SEISMIC DESIGN: BENEFIT/COST FOR OVERALL SERVICE TIME VERSUS PER UNIT SERVICE TIME

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ABSTRACT :

The maximum expected monetary benefit or minimum expected cost rule are often adopted in assessing the optimal seismic design levels for structures under infrequent large earthquakes. Use of the obtained optimal design levels for structural designs ensures the most economic use of the resources and the maximum profit. It is noted that in such an assessment the monetary benefit or cost functions are frequently established by considering the overall benefit or lifecycle cost at present value for a given structural design life. Yet, the selection of the structural design life is somewhat arbitrary, and in many cases one is interested in maximizing the structural service time or the benefit per dollar spent. The consideration of the benefit (or cost) at present value per lifecycle or per unit service time may lead to different optimal design levels for a given planning time horizon. The difference between the obtained optimal design levels by considering the former and the latter are investigated. Analyses are carried out for structures located in Mexico City under seismic loading with earthquake occurrence modeled as a Poisson or non-Poisson process. The implication of the results for the codified designs is discussed.

KEYWORDS: Optimal design, seismic hazard, seismic risk, decision rule.

1. INTRODUCTION

The selection of optimal seismic design levels for buildings is a decision-making problem under uncertainty and has been dealt with by many researchers including Rosenblueth (1976, 1987), Liu et al. (1976), Rosenblueth and Jara (1991), Rackwitz (2000), Kang and Wen (2000), Ellingwood and Wen (2005), Goda and Hong (2006). In all these studies, the selection of the seismic design level is viewed as safety and economic issue that balances benefit and cost for the structural life cycle. For selecting the optimal design level, most studies consider the maximum expected benefit or minimum expected cost rule, although it does not take into account higher order statistical moments of the benefit or the cost and, decision makers' risk attitudes. To incorporate the risk attitude, Rosenblueth (1987) took the view that utility is a logical scale measure of the intensity of happiness, and applied it to select seismic design level using a specific utility function. Goda and Hong (2006) employed the stochastic dominance criteria coping with classes of utility functions which reflect risk attitudes for selecting the optimal seismic design level. Their analysis results showed that the optimal seismic design level for a risk-neutral decision maker coincides with that obtained based on the maximum or minimum expected value (MEV) rule and that this optimal design level defines the upper and lower bounds on the efficient or optimal seismic designs for risk-seeking and risk-averse decision makers, respectively.

In all the above-mentioned studies, the (functions of the) total benefit or the total cost per structural design life are employed. However, the selection of the design life is somehow arbitrary and periods of 30, 50 and 75 years have been adopted for calibrating building and bridge design codes. Furthermore, in many cases one is interested in maximizing the benefit per dollar spent or minimizing the cost spent on the structure per unit service time. Therefore, rather than applying the MEV rule considering the total benefit or cost per design life or planning time horizon, one could consider the selection of the optimal seismic design level by using the MEV rule considering the cost per unit service time. As will be seen, the use of the former and the latter can lead to the different optimal design levels. The use of Poisson process for earthquake occurrence (Cornell 1968; McGuire 2004) is usually considered for seismic zones that do not have very clear identified faults, while use of renewal process could be considered adequate for subduction earthquakes such as characteristic earthquakes along Mexican subduction region (Singh et al. 1983, Hong and Rosenblueth 1988).

In this study, we adopt the MEV rule for selecting the optimal seismic design level. Both linear and nonlinear structural responses under seismic excitations are used to define the partial damage and incipient collapse, and to evaluate the expected damage cost and/or the expected annual average cost (at present value). For the analysis, both Poissonian and non-Poissonian earthquake occurrence modeling are considered. One of the main objectives of the study is to compare the differences between the optimal seismic design levels if the expected benefit (or cost) per life cycle or per unit service time is employed in the MEV rule. For the numerical analysis, structures located in Mexico City are considered and, a set of newly developed attenuation relations, and displacement ductility demand based on Mexican ground motion records are employed.

