Post-Earthquake Reliability Assessment of Structures subjected to Aftershocks: An Application to L'Aquila Earthquake, 2009

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ABSTRACT: The post-earthquake assessment of existing structures can be further complicated by the progressive damage induced by the occurrence of a sequence of after-shocks. This work presents a simple methodology for the calculation of the probability of exceeding a certain limit state in a given interval of time. The time-decaying mean daily rate of occurrence of significant after-shock events is modeled by employing a site-specific after-shock model for the L'Aquila 2009 after-shock sequence (central Italy). The occurrence of after-shock events is modeled using a non-homogenous Poisson model. An equivalent single-degree of freedom structure with cyclic stiffness degradation is used in order to evaluate the progressive damage caused by a sequence of after-shock events. Given the time history of the main-shock and the residual damage caused by it, the probability of exceeding a set of discrete limit states in a given interval of time is calculated. Of particular importance is the time-variant probability of exceeding the limit state in a 24-hour (a day) interval of time which can be used as a proxy for the life-safety considerations regarding the re-occupancy of the structure. The method presented herein can also be used in an adaptive manner, progressively conditioned on the time-histories of after-shock events following the main-shock and on the corresponding residual damage caused by them.

1 INTRODUCTION

The inspection and management of civil structures after the occurrence of a severe earthquake event is subjected to considerable challenges. The postearthquake deterioration as a result of the sequence of after-shocks may obstacle significantly eventual inspections and/or re-occupancy of these structures. In fact, a significant main-shock is often followed by a number of after-shock events (usually smaller in moment magnitude) which take place in a limited area (i.e., the after-shock zone) around the epicenter of the main event. This sequence of after-shock events can last in some cases for 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 after-shocks (in some cases up to 6000) and 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-occupancy of the building (for rescue or for inspection) while the mean daily after-shock rate is quite considerable.

The present study presents a procedure for calculating the time-dependent probability of exceeding the limit states corresponding to various discrete performance objectives. A simple cyclic stiffness deteriorating single degree of freedom (SDOF) model of the structure is used in order to study the damages induced as a result of a sequence of after-shocks. The timedecaying model parameters are estimated for the L'Aquila 2009 after-shock sequence using a Bayesian updating framework based on the Italian generic sequence as prior information. As a criteria for assessment of the decisions regarding re-entrance for inspections purposes, the (time-dependent) probability of exceeding the limit state of life-safety in a 24-hour interval is compared to an acceptable threshold. The less severe limit states of severe damage and onset of damage can be used in a similar manner in order to make decisions regarding the re-occupancy and serviceability of the structure.

2 METHODOLOGY

The objective of this methodology is to calculate the time-dependent probability of exceeding various discrete limit states in a given interval of time for a given structure subjected to a sequence of aftershocks. The methodology presented herein for the evaluation of the limit state probability in a given time interval can be used for decision making between different viable actions such as, re-entry/evacuation, re-occupancy/shutting down. This methodology starts from the state of the structure after it is hit by a mainshock. Therefore, given that the main shock waveforms are available, the damages undergone by the structural model can be evaluated. The clustering of earthquakes usually occurs near the location of the main-shock also referred to as the after-shock zone. Therefore, it is assumed that for the sequence of earthquakes including the main-shock and after-shock events, each point within the after-shock zone is equally likely to be the epicenter of an earthquake event. The aftershock clusters should be eventually classified based on their generating source, should they belong to different fault structures, as in the case of L'Aquila Earthquake. An important characteristic of the sequence of after-shocks following the mainshock is that the rate of after-shocks dies off quickly with time elapsed since the main-shock. The timedecaying parameters of the aftershock sequence are estimated by applying a Bayesian updating framework to the L'Aquila 2009 sequence based on the Italian generic after-shock model as prior information. The methodology presented is of an adaptive nature; that is, with occurrence of more after-shock events, the state of the structure can be updated by evaluating the damages undergone by the structural model subjected to the sequence of main-shock and after-shocks.

