Structural Safety 31 (2009) 443–447

Contents lists available at ScienceDirect

Structural Safety

journal homepage: www.elsevier.com/locate/strusafe

Equivalent constant rates for post-quake seismic decision making

Gee Liek Yeo *, C. Allin Cornell

Department of Civil and Environmental Engineering, Formerly Stanford University, Stanford, CA 94305, USA

ARTICLE INFO

Article history:
Received 17 April 2008
Received in revised form 11 November 2008
Accepted 24 November 2008
Available online 1 January 2009

Keywords:
Mainshocks
Aftershocks
Life-safety considerations
Equivalent constant rates
Building tagging

ABSTRACT

The objective of this paper is to develop a methodology to quantify the time-varying post-quake aftershock threat in a manner consistent with the development of conventional design ground motion levels, which are typically based on a constant probability of exceedance of a given ground motion intensity. We use the frequency of collapse of a mainshock-damaged building in the aftershock environment as a proxy for the life-safety risk faced by an arbitrary building occupant. We demonstrate that the time-varying aftershock threat can be transformed to an equivalent constant collapse rate for a mainshock-damaged building by considering the total expected number of aftershock collapse events in the time interval of interest and a 'social discount factor', typically taken to be from 3% to 5%. The ability to quantify the time-varying life-safety risk in the aftershock environment in a manner compatible with existing building code provisions will allow informed building evacuation and re-occupancy decisions to be made. While this concept has been introduced for the aftershock environment, it may also be a potential solution to a wider class of safety problems involving transient life threats.

1. Introduction

After an earthquake of large magnitude (referred to as the mainshock), many induced events or aftershocks will occur. The mean rate of aftershocks, which is mainshock magnitude, m_m, dependent, decreases with increasing elapsed time t from the occurrence of the mainshock. For a given mainshock magnitude, Reasenberg and Jones [1,2] study the generic California aftershock sequence using the mainshock magnitude, m_m, as the upper bound magnitude of the aftershocks. Using $a = -1.67, b = 0.91, p = 1.08$ and $c = 0.05$ from [1,2], the instantaneous daily rate density of aftershocks with magnitudes between m_l and m_m at time t following a mainshock of magnitude m_m, $\mu(t; m_m)$, can be calculated using Eq. (1). m_l represents the minimum aftershock magnitude of engineering interest. Aftershocks may pose a significant life-safety threat to the occupants of a building due to their increased frequency of occurrence after the mainshock, especially if the building has suffered significant structural damage due to the main quake itself.

$$\mu(t; m_m) = \frac{10^{a+b(m_m-m_l)} - 10^a}{(t+c)^p}. \quad (1)$$

As an example, assuming a mainshock with $m_m = 7.0$ and $m_l = 5.0$, we use Eq. (1) with the generic California aftershock sequence parameters to obtain the instantaneous daily aftershock rates with magnitudes between m_l and m_m as a function of elapsed time from the mainshock. The results are shown in Fig. 1. The instantaneous daily aftershock rates can be significant; for example, the total expected number of aftershocks starting 10 days after the mainshock into the future is equal to seven, i.e., the shaded area shown in Fig. 1.

Such dependence of the aftershock occurrence rates is not taken into account in studies related to the establishment of reliability-based seismic design criteria. For example, seismic hazard maps are typically developed for PGA and other spectral acceleration levels for 2%, 5% and 10% probabilities of exceedance in 50 years (see [3] and [4]). Also, design ground motion levels for new buildings may be set at a probability of exceedance of 0.0004/year (or 2% in 50 years) as an estimation of rare but possible ground motion level to ensure a low level of collapse probability (see [5]). Such probabilities of exceedance are implicitly assumed to be constant and extend indefinitely into the future.

