing structure, the above-mentioned procedure must be modified, because in addition to other constraints, the bare frame properties are generally fixed. It may be noted from Table 1 that, in most cases, α has an insignificant influence on the set of η and $\mu_{\max}$ that can be chosen to satisfy the structural fuse concept. Therefore, in that case, η and $\mu_{\max}$ may be selected from Table 1 regardless of α, 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:

$$\alpha = \frac{V_{yf}}{\eta \mu_{\max} m \ddot{u}_{g\,\max}} \tag{15}$$

where α shall not be greater than $(T_L^2 K_f)/(4\pi^2 m)$ to satisfy the drift limit defined in Eq. (9). Accordingly, the elastic period, T, may be calculated, respectively, as

$$T = 2\pi \sqrt{\frac{\Delta_{yf}}{\eta \mu_{\max} \ddot{u}_{g\,\max}}} \tag{16}$$

where T shall not be greater than T_L to satisfy the drift limit defined in Eq. (9).

As a result of the above-mentioned constraints, only the structural fuses properties can be modified to satisfy the retrofit design requirements. Note that η and $\mu_{\max}$ are the only parameters that can be arbitrarily specified, as α and T depend directly on the existing frame properties.

Note that the structural fuse concept objective can be achieved with this general design procedure for earthquakes that do not exceed the level of design specifications. High variability of earthquake records makes it possible that the target design objective may be violated for a given earthquake exceeding the design spectrum, but it should be recognized that in such cases, response of the system remains ductile and safe. However, to minimize such probability of exceeding the design level, it is recommended to use target design spectra at maximum credible earthquake level (e.g., 2% of probability of being exceeded in 50 years). Subsequent sections present an example of how the structural fuse concept can be applied to design MDOF systems.

Design Example

The selected SAC project consists of a three-story steel building with seven frames in the north–south (NS) direction and five frames in the east–west direction, as shown in Fig. 4. Moment-resisting frames are represented by solid lines on the perimeter, and gravity frames are shown as dotted lines. According to FEMA 355-C, the project is a standard office building located on stiff soil (soil type B as per FEMA 450). As reported in FEMA 355-C, designs of the moment-resisting frames in the two orthogonal directions were very similar, therefore, only half of the structure is considered in the analysis. In this study, one single bay of the exterior frames in the NS direction is considered as a substructure for design purposes. This prototype substructure is designed following the procedure presented for MDOF buildings using BRBs as structural fuses.

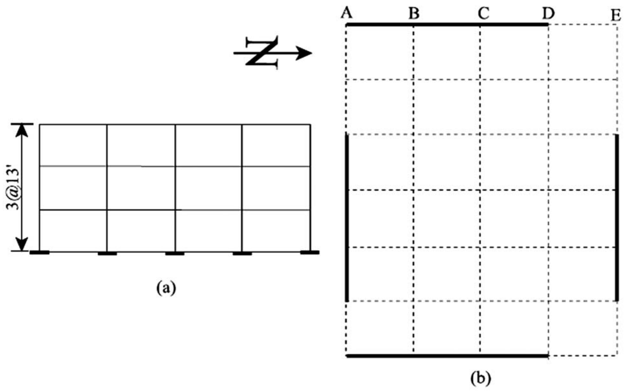

Fig. 4. Prototype: (a) elevation view; (b) plan view (FEMA 2000)

Based on the loading definition described on FEMA 355-C, the seismic mass of the entire structure is 0.9565 kN s^2/mm (65.53 kip s^2/ft) for the typical floors, and 1.0349 kN s^2/mm (70.90 kip s^2/ft) for the roof. The total mass of the building is 2.9480 kN s^2/mm (201.96 kip s^2/ft), which corresponds to a total weight of 28.93 MN (6,503 kips). As only one bay of the exterior frame is considered for the analysis, one-sixth of the total mass is assigned to the substructure as 0.4913 kN s^2/mm (34.49 kip s^2/ft), which corresponds to a weight of 4,822 kN (1,084 kips). Fig. 5 shows the geometry and mass distribution for the studied frame. Note that the BRBs are placed in diagonal configuration at every story.

