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Life Cycle Cost Analysis for Retrofit of Critical Infrastructure Subject to Multiple Hazards

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Abstract

The life-cycle cost can be regarded as a benchmark variable in decision making problems involving the retrofit of existing structures. A critical infrastructure is often subject to more that one hazard during its life time. Therefore, the problem of evaluating the life-cycle cost involves uncertainties in both loading and structural modeling parameters. The present study is a preliminary study aiming to calculate the expected life-cycle for a critical infrastructure subject to more than one hazard in its service life time. A methodology is presented which takes into account both the uncertainty in the occurrence of future events due to different types of hazard and also the deterioration of the structure as a result of a series of events. In order to satisfy life safety conditions, the probability of exceeding the limit state of collapse is constrained to be smaller than an allowable threshold. Finally, the methodology is implemented in an illustrative numerical example which considers a structure subject to both seismic hazard and blast hazard in both retrofitted and non-retrofitted configurations. It is demonstrated how expected life-cycle cost can be used as a criterion to distinguish between the two choices while satisfying the life safety constraint.

1 INTRODUCTION

The rescue operations, inspection and management of the civil structure, after the occurrence of a severe earthquake event is subject to considerable challenges. The post-main shock deterioration as a result of the sequence of aftershocks threaten significantly eventual inspection and/or reuse of these structures. A significant main shock is often followed by a number of aftershock events (usually smaller in moment magnitude) which take place in a limited area (i.e., the aftershock zone) around the epicenter of the main event. This sequence of aftershock events can last in some cases for more months. Although these events are smaller in magnitude with respect to the main event, they can prove to be destructive on the structure. This is due to both the significant number of main-shocks (in some cases up to 6000) and also due to the fact that the structure has probably already suffered damage from the main event.

The occurrence of main-shock events is often modeled by a homogenous poisson stochastic process with time-invariant rate. However, the sequence of aftershocks are characterized by a rate of occurrence that decreases as a function of time elapsed after the earthquake. Therefore, the occurrence of the aftershocks are modeled by a non-homogenous poisson process with a decreasing time-variant rate. The first few days after the occurrence of main-shock can be very decisive as there is urgent need for re-entrance in the buidling (for rescue or for inspection) while the daily aftershock rate is quite considerable.

Design and assessment of critical civil infrastructure can be considered as a decision making problem in which the desired performance objectives, defined in terms of a set of design parameters, are optimized subject to a number of constraints. In the context of performance-based design, several performance objectives (e.g., minimize initial cost of construction, ensure life-safety in case of extreme and rare events) can be considered for a set of [discrete] limit states. In order to implement the performance objectives in a decision making framework, it is desirable to quantify and measure these objectives in terms of a common benchmark variable. The life-cycle cost has been proposed by many (Wen, 2001), (Faber and Rackwitz, 2004), (Yeo and Cornell, 2008) as a suitable benchmark performance variable. Life-cycle cost, which is historically identified as an economic term expressed in monetary units, accounts for initial costs of construction of facility, the regular costs of its maintenance and functionality over time, loss of revenue in case of damage, re-pair/replacement costs, social losses including eventual loss of life and end-of-life recycling costs. The evaluation of life-cycle cost is subject to several sources of uncertainty, such as the occurrence and the intensity of critical hazards, the resistance of the infrastructure and the service life itself. Therefore, the life-cycle cost is generally evaluated in terms of its expected value over the life-time of the infrastructure.

The present study aims to apply the life-cycle cost criteria to retrofit design of an existing critical infrastructure located in a seismic zone. Given the importance of the infrastructure, an unexpected strong explosion is considered to be plausible through its life-time. Hence, the performance-based retrofit design of the infrastructure needs to be conducted on a multiple-hazard basis (i.e., earthquake and blast in this case). The retrofit design involves decision making between a set of viable options which can be evaluated and compared in terms of their corresponding life-cycle cost and subject to reliability constraints. In particular, for each retrofit option, the corresponding life-cycle cost is evaluated by calculating in monetary terms, the direct cost of the installation of the retrofit solution, the maintenance cost of the retrofitted system, the repair/replacement costs in case of damage, and the social costs including eventual loss of life and end-of-life recycling costs. After the low-cost option is identified among the set of options, the system reliability for the corresponding retrofitted infrastructure needs to be verified against a acceptable threshold. In this work, the system reliability is calculated taking into account both blast and earthquake hazards (Asprone et al., 2008a). It should be mentioned that the methodology presented in this work focuses on decision making between viable retrofit options for existing buildings. Thus, it has a different scope from methodologies (e.g., (Yeo and Cornell, 2008)) presented for decision making between a set of actions involving a structure in a post-main shock environment.

