RISK-BASED MULTI-HAZARD OPTIMIZATION OF PASSIVELY DAMPED STRUCTURES USING EVOLUTIONARY ALGORITHMS

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ABSTRACT:
A new computational methodology is developed for conducting Risk-based Optimal Multi-hazard Design (ROMD) of seismic- and wind-excited structures retrofitted with passive energy dissipation (PED) devices. In addition to designing in compliance with the relevant codes of practice, it is important to consider that the performance of PED devices for reducing the structural responses depends on the type, size, and distribution of dampers. The proposed framework provides a genetic algorithm (GA) based methodology to address these optimization issues of multi-hazard design within the context of nonlinear steel frame structures. Steel buckling restrained braces, viscous fluid dampers and solid viscoelastic dampers are all considered as possible design alternatives within this framework. In the proposed algorithm, passively damped structural designs evolve toward configurations that limit damage associated with inter-story drift and absolute floor acceleration, while considering essential conflicts in dynamic response demands of the structures under multi-hazard environments, involving earthquakes and strong windstorms. Unlike previous work in PED optimization, the ROMD approach compares the life cycle costs and benefits of the alternative design or retrofit strategies using an optimization criterion that reflects physical and economical uncertainties, as well as the attitudes of decision makers through the introduction of a risk measure. This measure characterizes the risk tolerance of the decision makers and allows the evolution of rational design or retrofit strategies that depend upon the level of risk aversion to low frequency, high consequence events. Besides providing an outline of the evolutionary risk-based algorithm, this paper includes an example to emphasize the potential benefits of the proposed computational multi-hazard design approach.

KEYWORDS:
Optimal structural design, multi-hazard mitigation, life-cycle cost, risk aversion, genetic algorithms

1. INTRODUCTION

The extensive damage sustained in recent natural hazards has highlighted the need to extend structural design and retrofit procedures to address performance under several hazard scenarios. Passive energy dissipation (PED) systems have been proposed as an effective solution to protect structures against hazards (*e.g.*, Soong and Dargush, 1997). However, performance depends on the distribution, type, size, and number of dampers in the structure. For the corresponding optimization problem, many researchers have suggested different algorithms, such as sequential search schemes, control theory method, single point substitution method, steepest descent search algorithms and genetic algorithms (GA). For example, Furuya *et al*. (1998) examined the application of GA for damper distribution under wind-induced vibrations, while Moreschi and Singh (2003) and Dargush and Sant (2005) have considered nonlinear behavior for aseismic design using genetic algorithms. On the other hand, Agrawal and Yang (1999) applied combinatorial algorithms to determine the optimal placement of PED devices in seismic/wind-excited linear buildings. However, few attempts have been made to solve the optimum sizing and placement of PED devices with nonlinear dynamic analysis for both wind and earthquake hazards.

During the same period, within the context of Performance-Based Design (PBD) (Bertero, 1997; Priestley, 2000), a relatively large number of research studies have been directed towards design of buildings considering damage cost estimation methodologies. The idea is to establish an overall framework (*e.g.*, Cornell and Krawinkler, 2000; Krawinkler, 2002) and well-defined quantitative measures.

In the present paper, we develop a computational approach for Risk-based Optimal Multi-hazard Design (ROMD) of structures with PED dampers within the context of PBD. Here the structure may contain a number of PED devices including metallic yielding dampers, viscous fluid dampers and/or viscoelastic solid dampers over a range of sizes. The proposed genetic algorithm-based methodology allows decision makers to achieve a specific performance objective, such as minimum life-cycle cost of a building. The introduction of a weighted life-cycle cost objective function permits consideration of risk aversion. The resulting methodology can provide insight into multi-hazard performance of passively controlled structures, an estimate of the costs and benefits of PED retrofit strategies and information on the sensitivity of these designs to the risk tolerance of the decision makers. An example is provided for the simplified case of a risk neutral decision maker by minimizing expected life-cycle cost of a sixteen story steel structure under both seismic and wind hazards.

