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SEISMIC DESIGN OF MULTI-STORY BUILDINGS WITH METALLIC STRUCTURAL FUSES

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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 is to concentrate damage on disposable and easy to repair structural elements (i.e., "structural fuses"), while the main structure is designed to remain elastic or with minor inelastic deformations. A systematic procedure is proposed in this paper to design buildings with metallic structural fuses. The proposed procedure has been illustrated as examples of application using Buckling-restrained braces working as metallic structural fuses.

Introduction

To achieve stringent seismic performance objectives for buildings, one design approach is to concentrate damage on disposable and easy to repair structural elements (i.e., "structural fuses"), while 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. Furthermore, 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. In 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, particularly 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.,

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T > 4 s) (e.g., Shimizu et al. 1998; Wada and Huang 1995), or for systems with friction brace dampers intended to act as structural fuses (e.g., Filiatrault and Cherry 1989; Fu and Cherry 2000).

A systematic and simplified design procedure to achieve and implement a structural fuse concept that would limit damage to disposable structural elements for any general structure, without the need for complex analyses, can be helpful. One such procedure is presented on the NEHRP Recommended Provisions (FEMA 450) in the perspective of dampers. Another procedure is proposed here focusing solely on hysteretic energy dissipation fuses for designing purposes.

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 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. The proposed design procedure has been illustrated using Buckling-restrained braces as metallic structural fuses.

Parametric Formulation

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

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

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

The maximum displacement ductility, μ_{max}, is the ratio of the frame yield displacement, Δ_{yf}, with respect to the yield displacement of the fuses system, 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:

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

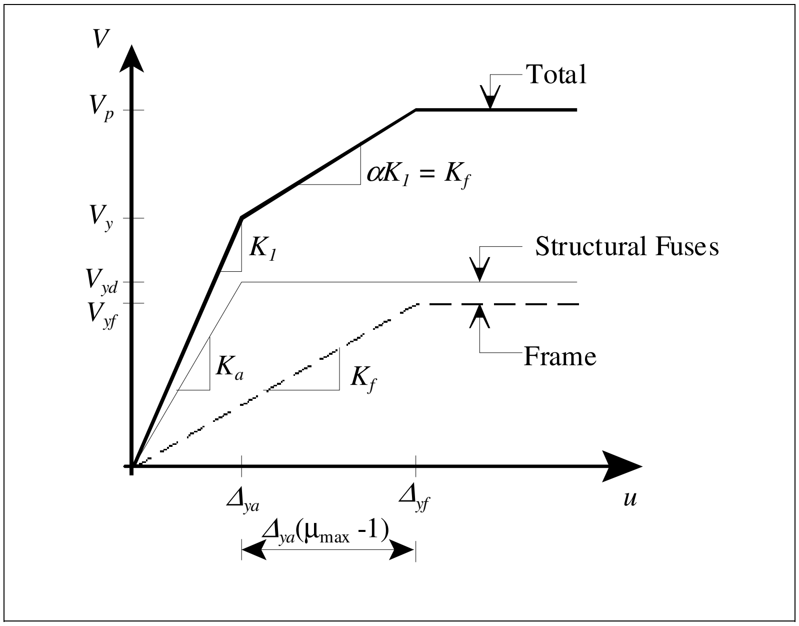

Figure 1. General Pushover Curve

In Fig. 1, V_{yf} and V_{yd} are the base shear capacity of the bare frame and the fuses system, respectively; and V_y and V_p are the total system yield strength and base shear capacity, respectively.

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 below:

$$\ddot{\mu}(t) + \frac{4\pi\xi}{T}\dot{\mu}(t) + \frac{4\pi^2}{T^2}\rho(t) = -\frac{4\pi^2}{T^2\eta}\left[\frac{\ddot{u}_g(t)}{\ddot{u}_{g\,\max}}\right] \tag{3}$$

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

$$\eta = \frac{V_y}{m\ddot{u}_{g\,\max}} \tag{4}$$

where $\ddot{u}_{g\max}$ is the peak ground acceleration. For a specific ground motion, $\ddot{u}_g(t)$, Eq. 3 can be solved in terms of the above parameters, assuming a damping ratio, ξ, of 0.05 in this study.

Nonlinear Dynamic Response

Design response spectrum was constructed based on the National Earthquake Hazard Reduction Program Recommended Provisions (NEHRP 2003) for a structure located in California on 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_{D1} = 0.58 g (i.e., $\ddot{u}_{g\max} = 0.40 S_{DS}$ = 0.52 g). Using the Target Acceleration Spectra Compatible Time Histories (TARSCTHS) code (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 using the following parameters: α = 0.05, 0.25, 0.50; $\mu_{\max}$ = 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, 1.5 s, 2.0 s.