Steel yield strength of 345 MPa (50 ksi) and 290 MPa (42 ksi) is used to design frame elements and the BRBs, respectively. The prototype is designed for the set of previously described ground motions scaled to a peak ground acceleration of 0.375 g. This value was selected to satisfy the capability of the shake tables at University at Buffalo, which will be used in the companion paper (Vargas and Bruneau 2009) to experimentally validate the procedure. Mass matrix for this building can be obtained from Fig. 5 as

$$\mathbf{M} = \begin{bmatrix} 0.1594 & 0 & 0 \\ 0 & 0.1594 & 0 \\ 0 & 0 & 0.1725 \end{bmatrix} \frac{\text{kN s}^2}{\text{mm}}$$

In order to determine the system dynamic properties with sufficient accuracy, a linear mode shape was assumed as

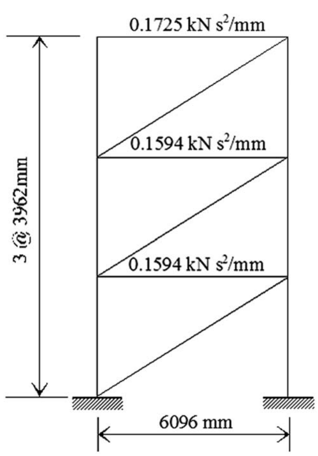

Fig. 5. Geometry and mass distribution for the prototype

Table 2. Summary of Components for the Prototype System

Story	Beams	Columns	BRB (mm)
3	W16×36	W14×74	PL 73×10
2	W24×62	W14×74	PL 121×10
1	W24×76	W14×74	PL 143×10

$$\boldsymbol{\phi}_1^T = [0.33 \quad 0.67 \quad 1.00]$$

The elastic period limit, T_L, and the modal participation factor, Γ_1, are obtained from Eqs. (9) and (10), respectively. In this particular case, $T_L = 1.80$ s, and $\Gamma_1 = 1.27$, which corresponds to an allowable story drift of 2% (i.e., $\Delta_{ar} = 238$ mm). Based on the prior observation, values of $\alpha \geq 0.25$ and $\mu_{max} \geq 5$ provide systems with appropriate seismic performance. Therefore, values for parameters α and μ_{max} are arbitrarily selected as 0.25 and 5, respectively, as target parameters. Note that other parameter combinations could be selected (e.g., $\alpha = 0.40$ and $\mu_{max} = 8$) provided they are in the target range. Recognizing that the elastic period, T, needs to be shorter than 1.80 s, $\eta = 0.25$ is chosen from Table 1 for $\alpha = 0.25$ and $\mu_{max} = 5$, assuming that the actual period will be close to 1 s.

Analytical Results for the Prototype

From the target parameters (i.e., $\alpha = 0.25$, $\mu_{max} = 5$, $\eta = 0.25$, and $T \leq 1.80$ s) and using Eqs. (11)–(14), the required yield base shear, V_y, total base shear, V_p, base shear capacity for the frame, V_{yf}, and for the damping system, V_{yd}, are calculated as 452, 903, 565, and 339 kN, respectively, for the design earthquake scaled to a peak ground acceleration of $0.375g$. Consequently, frame members and BRBs are designed for their required base shear capacities, and their properties are shown in Table 2. Note that the cross-sectional area of braces consists of rectangular steel plates (in Table 2 only the braces core properties are presented).

Actual parameters and elastic period are determined from pushover and eigenvalue analyses, respectively, as $\alpha = 0.42$, $\mu_{max} = 2.92$, $\eta = 0.30$, and $T = 0.98$ s, which are in fair agreement with the previously calculated target parameters, recalling that some deviations from target parameters may result from the selection of available structural elements for the actual system, and other underlying assumptions and approximations (e.g., the use of a SDOF based procedure). However, actual parameters for the prototype system result in a behavior that still falls within the region of admissible solutions according to the graphic representation of Fig. 3. Fig. 6 shows the pushover curves corresponding to the bare frame, BRBs, and the total base shear capacity of the system. Yield displacements of 40 and 118 mm for the BRBs and