2.1 Bayesian after-shock sequence parameter estimation

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The aftershock sequence is modeled using a nonhomogenous Poisson process in which the timedecaying rate of the occurrence of aftershocks is modeled by a modified Omori law:

$$N(t) = \frac{K}{(t+c)^p}$$
(1)

where N(t) is the total (daily) number of after-shock events at time t elapsed after the main-shock and K, pand c are constants. The magnitude distribution for the aftershocks is modeled using the Gutenberg-Richter law:

$$N(m) = A \cdot 10^{-bm} \tag{1}$$

where N(m) is the number of events with magnitude greater than or equal to *m* and *A* and *b* are constants. Therefore, the mean daily rate of aftershocks with magnitude equal to or greater than m and equal to or smaller than the main-shock magnitude M_m at time t elapsed after the main-shock is equal to:

$$\lambda(t,m) = \frac{10^{a+b(M_m - m)}}{(t+c)^p}$$
(2)

Finally, the mean daily rate of aftershock with magnitude equal to m following a mainshock of M_m can be calculated, by differentiating Equation 2 with respect to magnitude, as (Yeo & Cornell, 2006):

$$\mu(t) = b \log(10) \frac{10^{a+b(M_m-m)}}{(t+c)^p}$$
(3)

The uncertain parameters to be estimated are a, c and p where a measures the likelihood of aftershocks occurring and p is measure of the after-shock sequence decay with time and c is a time off-set parameter. Therefore, the posterior joint probability distribution for uncertain parameters a, c and p can be calculated by implementing the Bayes formula:

$$p(a, p, c \mid data) = \frac{p(data \mid a, p, c)p(a, p, c)}{\sum_{a} \sum_{p} \sum_{c} p(data \mid a, p, c)p(a, p, c)}$$
(4)

where p(data|a,p,c) is the likelihood function for the aftershock data observed and p(a,p,c) is the joint prior probability distribution for parameters a and p. The parameter b decides the magnitude distribution of the after-shock events and is estimated separately (later in this section) using the Gutenberg-Richter magnitude distribution.

2.2 Deriving the likelihood function given a, p and c

The probability that at least one after-shock of magnitude equal to m occurs in time interval [0,T] elapsed after the occurrence of the main-shock can be calculated from a non-homogeneous Poisson process (Reasenberg & Jones 1989, 1994):

$$P(AS \mid T, m) = 1 - e^{\left[-\int_0^T \mu(t)dt\right]}$$
(5)

The cumulative distribution function (CDF) for the inter-arrival time IAT between the aftershock events occurring at times $t_i : i = 1, ..., N$ can be written as:

$$P(IAT \le (t_i - t_{i-1}) \mid t_{i-1}, m) = 1 - e^{\left[-\int_{t_{i-1}}^{t_i} \mu(t)dt\right]}$$
(6)

where N is the total number of aftershocks occurring in time T. Replacing $\mu(t)$ from Equation 3 in Equation 6, a closed-form solution for the probability $P(IAT \le (t_i - t_{i-1}) | t_{i-1}, m)$ can be obtained as following:

$$P(IAT \le (t_i - t_{i-1}) \mid t_{i-1}, m) = 1 - e^{-b\log(10)\frac{10^{a+b(M_m - m)}}{1 - p} \left[\frac{1}{(t_i + c)^{p-1}} - \frac{1}{(t_{i-1} + c)^{p-1}}\right]}$$
(7)

The probability density for the inter-arrival time between aftershock events with magnitude m can be calculated by calculating the derivative of Equation 7 with respect to time:

$$p((t_{i}-t_{i-1}) \mid t_{i-1},m) = \mu(t_{i})e^{-\log(10)\frac{10^{a+b(M_{m}-m)}}{1-p}\left[\frac{1}{(t_{i}+c)^{p-1}} - \frac{1}{(t_{i-1}+c)^{p-1}}\right]}(8)$$

Hence, the likelihood function for a sequence of N aftershock events with magnitude equal to $m_i : i = 1, ..., N$ occurring at times $t_i : i = 1, ..., N$ can be obtained.