2. OBJECTIVE FUNCTIONS

Consider that the benefit at present value derived from the service and existence of an engineered structure up to the time t is denoted by $B(A,t)$, where A is a set of design parameters. The initial capital investment to build such structure is denoted by $C_0(A)$. If it is damaged or collapsed due to a large earthquake at a time there would be a corresponding damage cost at the present value, that represents structural and nonstructural damage cost, cost of lost life and limb and, cost of demolition and removal. Consider that the structure is immediately repaired or reconstructed upon damage or collapse without modifying the design and construction rules (i.e., systematic reconstruction after failure), and that the total damage cost and, the repair and replacement cost is denoted by $C_{DT}(A,t)$ for service until the end of a planning time horizon t. The optimal design dictated by the MEV rule is obtained by maximizing the following objective function $O(A,t)$ (Rosenblueth 1976),

$$O(A,t) = B(A,t) - C_0(A) - C_{DT}(A,t), \tag{2.1}$$

The value of $O(A,t)$ at the optimum must be positive for the structure to be viewed as of benefit.
In the following it is considered that the possible failure at the completion of the structure can be ignored. If there are n seismic source zones that affect the structure, and the earthquakes occur randomly in time τ_{ij}, $i = 1,\ldots,N_j(t)$, $j = 1,\ldots,n$, where $N_j(t)$ denotes the total number of earthquakes originated from the j-th source zone in the time interval 0 to t, $O(A,t)$ shown in Eqn. (2.1) can be written as,

$$O(A,t) = B(A,t) - C_0(A) - \sum_{j=1}^{n} \sum_{i=1}^{N_j(t)} \left(C_D(A|x_{ij}) + C_R(A|x_{ij}) \right) e^{-\gamma\tau_{ij}}, \tag{2.2}$$

where $C_D(A|x_{ij})$ and $C_R(A|x_{ij})$ represent the damage cost and repair/reconstruction cost given that the damage state induced by the earthquake occurred at τ_{ij} is x_{ij}, and γ is a discount rate adjusted for inflation which is often set to 5%. If the annual average benefit is of interest, the objective function presented in Eqns. (2.1) and (2.2), is replaced by $O_a(A,t)$, which is defined by,

$$O_a(A,t) = O(A,t)/t. \tag{2.3}$$

It is noteworthy that only under very special circumstances (i.e., the benefit derived from the existence of the structure per unit service time is a constant, b, the seismicity can be described by a single statistically homogeneous seismic source zone, the earthquake occurrence can be modeled as a homogeneous Poisson process, and only the collapse damage state needs to be included in assessing $C_{DT}(A,t)$), a simple equation for $E(O(A,t))$, where $E(\ )$ represents the expectation, can be obtained as follows (Rosenblueth 1976),

$$E(O(A,t)) = \frac{b - \gamma C_0(A) - \lambda E(C_{DR}(A))}{\gamma} - \frac{b - \lambda E(C_{DR}(A))}{\gamma} e^{-\gamma t}. \tag{2.4}$$

However, in general, analytical expressions for $E(O(A,t))$ as well as for $E(O_a(A,t))$ are not available; their evaluation can be carried out using simulation techniques. Furthermore, $E(O_a(A,t))$ is not only a function of A but also a function of the planning time horizon t. This implies that the maximization of $E(O_a(A,t))$ for a given planning time horizon t, leads only to a suboptimum since it does not ensure that $E(O_a(A,t))$ is maximum for all possible t values. Therefore, it is expected that use of one or the other objective function shown in Eqns. (2.2) and (2.3) could lead to different optimal design level. In the former, the direct comparison of $E(O(A,t))$ values for two different t values is not meaningful, resulting in difficulty in selecting the optimal design for all possible t values. In the latter, t is treated as a decision parameter; one finds the optimal design level by maximizing $E(O_a(A,t))$ which emphases the expected benefit per dollar spent.