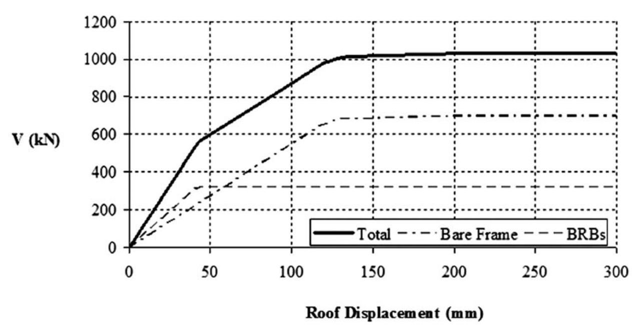

Fig. 6. Pushover curve for the prototype system

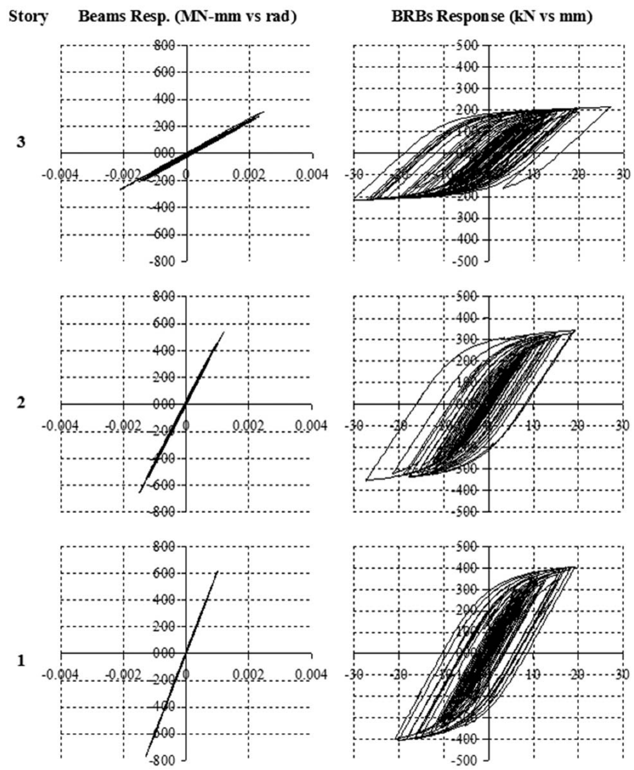

Fig. 7. Hysteresis loops for the prototype system

the bare frame, respectively, may be observed on this plot. A yield base shear of 535 kN and a total shear capacity of 1,032 kN may also be noted, which exceed the minimum values required by the design procedure (i.e., $V_y = 452$ kN and $V_p = 903$ kN).

Seismic response of the system is then evaluated by nonlinear time history analysis to verify that the structural fuse objective is fully satisfied. Fig. 7 shows the maximum response in terms of hysteresis loops of beams and BRBs at each story. Note that beams respond elastically, whereas hysteretic energy is completely dissipated by inelastic behavior of BRBs at every story. A maximum roof displacement of 85 mm was obtained from the analysis. Note that this roof displacement corresponds to a frame ductility of 0.71 (i.e., $\mu_f < 1.0$, which is required to avoid inelastic deformations of the frame members). Further, the maximum observed story drift was 0.74%, which is less than the limit of 1% determined from the pushover curve to fully satisfy the structural fuse concept (see Fig. 6).

Conclusions

In this study, clarification has been provided to the previously used definition of structural fuses. This study specifically defines structural fuses as sacrificeable and easy-to-repair elements designed to protect the primary structure of a building, simultaneously allowing automated self-centering of the frame during fuse replacement (hence the fuse analogy). The parameters that govern the seismic behavior of buildings designed or retrofitted with metallic dampers working as structural fuses were identified. Seismic response was validated through parametric analyses of the studied systems, and design guidance was provided for the sizing of the fuse system as a function of the total system strength.

The validity of the proposed design procedure was thoroughly verified through several analytical examples of MDOF systems designed and retrofitted with different types of structural fuses (Vargas and Bruneau 2006a). However, for brevity purposes, only one example has been presented in this paper. Therefore, it is concluded that the proposed procedure is sufficiently robust and reliable to design structural fuse systems with satisfactory seismic performance. Further, the procedure was also experimentally validated, and results are presented in the companion paper (Vargas and Bruneau 2009).

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. It may be observed that systems having $\mu_{max} \geqslant 5$ 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 amounts of hysteretic energy can be dissipated by system having identical ductility demands.

As demonstrated in the examples of application, by using the listed procedure, buildings can be systematically designed or retrofitted using metallic fuses to protect beams and columns from inelastic deformations. From Fig. 2 it may be noted, on one hand, that systems having $\alpha < 0.25$ require large fuse elements (i.e., large metallic fuses) to meet the objectives of the structural fuse concept. On the other hand, systems having $\mu_{max} < 5$ also require large fuse elements and high values of F_{yd}, which may be difficult to implement (not to mention that having $\mu_{max} < 5$ implies less ductile behavior of the structural fuse, which is less desirable. Therefore, it is recommended for best seismic performance to use $\alpha \geqslant 0.25$ and $\mu_{max} \geqslant 5$ as target parameters (this region of viable designs is boxed in Fig. 3, along with the cutoff target of $\mu_f \leqslant 1$ for the elastic frame).

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