Seismic Retrofit Decision-Making based On Life Cycle Cost Criteria

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SUMMARY:
The life-cycle cost can be regarded as a benchmark variable in decision making problems involving the seismic retrofit and upgrading of existing structures. A methodology is presented which takes into account both the uncertainty in the occurrence of future earthquake events and also the deterioration of the structure as a result of a series of earthquake 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. The presented methodology is applied to retrofit decision-making for a set of viable upgrade options for upgrading of an existing RC building. The upgrade solutions considered include (a) reinforcing with fiber reinforced polymers (FRP), (b) addition of buckling restrained axial dampers (BRAD), (c) reinforcing with steel angles and steel ribbons (the CAM system), (d) applying both the FRP reinforcement and the BRAD, (e) applying both the CAM solution and the BRAD.

Keywords: performance-based design, life-cycle cost, seismic retrofit, structural reliability, Poisson model

1. INTRODUCTION

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, Porter et al. 2001, Franchin et al. 2006, Goulet et al. 2007) 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 its 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 subjected to several sources of uncertainty, such as the occurrence and the intensity of future earthquake events, the structural resistance and the service life itself. 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 structure located in a seismic zone. 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 subjected to reliability constraints. In particular, for each upgrade option, the corresponding life-cycle cost is evaluated by calculating in monetary terms, the direct cost of the installation of the upgrade solution, the maintenance cost of the upgraded system, the repair/replacement costs in case of damage, and the costs including eventual loss of life and end-of-life recycling costs.

The methodology for calculating the life-cycle cost takes into account the time-varying profile of the probability of exceeding a set of structural limit states. The homogeneous Poisson process is used to model the probability that a certain number of earthquakes take place over the structural life cycle. Given the number of earthquake events, the probability of exceeding different structural limit states is calculated by considering the probability that the structure is going to be repaired before the next events takes place. For the simplicity of calculations, a degrading single degree of freedom model is adopted based on the results of the pushover analysis for the multi-degree of freedom structure. The calculated time-dependent limit state probabilities are then used in order to calculate the expected life-cycle cost for each retrofit option considered. After the low-cost option is identified among the set of options, the structural reliability for the corresponding upgraded infrastructure needs to be verified against a acceptable threshold. The presented methodology is applied to retrofit decision-making for a set of viable upgrade options for upgrading of an existing RC building. The upgrade solutions considered include (a) reinforcing with fiber reinforced polymers (FRP), (b) addition of buckling restrained axial dampers (BRAD), (c) confining the columns using steel angles and steel ribbons (the CAM system), (d) applying both the FRP reinforcement and the BRAD, (e) applying both the CAM solution and the BRAD. The optimal solution is highlighted based on the minimization of the life cycle cost satisfying the acceptable reliability-based criteria.

2. METHODOLOGY

The objective of this methodology is to evaluate the expected life-cycle cost for a structure that is subjected to seismic action during its life-time. First, the probability of exceeding a set of given structural limit states is calculated during the structural 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 set of assumption described in the following section The methodology presented herein for the evaluation of expected life-cycle cost can be used for decision making between different upgrade options while satisfying prescribed reliability constraints.

2.1. The set of assumptions

It is assumed that once a seismic 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. Assessment of the Limit State Probabilities

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:

$$P(LS;T_{\max}) = \sum_{i=1}^{N} P(LS \mid i) P(i;T_{\max}) \qquad (2.1)$$

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 $P(i; T_{max})$, it is assumed that the earthquake occurrence in the life-time of the structure is expressed by a stationary Poisson probability distribution. Therefore, $P(i; T_{max})$ the probability of having exactly i earthquake events in time T_{max} can be calculated as:

$$P(i;T_{\max}) = \frac{\left(\upsilon T_{\max}\right)^{i} e^{-\upsilon T_{\max}}}{i!} \qquad (2.2)$$

