Practical Seismic Design Criteria and Life-Cycle Optimization for Structures with Hysteretic Energy-Dissipating Devices

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ABSTRACT

A review is presented of several topics related to seismic design criteria, response and damage estimation methods and life-cycle optimization analysis of structural frames provided with hysteretic energy dissipating devices. A ductility-based design method is proposed, adequate for practical applications. Stiffness-and-strength degrading hysteretic moment-rotation models are used to represent the behavior of critical sections of reinforced concrete beams and columns. These models are used to study the process of damage accumulation, as well as the influence of some seismic-design parameters on the performance and reliability of multistory frames.

INTRODUCTION

One approach of modern earthquake engineering to the control of structural damage produced by earthquakes is the use of energy-dissipating devices (EDD's). These are elements capable of undergoing a large number of high amplitude deformation cycles without experiencing any significant degradation in the strengths and stiffnesses of their constitutive functions (stress-strain or forcedeformation curves). Our discussion will be oriented to those devices that base their energy-dissipation capability on the hysteretic behavior of the materials or elements that constitute them. However, many of the concepts and criteria presented are directly applicable to systems provided with other types or devices, or may be easily adapted to their specific conditions.

Up to now, a large portion of the applications of EDD's in earthquake engineering problems have been addressed to the strengthening or retrofitting of existing structures, including both cases, when the system considered is in an undamaged state and when it shows some initial damage. The latter may arise as a result of previous earthquakes or of other perturbations, such as differential settlements or excessive stresses produced by gravitational loads.

Here we devote attention to this type of applications, as well as to those when the use of EDD's is considered at the design stage of a new construction as a means for attaining an efficient structural system from a life-cycle perspective. For this purpose, the mentioned devices must be considered as ordinary structural elements, described in terms of their cyclic constitutive functions. In addition, the decisions regarding design values and repair and maintenance policies must be made in terms of a costbenefit analysis that accounts for the manners in which the initial costs and those expected to be generated in the future may be affected by those decisions. In order to permit the comparison of initial costs with those that may be generated at unknown times in the future, the latter must be transformed into their equivalent values at the moment for which initial costs are estimated.

The main benefits of the use of EDD's in structures exposed to severe earthquakes arise from (a) their capability to control damage on the structural and non-structural elements of the main system, and (b) the possibility of being easily replaced when damage accumulated on them reaches unacceptable levels. However, these objectives are not explicitly considered in quantitative terms in the conventional practice of earthquake resistant design, which emphasizes the additional damping supplied by EDD's and the local ductility demands experienced by them, as well as by the members of the main structural system at their critical sections. But talking about additional equivalent damping ratio when dealing with hysteretic dampers may be inadequate, or at least ambiguous, since that value is a function of the response amplitudes and frequencies, which are in turn correlated.

The need for a simple and realistic, performance-based, seismic design criterion is obvious. The approach proposed here is an attempt to contribute to satisfy this need. It is based on representing the EDD's as conventional structural members, characterized by their constitutive functions for alternating load cycles. The main frame system is described also in terms of its constitutive functions, expressed in terms of either the base shear vs. top displacement curves or of the shear vs. drift at each story. In both cases, these curves correspond to monotonic load application, and are modified by simplified rules that account for stiffness and strength degradation resulting from damage accumulation.

In order to establish optimum design criteria for systems of the type described above, it is necessary to formulate the problem within a life-cycle optimization framework that permits the comparison of alternatives regarding design requirements and maintenance policies. This creates the need for damage accumulation models and indices for a given system, as well as of simple relations that can be readily used to transform those indices into the corresponding variations in the mechanical properties and reliability functions of the system. These problems are the subject of the last sections of this paper.