2 METHODOLOGY

The objective of this methodology is to evaluate the expected life-cycle cost for a civil infrastructure that is subject to multiple critical events/hazards during its life-time. First, the probability of exceeding a set of given structural limit states is calculated during the infrastructure's life time. Then, the expected life-cycle cost is calculated by taking into account the initial construction costs, the repair costs, the loss of revenue due to down time, and the eventual end-of-life recycling cost. The calculations involved in this methodology are based on a presumed specific set of rules for the management of the structure. The methodology presented herein for the evaluation of expected life-cycle cost can be used for decision making between different retrofit options while satisfying prescribed reliability constraints.

2.1 The Management Rules

A number of rules are introduced in order to outline the set of actions pursued by the management in case a critical event takes place and depending on the course of events experienced by the structure. It is assumed that once a critical event hits the structure, the structure is going to be immediately shutdown and repaired. The repair operation is supposed to restore the structure to its intact initial state. Moreover, it is assumed that the time of repair, which is also equal to the down-time for the structure, only depends on the state of the damaged structure. Furthermore, it is assumed that once the structure goes beyond the collapse limit state, it needs to be rebuilt/recycled. In case the structural repair in the aftermath of a critical event endangers the future repair operations, it is assumed that the structure is going to be replaced/recycled. The same decision is going to be taken when the cost of repair operations exceed the replacement costs.

2.2 Multi-Hazard Assessment of the Limit State Probability

Let T_{max} denote the life time of the structure, N the maximum number of critical events that can take place during T_{max} * and τ the repair time for the structure. The probability $P(LS; T_{max})$ of exceeding a specified limit state LS in time T_{max} can be written as:

$$P(LS; T_{max}) = \sum_{i=1}^{N} P(LS|i)P(i; T_{max}) \tag{1}$$

Where $P(LS|i)$ is the probability of exceeding the limit state given that exactly i events take place in time T_{max} and $P(i; T_{max})$ is the probability that exactly i events take place in time T_{max}. In order to calculate the term $P(i; T_{max})$ in a multi-hazard context, it is assumed that every type of event/hazard in the life-time of the structure is expressed by a *Poisson* probability distribution and that it is independent from other types of events. Therefore, $P(i; T_{max})$ can be calculated from a Poisson probability distribution with a rate equal to the sum of the rates for all N_h events/hazards considered:

$$\nu = \sum_{l=1}^{N_h} \nu_l \tag{2}$$

The probability of having exactly i events in time T_{max} can be calculated as:

$$P(i; T_{max}) = \frac{(\nu T_{max})^i e^{-\nu T_{max}}}{i!} \tag{3}$$

The term $P(LS|i)$ can be calculated by taking into account the set of mutually exclusive and collectively exhaustive (MECE) events that the limit state is exceeded at one and just one of the previous events:

$$\begin{aligned} P(LS|i) = P(C_1 + \overline{C_1}C_2 + \cdots + \\ + \overline{C_1C_2 \cdots C_{i-1}}C_i|i) \end{aligned} \tag{4}$$

*The number of possible events N in time T_{max} is unbounded.