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:

$$P(LS \mid i) = P(C_1 + \overline{C_1}C_2 + \overline{C_1C_2}C_3 + ... + \overline{C_1C_2....C_{i-1}}C_i \mid i) \qquad (2.3)$$

where Cj ; j = 1 : i indicates the event of exceeding the limit state LS due to the jth event and Cj indicates the negation of Cj. Since the events $\overline{C_1C_2....C_{j-1}}C_j$, j=1:i, are MECE, P(LS|i) can be calculated by summing up the probabilities for each separate term in **Equation 2.3**:

$$P(LS \mid i) = P(C_1 \mid i) + P(C_2\overline{C_1} \mid i) + ... + P(C_i\overline{C_1C_2....C_{i-1}} \mid i) \qquad (2.4)$$

The expression in **Equation 2.4** can be expanded as follows:

$$P(LS \mid i) = P(C_1 \mid i) + P(C_2 \mid \overline{C_1}, i)P(\overline{C_1} \mid i) + ... + P(C_i \mid \overline{C_1C_2....C_{i-1}}i)P(\overline{C_1C_2....C_{i-1}} \mid i) \quad (2.5)$$

For example, the second term can be read as: "*the probability that the structure exceeds the limit state threshold after the occurrence of the second event given that the structure has not exceeded the limit state after the occurrence of the first event times the probability that the structure has not exceeded the limit state after the first event*". It can be shown the general term j (j=1:i) in **Equation 2.5** can be calculated in a recursive manner (i.e., employing the preceding terms in the sequence) from the following expression:

$$P(\overline{C_1C_2....C_{j-1}}C_j \mid i) = P(C_j \mid \overline{C_1C_2....C_{j-1}}i)\prod_{k=1}^{j-1}(1 - P(C_k \mid \overline{C_1C_2....C_{k-1}}, i)) \qquad (2.6)$$

Note that $P(\overline{C_k} \mid \overline{C_1C_2....C_{k-1}}, i)$ is calculated as $1 - P(C_k \mid \overline{C_1C_2....C_{k-1}}, i)$. The probability $P(C_j \mid \overline{C_1C_2....C_{j-1}}, i)$ can be further broken down into the sum of the probabilities of two MECE events that event j hits the intact structure (denoted by D_0) and that the event j hits the damaged structure (based on the Total Probability Theorem):

$$P(C_j \mid \overline{C_1C_2....C_{j-1}}, i) = P(C_jD_0 \mid \overline{C_1C_2....C_{j-1}}, i) + P(C_jD \mid \overline{C_1C_2....C_{j-1}}, i) \qquad (2.7)$$

Equation 2.7 can be further expanded as follows:

$$\begin{aligned} P(C_j \mid \overline{C_1C_2....C_{j-1}}, i) &= P(C_j \mid D_0, \overline{C_1C_2....C_{j-1}})P(D_0 \mid i, \overline{C_1C_2....C_{j-1}}) \\ &+ \sum_{k=1}^{j-1} P(C_j \mid D_k, \overline{C_1C_2....C_{j-1}})P(D_k \mid i, \overline{C_1C_2....C_{j-1}}) \end{aligned} \qquad (2.8)$$

where $P(D_0 \mid i, \overline{C_1C_2....C_{j-1}})$ is the probability that the structure is re-pristined back to its intact state right before the last jth event takes place; $P(D_k \mid i, \overline{C_1C_2....C_{j-1}})$ is the probability that the structure is subjected to exactly k events while it has been under repair; $k = 1 : (j-1)$ indicates the number of times the structure has been subjected to events while it has been under repair. The sequence probability terms $\left\{P(C_j \mid D_k, \overline{C_1C_2....C_{j-1}}, i) \mid k = 0,1,...,(j-1)\right\}$ are the limit state probabilities given that the structure is hit k times by earthquake events without exceeding the limit state threshold in the previous events.[1] The formulation in **Equation 2.8** is based on the consideration that an event can hit a structure already damaged by one or more previous event(s). In order for a structure to be damaged k times (without being repaired) before reaching the limit state threshold, it is necessary that for all of the preceding k events the inter-arrival time (IAT) between the events is smaller than the time needed for repairing the structure τ and that the structure is intact (i.e., has already been repaired) before the ultimate k events take place. Therefore, the probability that the structure has experienced exactly k events (in sequence) before reaching the limit state can be calculated as follows:

$$P(D_k \mid \overline{C_1 C_2 \ldots C_{j-1}}, i) = e^{-\nu\tau}(1-e^{-\nu\tau})^k \qquad (2.9)$$