DUCTILITY-BASED DESIGN

The design criterion that follows is applicable to cases where the energydissipation system (EDS) is assumed to act in parallel with the conventional frame (CF) at each story. Thus, the design parameters are the contributions of both systems to the story strengths and stiffnesses (R_d , R_c , K_d and K_c , res-

pectively). These are conveniently expressed in terms of the resulting story strengths and stiffnesses (R and K), and of the ratios $\alpha = K_d / K_c$ and $\beta = R_d / R_c$. The design conditions are expressed in terms of μ_d^* , μ_c^* and δ^* , the allowable values of the story ductility demands μ_d and μ_c , for the energy-dissipating and conventional systems, and of the story drift δ , respectively. As a first approximation for preliminary design, a set of values K, R, α and β are chosen so that the combined system (CS) satisfies the conditions mentioned above for the set of nonlinear response spectra of the design earthquake for different ductility values. For this purpose, the global properties of the CS (K and R at each story, as well as the allowable ductility value, μ^*) are expressed in terms of the parameters defined above. For an elasto-plastic system, the following relations are obtained from Fig. 1:

$$\mu_{c} = \frac{\delta}{\delta_{yc}}, \quad \mu_{d} = \frac{\delta}{\delta_{yd}}, \quad \mu = \frac{\delta}{\delta_{y}}$$
(1a, 1b, 1c)

Here, $\delta_{yc} = R_c / K_c$, $\delta_{yd} = R_d / K_d$ and $\delta y = R / K$ are the yield deformations of the CF, the EDS and the CS, respectively. It is convenient to define a new parameter $\varepsilon = \alpha / \beta$, which is equal to $\mu_d^* / \mu_c^* = \delta_{yc} / \delta_{yd}$.

From the foregoing concepts it is easy to show the following relation between the ductility ratios developed by the CS and the CF:

$$\mu = \mu_c \frac{1+\alpha}{1+\beta} \tag{2}$$

The design earthquake is supposed to be defined by a set of elasto-plastic response spectra for different ductility ratios. For the linear-behavior natural period of the system to be designed,



Fig. 1 Idealized shear *vs.* drift function for a story of a building frame

the ordinates of the acceleration response spectra are assumed to be $S(T, \mu)$. The ratio $S(T, 1) / S(T, \mu)$ will be denoted by Q, and the ratio Q / μ will be denoted by γ . Design proceeds in accordance with the following steps:

- 1. Target values of μ_c^* and μ_d^* are established. In some cases, these values are established independently of each other; that is, there are no restrictions to the possible values of ε . In many cases, either K_c / R_c or K_d / R_d , or both, can only adopt values contained within narrow intervals, which imposes restrictions to the possible values of ε . Under these conditions, a new set of allowable ductility values $\mu_c' < \mu_c^*$, and $\mu_d' < \mu_d^*$ is adopted, such the ε lies within the range of its acceptable values.
- 2. In any of the previous cases, the design is started, for instance, by assuming reasonable values for K_c and K_d . This determines α and T.
- 3. Once a practically feasible value of ϵ has been adopted, β is taken equal to α / ϵ .
- 4. Taking into account Eq. (2), the target ductility value for the CS is equal to μ' = μ_c' (1 + α) / (1 + β). The required base-shear ratio is determined from

the ordinate of the design response spectrum corresponding to this ductility value. This determines the required lateral strength R at each story.

5. The peak story displacement δ is obtained as $\mu' R / K$, and compared with δ^* . If $\delta < \delta^*$, the design has been completed at the level of global system parameters; otherwise, a new set of tentative parameters must be investigated.

In ordinary multi-story frames, EDD's are connected to the beams or to the beam-column joints by means of diagonal members. The axial forces acting on the columns will be the result of the story shear forces taken by both, the CF and the EDS.

CALIBRATION OF DESIGN CRITERION

The design criterion sketched above is consistent with modern ideas about performance-based design of conventional systems. Because the EDD's are explicitly dealt with as structural elements with known force-deflection functions, peak story drifts and ductility demands on those elements and on the CF are estimated with the same tools applied in the design of conventional systems. However, before adopting this criterion for routine practical design, it seems convenient to calibrate it. comparing the safety levels attained through its use with those implicitly accepted for conventional systems according to current design procedures. The need to do this arises from two main considerations. First, an approximation is implied in the replacement of the CS, made of elements capable of developing different ductility levels, by a system with an equivalent ductile capacity; second, the strength and stiffness degradation properties of members of the CF differ from those typical of the EDD's.

The calibration proposed is underway. It has consisted in comparing the ratios of actual ductility demands (computed by means of step-by-step integration) to their target values, for both, conventional frames and systems provided with EDD's, assuming they are all designed on the basis of allowable values of story ductility demands.