Individual life-safety criteria are also normally stated in terms of annual frequency of fatalities. For example, Norwegian Petroleum Directorate [6] sets the maximum annual individual fatality risk per worker to 10^{-4} on new platforms for the offshore oil and gas industry. Again, such criteria implicitly assume time-independent fatality risks that are constant and extend indefinitely into the future. In order to compare or calibrate safety criteria in a time-varying environment (such as in the post-quake aftershock environment) to the standard time-invariant situation, some means must be identified to transform the former into the latter. Only then can we compare the threat in the aftershock environment with conventional acceptable limits on a consistent basis.

* Corresponding author. Tel.: +1 650 714 2862; fax: +1 212 713 8800.
E-mail address: geeliek@stanfordalumni.org (G.L. Yeo).

0167-4730/$ - see front matter
doi:10.1016/j.strusafe.2008.11.002

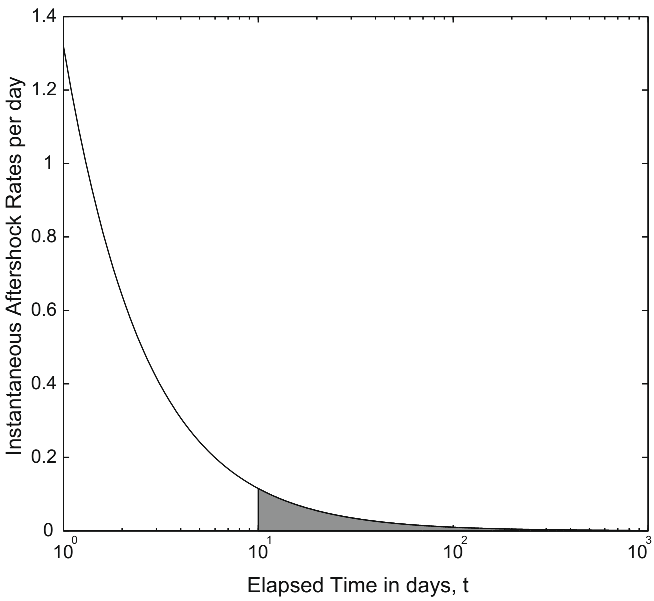

Fig. 1. Instantaneous daily aftershock rate as a function of elapsed time due to a mainshock of magnitude 7.0.

Here, we shall develop such a methodology where time-varying rates or frequencies are transformed to equivalent constant rates with the desired time-invariant characteristics by considering an implied discounted investment in life-safety technologies for both the constant (or homogeneous) mainshock case and the time-varying (or nonhomogeneous) aftershock case on the basis of social equity. See [7] for a discussion of the notion of social equity and investment in life-safety technologies in the context of structural safety in the typical constant rate case. Such equivalent constant rates which represent the time-varying aftershock rates are referred to as equivalent constant rates, or *ECR*s.

2. Formulation of equivalent constant rates (*ECR*s) in the aftershock environment

Aftershocks are typically modeled as a nonhomogeneous Poisson process with intensity function $\mu(t; m_m)$ given by Eq. (1). In this paper, we use the frequency of collapse as a proxy for the life-safety risk faced by an arbitrary building occupant. We assume that a building has deteriorated from the intact state to damage state i after the occurrence of the mainshock. We assume further that this mainshock-damaged building in damage state i will collapse in an aftershock with probability Q_{in}, where damage state n is defined as the collapse state.

In this study, we adopt the procedure proposed in [8] and [9] to compute Q_{in} for a conventional steel frame structure. Post-mainshock damage states are defined based on peak mainshock roof drift ratios. The damage sustained by the structure due to the mainshock is associated with increasing values of peak roof drift ratios after an earthquake, denoted as θ. We assume that we can classify several levels of damage sustained by the structure into discrete damage states j, where θ associated with damage state j is θ_j. Note that we can also choose to define damage states based on other engineering demand parameters such as the residual roof drift ratios as well. The four discrete damage states considered are:

(1) Damage state 1 at a peak roof drift ratio of $\theta_1 = 0.009$ corresponds to onset of nonlinear behavior in the building.
(2) Damage state 2 at a peak roof drift ratio of $\theta_2 = 0.016$ corresponds to fracture of exterior beam-column connections of the first floor in the building.
(3) Damage state 3 at a peak roof drift ratio of $\theta_3 = 0.024$ corresponds to fracture of interior connections of the frame (in addition to exterior connections).
(4) Damage state 4 at a peak roof drift ratio of $\theta_4 = 0.048$ corresponds to local failure of a shear-tab, and it represents local collapse.