Where the inter-arrival time (IAT) is described by the Exponential probability distribution. Thus, the probability that the IAT is less than or equal to the repair time τ is expressed as 1-exp(-$\nu\tau$); the probability that the IAT is larger than the repair time (i.e., that the structure is re-pristined back to its original state before the occurrence of the ultimate k events) is expressed as exp(-$\nu\tau$). In the same manner the probability that the structure is intact before being subjected to the jth event can be written as:

$$P(I \mid \overline{C_1 C_2 \ldots C_{j-1}}, i) = e^{-\nu\tau} \qquad (2.10)$$

which is equal to the probability that the IAT is greater than the repair time τ and IAT is described by the Exponential distribution.

The sequence of probability terms $\left\{P(C_j \mid D_k, \overline{C_1 C_2 \ldots C_{j-1}}, i) \mid k = 0,1,\ldots,(j-1)\right\}$ denoting the probability that the structure exceeds a given limit state threshold after the jth event given that it is subjected to k events without being repaired is calculated using non-linear dynamic analyses for an equivalent SDOF system. The next section is going to describe in detail how the abovementioned sequence of probability terms is calculated.

2.1. Calculation of the sequence of limit state probabilities

The sequence of limit state probabilities are calculated based on the following procedure (Jalayer et al. 2011, Yeo and Cornell 2009):

1. The structural pushover curve is constructed by carrying a nonlinear static analysis. The onset of various structural limit states is marked on the curve in terms of the maximum roof displacement.
2. The structural pushover curve is transformed into that of an equivalent SDOF system.
3. The SDOF pushover curve is used in order to construct the hysteresis model of the equivalent SDOF system for nonlinear time-history analyses.
4. A suite of ground motion records is chosen for the site of the structure.

for k=1: N_{events}
(N_{events} is the maximum number of events realistically taking place in the lifetime of the structure)

5. Each record in the set is "cloned" k times (repeated k times in sequence)
6. The equivalent SDOF structure is subjected to the suite of the "cloned" GM records. However, the ground motion records that have caused the structure to exceed the limit state threshold -in the previous (k-1) steps - are taken out from the suite of records.
7. The maximum and residual displacements and residual strength of the equivalent SDOF system in response to the suite of records are registered.

[1] Note that the fact that a total i events have taken place does not offer additional information with respect to the knowledge that the structure has not exceeded the limit state in the previous k events. Therefore, for simplicity the information on i events taking place is dropped from the notation.

8. The structural response to the suite of records is then used to conduct a linear regression analysis on the (natural logarithm of) SDOF maximum response versus (natural logarithm of) spectral acceleration at the secant period T_{k-1} calculated in the previous step (see the Cloud Method, Jalayer and Cornell 2008). This provides the conditional mean $\eta_k(S_a(T_{k-1}))$ and standard deviation β_k of the (natural logarithm of the) structural response versus spectral acceleration.
9. The residual displacement for each record is used to calculate the secant period T_k of the equivalent SDOF system.
10. Steps 5–9 are repeated for the same suite of ground motion records until $k=_{Nevents}$.

If the limit state threshold marking the onset of limit state LS is denoted by $d_{cap}(LS)$, the structural fragility or the probability of exceeding the limit state given spectral acceleration and given that the structure is subjected to j events (without exceeding the limit state threshold and without being repaired after the previous k-1 events) can be calculated as:

$$P(C_j \mid S_a(T_{k-1}), D_k, \overline{C_1 C_2 .... C_{j-1}}) = 1 - \Phi\left(\frac{\log\left(\frac{\eta_k(S_a(T_{k-1}))}{d_{cap}(LS)} \right)}{\beta_k} \right) \qquad (2.11)$$

where

$$\log \eta_k(S_a(T_{k-1})) = a + b \log(S_a(T_{k-1}))$$

$$\beta_k = \sqrt{\frac{\sum_{i=1}^{N(k)} \left(\log(d_{\max,k}(i)) - a - b \log(S_a(i, T_{k-1})) \right)^2}{N(k) - 2}} \qquad k = 1 : N_{events} \qquad (2.12)$$