Figures 2 and 3 show ductility spectra for systems designed in accordance with the algorithm described above for prescribed values of the allowable ductilities, $\mu_{c'}$ and $\mu_{d'}$, for different values of α and ε . The case $\alpha = 0$ corresponds to the ordinary frame without EDD's. These curves are representative of a number of curves that were obtained for systems with different values of those parameters. Each curve corresponds to a



Fig. 2 Ductility demand for 1dof conventional system



Fig. 3 Ductility demand for 1dof system with EDD; $\alpha = 0.5$, $\varepsilon = 2.0$

value of μ' , the target ductility demand for the CS. The ordinates show the expected values of the ductility demands on the CF for systems subjected to a sample of simulated acceleration time histories with statistical properties equal to those of the EW component of the SCT (Mexico City) record of September 19, 1985 The behavior of the (SCT850919EW). EDD's was assumed to be elasto-plastic, while the CF was assumed to behave in accordance with a Takeda model, with representative parameters of those obtained in laboratory tests of ductile moment-resisting reinforced concrete frame components [1].

A comparison of Figs. 2 and 3 shows that, for the two values of the target

ductility μ' that were studied, the ductility demands on the CF are largest for $\alpha_T = 0.4$ and $\beta_T = 0.33$. Here, α_T and β_T are the parameters of Takeda's model, such that $\alpha_T = 0$ means that the stiffness of the unloading branch is not reduced, while β = 1.0 corresponds to the case where the re-loading branch reaches the original force-deflection curve at the yield-point for the first load cycle. For $\mu' = 2$, the use of EDD's reduces in 44 percent the maximum ductility demand, which occurs for a natural period equal to 2.5s. Also, for both values of μ' the use of EDD's causes an increase of the response for systems with $\alpha_T = 0$, regardless of the value of β_T , for natural periods shorter than 1.5s. Thus, the change in dynamic properties resulting from the use of EDD's is not always beneficial to the expected behavior of a structure. More systematic studies are necessary to clarify this point.

The results discussed in the foregoing paragraphs correspond to deterministic systems subjected to random earthquake ground motion of a given intensity. In the design conditions reality, are established in terms of nominal values of the design variables (loads, strengths, stiffnesses, safety factors, etc.). The expected or the most likely value of the base-shear lateral capacity of a given structure are usually significantly greater than the value implied by the nominal base-shear coefficient used in design. Therefore, before using graphs similar to those included in Figs. 2 and 3 for the purpose of estimating expected ductility demands of actual systems, adequate criteria must be developed to transform the nominal values of lateral strengths into their expected or most likely values. Table 1 gives an idea of the order of magnitude of the ratios between expected and nominal lateral strength coefficients for some typical multistory frames

designed in accordance with the Mexico City seismic regulations of 1993. The variables considered are *N*, the number of stories, and the nominal value of the base-shear coefficient used in design. The results show that the ratios between expected and nominal values of lateral capacities, measured by base-shear coefficients, are ordinarily greater than 2, and can be very high for low values of the nominal design coefficient.

Table 1 Comparison of nominal and expected values of base shear ratios (\bar{c}, c)

N	С	<i>ī</i> / c	
	0.05	3.88	
5	0.10	2.42	
	0.15	2.07	
10	0.05	3.10	
	0.10	2.37	
	0.15	2.31	
15	0.05	2.52	
	0.10	2.32	
20	0.05	2.46	
	0.10	2.38	

DAMAGE ACCUMULATION MODELS

The behavior models described in the following paragraphs were used in the study on ductility and damage distribution in multistory frames presented below. They are representative of cases frequently found in engineering practice.

Behavior Models for Members of Conventional Frames

The nonlinear behavior of the conventional frames is assumed to be

concentrated at the critical sections located at their member ends. For each of these sections, the plastic component of the curvature varies with the acting internal moment as shown in Fig. 4. The constitutive law shown [2] was derived starting from one previously proposed by Wang and Shah [3], which appeared to predict excessive stiffness-and-strength degradation. In its present version, moments vary within two enveloping straight lines that intersect the vertical axis at the values of the corresponding yield moments. A damage index D is defined, which is associated with the residual stiffness and capacity of the critical section each in loading direction. This index is obtained as a conventional fatigue index: $D = \Sigma (\theta / \theta_F)$, where θ and θ_F are respectively the plastic-hinge rotation and the rotation at failure (the point where the lateral load reaches its maximum) under monotonic loading conditions. The reloading branch in a given direction is a straight line that intersects the vertical corresponding to the maximum rotation amplitude previously reached in that direction at an ordinate M' smaller than