2. PROBABILISTIC HAZARD LOSS ESTIMATION

2.1. Performance based design framework

Cornell and Krawinkler (2000) proposed a general probabilistic framework for PBD. Within that framework, Krawinkler (2002) then decomposed the problem of seismic assessment into four separate tasks, which can be identified with hazard assessment, structural response analysis, damage evaluation, and loss estimation. Applying this methodology for the multi-hazard scenarios, the comprehensive probabilistic equation can be written:

$$G_C(c) = \int_c^{\infty} \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} f_{C|S}(\xi,s)\, f_{S|X}(s,x)\, f_{X|L}(x,l)\, f_L(l)\, dl\, dx\, ds\, d\xi \tag{2.1}$$

Here the symbol $f_Y(y)$ represents the probability density function (pdf) for a random variable Y evaluated at y, while $f_{Y|Z}(y,z)$ denotes the conditional pdf for Y evaluated at y given $Z = z$. Furthermore, $G_Y(y)$ is the complementary cumulative distribution function (CCDF) of Y evaluated at y. Thus, $G_Y(y)$ is the probability that $Y > y$, *i.e.*, $G_Y(y) = \mathrm{P}(\mathrm{Y} > \mathrm{y})$. Meanwhile, the specific random variables appearing in Eqn. 2.1 characterize the environmental loads L, the structural response X, the damage S and the costs C over the life or remaining life of the structure, where in general each of these random variables could actually be vector functions.

Notice that Eqn. 2.1 is a statement of the total probability theorem for a hierarchical sequence of random variables. Thus, within this hierarchy,

$$f_X(x) = \int_{-\infty}^{\infty} f_{X|L}(x,l)\, f_L(l)\, dl; \quad f_S(s) = \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} f_{S|X}(s,x)\, f_{X|L}(x,l)\, f_L(l)\, dl\, dx \tag{2.2a,b}$$

$$f_C(c) = \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} f_{C|S}(c,s)\, f_{S|X}(s,x)\, f_{X|L}(x,l)\, f_L(l)\, dl\, dx\, ds \tag{2.2c}$$

Various functions of the random variables can also be introduced in these relations. For example, following Cornell and Krawinkler (2000), the function $\lambda_L(l)$ can be incorporated in the formulation to find the mean annual rate of exceeding the hazard level l, although some caution is needed when dealing with non-ergodic uncertainties, as discussed in Der Kiureghian (2005). In any case, one can extract expected life-cycle cost and the corresponding standard deviation:

$$E[C] = m_C = \int_{-\infty}^{\infty} c\, f_C(c)\, dc; \quad \sigma[C] = \sigma_C = \left(\int_{-\infty}^{\infty} (c - m_C)^2\, f_C(c)\, dc \right)^{1/2} \tag{2.3a,b}$$

Then, the objective of performance-based design could focus on minimizing a risk-based life-cycle cost measure $R[C]$. One such measure could be formed from the linear combination of Eqn. 2.3a and 2.3b, as in

$$R_\alpha[C] = m_C + \alpha\, \sigma_C \tag{2.4}$$

where α represents a risk aversion parameter. Positive α corresponds to a risk averse decision maker, while $\alpha = 0$ implies a risk neutral decision.

The formulation has been written for continuous random variables. However, similar relations exist for discrete random variables. Next, each individual tasks involved in the formulation is described in separate subsections.

2.2. Hazard assessment

For seismic hazard analysis, representative ground motion data for the structure location are used. These motions include suites of sufficient number of ground motion records representing the hazard intensity of the site at different hazard levels (*e.g.*, particular probabilities of exceedance, such as *2%*, *10%*, and *20%* in 50 years). Note that, engineering judgment and the site hazard curve should be used to select the discrete hazard levels for which the structure will be evaluated. For wind hazard analysis, a spectral approach using the inverse fast Fourier transform is used to generate sets of time histories of the wind velocity component along the approaching flow direction. Mean wind speed (3-sec. gust wind speed) at a given location for different wind hazard levels with probabilities of exceedance of *2%*, *10%*, and *20%* in 50 years, is estimated from the wind dataset by the Gumbel method (Holmes, 2001). Then these mean wind speed values are added to wind fluctuation data that is generated from the well known Davenport spectrum (Davenport, 1961), which has energy contents varying with altitude. The wind load is then calculated from the generated time history of the wind velocity data, which is translated to each floor level of the building with a power law model.