where N(k) is the number of ground motion records in the suite of records at each step k; $d_{max,k}(i)$ is the maximum displacement response of the SDOF system in response to record i after it has been subjected to the suite of GM records k times; a and b are the coefficients of the linear regression of $\log(d_{max,k})$ on $\log(S_a(T_{k-1}))$ (see Jalayer et al. 2011 for more details). $S_a(T_{k-1})$ is the spectral acceleration at the average (softened) period of the SDOF system after it has been subjected to the suite of records k−1 times. Finally, the limit state probability $P(C_j \mid D_k, \overline{C_1 C_2 .... C_{j-1}})$ is calculated by integrating the structural fragility, denoted by $P(C_j \mid S_a(T_{k-1}), D_k, \overline{C_1 C_2 .... C_{j-1}})$, and the spectral acceleration hazard, denoted by $\lambda(S_a(T_{k-1}))$ as follows[2]:

$$P(C_j \mid D_k, \overline{C_1 C_2 .... C_{j-1}}) = \int_{S_a(T_{k-1})} P(C_j \mid x, D_k, \overline{C_1 C_2 .... C_{j-1}}) \left| d\lambda(x) \right| \qquad (2.13)$$

where $\lambda(S_a(T_{k-1}))$ is the mean annual frequency of exceeding spectral acceleration (aka. spectral acceleration hazard) at a period equal to T_{k-1} which is the period calculated after the SDOF system has been subjected to the suite of ground motion records k−1 times.

2.2. The probability of exceeding the limit state in a year

In the previous section, it is explained how the probability of exceeding the limit state LS can be calculated. However, in order to allow for discounting of the future costs into present, it is of interest

[2] Strictly speaking, the left-hand side of Equation 2.13 is the rate of exceeding the limit state and not the probability of exceeding the limit state. Herein, it is treated as a probability term.

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 +ΔT] can be calculated as:

$$P(LS;[T,T+\Delta T]) = P(LS;T+\Delta T) - P(LS;T) \qquad (2.14)$$

Therefore, the probability of exceeding the limit state in a year can be calculated from Equation (2.14), by setting ΔT equal to 1.

2.3. Expected Life Cycle Cost

The expected life-cycle cost is calculated from the following equation (Wen 2001, Porter et al. 2001, Miranda and Aslani 2003):

$$E[L;T_{\max}] = C_0 + C_R + C_M \qquad (2.15)$$

where C_O is the initial construction/upgrade 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:

$$C_R = \sum_{n=1}^{N_{LS}} \sum_{t=0}^{T_{\max}-1} L_n e^{-\zeta t} \left[P(LS_{n+1};[t,t+1]) - P(LS_n;[t,t+1]) \right] \qquad (2.16)$$

Where $P(\mathrm{LS}_n;\ [t,\ t{+}1])$ is the probability of exceeding the limit state LS_n in the one-year time interval $[t,\ t{+}1]$ from **Equation 2.14**, 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. ζ is the discount rate and the term in the brackets of **Equation 2.16** 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:

$$C_M = \int_0^{T_{\max}} C_m e^{-\zeta t} dt = \frac{C_m}{\zeta}[1 - e^{-\zeta T_{\max}}] \quad (2.17)$$

where C_m is the (constant) annual maintenance cost.

3. NUMERICAL EXAMPLE

The methodology described in the previous section is applied to performance-based retrofit design of an existing RC building in order to find the most suitable retrofit solution according to life cycle cost and reliability criteria.

2.3. Building Description

The RC frame existing building considered in this study was built in the late 1930s and is characterized by a rectangular plan layout whose sides are 54.5 m and 18.5 m long and the total height is 19.2 m, as shown in **Figure 1**. The structural system consists of frames aligned in one direction only and the stairs are located in a slightly eccentric position, as shown in the **Figure 1**. The 4-storey building has large interstorey heights in the range between 4.58 m and 5.10 m, as shown **in Figure 1**, and the floors are placed at a height of 5.10 m, 9.86 m, 14.62 and 19.2 m. The floor slabs consist of 21 cm and 23 cm deep cast in situ concrete and brick decks at the first floor and all the other floors, respectively. The solid slab thickness is 5 cm at all floors; thus diaphragmatic behavior may be assumed for the sample frame. The as-built framed system employs deep foundations consisting of plinths on piles, connected each other's.