2.3 The prior probability distributions for parameters a, p and c

It is assumed that the prior probability distributions for a, p and c are independent. Regarding the choice of prior, the probability distributions recommended by Lolli and Gasperini, 2003 the generic Italian aftershock model, reported in Table 1, are used:

Table 1: A priori predictions of Italian aftershock occurrence from 1981 to 1996

Parameter	Mean ValueStandard		Туре
		Deviation	
р	0.93	0.21	Logormal
Log10(c)	-1.53	0.54	Normal
b	0.96	0.18	Lognormal
а	-1.66	0.72	Lognormal

2.4 Estimating the b paremeter

The parameter b is estimated separately from the Gutenberg-Richter formula for the distribution of magnitude. The probability of having an aftershock with magnitude equal to m which is smaller than or equal to the main-shock magnitude M_m and larger than a lower limit magnitude M_l is equal to:

$$p(m \mid b) = \frac{b \log_{10} e^{-b \log_{10} m}}{e^{-b \log_{10} M_1} - e^{-b \log_{10} M_m}}$$
(9)

Therefore, the posterior probability distribution for parameter b given the sequence of aftershock event with magnitude **Errore.** can be calculated using the Bayes formula:

$$p(b \mid data) = \frac{p(data \mid b) p(b)}{\sum_{b} p(data \mid b) p(b)}$$
(10)

where p(data|b) is the likelihood function and can be calculated using Equation 10:

$$p(data | b) = \prod_{i=1}^{N} p(m_i | b)$$
 (11)

As the prior distribution p(b), the ignorance prior is chosen:

$$p(b) = \frac{const.}{b} \tag{12}$$

The lower cut-off magnitude for estimation of the b parameter is chosen by establishing the completeness criteria in the database. That is, the b-value is estimated by varying the lower cut-off magnitude. It can be observed that the estimated b-value increases with increasing cut-off magnitude up to around $M_l=2.5$; for magnitude values larger than that, it was estimated to be roughly constant.

2.5 The probabilistic seismic after-shock hazard

The probability that the structural acceleration at the fundamental period of the structure S_a exceeds a given level *x* given that a significant after-shock event with a source-to-site distance *R* has taken place denoted by $P(S_a > x | as)$ can be calculated as:

$$P(S_a > x \mid as) = \int_{M_l}^{M_m} \int_{x_{\min}}^{x_{\max}} \int_{y_{\min}}^{y_{\max}} P(S_a > x \mid m, R) p(m) dm dx dy$$
(13)

where *xmin*, *xmax*, *ymin* and *ymax* are the minimum and maximum coordinates of the after-shock zone with respect to the site and $R^2 = x^2 + y^2$. It is assumed that every point inside the after-shock zone is equally likely to be the epicenter of an after-shock. *Mm* is the moment-magnitude for the main-shock event and *M1* is the lower-bound for the moment magnitude for the earthquake events of engineering interest. The term $P(S_a > x | m, R)$ can be calculated using the parameters of the ground motion prediction relation for the site and p(m) is the truncated Gutenberg-Richter probability density function for moment magnitude:

$$p(m) = \frac{\beta e^{-\beta m}}{e^{-\beta M_l} - e^{-\beta M_m}}$$
(14)

 β =blog10 where *b* is related to the seismicity of the site. The mean daily rate of exceeding a given spectral acceleration level can be calculated by multiplying Equation 14 by the average daily rate of occurrence of after-shock events:

$$H(S_a > x) = v(t) \cdot P(S_a > x | as)$$
(15)

where v(t) is the time-dependent average daily rate of occurrence of after-shocks after *t* days are elapsed from the main-shock.

2.6 Updating the hazard after the occurrence of the main-shock

After the occurrence of a main-shock, assuming that its wave-form is known, the probability of exceeding a given value of spectral acceleration can be updated using the Bayes formula taking into account the spectral acceleration at the fundamental period of the structure for the main-shock, $S_{a,ms}$:

$$p(S_{a,as} = x \mid S_{a,ms}, as) = \frac{p(S_{a,ms} \mid S_{a,as} = x, as)p(S_{a,as} = x \mid as)}{\sum_{x} p(S_{a,ms} \mid S_{a,as} = x, as)p(S_{a,as} = x \mid as)}$$
(16)

where $p(S_{a,as} = x | S_{a,ms}, as)$ denotes the probability density function (PDF) for the spectral acceleration of the after-shock given that the spectral acceleration of the main-shock is known, $p(S_{a,ms} | S_{a,as} = x, as)$ is the probability density function for main-shock given the after-shock spectral acceleration is known and $p(S_{a,ms} | S_{a,as} = x, as)$ is the PDF for after-shock spectral acceleration before having the extra information. Having calculated the updated PDF, the updated probability of exceeding a given after-shock spectral acceleration can be calculated using the following relationship:

$$P(S_a = x) = -\frac{P(S_a > x)}{dx}$$
(17)