2.3. Structural fragility

Fragility curves are used to investigate the probability of exceeding a specific damage state for a given hazard level to represent the relationship between the probability of structural damage and intensity of each hazard. For this study, different performance groups are assigned for the building components according to their performance similarity under hazard loading as: (1) global structural system (2) drift-sensitive nonstructural components, (3) acceleration-sensitive nonstructural components, (4) building contents. Each of these performance groups are assumed to have a specific single fragility curve. For the global structural system, non-linear transient dynamic analysis is carried out under a number of hazardous loading ensembles that represent the region of interest. For example, the structural damage, in terms of inter-story drift and absolute floor acceleration, is investigated through Monte Carlo simulation of different ground motions in the horizontal direction. The probability of exceeding each damage state at each hazard level is found by fitting a lognormal distribution from which the probability of exceeding each damage state is calculated and a lognormal cumulative distribution is fitted to the available data (Singhal and Kiremidjian, 1996).

Nonstructural components (NSCs), either sensitive to interstory drift or peak floor acceleration, and building contents are assumed to have a single fragility curve representing each performance group. These performance group specific fragility curves are adapted with interpolation for each defined damage state from those in the HAZUS-MH technical manual (NIBS, 2003). Note that, the fragility curves are conditional, in the sense that the failure sequences, initiating from non-structural component failure and developing into partial structural system damage and/or complete collapse of the building, are not considered.

Fragilities of each performance group consist of a sufficient number of discrete damage states that are introduced to completely describe the range of damage to the global structure, NSCs and contents. In order to estimate economical losses, the expected damage cost associated with each of these damage states are defined as a percentage of the cost of the total structure. These values, along with drift and acceleration limits, are assigned according to engineering judgment and rational reasoning; however available data should be used when determining the damage states and costs, as they vary depending on the type, age, and condition of the building.

2.4. Loss estimation

With the chosen hazard type and hazard level, engineering response parameters (*i.e.*, interstory drift ratio and absolute floor acceleration) are computed by nonlinear transient dynamic analysis to evaluate the structural loss associated with the direct damage to the structure. Since nonlinear transient dynamic analysis is computationally expensive, Monte-Carlo simulation is employed to generate several realizations of structural response and damage. This technique can sample correlated response variables from a few dynamic analyses and, therefore, is an efficient procedure to estimate the entire distribution of computed response variables in order to estimate structural, nonstructural and content damage and associated damage costs (Yang, 2006).

The present study also includes the loss associated with business interruption, which occurs when building damage disrupts commercial activity. For the estimation of business interruption, two parameters play a crucial role: (1) the level of income generated by the enterprise, (2) the loss of function time for the facility. In this study, the business interruption model for commercial buildings is adapted from the model utilized in the HAZUS-MH technical manual. Additionally another specific business interruption model is proposed for wind hazard. In general, earthquake-induced motions, even when they are more severe than those induced by wind, cause a totally different human response: First, because wind storms occur much more frequently than earthquakes, and second, because the duration of motion caused by a wind storm is generally longer. Thus, wind-induced motions can usually cause occupant discomfort in low intensities and can endanger structural safety in high intensities, whereas earthquake-induced motions endanger human safety rather than human discomfort. So, it is assumed that people are not able to work during wind storms due to motion sickness symptoms and creaking noise, if the peak floor acceleration of the building exceeds a value in the range between *0.4%g* to *1.2%g*, according to values suggested by Simiu and Scanlan (1996). A triangular probability density function with minimum and maximum values of human discomfort thresholds are assigned to represent the business interruption loss probability function of a structure under wind hazard. It should be noted that in this model no consideration is given to the frequency dependence of human perception to acceleration.

2.5. Risk-based life-cycle cost formulation

Regarding cost efficient retrofit strategies; this study aims at robust design of passively damped structural systems, while minimizing a risk-based life-cycle cost of the structure. The life-cycle cost of a structure can be defined as economic losses (*i.e.*, monetary-equivalent losses) due to single or multiple-hazards that are expected to occur during the lifetime of a new structure or the remaining life of a retrofitted structure. Here, life-cycle cost is assumed to be the sum of the retrofit cost, C_R and five damage cost components: (1) global structure damage cost C_S corresponding to the damage defined by interstory drift; (2) drift-sensitive nonstructural component damage cost C_{NSD} corresponding to the damage measured by interstory drift; (3) acceleration-sensitive nonstructural component damage cost C_{NSA} corresponding to the damage defined by peak floor acceleration; (4) building contents damage cost C_C corresponding to the damage measured by peak floor acceleration; and (5) cost associated with the business interruption C_{BI} dependent on the global structure damage level, which is defined by interstory drift. The loss of human life, business inventory losses, rental income losses, and relocation expenses, which in general are also associated with the life-cycle cost of a structure, are not accounted for in the present study. Thus, the overall life-cycle cost random variable C can be defined as follows:

$$C = C_R + C_S + C_{NSD} + C_{NSA} + C_C + C_{BI} \tag{2.5}$$

Included in this life-cycle cost is a discount rate per year over the lifetime or remaining lifetime of the building. Although the discount rate is prone to variation, it is usually chosen in the range of *3%* to *6%* according to many studies.