3. SEISMICITY AND DESIGN CONSIDERATIONS

Evaluation of $E(O(A,t))$ and $E(O_a(A,t))$ requires information on the probabilistic characterization of the seismic hazard; on an adopted seismic design level and; on the cost of initial construction and, cost of damage and collapse. The characterization of the seismic hazard is based on the earthquake occurrence modeling, magnitude-recurrence relation, attenuation relations and seismic source zones. The most often employed methodology for assessing the seismic hazard is the one proposed by Cornell (1968).
To characterize the seismic hazard for Mexico City, we consider that the seismic source zones given by Ordaz and Reyes (1999) for near Mexico's Pacific Coast shown in Figure 1 are sufficient accurate, and that other source zones can be ignored. We note that the source zones shown in Figure 1 are separated into two groups, the first one for small and moderate earthquakes with moment magnitude, M_W, less than 7 and the second one for earthquakes with magnitude greater than 7. For the former, the moment magnitude-recurrence relation $\lambda(M_W)$ is defined by,

$$\lambda(M_W) = \lambda_0 \frac{\exp(-\beta M_W) - \exp(-\beta M_U)}{\exp(-\beta M_L) - \exp(-\beta M_U)}, \tag{3.1}$$

for $M_W < 7$, while for the latter,

$$\lambda(M_W) = \lambda(7)\left(1 - \Phi\left((M_W - m_M)/s_M\right)\right), \tag{3.2}$$

for $M_W > 7$, where $\Phi(\ )$ denotes the standard normal probability distribution function, $M_L = 4.5$, $M_U = 7$, $m_M = 7.5$, $s_M = 0.3$ and, λ_0, β, and $\lambda(7)$ for the considered source zones are given in Ordaz and Reyes (1999). Note that their study implies that the earthquake occurrence in each source zone can be modeled as Poisson process. The assumption of homogeneous Poisson occurrence could be relaxed for one or two zones for $M_W > 7$ which are associated with Michoacan segment and Guerrero Gap (Rosenblueth and Jara 1991).

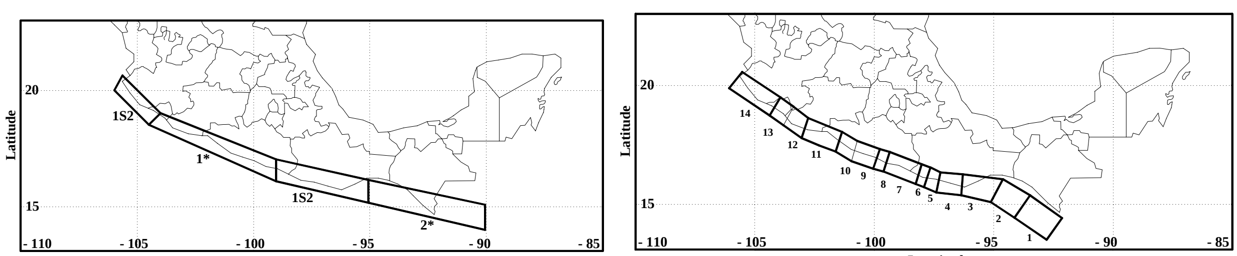

Figure 1 Seismic source zones suggested by Ordaz and Reyes (1999): a) small and moderate earthquakes with magnitude less than 7 and b) earthquakes with magnitude greater than 7.

For the ground motion measure for Mexico City, we note that the attenuation relation of the following form is considered by Reyes et al. (2002) for a firm site such as the recording site at the University City (i.e., CU site),

$$\ln\big(S_A(T_n,\xi)\big) = c_1 + c_2(M_W - 6) + c_3(M_W - 6)^2 + c_4\ln(R) + c_5 R + \varepsilon\,, \qquad (3.3)$$