3 THE ASSESSMENT OF TIME-DEPENDENT LIMIT STATE PROBABILITY

Let T_{max} denote a given interval of time elapsed after a main-shock has taken place, *N* the maximum number of after-shock 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:

$$P(LS;T_{\max}) = \sum_{i=1}^{N} P(LS \mid i) P(i;T_{\max})$$
(18)

Where P(LS|i) is the probability of exceeding the limit state given that exactly *i* after-shocks take place in time T_{max} and P(i;T_{max}) is the probability that exactly i after-shock events take place in time T_{max}. It is assumed that the after-shock hazard for the site of the structure is expressed by a non-homogenous *Poisson* probability distribution with the time-decaying rate denoted by v(*t*). The probability of having exactly *i* events in time T_{max} can be calculated as:

$$P(i;T_{\max}) = \frac{\left(\int_{0}^{T_{\max}} v(t)dt\right)^{l} e^{-\int_{0}^{l_{\max}} v(t)dt}}{i!}$$
(19)

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 after-shock events:

$$P(LS|i) = P(C_1 + \overline{C_1} C_2 + \dots + , + \overline{C_1 C_2 \cdots C_{i-1}} C_i|i)$$

$$(20)$$

where C_j , j = 1,...,i indicates the event of exceeding the limit state *LS* due to the *j*th event and 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 (i.e., damaged only by the main-shock) and that the event *j* hits the damaged structure:

$$P(C_{i}|i) = P(C_{i}I|i) + P(C_{i}D|i)$$
(21)

Equation 21 can be further expanded as follows:

$$P(C_j \mid i) = P(C_j \mid I, i)P(I \mid i) + \sum_{k=1}^{j-1} P(C_j \mid k, 1)P(k \mid i)$$
(22)

where $\{k:k=1,2,\dots,i-1\}$ indicates the number of times the structure has been damaged by an after-shock before reaching the target limit state, implying that the structure deteriorates with the occurrence of each event. The formulation in Equation 22 is based on the consideration that an event can hit a structure already damaged by one or more *previous* event(s). 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 structure is damaged *k* times before reaching *LS* is equal to:

$$P(k \mid i) = e^{-\int_0^\tau v(\tau)dt} \left(1 - e^{(-\int_0^\tau v(\tau)dt)}\right)^k$$
(23)

Assuming that the structure under repair is hit by another after-shock 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:

$$P(I \mid i) = e^{(-\int_{0}^{t} V(\tau)dt)}$$
(24)

Observing Equation 22, one can identify the sequence of the limit state probability terms, namely, $P(C_j | I, i)$ and $P(C_i | k, i)$ where $k=1, \dots, (j-1)$.

3.1 Estimation of limit state probabilities

In order to calculate the sequence of limit state probability terms $P(C_i | D_k, i)$ where $k=1, \dots, (j-1)$, the following procedure is applied. A selection of nearthquake records (consisted of main-shocks and after-shocks) is selected. In order to emulate the deterioration caused by the sequence of after-shocks, each ground motion is applied k times in sequence to the structural model. The maximum displacement response of the structure due to the sequence of k events denoted by Y(k) is related to the spectral acceleration at the fundamental period of the damaged structure, after being subjected k-1 times in sequence to the selected ground motion record using the linear least squares (in the logarithmic scale). That is, the median for maximum displacement is described by $\eta_{Y|S_a(T_k)}(k) = a \cdot S_a(T_k)^b$ and that the standard deviation (of the togarithm) of Y(k) given S_a is calculated as:

$$\sigma_{\ln Y(k)|S_a(T_k)} = \sqrt{\frac{\sum_{1}^{n} (\ln Y(k) - \ln a \cdot S_a(T_k)^b)^2}{n-2}}$$
(25)

where a and b are regression coefficients calculated as:

$$a = \frac{\sum \log Y_{i} \sum \log S_{a,i}^{2} - \sum \log S_{a,i} \sum \log Y_{i} \log S_{a,i}}{n \sum \log S_{a,i}^{2} - \left(\sum \log S_{a,i}\right)^{2}}$$

$$b = \frac{n \sum \log Y_{i} \sum \log S_{a,i} - \sum \log S_{a,i} \sum \log Y_{i}}{n \sum \log S_{a,i}^{2} - \left(\sum \log S_{a,i}\right)^{2}}$$
(26)