where $C_j, j = 1 : i$ indicates the event of exceeding the limit state LS due to the jth event and $\overline{C_j}$ indicates the negation of C_j. The probability $P(C_j|i)$ can be further broken down into the sum of the probabilities of two MECE events that event j hits the intact structure and that the event j hits the damaged structure:

$$P(C_j|i) = P(C_jI|i) + P(C_jD|i) \tag{5}$$

Equation 5 can be further expanded as follows:

$$\begin{aligned} P(C_j|i) = P(C_j|I, i)P(I|i) + \\ + \textstyle\sum_{k=1}^{j-1} P(C_j|k, i)P(k|i) \end{aligned} \tag{6}$$

where $\{k : k = 1, 2, \cdots, i-1\}$ indicates the number of times the structure has been damaged before reaching the target limit state, implying that the structure deteriorates with the occurrence of each event. The formulation in Equation 6 is based on the consideration that an event can hit a structure already damaged by one or more *previous* event(s). In the framework of the rules described in the previous section, this situation occurs only if the inter-arrival time IAT for events is smaller than the repair time τ. Moreover, since the inter-arrival time can be described by the *Exponential* probability distribution, the probability that the IAT is less than or equal to the repair time τ can be expressed as $1-\exp(-\nu\tau)$ times the probability $\exp(-\nu\tau)$ that the structure is intact before k events. Therefore, the probability that the structure is damaged k times before reaching LS is equal to:

$$P(k|i) = e^{\nu\tau}(1 - e^{\nu\tau})^k \tag{7}$$

Assuming that if structure under repair is hit by another event, the repair operations are going to resume from zero. Thus, the probability that the structure is intact when hit by an event can be calculated as the probability that the IAT is greater than the repair time:

$$P(I|i) = e^{\nu\tau} \tag{8}$$

Observing Equation 6, one can identify the sequence of the fragility terms, namely, $P(C_j|I, i)$ and $P(C_j|k, i)$ where $k = 1, \cdots, (j-1)$. These fragility terms can be further expanded, assuming that the event j can only take place due to one of the set of N_h hazards considered in Equation 2. Therefore, the $P(C_j|I,i)$ can be expanded as following:

$$P(C_j|I,i) = \sum_{l=1}^{N_h} \frac{\nu_l}{\nu} P(C_j|H_l,I,i) \tag{9}$$

where ν_l is the mean annual rate of occurrence for hazard H_l, the ratio ν_l/ν is the probability that the next event is of type l and $P(C_j|H_l,I,i)$ is the probability that the intact structure exceeds the limit state LS due to a hazard of type l. The same approach can be used for expanding $P(C_j|k,i)$:

$$P(C_j|D_k,i) = \sum_{l=1}^{N_h} \frac{\nu_l}{\nu} P(C_j|H_l,k,i) \tag{10}$$

where $P(C_j|H_l,k,i)$ denotes the probability of exceeding the limit state LS due to an event of kind H_l given that the structure has been damaged due to $k = 1, \cdots, (j-1)$ events before reaching the limit state LS (without being repaired, i.e., $IAT < \tau$ for each event).

2.2.1 Estimation of Fragilities

In order to calculate the sequence of fragility terms $P(C_j|D_k,i)$ where $k = 1, \cdots, (j-1)$, the average number of events needed to make the structure exceed the target limit state LS can be estimated as:

$$\overline{s} = \sum_{l=1}^{Nh} \frac{\nu_h}{\nu} s_h \tag{11}$$

where s_h is the average number of events of the type H_k needed to make the structure exceed the target limit state. Therefore, the kth term in the sequence of fragilities $P(C_j|k,i)$ can be calculated as follows:

$$P(C_j|k,i) = \sum_{l=1}^{N_h} \frac{\nu_h}{\nu} P(C_j|H_l,k,i) \tag{12}$$

where $P(C_j|H_l,k,i)$ is the probability of exceeding the limit state due to hazard H_l, which can be approximated from an empirical formula as the probability of exceeding the limit state due to hazard H_l given the structure is initially in its intact state plus an additional term:

$$\begin{aligned} P(C_j|H_l,k,i) = P(C_j|H_l,I,i) + \\ + \min(1, \frac{k}{\overline{s}})[1 - P(C_j|H_l,I,i)] \end{aligned} \tag{13}$$

when the number of times the structure is damaged k exceeds $\overline{s}$, it is set equal to $\overline{s}$.