Fig. 4 Stiffness-degrading functions for plastic-hinge rotation

that associated with the virgin momentrotation curve for monotonic load application (M). The original and the reduced values are related by the equation $M' = M \cdot (1 - \lambda)$, where $\lambda = 1 - \lambda$ $\exp(-\kappa D)$. Moment-rotation curves of this form are used in this paper. A value of κ equal to 0.0671 was adopted, after fitting the mathematical models to some experimental results reported by Ma, et al., [4], Wang and Shah [3], Townsend and Hanson [5], Scribner and Wight [6] and Uzumeri [7].

Behavior Models for EDD's

The constitutive functions proposed for EDD's are based on some derived from the results of laboratory tests carried out by Aguirre and Sánchez [8] in U-shaped hysteretic EDD's. A non-degrading bilinear moment-rotation curve is assumed. Failure of one of these elements is assumed to occur when the fatigue index D_F exceeds unity, where $D_F = \sum N_i^{-1}$, and N_i is the expected number of constantamplitude deformation cycles that lead to fatigue failure. This number is given by the equation $N_i = \exp(121 (\xi^{-0.02} - 1))$, where ξ_i is the ratio of the amplitude of the deformation cycle to the deformation at failure under monotonic load.

Story Damage Index

At any individual story, the damage index D_k in a given load direction is defined as the ratio $(K_0 - K_s) / K_0$, where K_s is the secant stiffness associated with the maximum story drift in that direction, and K_0 is the corresponding initial tangent stiffness for small deformations. The life-cycle optimization formulation presented at the end of this paper makes use of conditional probability distributions of the damage on both, the CF and the EDD's, at the end of an earthquake, in terms of the intensity and

the initial damage values on both groups Thus, it was deemed of elements. convenient to obtain curves displaying the expected final value of the damage index D_k as a function of the initial value, for different intensities [9]. Figures 5 and 6 show some of those curves for systems with natural periods equal respectively to 1.0 and 1.5s, built on the soft soil area in Mexico City, and designed according to the current seismic code for a nominal base shear ratio of 0.10. The stiffness and strength ratios, α and β , are both equal to 1.0. The intensity of the excitation is represented by its normalized



Fig. 5 Cumulative damage functions for frame with EDD; T = 1.0s, $r_k = 0.5$



Fig. 6 Cumulative damage functions for frame with EDD; T = 1.5s, $r_k = 0.5$

value, y / y_m , with respect to that of SCT850919EW. Use was made of the modified Wang and Shah model, with strength-degradation parameters obtained from the results published by them. Because these parameters seem to predict excessively high degradation values, the damage indexes shown in Figs. 5 and 6 may be too high.

DUCTILITY AND DAMAGE DISTRIBUTION IN MULTISTORY FRAMES

A study on the spatial distributions of ductility demands and damage indexes was carried out in a group of several building frames, including one that did not have EDD's (see Table 2, taken from Campos [10]). As in the sdof systems studied in the previous section, the responses computed for the system without EDD's serve as a basis of comparison for the responses of the other cases. All the frames studied in this series were fourteen-story high, with their fundamental periods of vibration ranging between 1.34 and 1.41s. The variables studied were the nominal value μ of the target design ductility for the combined system, the ratio ψ of the yield deflection of the EDD to that of the CF at each story, and the ratio r_k between the stiffnesses of the EDD and the CS at each story. The last three cases in Table 2 correspond to systems where EDD's were placed only in the first four stories, with the objective of concentrating all energy dissipation in those stories. In case g, the members in the stories above the first four were designed for the same load factor used for the lower ones. In cases h and i the load factors used for the design of the upper stories were respectively 1.1 and 1.2 times those applied for the design of the

System	Stories with EDD's	T (s)	r_k	ψ	μ	r
а	None	1.41			4	
b	all	1.33	0.25	0.50	4	
с	all	1.38	0.25	0.50	5	_
d	all	1.37	0.50	0.50	4	
е	all	1.45	0.50	0.50	5	_
f	all	1.34	0.50	1.00	5	
g	1 ~ 4	1.36	0.50	1.00	5	1.0
h	1~4	1.34	0.50	1.00	5	1.1
i	1~4	1.34	0.50	1.00	5	1.2