The response of the system is expressed in terms of the frame ductility, μ_f, and the global ductility, μ, defined as $\mu_f = u_{\max} / \Delta_{yf}$ and $\mu = u_{\max} / \Delta_{ya}$, respectively. In these expressions, $u_{\max}$ is the maximum absolute displacement of the system, taken as the average of the maximum absolute responses caused by each of the applied ground motions. Fig. 2 shows the matrix of results corresponding to all nonlinear analyses conducted in terms of average frame ductility, μ_f, as a function of the elastic period, T. All the points having $\mu_f < 1$ in Fig. 2 (shown as shaded areas) represent elastic behavior of the frame (which is the objective of the structural fuse concept).

In some instances, 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 the following relationship:

$$T_L = \frac{4\pi^2 \Delta_{ar}}{\Gamma_1 \phi_{r1} S_{D1}} \tag{5}$$

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

$$\Gamma_1 = \frac{\Phi_1^T M \tilde{1}}{\Phi_1^T M \Phi_1} \tag{6}$$

where $\boldsymbol{M}$ is the known mass matrix, $\boldsymbol{\Phi_1}$ is the vector corresponding to the first mode shape, and $\tilde{1}$ is a vector of unit values.

In summary, the structural fuse concept is fully satisfied when the frame remains elastic (i.e., $\mu_f \leq 1.0$), and the building is designed to have a period shorter than the limit period associated with the story drift limit (i.e., $T \leq T_L$). Minimum η values that satisfy the structural fuse concept are presented in Table 1, which was built based on the results shown in Fig. 2.