The procedure described in this section for the calculation of the probability of exceeding limit state LS can be employed to calculate the limit state probabilities for an increasing sequence of limit states, e.g., from serviceability to collapse. It should be mentioned that the average number of events that takes for a structure to exceed a limit state depends on the severity of the so-called limit state. Moreover, the limit states can be defined in terms of different engineering demand parameters depending on the type of event/hazard H_k.

2.3 The Probability of Collapse in a Year

In the previous section, it is explained how the probability of exceeding the limit state LS can be calculated from Equation 1. However, in order to allow for discounting of the future costs into present, it is of interest to calculate the probability of exceeding the limit state in a year. The probability of exceeding the limit state in the time interval $[T, T+\Delta T]$ can be calculated as:

$$P(LS;[T,T+\Delta T]) = P(LS;T+\Delta T) - P(LS;T) \tag{14}$$

Therefore, the probability of exceeding the limit state in a year can be calculated from Equation 14, by setting ΔT equal to one.

2.4 Expected Life-cycle Cost

The expected life-cycle cost is calculated from the following equation (Wen, 2001):

$$E[L;T_{max}] = C_O + C_R + C_M \tag{15}$$

where C_O is the initial construction/retrofit installation costs, C_R is the repair/replacement costs taking into account also the loss of revenue due to downtime and C_M is the annual maintenance costs. The repair cost C_R can be calculated from the following equation:

$$C_R = \sum_{n=1}^{N_{LS}} \sum_{t=1}^{T_{max}} L_n e^{-\lambda t} [P(LS_{n+1}; t) - P(LS_n; t)] \tag{16}$$

where N_{LS} is the number of limit states ranging from the intact state of the structure up to the limit state of collapse, L_n is the expected cost of restoring the structure from the limit state LS_n back to its intact state including eventual loss of revenue caused by interruption for repair operations. In the case of collapse limit state, L_n is equal to the end-of-life replacement cost. λ is the discount rate and the last term in Equation 16 is the probability that the structure is between limit states n and $n+1$. The cost of maintenance C_M can be calculated from the following equation:

$$C_M = \sum_{t=1}^{T_{max}} C_m e^{-\lambda t} \tag{17}$$

where C_m is the constant annual maintenance cost.

3 NUMERICAL EXAMPLE

The methodology presented in the previous section is applied to an existing structure as a case study.

3.1 Structural Model

The case-study building is a generic five-story RC frame structure. The structural model is illustrated in Figure 1, presenting a plan of the generic storey. Each storey is 3.00m high, except the second one, which is 4.00m high. The non-linear behavior in the sections is modeled based on the concentrated plasticity concept. It is assumed that the plastic moment in the hinge sections is equal to the ultimate moment capacity in the sections which is calculated using the Mander (Mander et al., 1988) model for concrete and elastic-plastic model for steel rebar. The retrofit intervention consists of steel brace couples, installed in the panels indicated with a bold line in Figure 1, at every floor; in particular, from first to third floor braces with 10cm^2 of area and in the last two floors braces with 6cm^2 of area are considered.

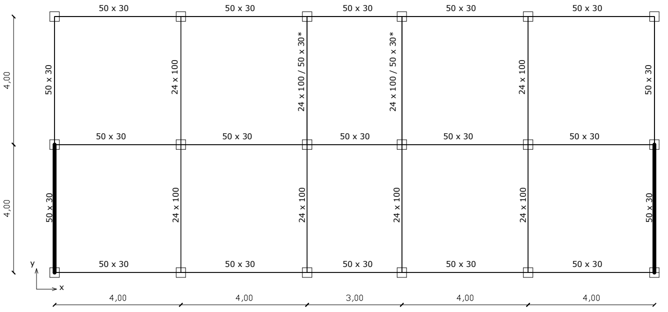

Figure 1: Storey view (dimensions in m) *Beam frame labels indicate the section dimensions in cm; column sections are all (30×30) *this frame represents both storey beams (24×100) and stair knee beams (50×30)*