Table 2 Properties of systems studied

members of the lowest stories (see overstrength factor r, in the last column in Table 2). In all cases, the excitation was represented by a sample of simulated acceleration time histories with statistical properties equal to those of SCT850919EW [11], normalized to the same spectral intensity I:

$$I = \frac{1}{2\pi} \int_{0}^{T_{1}} T \cdot S_{a}(T, \zeta) dT$$
 (3)

Figure 7 shows the average ordinates of the elasto-plastic response spectra for different values of the ductility value. The cyclic behavior of the EDD's was assumed to be bilinear, with a postyielding stiffness equal to 3.2 percent of that corresponding to the linear range. The nonlinear behavior of the conventional frame was assumed to be concentrated at plastic hinges forming at its member ends, which were represented by stiffness-and-strength-degrading elements following the model proposed by Shah and Wang [3], modified by Esteva and Díaz [2], and summarized above. The results of the analysis are summarized in Figs. 8 and 9, which represent the mean values of the story damage indexes and of the story ductility demands, respectively.



Fig. 7 Mean response spectra for simulated accelerograms

The results show that story ductility demands never exceeded significantly of 3.0, and were usually substantially smaller, in spite of the fact that the target design ductilities for the CS were equal to 4.0 or 5.0. This is the result of two effects that tend to compensate each other. On one hand, the degrading behavior of the CF tends to produce larger ductility demands than expected for the non-degrading bilinear model; on the other, nominal values of design loads and member capacities lie on the conservative side of their most probable values. This leads to larger expected ductility demands than their nominal values assumed in design.

The beneficial influence of the nondegrading behavior of the EDD's, as compared with the degrading behavior of the CF, is clearly shown in Figs. 6 and 7. Thus, a comparison of curves corresponding to systems a, b and d show that mean story damage indexes and ductility demands are significantly reduced when fractions of story stiffnesses equal to 0.25 and 0.50 are provided by EDD's, while



Fig. 8 Story damage indexes in multistory frames



Fig. 9 Story ductility demand in multistory frames

keeping target ductility values for the CS equal to 4.0. By means of curves a, c, e and f a comparison is made of the behavior of conventional frames designed for a target ductility value of 4.0 with that of systems with EDD's that had been designed for a system ductility of 5.0. It

can be appreciated that damage indexes and ductility demands are lower along a large portion of the structure's height for the systems with EDD's than for the conventional frame system.

Frames g, h and i were studied in order to obtain some information about the efficiency of EDD's used only in the lowest stories of a frame. The design values of the lateral shear forces acting on the stories of the CS were derived from the design spectrum corresponding to a nominal target ductility of 5.0. However, an additional load factor greater than unity was applied to the design of the structural members in stories above the fourth one for systems *h* and *i*. Thus, the stories having EDD's were conceived as subsystems providing partial base isolation. The figures show that for the lowest four stories the damage indexes and ductility demands were lower for the systems with EDD's than for the conventional frame designed for a nominal target ductility value of 4.0. At the upper stories, ductility demands in systems g and h were significantly larger than for frame *a*, but remained in general below the maximum value experienced by the conventional frame anywhere along its height.

The response studies of the systems in Table 2 were also used to assess the influence of the EDD's on the structural reliability levels. For this purpose, the reliability for the family of ground motion records used to represent the random excitation for a given intensity was measured by means of index β_c (Cornell's beta). This was defined in terms of the random variable Z, equal to the natural logarithm of the minimum value, along the system's height, of the ratio of the available story ductility to the corresponding ductility demand that results from the dynamic response analysis: $\beta_c = E(Z) / \sigma_Z$, where $E(\cdot)$ and σ stand for expected value and standard deviation, respectively. For frame a, β_c was equal to 3.18. The largest value adopted by any of the systems with EDD's corresponded to systems d, f and i (4.76, 7.88 and 5.81, respectively), while the lowest values corresponded to systems e and h (3.62 and 3.27, respectively).