3. STRUCTURAL MODELING AND OPTIMIZATION

The framework uses nonlinear transient dynamic analysis with an explicit state-space approach to determine the response of the structure over a range of multiple-hazard load intensities during its lifetime. A uniaxial version of a two-surface cyclic plasticity model is implemented for the primary structure, which is simplified by lumped parameter models. The primary structure may contain a number of metallic yielding dampers, viscoelastic solid dampers and/or viscous fluid dampers. For metallic yielding dampers, the same two-surface cyclic plasticity model has been applied. A coupled thermo-viscoelastic Maxwell model and strictly linear Newtonian model are used for viscoelastic solid and viscous fluid dampers, respectively.

This study develops an automated performance based design algorithm that can identify an optimized design or retrofit of structures under seismic or wind excitations, using genetic algorithms. Genetic algorithms, which were originally developed by Holland (1975), are based on the evolutionary theory of natural selection and genetics of all species, commonly referred to as "survival of the fittest." The GA starts with a set of initial structural designs that are generated from a random combination of different damper sizes and/or types. Within each generation, each structure is subjected to a specified number of seismic and/or wind load conditions and evaluated to determine the level of damage. This damage is then used to estimate the risk-based life-cycle cost for this design from Eqn. 2.4. Finally, the GA fitness ϕ for each design is evaluated as follows:

$$\phi = \frac{R_\alpha^{\text{bare}}[C] - R_\alpha^{\text{ped}}[C]}{R_\alpha^{\text{bare}}[C]} \tag{3.1}$$

with $R_\alpha^{\text{bare}}[C]$ and $R_\alpha^{\text{ped}}[C]$ representing the weighted life cycle cost for the bare and passively damped structure, respectively. The fitness values, along with genetic operators modeling selection, crossover, and mutation processes, provide the basis for defining the next generation of structures.

4. EXAMPLE

A 16-story, 58.4*m* high, uniform steel frame building, situated on firm ground and open terrain in Los Angeles, CA with a lifetime of 50 years, is evaluated to illustrate the application of the proposed optimization framework. The structure is classified as a professional business service building (COM4 in HAZUS-MH) with a total floor area of 6020m^2. The total cost of the baseline building without regard to its location, is estimated to be approximately \$740 per square meter. The baseline lumped-mass steel frame model is assumed to be initially undamped with a story weight of *2500kN* and lateral story stiffness of *100kN/mm*, resulting in the first two periods as *3.3sec* and *1.1sec*, respectively. The yield forces on the inner loading surface and outer loading surface needed for the previously mentioned two-surface cyclic plasticity structural model for each story of the building are set to *1100kN* and *4300kN*, respectively. It is assumed that 12% of the total cost of the structure is associated with structural components, 35% of the cost is associated with drift-sensitive NSCs, another 35% of the cost is associated with acceleration-sensitive NSCs, and the remaining 18% of the cost is associated with the building contents. Dampers are available in four different discrete sizes and their costs are assumed to vary from $3\$/m^2$ to $20\$/m^2$ depending on size. For business interruption cost estimations, the income from the building is assumed to be $9.9\$/m^2/day$ (HAZUS-MH). The maximum interstory drift ratio and peak floor acceleration are chosen as the engineering response parameters and $\alpha = 0$ is used to represent a risk-neutral decision-maker.