where $S_A(T_n,\xi)$ denote the pseudo-spectral acceleration (PSA) (g), T_n is the natural vibration period in seconds, ξ is the damping ratio considered to be 5%, R (km) is the closest distance to the fault surface, c_i, i = 1,…,5, are regression coefficients, ε is the error term which is considered to be zero mean normal variate. c_i and ε depend on the value of T_n. If the geometric mean is employed for regression analysis, the evaluated standard deviation of ε, σ, needs to take into account the intra- and inter-event variability, and the random orientation variability (Boore *et al*., 1997). We prefer to use the attenuation relation developed based on the geometric mean rather than quadratic mean because it leads to the predicted PSA and the obtained σ values representing along a random orientation. By using the 40 record components considered by Reyes et al. (2002) for the same CU site, which are from events with $M_W > 6.0$, occurred near Mexico's Pacific Coast from 1965 to 1995, and using Joyner-Boore algorithm, the regression coefficients and σ are calculated, and illustrated in Table 3.1 for a few T_n values.

Table 3.1 Model coefficients for Eqn. (3.3) considering Mexican Subduction earthquake records at the CU site.

T_n(s)	c_1	c_2	c_3	c_4	c_5	σ
0.0	6.108	0.783	0.0666	-0.5	-0.00624	0.312
0.5	6.953	0.987	0.004	-0.5	-0.00725	0.332
1.0	6.634	1.113	-0.001	-0.5	-0.00617	0.327
3.0	5.345	1.816	-0.264	-0.5	-0.0052	0.449

Based on the above, the obtained uniform hazard spectra (UHS) through seismic hazard analysis using simulation-based analysis procedure (Hong et al. 2006) for the CU site are shown in Figure 2 for three selected return periods. For the analysis, it is considered that Eqn. (3.3) is applicable for M_W less than 8.1, and for earthquakes with magnitude greater than 8.1, their magnitude is set equal to 8.1 (Reyes, personal communication 2007). The UHS are compared with the design spectrum for hill zone, recommended by Mexico Federal District Code (MFDC) (DDF 2004). This comparison indicates that the recommended design spectrum is not probability level consistent.

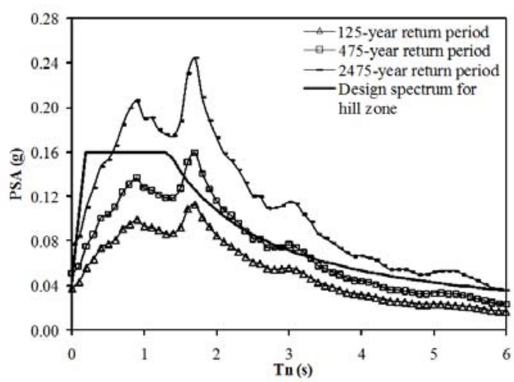

Figure 2 Uniform hazard spectra for the recording site at CU site for different return periods.

To investigate the inelastic demand bilinear hysteretical SDOF system, we use the above-mentioned set of records, and concluded that the empirical expression given in Hong and Hong (2007),

$$m_\mu = \exp\left((-\alpha_1 \ln\phi)^\beta\right), \qquad (3.4)$$

for the expected seismic displacement ductility demand, μ, m_μ, is equally applicable for the considered records. In this equation, ϕ is the normalized yield strength, and the obtained α_1 and β are model parameters shown in

Table 3.2 that differ from those for California records (Hong and Hong 2007).

Table 3.2 Parameters for Eqn. (3.4) an elastic-perfect-plastic hysteretical SDOF system.

$\alpha_1 = a_1 \exp(a_2 / T_n^{a_3})$			$\beta = \begin{cases} b_1 - b_2 \ln(T_n/0.3)/\ln(0.1/0.3) & 0.05 \le T_n < 0.3 \\ b_1 + (b_3 - b_1)\ln(T_n/0.3)/\ln(15/0.3) & 0.3 \le T_n < 5 \end{cases}$	
Parameter	$0.05 < T_n < 0.2$	$0.2 < T_n < 5$	Parameter	$0.05 < T_n < 5$
a_1	0.17	0.17	b_1	1.29
a_2	2.10	1.47	b_2	0.55
a_3	0.25	-0.19	b_3	1.05

A distribution fitting was carried out. The results show that for T_n equal to 1.0 s, samples of μ can be modeled as a Frechet variate with the coefficient of variation of about 0.35.