Storey-dependent concrete compression strengths are adopted: fcc1=19.16 MPa, fcc2=18.51 MPa, fcc3=13.44 MPa and fcc4=22.50 MPa, for the first, the second, the third and fourth floor, respectively; further details are available in Chiodi et al. (2011).Tensile tests were also carried out on steel reinforcement smooth bars; the laboratory tests showed quite uniform yield strength fy and ultimate strength fu for the first, the second and the fourth floor and rather lower yield strength for the third floor. Thus, it is assumed values of fy=320.38 MPa and fu=418.18 MPa for the third floor and values of fy=393.96 MPa and fu=479.72 MPa for all the other floors. On average, the estimated material overstrength is about 1.24 and the ultimate elongation is higher than 10%, demonstrating a good ductility of the steel reinforcement.

Refined three-dimensional (3D) finite element models using SismiCad vers. 11.10 were employed to analyze the as-built and retrofitted structures. A conventional viscous damping coefficient equal to 4% has been assumed for the as-built structure. The as-built structure was characterized by a fundamental mode of vibration along the longitudinal axis whose period is equal to 0.94 s. The second mode of vibration, whose period is equal to 0.77 s, has a strong torsional coupling due mainly to the difference between the mass centroid and stiffness centroid due to the stairs. In order to calibrate the finite element model and test its reliability with respect to the quality of vibration modes, the natural frequencies and the damping ratios are experimentally investigated by operational modal analysis tests (Chiodi et al., 2011).

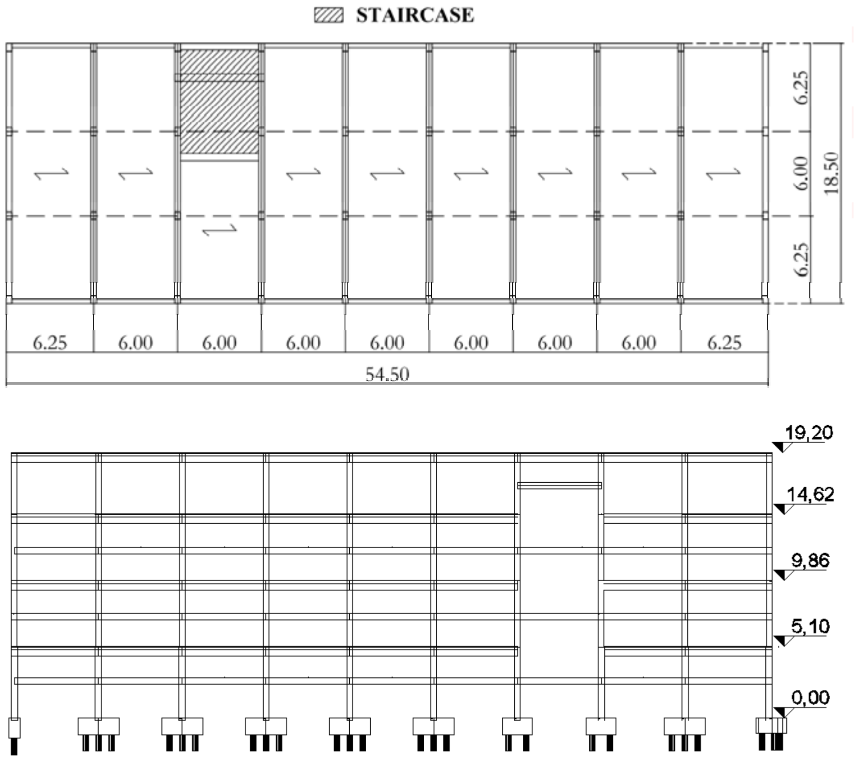

Figure 1. The plan and elevation views of the as-built existing RC building (dimesions are in meters)

3.2 Alternative Retrofit Solutions

The evaluation of the structural safety of the structure was carried out with nonlinear static (pushover) analyses both for the as-built and retrofitted structural systems. Two lateral force patterns were employed for the seismic structural assessment:

- a modal pattern, proportional to lateral forces consistent with the mode of vibration determined by the eigenvalue analysis (T);
- a uniform pattern, based on the distribution of mass along the height (R).