3.2 Retrofit Decision Making Using Life-cycle Cost Analysis

The case-study structure is analyzed for both seismic and blast hazards in the two cases a)original structure and b) structure retrofitted with braces in a previous work by the authors (Asprone et al. 2008b). The probability of exceeding the limit state of collapse has been calculated for both hazards. In the case of seismic hazard, the limit state of collapse has been defined in relation to the maximum rood displacement. In the case of blast, the collapse limit state has been identified in relation to the required service load multiplier to achieve global instability in the structure. It was demonstrated that the seismic retrofit strategy of adding braces to the structure leads to an increase in both seismic reliability and blast reliability. The mean annual rate of significant earthquake events is assumed to be equal to $\nu_{earthquake} = 0.10$ and the mean annual rate of blast events is assumed to be equal to $\nu_{blast} = 0.005$. In this study, the objective is to compare the two decisions, namely, retrofit with braces and no retrofit

Table 1: Equivalent SDOF maximum displacement [meters]

LS	Retrofit	No Retrofit
Serviceability	0.02	0.01
Onset of damage	0.03	0.02
Severe Damage	0.07	0.06
Collapse	0.10	0.10

Table 2: The load multipliers for blast

LS	Retrofit	No Retrofit
Serviceability	4.2	4.2
Onset of damage	4	4
Severe Damage	2	2
Collapse	1	1

based on life-cycle cost analysis subject to reliability constraints. The sequence of structural limit states $LS_n, n = 1, \cdots, N_{LS}$ from the intact state to collapse are discretized as, *intact*, *serviceability*, *onset of damage*, *severe damage* and *collapse*. Tables 1 and 2 illustrate the relation between each limit state and the corresponding EDP for earthquake and blast hazards respectively. In the case of earthquake hazard, the limit states are distinguished in terms of increasing levels of the maximum displacement for the equivalent SDOF system. In the case of blast, the limit states are identified by decreasing levels of the load multiplier that, once applied on acting loads, would result in global instability. The probabilities of exceeding each limit state, from intact state I, are calculated for earthquake and blast hazard (Asprone et al. 2008b) and tabulated in Tables 3 and 4, respectively. The expected loss is calculated assuming a service life of $T_{max} = 100$ years and a maximum number of events $N = 50$ and a discount rate equal to $\lambda = 0.05$. It is assumed that the duration of the repair operations depends only on the structural limit state. It is further assumed that the repair costs constitute a fraction of end-of-life replacement cost R depending on the structural limit state. The average number of events that make the structure exceed the collapse limit state for both blast and earthquake hazards, namely, s_b, s_e, is assumed to depend on the structural limit state. It is assumed that the annual maintenance cost M is equal to a fraction of the initial cost of construction/installation C_O. Table 5 and Table 6 outline the parameters used in the case-study.

Table 3: The $P(LS_n|\text{earthquake}, I)$

LS	Retrofit	No Retrofit
Serviceability	4.72×10^{-2}	2.75×10^{-1}
Onset of damage	1.94×10^{-2}	6.02×10^{-2}
Severe Damage	3.65×10^{-3}	6.50×10^{-3}
Collapse	1.11×10^{-3}	1.76×10^{-3}

Table 4: The The $P(LS_n|\text{blast}, I)$

LS	Retrofit	No Retrofit
Serviceability	1	1
Onset of damage	0.20	0.21
Severe Damage	0.10	0.18
Collapse	0.04	0.12

Table 5: LCA parameters

LS	Repair Time [months]	Repair Cost	s_b	s_e
serviceability	2	$(1/3)R$	1	2
onset of damage	6	$(2/3)R$	2	4
severe damage	12	R	3	5
collapse	12	R	4	6

Table 6: The LCA constant parameters

Decision	C_O [M euro]	R	M [M euro/yr]	DT
Retrofit	1.075	$1.1C_O$	$0.01C_O$	0.10
No Retrofit	1.0	$1.1C_O$	$0.01C_O$	0.10