The limit state probability $P(C_j | D_k, i, S_a(T_k))$ can be calculated as:

$$P(C_{j} | D_{k}, i, S_{a}(T_{k})) = 1 - \Phi\left(\frac{\log Y_{C_{j}} - \log(\eta_{Y|S_{a}(T_{k})}(k))}{\sigma_{\ln Y(k)|S_{a}(T_{k})}}\right) \quad (27)$$

In order to calculate $P(C_j | D_k, i)$, the expression in Equation 27 needs to be integrated with the probability density function (pdf) for the spectral acceleration given that a significant after-shock has taken place, calculated by the differentiation of the complementary cumulative distribution function for spectral acceleration given a significant after-shock has taken place. Therefore:

$$P(C_{j} | D_{k}, i) = \int_{0}^{\infty} P(C_{j} | D_{k}, i, S_{a}(T_{k})) \cdot p(S_{a}(T_{k}) | as)$$
(28)

Where the hazard curve is calculated at the fundamental period of the damaged structure after it is being heat by k-1 ground motion records. 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.

3.2 The limit state probability in a 24-hour interval

In the previous section, it is explained how the probability of exceeding the limit state *LS* in a given interval of time can be calculated. However, it is of interest to calculate the probability of exceeding the limit state in a reference time interval(e.g., 24 hours). The probability of exceeding the limit state in the reference time interval $[T,T+\Delta T]$ can be calculated as:

$$P(LS;[T,T+\Delta T]) = P(LS;T+\Delta T) - P(LS;T)$$
(29)

Therefore, the probability of exceeding the limit state in one day can be calculated from Equation 29, by setting ΔT equal to one.

4 NUMERICAL EXAMPLE

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

4.1 Structural model

The case-study building is a generic five-story RC frame structure designed to resist seismic action. 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. More details can be found in a previous work by the authors (Asprone et al. 2010). In order to simplify the structural analyses, an equivalent degrading (SDOF) system is used as the structural model. In order to model the the non-linear characteristics of the equivalent SDOF system, a nonlinear static analysis on the case-study structure is The resulting pushover performed. curve is transformed into that of an equivalent SDOF system with $T_1=0.58sec$ and yield displacement equal to $d = 0.03^{4}$ calculated based on the first mode shape of the structure. Based on the resulting equivalent pushover curve, a non-linear degrading hysteresis model for the equivalent SDOF system is constructed.

4.2 The L'Aquila aftershock sequence

The Bayesian updating framework is used in order to calculate the parameters of the aftershock occurrence rate for the sequence of after-shocks following the L'Aquila earthquake of 6th April 2009 with moment magnitude equal to $M_m = 6.3$. In order to estimate the parameters of the L'Aquila sequence, the lower cut-off level for magnitude is set at M=3 which is above the completeness level as discussed before. The b is calculated as the maximum likelihood estimate (MLE) value for the posterior probability distribution updated given the L'Aquila sequence magnitude values. It can be seen the MLE for b value is equal to b=1.03. This is while the b value for the generic California after-shock sequence is calculated to be equal to b=0.91. The parameters a, p and c are estimated using the procedure described in previous section using a cut-off magnitude equal to M=3. The joint posterior probability distribution for a, p and c is also calculated.

4.3 Calculation of failure probabilities

In order to calculate the failure probabilities due to the sequence of after-shock events, a set of 50 ground motion records (consisting of main-shocks and after-shocks) are chosen. Each ground motion record is applied sequentially k times on the equivalent SDOF model with cyclic stiffness degrading behavior. The

probability of failure given that a sequence of k aftershocks has occurred is calculated following the procedure explained in previous section. For each sequence of k earthquakes, the maximum displacement response of the equivalent SDOF system is calculated. A linear least squares method is used to estimate the median and the standard deviation of maximum displacement as a function of spectral acceleration at the fundamental period of the damaged structure being subjected to k-1 ground motion records. The median and standard deviation of the maximum displacement of the k-times damaged structure are then used to calculate the structural fragility assuming that it is lognormal. The failure probability for the damaged structure can be calculated by integrating the structural fragility and the spectral acceleration hazard at a period close to the fundamental period of the damaged structure.