The performance points at *serviceability*, *onset of damage*, *severe damage* and *collapse* limit states are obtained based on the pushover results (**Table 3.1**). The computed results showed that the as-built system is characterized by a low stiffness and ductility along X-direction, that is the weaker direction due to the lack of frames. In order to perform further analyses on life cycle costs only two pushover curves have been considered, one referred to modal load distribution in the X direction (T+X) and the other to the uniform load distribution in the Y direction (R+Y). The seismic strategies were aimed at enhancing the global lateral stiffness, the strengthening, the ductility or a combination of them. The inelastic seismic performance of the retrofitted structure was investigated through nonlinear static analyses. Such analyses were performed with respect to different retrofitting strategies:

- Buckling restrained braces placed along the perimeter frames (BRAD);
- Buckling restrained braces placed along the perimeter frames and local strengthening with CFRP (BRAD+FRP);
- Buckling restrained braces placed along the perimeter frames and local strengthening with CAM system (BRAD+CAM);
- local strengthening with CFRP (FRP);
- local strengthening with CAM system (CAM).

For the first three strategies, a conventional viscous damping coefficient equal to 10% has been assumed and the results showed a significant increase of the base shear and the global stiffness that is produced by the diagonal braces; however, when the BRAD's retrofitting strategy was uncoupled from local strengthening, many brittle failures occurred and the global ductility was very low. Although the local strengthening did not enhance the global strengthening and stiffness, a high global ductility was achieved. This is true both for FRP strategy and CAM strategy that were about identical in terms of seismic performance.

Table 3.1. Limit state threshold d_{cap}(LS) for the four limit states and the two loading cases considered

T+X						
LS	CAM+BRAD	FRP+BRAD	BRAD	FRP	CAM	AS IS
	D(m)	D(m)	D(m)	D(m)	D(m)	D(m)
Service	0.030	0.030	0.001	0.30	0.030	0.001
Onset	0.040	0.040	0.001	0.035	0.035	0.001
Severe	0.100	0.100	0.001	0.060	0.060	0.001
Collapse	0.150	0.150	0.08	0.150	0.150	0.12
R+Y						
Service	0.030	0.030	0.001	0.030	0.030	0.001
Onset	0.040	0.040	0.001	0.035	0.035	0.001
Severe	0.100	0.100	0.001	0.060	0.060	0.001
Collapse	0.15	0.150	0.090	0.150	0.150	0.0083333

3.3 The Numerical Results

The expected cost in the life-time of the structure is calculated for the 5 retrofit options described in the previous section using the methodology described herein. Table 3.2 summarizes the parameters used in the life cycle analysis.

Table 3.2. Life cycle cost analysis parameters

LS	Repair time (months)	Ln (Repair Cost)
Service	2	(1/3)R
Onset	6	(2/3)R
Severe	12	R
Collapse	12	R

*The initial construction costs C_o also include the initial costs of building construction.

Retrofit Option	C_O* (Euro)	R (*C_o)	Cm (*C_o/year)	DT (Euro/year)
As Is	7000000	1.1	0.01	100000
CAM+BRAD	8140000	1.1	0.01	100000
BRAD+FRP	8210000	1.1	0.01	100000
FRP	7920000	1.1	0.01	100000
BRAD	7290000	1.1	0.01	100000
CAM	7850000	1.1	0.01	100000