Note that the MFDC recommends a vibration period dependent factor *Q'* to reduce the seismic design base shear force. By using this factor, for a structure modeled as a bilinear hysteretical single-degree-of-freedom (SDOF) system whose design is governed by strength criterion and the MFDC (expect the elastic seismic design base shear coefficient), the normalized yield strength ϕ can be expressed as,

$$\phi = \min\left(\left(S_{AEf} / S(T_n)\right) \times \left(R_n / Q'\right), 1\right), \tag{3.5}$$

where $S(T_n)$ (g) represents the PSA of a seismic event. In writing this equation, it is considered that the ratio of the yield capacity of the designed structure to the minimum required design base shear force can be represented by R_n, and the elastic seismic design level (in terms of PSA) recommended in the code is replaced by S_{AEf}. Note that the design base shear coefficient C_S equals $(S_{AEf}/W)/Q'$, where W represents the weight of the structure.

4. COST INFORMATION, ANALYSIS PROCEDURE AND NUMERICAL RESULTS

The initial construction cost for structures to be located in Mexico City has been discussed by several authors including Rosenblueth and Jara (1991). Based their study, the initial cost function of the structural components $C_{0S}(C_s)$ can be expressed as,

$$C_{0S}(C_s) = C_{00} \times \max\left(1, 1 + c_c (C_s - a_c)^{b_c}\right), \tag{4.1}$$

where C_{00} represents the cost if it were not designed against earthquakes, a_c, b_c and c_c are model parameters taking the values 0.05, 1.1 and 1.4 respectively, and C_s is the seismic design base shear coefficient. This initial cost function is adopted in the present study. By including the cost of the nonstructural components, the initial total cost of the structure $C_0(C_s)$ could be defined as,

$$C_0(C_s) = C_{0S}(C_s) + C_{0S}\left(C_{s,ref}\right)/\kappa, \tag{4.2}$$

where $C_{s,ref}$ is a reference seismic design base shear coefficient and κ represents the ratio of the cost of the structural to the nonstructural components. The cost due to damage $C_D(\delta)$ which considers loss of contents, relocation, rental and income; the cost due to injury and fatality $C_F(\delta)$ and the cost due to repair/reconstruction $C_R(\delta, C_s)$ can be expressed as fractions or ratios of their values corresponding to collapse state (i.e., for δ=1), where the damage factor δ equals max(min((μ-1)/((μ_C-1),1),0), μ and μ_C denote the seismic displacement ductility demand and the ductility capacity of the structure, respectively.

For the above mentioned cost, we adopt simple to use functions developed by Goda (2007) using information documented in HAZUS (FEMA/NIBS 2003) (and references thereafter) for a variety of structures. For concrete and commercial use category structures, this leads to,

$$C_D(\delta) = C_{00}\left(\alpha_{BC}\delta^{0.64} + \alpha_{BI}\delta^{0.62} + \alpha_F \delta^{9.9}\right), \text{ and } C_R(\delta, C_s) = C_0(C_s)\delta^{0.77}, \tag{4.3}$$

where α_{BC}, α_{BI} and α_F are coefficients relating the damage cost with contents-related loss, business-interruption-related loss and fatality, respectively.

Eqns. (4.1) to (4.3) can be used in Eqns. (2.2) and (2.3) in establishing the objective functions, where the damage state is replaced by the damage factor δ. In doing so, it is considered that the only design parameter A that needs to be considered is S_{AEf} (i.e., A represents S_{AEf}) which is directly related to C_s. For the identification of the optimal seismic design level, the simulation procedure (Goda and Hong 2006) is employed to evaluate $E(O(A,t))$ and $E(O_a(A,t))$. Basically the procedure consists of: 1) carrying out probabilistic seismic hazard analysis; 2) selecting fractile of the PSA, S_{AEf} for seismic design level; 3) sampling $O(A,t)$ and $O_a(A,t)$ and estimating their statistics for the considered structure whose probabilistic characteristics are given. Results of Step 1) for the CU site are used to develop the UHS as already shown in Figure 2. Steps 2) and 3) are repeated for a range of considered design levels to identify the optimal design level. It must be emphasized that for the numerical results presented in the following the designed structure is approximated by an elasto-perfectly-plastic hysteretical SDOF system with damping ratio of 5% and displacement ductility capacity μ_C, and the (elastic) seismic design level is based on S_{AEf}. The estimation of ϕ value, which is needed for evaluating the mean of the ductility demand μ (see Eqn. (3.4)) and the damage factor δ, is to be carried out using Eqn. (3.5).