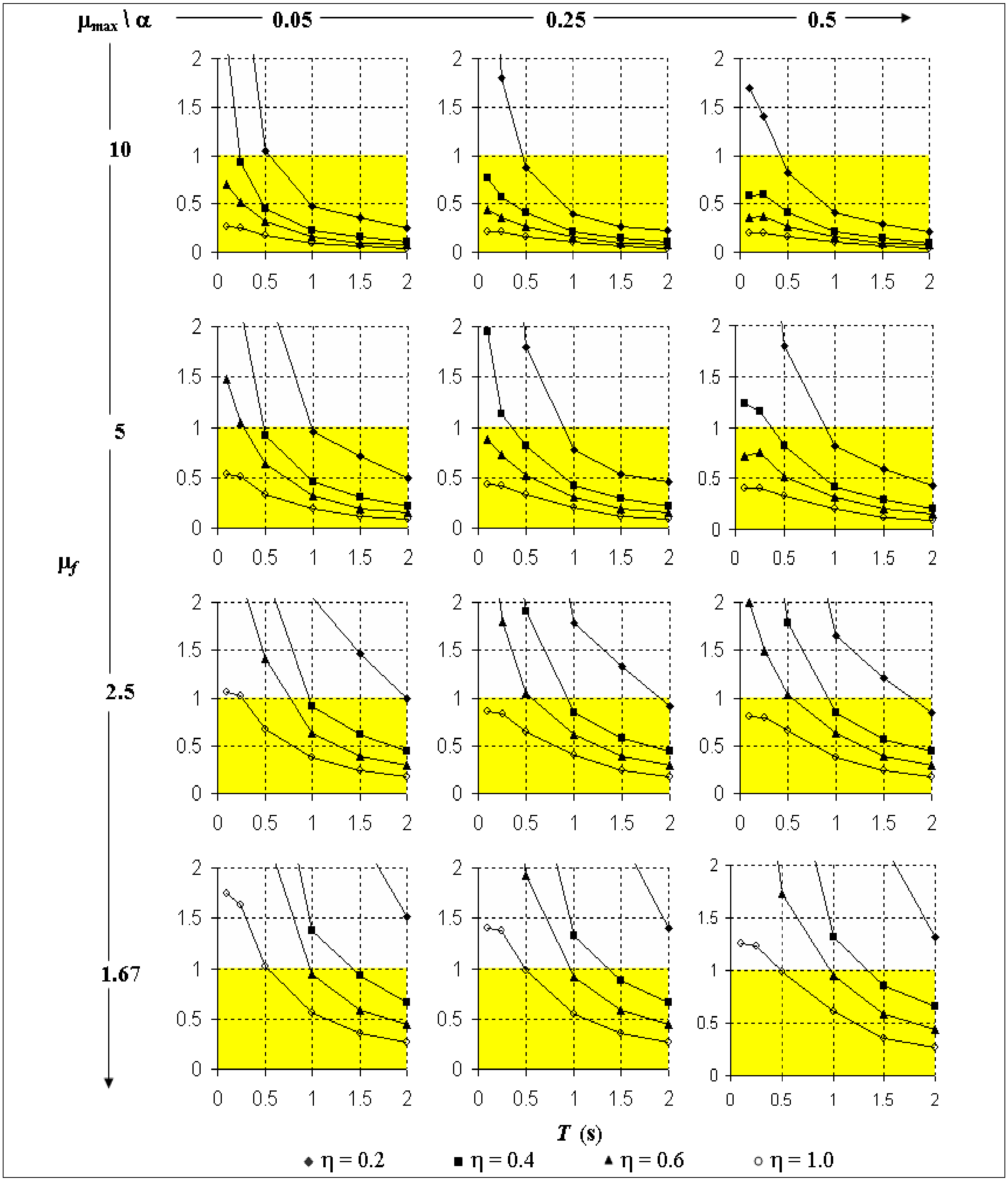

Figure 2. Response in terms of Frame Ductility

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

μ_{max} \ T (s) (1)	0.1 (2)	0.25 (3)	0.5 (4)	1 (5)	1.5 (6)	≥ 2 (7)
			$\alpha = 0.05$			
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

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 below shows how satisfactory designs can be obtained for a given frame.

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 (Eq. 5).

Step 3. A minimum η value may be selected from Table 1 for a given set of target parameters α and μ_{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 $0.25 \leq \alpha \leq 0.50$ 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 μ_{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 $\mu_{max} \geq 5$ were found to be appropriate for most cases.

Step 4. Given the mass, m, and the peak ground acceleration, $\ddot{u}_{gmax}$, calculate the required yield base shear, V_y as:

$$V_y = \eta m \ddot{u}_{g\max} \tag{7}$$

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

$$V_{yf} = \alpha \mu_{\max} V_y \tag{8}$$

$$V_{yd} = (1-\alpha) V_y \tag{9}$$

In this study, these specific shears are vertically distributed through the height of the building, using a vertical distribution function proportional to the assumed mode shape, $\mathbf{\Phi_1}$.

Step 6. Design frame members and metallic fuses for V_{yf} and V_{yd}, respectively. Capacity design principles should be followed to protect beams and columns against undesirable failure mechanisms.

Step 7. Determine the actual parameters (i.e., α, μ_{max}, and η) for the designed system from a static pushover analysis, conducted using a load pattern proportional to $\mathbf{\Phi_1}$.

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. 2), or using approximate closed form solutions (Vargas and Bruneau 2005).

Step 10. Verify that the system response is still satisfactory. If the structural fuse concept is not satisfied, increase frame and fuse 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}).

Design Example

Presented example consists of a transverse moment-resisting frame from the MCEER Demonstration Hospital, which is a four-story building modeled with masses lumped at floor levels (Yang and Whittaker 2002). For this particular example, the design was conducted using BRBs as metallic fuses. For expediency, a linear mode shape is assumed since it showed to be sufficiently accurate to determine the system dynamic properties.

According to Step 3 of the design procedure, intermediate values of $\alpha = 0.25$ and $\mu_{max} = 5$ are used in this example to satisfy capacity design principles and yet provide adequate ductility. Then from Table 1, a value of $\eta = 0.20$ is chosen for $\alpha = 0.25$ and $\mu_{max} = 5$, assuming that the period will be close to 1 s.

From the target parameters frame members and BRBs are designed for their required base shear capacities, using steel with a specified yield stress of 345 MPa (50 ksi) for the frame and 248 MPa (36 ksi) for the BRBs. Frame and BRBs 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).

Table 2. Frame and BRB Properties (Design Example)

Story (1)	Beams (Ext. and Int.) (2)	Ext. Cols. (3)	Int. Cols. (4)	BRB (mm) (5)
4	W12 x 26	W14 x 38	W14 x 74	PL 67 x 13
3	W16 x 50	W14 x 38	W14 x 74	PL 127 x 13
2	W21 x 62	W14 x 68	W14 x 109	PL 171 x 13
1	W24 x 68	W14 x 68	W14 x 109	PL 200 x 13

Actual parameters and elastic period are determined from pushover and eigenvalue analyses, respectively, as $\alpha = 0.27$, $\mu_{max} = 4.58$, $\eta = 0.20$ and $T = 0.97$ s, which are in good agreement with the previously calculated target parameters.

Seismic response of the system is then evaluated by nonlinear time history analysis to verify that the structural fuse concept is fully satisfied. Fig. 3 shows the maximum response in terms of hysteresis loops of beams and BRBs at each story. Note that beams respond elastically, while hysteretic energy is completely dissipated by inelastic behavior of BRBs at every story. A maximum roof displacement of 155 mm was obtained from the analysis, which corresponds to a frame ductility of 0.85 (i.e., $\mu_f < 1.0$). Further information can be found in Vargas and Bruneau (2005).