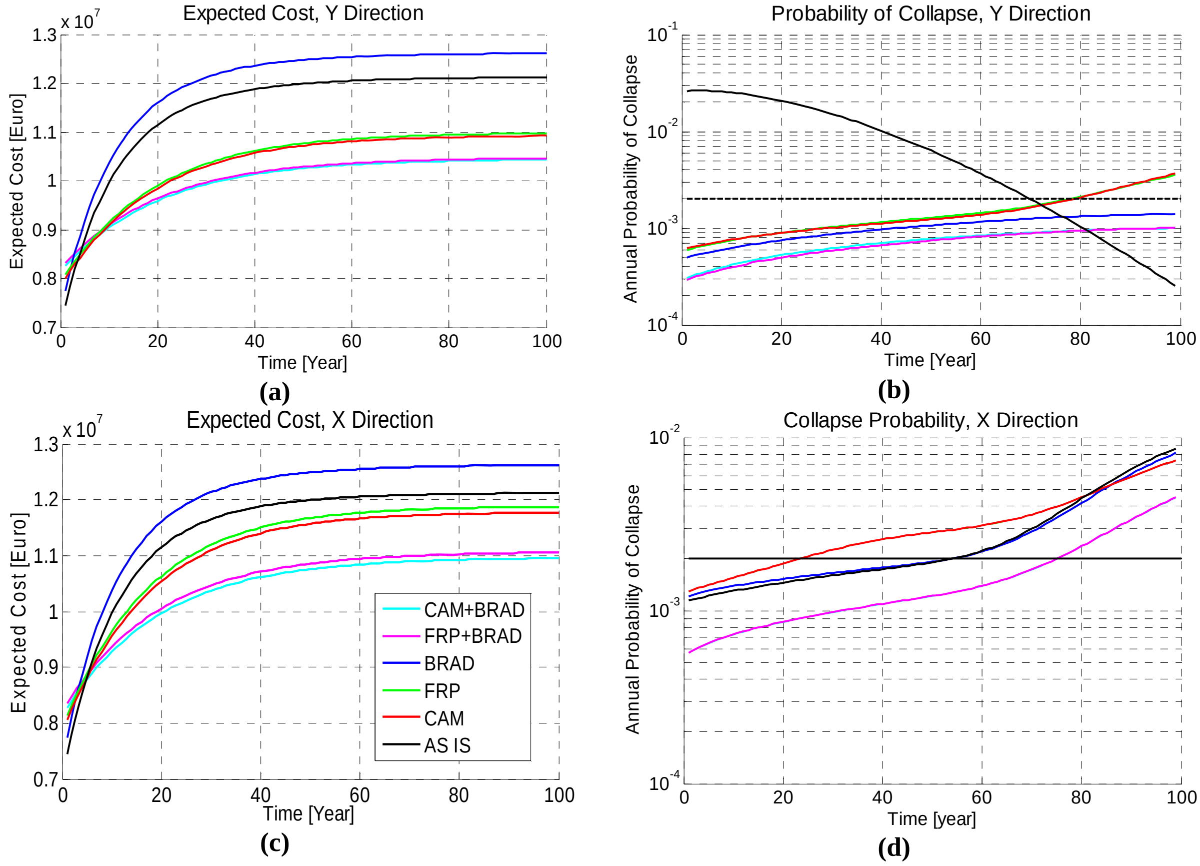

Figure 2. (a) The expected life cycle cost for the X direction; (b) The collapse probability in one year in the X direction; (c) the expected life cycle cost for the Y direction; (d) The collapse probability in one year in teh Y direction.

Figure 2 demonstrates the resulting expected life cycle cost and the annual rate of collapse for the 6 retrofit decisions considered for the two loading direction. It can be observed that the options CAM+BRAD has the lowest life cycle costs and also render the structure more reliable for the collapse limit state. Also the option FRP+BRAD renders very similar costs and structural performance. In order to take into account the structural reliability constraints, the annual collapse limit state probabilities are plotted against the 2% exceedance probability in 50 years (0.002) limit. As it can be observed, the structure is more vulnerable in the X direction where there have been no frames originally. In terms of cost criteria, although the CAM+BRAD and BRAD+FRP options are most expensive, it can be seen that after around 10 years, their corresponding expected costs fall beneath the other retrofit options.

4. CONCLUSIONS

A probability-based methodology is presented for calculating the time-dependent probabilities of exceeding discrete limit states for a structure subjected to earthquake events during the structural life-time. This methodology takes into account the history of the previous events taking place, the repair time needed to repristine the structure and eventual cumulative damage caused in the structure. These time-dependent limit state probabilities are then used to calculate the expected life-cycle cost taking into account the total initial construction costs, down time, repair/replacement costs, end of life recycling cost and the regular maintenance costs and the discount rate. This methodology is used as a decision-making tool for retrofit design of an existing RC frame structure. The expected life cycle is calculated for the existing structure (as is) and 5 different retrofit strategies. It is demonstrated that reinforcing using steel angles and steel ribbons and using buckling-restrained axial dampers in the perimeter frame (CAM+BRAD) is the option that leads to the least expected life cycle cost (although it is among the most expensive options in terms of installation costs) and manages to respect the structural reliability constraints for the collapse limit state.

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