DAMAGE ACCUMULATION UNDER RANDOM EARTHQUAKE SEQUENCES

For simplicity, the discussion that follows refers to a single-story system with one EDD. Just after the occurrence of the *j*-th earthquake, the damage accumulated on that element is equal to D_{dj} , while that affecting the conventional structural frame is equal to D_{ci} . After the (j + 1)-th event, these values become respectively $D_{d(j+1)} = D_{dj} + \delta_{d(j+1)}$ and $D_{c(j+1)}$ = D_{cj} + $\delta_{c(j+1)}$, where $\delta_{d(j+1)}$ and $\delta_{c(j+1)}$ are the corresponding damage increments. If Dcj exceeds a given threshold, designated here as D_{rc} , the frame is repaired in such a manner as to eliminate the damage accumulated, thus restoring its initial strength and stiffness, R_c and K_c . It is assumed that the damage level on the frame can be assessed from observation of the evidences of physical deterioration, while that on the EDD's is inferred from the estimated value of the low-cyclefatigue index. This information is used to implement the preventing strategy of replacing the EDD after the occurrence of a number of high-intensity earthquakes, on the basis of a threshold value D_{rd} , defined within the framework of a lifecycle optimization approach.

Whether the process of occurrence of earthquake ground motions with different characteristics involves some kind of correlation with previous history or is independent from it, the levels of damage accumulated D_{cj} and D_{dj} , $j = 1, ..., \infty$, at the end of the *j*-th earthquake occur as events of a Markov process. The transition probabilities from (D_{cj}, D_{dj}) to $(D_{c(j+1)}, D_{d(j+1)})$ are obtained from the probability density functions of $\delta_{c(j+1)}$ and $\delta_{d(j+1)}$, which depend on D_{cj} and D_{dj} , as well as on the probability density function of Y_{j+1} , the intensity of the (j + 1)-th event.

In order to determine the conditional probability density functions of $D_{c(j+1)}$ and $D_{d(i+1)}$, given the values corresponding to the end of the j-th earthquake, it is necessary both, to calculate the joint probability density function of the intensity of the (j + 1)-th and the waiting time to its occurrence, and to determine the damage states D_{ci} and D_{di} of the system's components after carrying out the operations of repairing the conventional frame members and/or replacing the EDD's. The conditional probability functions obtained in this manner are integrated recursively in order to obtain the marginal probability distributions of all D_{cj} and D_{dj} . Details are given by Díaz and Esteva [2]. In the life-cycle optimization studies reported in this paper, the direct approach described above was replaced by a Monte Carlo simulation procedure described by Esteva, et al. [9].

LIFE-CYCLE OPTIMIZATION

Let C_i be the initial construction cost of a system of interest, T_i , $i = 1, ..., \infty$ the (random) times of occurrence of earthquakes that may affect it, and L_i , i =1, ..., ∞ the losses associated with those earthquakes; they include damage and failure consequences, as well as repair and maintenance actions. The following objective function must be minimized:

$$U = C + E\left[\sum_{i=1}^{\infty} L_i^{-q T_i}\right]$$
(4)

Here, $E[\cdot]$ stands for expected value, and q is an adequate discount rate.

CASE STUDY

Esteva, et al., [9] applied the optimization analysis described above to three reinforced concrete buildings: five, ten and fifteen-story high, respectively. The following paragraphs make a brief description of the method used and present the main results for the tallest building. It has a square plan area of 346m², and a fundamental vibration period of 1.5s. It is assumed that it will be built at a soft soil site in the Valley of Mexico, where local soil conditions are similar to those of the recording site of the accelerogram SCT850919EW mentioned above. The seismic hazard at the site was expressed by an intensity-recurrence curve previously obtained.

The behavior of the reinforced concrete members was represented by Wang and Shah's model [3]. The modifications proposed by Díaz and Esteva [2] were not included. Therefore, according to the comments presented in the section on damage accumulation models, it is thought that the predicted values of the stiffness and strength degradation of those members are excessively thus large, leading to excessively large responses and damage levels. The case study is presented, however, because of the value of the qualitative conclusions extracted from its results.

On the basis of some approximate estimates of initial construction costs in terms of the seismic design coefficient, c,

it was concluded that the initial construction cost *C* of the frame system varies with that coefficient according to the equation $C = C_0 (1 + 0.14 \cdot Nc^{1.5})^{0.4}$, where *N* is the number of stories and C_0 is the value of *C* for c = 0. The cost of installing an EDD of the type considered, with a lateral yield strength *P*, is equal to $6.17 P^{0.68}$, where *P* is expressed in kilograms and the cost is expressed in US dollars.