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. It may be observed that systems having $\mu_{max} \geq 5$ offer a broader choice of acceptable designs over a greater range of η values.

As demonstrated in the example of application, by using the listed procedure, buildings can be systematically designed or retrofitted using metallic fuse elements to protect beams and columns from inelastic deformations. From the obtained results it was found 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 $0.25 \leq \alpha \leq 0.50$ and $\mu_{max} \geq 5$ as target parameters. Depending on the target period and the selected α and μ_{max} values, η may be accordingly selected from Table 1 to satisfy the structural fuse concept.

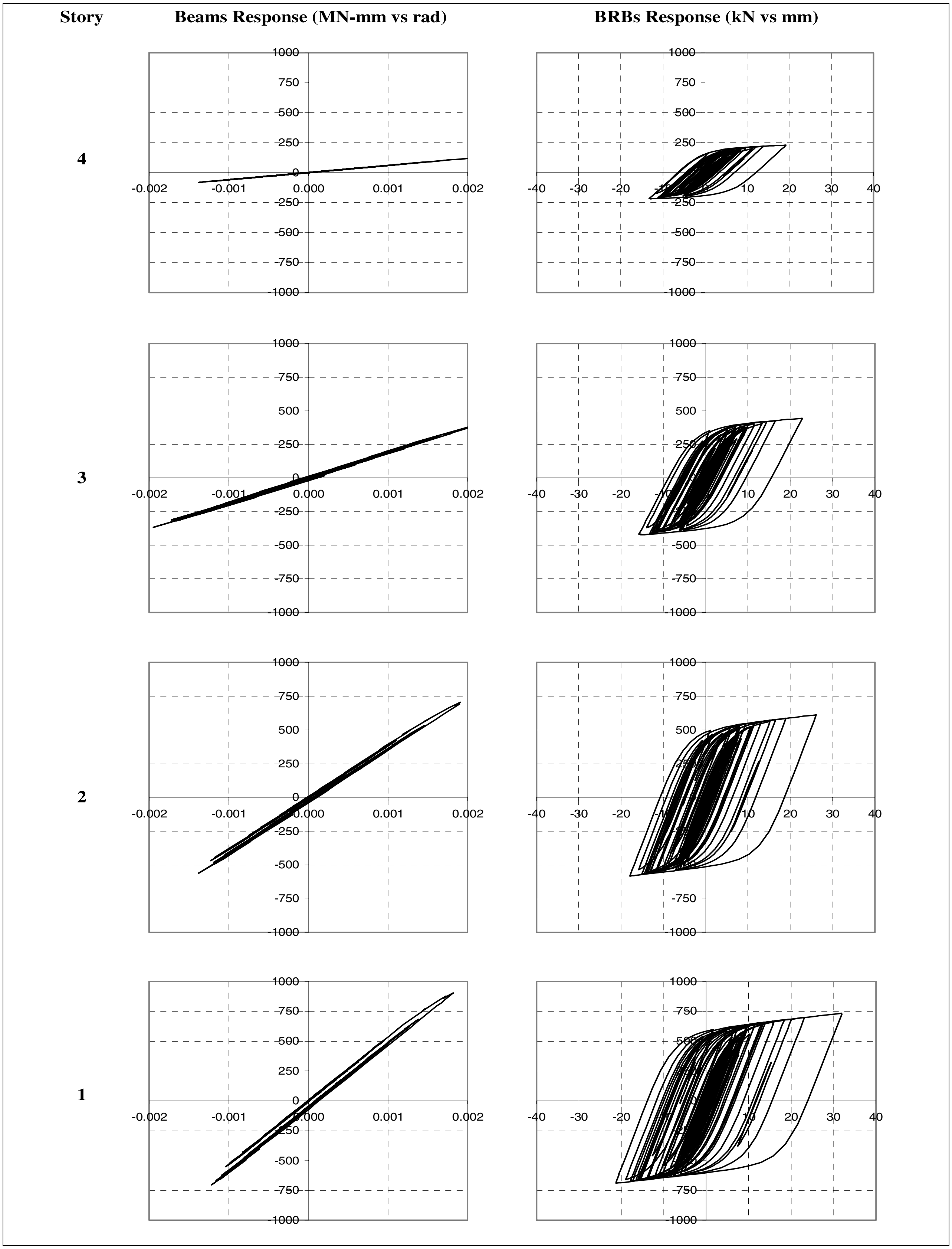

Figure 3. Hysteresis Loops from Design Example

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