For the numerical analysis, it is considered that there is need for a ten story reinforced concrete structure and that the benefit derived from the structure is always higher than the expected total cost. Therefore, one must select the value of b such that the expected values of $O(A,t)$ and $O_a(A,t)$ are positive at least when the design is near optimum. The information on the cost and structure for the considered numerical analysis is summarized in Table 4.1. Note that μ_c is assumed to be lognormally distributed with mean 2.5 and coefficient of variation (cov) of 0.3, these values are in agreement with those used by Esteva and Ruiz (1989) for a nine-story concrete structure located in Mexico City. R_n is treated as deterministic parameters with a value of 2.8 for the designed structure, which is within the values suggested by Alamilla and Esteva (2006). This deterministic treatment is justified since the impact of the coefficient of variation of R_n on the estimated structural reliability and damage cost statistics is usually small.

Table 4.1 Information employed for numerical analyses

Damage cost	$\gamma = 0.05$; $\kappa = 0.3$, $C_{s,ref}$ is based on 125-year return period value of PSA at T_n equal to 1.0 which equals 0.09 (g); α_{BC}, α_{BI} and α_F are set equal to 51.7, 163.9 and 540, respectively;
Structural design	Structure is for commercial use; Planning service period t is 30, 50 and 75 years; Structure is modeled as an elastic-perfectly-plastic SDOF system; T_n is 1.0 second; Damping ratio is 5%; μ_C is a lognormal variate with a mean of 2.5 and cov of 0.3; R_n is treated as a deterministic variable with value equal to 2.8. Q' equals 4.

For the analysis, it is considered that the earthquake occurrence in each source zone shown in Figure 2 can be modeled as Poisson process with parameters given in Ordaz and Reyes (1999), and that the optimal design is to be identified based on the objective function $O(A,t)$. If the planning period t is considered to be equal to 50 years, the obtained expected value of $(O(A,t)\text{-}B(A,t))/C_{00}$ is shown in Figure 3a for a range of seismic design levels. The figure shows that the optimal design level equals 0.177 (g), which represents the 1750-year return period value of the PSA. This optimal design level under the above consideration is independent of the value of b since $B(A,t)$ is considered to be independent of design level. The identified optimal seismic design level is higher than the one suggested in the MDFC (DDF 2004), which corresponds to a return period of about 1000 years (see Figure 2).

To investigate the impact of the service period t on the selected optimal seismic design level, we consider t equal to 30 years and 75 years. The obtained expected value of $O(A,t)$, $E(O(A,t))$, are shown in Figure 3b and compared with those for t = 50 years for a range of seismic design levels. For the plotting, a value of b (normalized with respect to C_{00}) equal to 150 is considered. Use of this value is to ensure that the expected values of $O(A,t)$ for the optimal design level for all considered t values are positive. It must be emphasized that as mentioned earlier the direct comparison of $E(O(A,t))$ for different curves shown in the figure is meaningless, since each curve corresponds to a different service period t. The figure shows that the optimal seismic design levels are 0.167 (*g*) (representing 1417-year return period value) for *t* equal to 30 years and 0.193 (*g*) (representing 2650-year return period value) for *t* equal to 75 years. In other words, the optimal seismic design level is affected by the arbitrarily selected planning period. The results do indicate that one should increase the seismic design level for the considered structure to achieve economic efficiency.