The seismic environment is described by three different hazard level ground motion records from the MCEER Northridge Earthquake ensemble (Filiatrault and Wanitkorkul, 2005) with probabilities of exceedance of 2%, 10%, and 20% in 50 years, respectively. Each seismic hazard level includes three horizontal ground motion records. For the wind environment, extreme wind speed data from 2002 to 2006, a total of five years of yearly maximum wind speeds collected by the National Climatic Data Center from the National Weather Service (NWS) station at the Los Angeles Downtown UCS Campus is used to estimate extreme wind speeds for Los Angeles. Accordingly, mean gust speeds are predicted by the Gumbel method as *35m/s*, *32 m/s*, and *28m/s* for three different wind hazard levels with probabilities of exceedance of 2%, 10%, and 20% in *50* years, respectively. Then, the inverse fast Fourier transform is used to generate three sets of time histories of the along-wind velocity components for each wind hazard level. Although a windstorm duration is assumed to be 2 hours, a duration of 45 seconds has been used to reduce the computational time, and such duration should be sufficient to establish the stationary response properties. Parameters for damage state definitions and associated damage costs for interstory drift and acceleration thresholds used here are shown in Tables 4.1 and 4.2, respectively (FEMA 273; FEMA 227; Lagaros *et al.*, 2006). It should be noted that no collapse cases are included for this study. Fragility curve parameters for NSCs and contents for the seven damage states of Tables 4.1 and 4.2 are developed for interpolated lognormal distribution values (*i.e.*, median and standard deviation) obtained from four damage states of HAZUS-MH. Monte-Carlo simulation is applied to the initial database of the structural response obtained from nonlinear transient dynamic analysis to generate 400 realizations of the seismic and/or wind loading response of the retrofitted structure. The damage cost information is analyzed to determine the expected life-cycle cost of each retrofit alternative with 5% discount rate. Because of the important differences between dynamic properties of seismic and wind excitations (*e.g.*, durations, predominant frequencies), different values of the limit states are proposed for damage under seismic and wind excitations. For seismic hazard design of the building, the interstory drift is limited to 2% of the height of the story of the buildings, whereas the absolute floor acceleration is only limited to 0.8 g. On the other hand, under severe wind environments, the main purpose of installing PED devices is to reduce the discomfort of the occupants and damage to sensitive equipment and nonstructural components in buildings. Thus, for the performance of the building deformation envelope to be adequate for serviceability requirements, the interstory drift is set not to exceed 1/300 of the story height (*i.e.*, interstory drift ratio of 0.33%) under wind-induced vibration. With these performance limits and $\alpha = 0$, the fitness function in Eqn. 3.1 seeks to minimize the expected life-cycle cost by defining an optimal distribution of passive energy dissipation devices. From baseline analysis without dampers, $R_{\alpha}^{\text{bare}}[C] = E^{\text{bare}}[C]$ is found to be \$200000, \$14000, and \$220000, when the structure is assumed to be subjected to only seismic, only wind, and both seismic and wind loading, respectively, during its lifetime.

Figure 4.1a shows robust design alternatives of the sixteen-story building when the building is allowed to be designed only with viscoelastic solid dampers along the height of the building for three hazardous environment scenarios: (1) seismic loading environment (the left hand design), (2) wind loading environment (middle design), (3) combined seismic and wind environments (the right hand design). Within each story, the dimension of the central blue symbol illustrates damper size. When compared to the structural design for only seismic environment, smaller and fewer numbers of viscoelastic dampers are selected for the design under only wind environment in the middle plot of Figure 4.1a. For the design under combined loading in the right hand plot of Figure 4.1a, the fitness value reduced slightly due to an increased size of the viscoelastic dampers in the lower stories compared to the left hand plot. The results of life-cycle cost analysis reported in Figure 1b show the exceedance curves for expected damage loss before and after optimal retrofit design of the structure due to combined hazards. With the optimum retrofit configurations, the life-cycle damage costs of the structure are reduced by 80%, 96%, and 81% for seismic hazard, wind hazard, and combined hazards, respectively. When the retrofit cost is taken into account, the fitness values (reduction in expected life-cycle cost after retrofit) are found to be 0.55, 0.18, and 0.53 for seismic hazard, wind hazard, and combined hazards, respectively. Thus, with a minor cost increase, the structure could be protected not only against seismic hazard, but also wind hazard.