4.4 *The probability of failure in a 24-hour time interval*

The probability of exceeding the limit state of collapse in a day (24 hours) has been calculated from Equation 29 setting ΔT =1. The results are plotted in Figure 1 where they are compared against an acceptable mean daily collapse rate of 2×10⁻³/365, as a proxy for life safety considerations. This threshold value is on average equivalent to an acceptable mean annual rate of collapse equal to 2×10⁻³. This verification is done for ensuring life safety for the building occupants.



Figure 1: Probability of exceeding the collapse limit state in a day

It can be observed that the low-residual structure is immediately below the acceptable threshold for lifesafety limit state; whereas, the high-residual case does not verify the acceptable threshold up to around one week elapsed after the occurrence of the main-shock. After 7 days, due to the decreasing rate of occurrence of after-shocks, the structure verifies against the lifesafety limit state threshold. It should be noted that such a time-variant performance assessment can be potentially useful for evaluation of the re-occupancy risk for the structure after a certain amount of time has passed from the occurrence of the main shock. In fact, the necessary time elapsed after the occurrence of main-shock in order for the structure to verify the lifesafety limit state is calculated for a range of residual to collapse displacement capacity ratios. Figure 2 illustrates (solid line) the time required in order to verify the collapse limit state for different residual percentiles for the L'Aquila after-shock sequence. It can be observed that the structure immediately verifies the life-safety limit state when the residual damage is minimal; whereas, it might take more than a year before the structure verifies in cases where the residual damage is very significant.



Figure 2: Time elapsed after the occurrence of the main-shock in order to verify the life-safety requirements

The figure also illustrates the time required for the structure in order to verify the collapse limit state for the generic California after-shock sequence (the dashed line) and the generic Italian after-shock sequence (the dotted line). It can be observed that for a given level of residual damage in the structure, more time is required in order to verify the collapse limit state when the parameters of the generic after-shock sequence are considered instead of those of the L'Aquila sequence.

7

5 CONCLUSIONS

This paper presents a preliminary effort for quantification of the time-variant probability of exceeding various discrete limit states for a structure in an after-shock prone environment. A simple methodology is presented for calculating the probability of exceeding a limit state in a given interval of time elapsed after the occurrence of the main-shock event. This procedure employs an aftershock model based on the modified Omori law in order to model the time-decay in the mean daily rate of the occurrence of significant after-shocks. The seismic after-shock hazard at the site of the structure is calculated by setting the main-shock moment magnitude as the upper limit for magnitude and is updated using the Bayes formula given that the smallamplitude spectral acceleration of the main-shock at the fundamental period of the structure is known. The progressive damage caused by the sequence of aftershock events is modeled in the form of a suite of different ground motion recordings that are applied (repeatedly) to the simplified structural model that includes cyclic stiffness degradation. Conditioned on the occurrence of a given number of after-shocks, the fundamental period of the damaged structure and its residual and maximum displacement response are calculated. The statistics of the structural response to the suite of records can then be used to calculate the probability of exceeding the limit state capacity. It can be observed that the probability of exceeding the limit state capacity increases as a function of the number of significant after-shocks until it reaches a plateau and remains constant afterwards. Conditioned on the occurrence of a given main shock event, the of exceeding the limit states probability of serviceability, onset of damage, severe damage and collapse in a given interval of time are calculated. It can be observed that the limit state probabilities increase as a function of time although they seem to reach a constant threshold at the end of a year passed from the occurrence of the main-shock. In order to better observe this effect, the collapse limit state probability in a 24-hour period is calculated as the increment of the time-variant limit state probability in a given interval of time (measured in days). In fact, comparing the time-variant probability of collapse in a 24-hour period of time against an acceptable threshold, it can be observed that the strongly damaged structure could be occupied after a certain amount of days has elapsed after the occurrence of the main-shock while the lightly damaged structure could be occupied immediately. This type of verification can be useful for evaluation of re-occupancy risk for the structures located in a zone prone to after-shocks, based on the life-safety criterion. In fact, the necessary time elapsed after the main-shock for the structure to verify the life-safety requirements is calculated as a function of different values of residual to collapse displacement capacity ratio. It is observed that time needed to verify against the life-safety limit state increases exponentially as a function of the level of residual damage undergone after the main-shock. The methodology presented in this work is adaptive in the sense that the limit state probability evaluations can be updated in time as more after-shock events take place. The proposed methodology could be used for postearthquake decision-making between a set of viable actions such as, evacuation, shut-down, repair and reoccupancy.

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