The probability of exceeding a specified limit state is calculated as a function of time $t = 1 : T_{max}$ by employing the procedure explained in Section 2.2 for both cases of retrofitted and not retrofitted structure. The results are plotted in Figures 2 and 3, respectively where the probabilities of exceeding the specified limit states are illustrated. As a proxy for life safety considerations, the acceptable threshold of 2×10^{-3} is set for the probability of collapse in a year (the reliability constraint). The probability of exceeding the limit state of collapse in a year has been calculated from Equation 14 for the structure before and after retrofit. The results are plotted in Figure 4, where they are compared against an acceptable mean annual collapse rate of 2×10^{-3}. This verification is done as a proxy for ensuring life safety for the building occupants. It can be observed that the non-retrofitted structure ceases to be safe after a certain point in time (10 years); whereas, the retrofitted structure remains safe throughout the entire life-time of the structure. It is interesting to note that both the retrofitted and non-retrofitted structure satisfied the safety constraint at the beginning of the structural life-time.

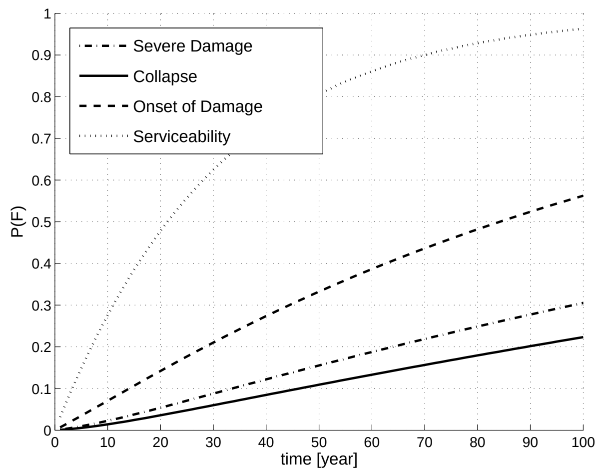

Figure 2: The limit state probabilities for the structure before retrofit

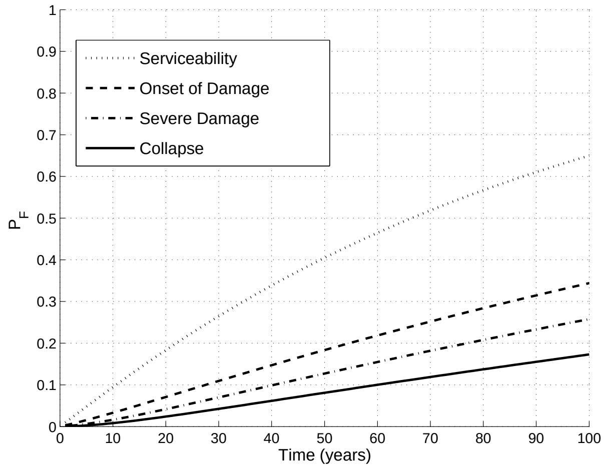

Figure 3: The limit state probabilities for the structure before retrofit

In order to examine the effect of considering the blast hazard in the assessments, the probability of collapse in a year is calculated for both retrofitted and non-retrofitted structures, considering only the seismic hazard. The results are reported on Figure 4. It can be observed that considering only the seismic hazard underestimates the probability of collapse in a year. Moreover, in both cases, the structure satisfies the safety criterion. The expected life-cycle cost is calculated