Table 4.1 Damage state interstory drift ratio limits and corresponding costs

Damage level	*Damage state*	*Interstory drift ratio, d (%)*	*Structural damage cost (% of building cost)*	*Cost of drift sensitive NSC (% of building cost)*
1	None	d<0.2	0.00	0.00
2	Slight	0.2<d<0.4	0.06	0.35
3	Light	0.4<d<0.6	0.60	1.40
4	Moderate	0.6<d<0.8	2.40	3.50
5	Extensive	0.8<d<2.5	5.40	17.50
6	Major	2.5<d<5	9.60	21.00
7	Destroyed	5<d	12.00	35.00

Table 4.2 Damage state floor acceleration limits and corresponding costs

Damage level	*Damage state*	*Peak floor acceleration, a (g)*	*Cost of acceleration sensitive NSC (% of building cost)*	*Cost of acceleration Sensitive Contents (% of building cost)*
1	None	a<0.1	0.00	0.00
2	Slight	0.1<a<0.25	0.66	0.36
3	Light	0.25<a<0.5	3.55	1.80
4	Moderate	0.5<a<1	10.53	9.00
5	Extensive	1<a<1.5	15.79	10.80
6	Major	1.5<a<2	26.32	12.60
7	Destroyed	2<a	35.00	18.00

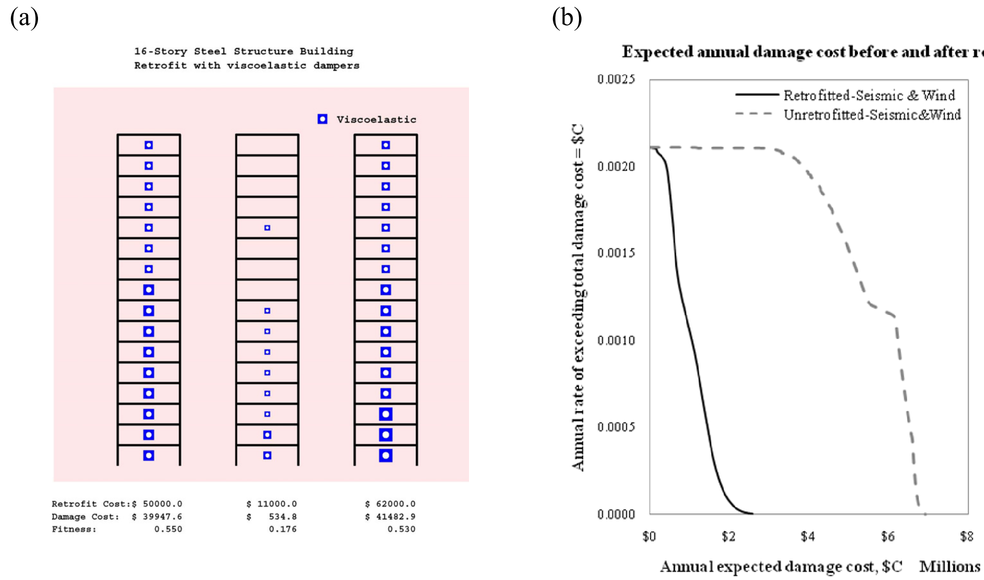

Figure 4.1 Sixteen-story steel frame (a) Robust designs due to different hazards; (b) Expected annual damage cost for combined hazards.

6. CONCLUSIONS

The results of the illustrative example suggest that the integrated Risk-based Optimal Multi-hazard Design framework can improve the performance of the structural system retrofitted with passive energy dissipation devices, while maintaining low expected life-cycle cost due to seismic and/or wind hazard. Optimum values of design variables are successfully determined by a genetic algorithm, which was used in this study as the optimization technique. Thus, it is concluded that the proposed method has advantages, not only from the viewpoint of performance under different hazard events, but also from an economic perspective. However, this study should be extended considering three important points. First, additional studies considering different structural systems and configurations must be carried out. Second, the cost data used in damage cost model is based on averages over metropolitan areas, and may not provide an accurate estimation of the cost of any specific building. Third, the results of the life-cycle cost study are sensitive to the assumptions made in developing the cost model. It should be noted that the simple expected life-cycle cost measure, used in the example, reduces the cost and probability information down to a single number and makes it difficult for decision makers to distinguish the risk characteristics of alternative designs. Therefore, full use of risk-based life-cycle cost models is recommended. In particular, the decision maker is usually assumed to be more concerned with the rare events that are presented in the right tails of the cost distribution functions. Thus, in future work, care must be exercised in defining appropriate tail distributions and in the selection of cost model parameters (*e.g.*, discount rate, design lifetime). Additionally, approaches that incorporate uncertainty in calculating probabilities should be integrated into the framework.

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