The repair-cost of the structural and non-structural elements at a given story was assumed to vary linearly with the damage index D_k at that story.

The variables considered in the study of the building selected were the seismic design coefficient and the pre-established threshold values of the story damage levels whose exceedance would lead to repair and/or replacement actions (D_{rc} and D_{rd} , for elements of the CF and of the EDS, respectively). At each story, the contribution of the EDS, both to the lateral stiffness and to the lateral strength of the CS, is equal to 75 percent. This value is higher than those found in typical practical cases.

The mechanical properties of the structure and the gravitational loads acting on it were taken equal to their expected values. Once these values were derived from the nominal design parameters and the assumed statistical models and safety factors, an equivalent sdof system was assumed to represent the real system for the purpose of performing a life-cycle optimization analysis. The free variables are the seismic design coefficient c and the threshold values for repair and replacement of the CF and the EDS: D_{rc} and D_{rd} . For each combination of these variables an estimate is made of the negative utility U, given by Eq. (4). The estimate is obtained by Monte Carlo simulation.

The estimation process starts by simulating a number of seismic histories, each consisting of the intensities and the times of occurrence of the corresponding events. Damage levels on the CF and the EDS at the end of each event are obtained by Monte Carlo simulation, taking into account the intensity of the ground motion and the damage levels on both elements at the beginning of the Repair or replacements earthquake. actions are taken, if adequate, after comparing the damage levels with the pre-established threshold values. The initial damage conditions for the next are established. earthquake The corresponding costs are then obtained and used to calculate the value of the term inside the parenthesis in Eq. (4). After doing this for a sufficiently large sample of seismic histories, the expected value appearing in that equation can be obtained. The results for the system considered are shown in Table 3. There, C is the initial construction cost for the structure; it is a function of the seismic design coefficient c. C_1 is the initial construction cost for a reference system, which in this case was a conventional frame, without EDD's, designed for c = 0.1. The figures reported in the table are values of U/C_1 . The minimum values of this variable are shown in bold type. It is easy to see that the normalized utility is not very sensitive to the repair and replacement thresholds. It is interesting to see that the optimum D_{rc} is very high, which may be a consequence of the fact that the contribution of the CF to the lateral stiffness and strength of the system is relatively low (0.25).

This example is only intended to show an approach to the life-cycle optimization of a system with EDD's on the basis of expected utilities. Another approach deserving attention is that of decision making on the basis of tolerable risk criteria.

с	D_{rd}	Drc			
C/C_1		0.2	0.4	0.6	0.8
0.05	0.2	1.470	1.493	1.508	1.507
1.168	0.4	1.472	1.493	1.510	1.501
	0.6	1.470	1.489	1.500	1.502
0.10	0.2	1.280	1.282	1.287	1.285
1.194	0.4	1.281	1.288	1.286	1.290
	0.6	1.281	1.283	1.285	1.289
0.15	0.2	1.257	1.252	1.250	1.252
1.2259	0.4	1.255	1.252	1.251	1.250
	0.6	1.255	1.252	1.251	1.250

Table 3 Influence of *c*, D_{rd} on C/C_1 and U/C_1

CONCLUDING REMARKS

Significant reductions in the expected seismic response of conventional structural frames may be obtained by replacing a portion of their lateral strength and stiffness with that provided by hysteretic energy-dissipating devices. This is a consequence of the capacity of the latter elements to sustain large numbers of large-amplitude deformation cycles without suffering significant degradation of their mechanical properties. The best way to estimate their possible influence on the response of specific nonlinear degrading systems is by dealing with both, EDD's and conventional frame members, as with nonlinear elements, each characterized by a constitutive function that accounts for the corresponding stiffnessand-strength degrading properties.

For systems where the EDD's act in parallel with the conventional frame members, an approach based on global ductilities can be used in practical design. Simplified approaches applicable to other configurations need to be developed. Because the reductions in response and damage achieved by the use of EDD's have a cost, it may happen that a stronger conventional frame, designed for the same tolerable lateral drifts as one with EDD's, has a cost lower than that of the latter. Optimum decisions in these cases have to be made under a life-cycle framework that accounts for the process of damage accumulation.

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