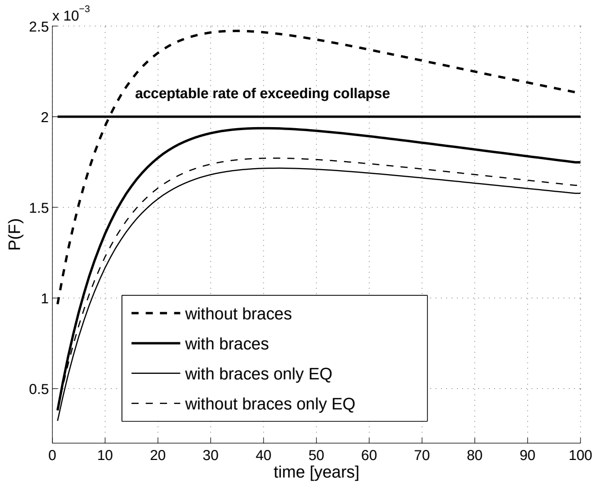

Figure 4: Probability of exceeding the collapse limit state in a year

employing the procedure described in Section 2.4 for the structure before and after retrofit based on the parameters reported in Table 3.2. The initial cost of construction/installation is larger for the retrofitted structure with respect to the non-retrofitted structure in order to take into account the cost of installation of the braces. Therefore, by association, also the maintenance cost is larger for the retrofitted structure. The expected life-cycle cost curves for both structures are plotted versus life-time in Figure 5. It can be observed from the figure that the expected life-cycle cost for the retrofitted structure after about 12 years is exceeded by the non-retrofitted structure. Therefore, the results confirm that for structure with a life-time longer that 12 years, the decision to retrofit is also justified by the life-cycle cost criterion. The expected life-cycle cost is calculated also considering only the seismic hazard for both the retrofitted and the non retrofitted structure. The results are plotted in Figure 6. It is evident that the expect costs are underestimated significantly if only the seismic hazard is being considered.

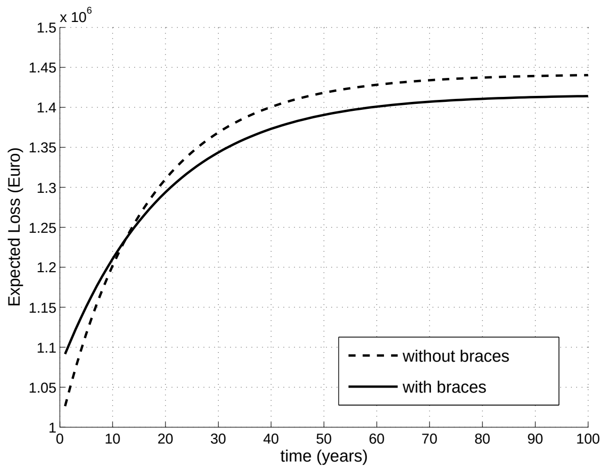

Figure 5: The expected life-cycle cost

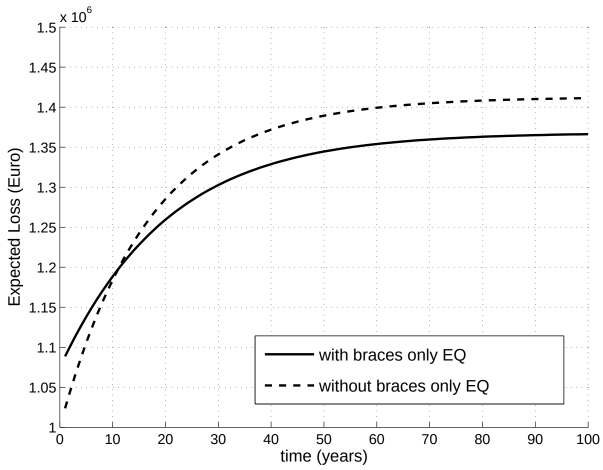

Figure 6: The expected life-cycle cost subject to seismic hazard only

4 Conclusions

This paper presents a preliminary effort for quantification of expected life-cycle cost for structures deteriorating under multiple events/hazards. The expected life-cycle cost is intended as a criterion for deciding between viable retrofit options. A methodology is presented for the estimation of the life-cycle cost taking into account the deterioration of structure in time as it is subject to a sequence of critical events. The case study presented in this work focuses on a structure subject to both earthquake and blast hazards, as an example of multi-hazard assessment. The decision between to retrofit or not to retrofit is made based on the minimization of the life-cycle cost subject to reliability constraint. It is demonstrated that the retrofitted structure not only has less expected life-cycle cost in the long run, but also it satisfies the life safety criterion. On the other hand, the non-retrofitted structure after a certain point in time ceases to satisfy the life safety criterion and starts to be more expensive. Hence, the presented methodology can be implemented in a decision making framework for retrofit design of existing structures based on minimum life-cycle cost considerations and accounting for multiple critical events.

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