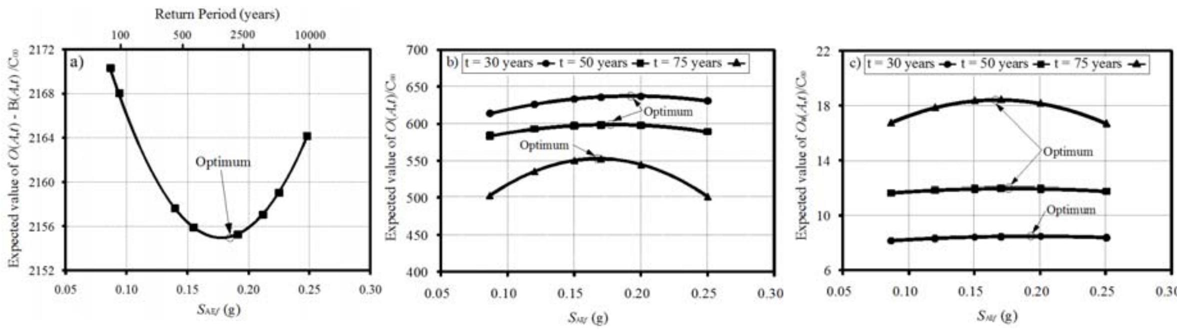

Figure 3 Expected value of the objective functions: a) Expected value of $O(A,t)$-$B(A,t)$ for t equal to 50 years normalized with respect to C_{00}, b) Expected value of $O(A,t)/C_{00}$; and c) Expected value of $O_a(A,t)/C_{00}$ for t equal to 30, 50 and 75 years.

To investigate the differences between the optimal seismic design levels obtained based on $E(O(A,t))$ and $E(O_a(A,t))$, we repeat the above analysis but using $E(O_a(A,t))$ instead of $E(O(A,t))$ to identify the optimal seismic design level. The obtained values of $E(O_a(A,t))$ as well as the identified suboptimal design levels are shown in Figure 3c. Note that in this case comparison of different curves can be carried out, and that the relative position of each curve has been changed as compared to that show in Figure 3b. However, the suboptimal design levels are identical to the optimal design levels for specified *t* values. Figure 3c along suggests that if only *t* equals 30, 50 and 75 years can be considered, the optimal seismic design level is 0.167 (g) since it provides the maximum benefit per unit time. A plot of $E(O_a(A,t))$ for these designs versus their corresponding design levels indicate that the optimal design level for all possible *t* values is likely to be less than 0.17 (g). Furthermore, the trend based on such a plot is significantly affected by the selected *b* value (i.e., benefit per unit time). Therefore, a further parametric study is warranted for a definite recommendation.

Also, preliminary parametric analyses for the considered numerical example by considering non-Poisson earthquake occurrence model for Michoacan segment and Guerrero Gap were carried out, the obtained optimal seismic design levels do not differ significantly from those mentioned previously.

4. CONCLUSIONS

The maximization of expected benefit rule is used in assessing the optimal seismic design level. For the assessment use of the overall benefit as well as the annual average benefit is considered. It is shown that, in general, the consideration of the former leads to different optimal design levels for different considered service periods, while the latter at least can be used to identify the optimal design level among the obtained suboptimal design levels. Therefore, the latter can be used at least to discriminate among the obtained suboptimal design levels for a range of possible service periods. More specific observations that can be drawn from the obtained numerical results include:

1) The current seismic design level recommended in the Mexico Federal District Code (MFDC) (DDF 2004) is not probability level consistent for different fundamental natural vibration period, and the shape of the design spectrum does not follow closely to that of the uniform hazard spectra. Therefore, a review and revision to the design spectrum may be warranted for reliability consistent design; and
2) The seismic design level recommended in the MFDC deviates from the optimal seismic design level. It is clear that a probabilistic cost/benefit based calibration of the seismic design spectrum is valuable by considering the hill zone as well as other zones in Mexico City, a range of natural vibration periods, and different possible cost model parameters.

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