Luco et al. [8] and Bazzurro et al. [9] propose a methodology to evaluate the (random) residual capacity of a building in post-mainshock damage state i to resist collapse, $Sa_{cap}^{i,n}$, based on back-to-back nonlinear time-history dynamic analysis (in this case, $n = 4$). $Sa_{cap}^{i,n}$ is defined as the minimum first-mode aftershock ground motion spectral acceleration that would induce the peak roof drift ratio θ_n associated with local collapse for a building originally in post-mainshock damage state i. Thus, a building in damage state i is assumed to have a capacity of $Sa_{cap}^{i,n}$ to resist local collapse, where the median $Sa_{cap}^{i,n}$ value is $\widehat{Sa}_{cap}^{i,n}$ and the dispersion (i.e., standard deviation of the logarithm of $Sa_{cap}^{i,n}$) is $\beta_{cap}^{i,n}$. Q_{in} is defined in Eq. (2) assuming that the post-quake aftershock hazard can be represented using a power-law approximation, similar to the approach in [10] for mainshock hazard.

$$Q_{in} = P(Sa_{site} > \widehat{Sa}_{cap}^{i,n} |\ \text{aftershock}) \exp\left(\frac{1}{2} k_a^2 \left[\beta_{cap}^{i,n}\right]^2\right) \tag{2}$$

$P(Sa_{site} > \widehat{Sa}_{cap}^{i,n} |\ \text{aftershock})$ is the probability of exceeding $\widehat{Sa}_{cap}^{i,n}$ given an aftershock of random magnitude at a random location resulting in a ground motion intensity, Sa_{site}, at the building location taken into consideration. This probability can be obtained from Aftershock Probabilistic Seismic Hazard Analysis (or APSHA) by integrating over the aftershock-magnitude distribution and the source-to-site distance distribution. The term $\exp(\frac{1}{2} k_a^2 [\beta_{cap}^{i,n}]^2)$ is a factor to account for the dispersion of the $Sa_{cap}^{i,n}$ value, where k_a is the slope of the linearly-approximated log–log aftershock hazard curve. See Yeo and Cornell [11] or Yeo [12] for details. This formulation does not take into account possible damage accumulation produced by consecutive aftershocks after the occurrence of the mainshock. This simplification understates the actual damage sustained by the structure, and deserves further research by the earthquake engineering community.

We define an aftershock which results in the local collapse of a building in damage state i as a fatality event. We describe local collapse as damage state 4 for a conventional steel frame structure which has a peak roof drift ratio of $\theta_4 = 0.048$. This corresponds to local failure of a shear-tab, thus resulting in the collapse of the slab and the consequent death of the occupants located under the area. The fatality events can be modeled as a nonhomogeneous Poisson process with intensity function $\varpi_i(t; m_m)$ in Eq. (3). We now wish to assess the threat due to fatality events for an arbitrary occupant of a building in damage state i in the time interval $[\tau, \tau + d\tau]$, where $\tau \in [0, \infty)$ and $d\tau$ is an infinitesimal increment from τ. The probability of a fatality event in $[\tau, \tau + d\tau]$ is approximately equal to $\varpi_i(\tau; m_m) d\tau$. The definition of $\mu(t; m_m)$ is given in Eq. (1).

$$\varpi_i(t; m_m) = \mu(t; m_m) Q_{in}, \quad t \in [0, \infty) \tag{3}$$

The assessment of the probability of a fatality event in the aftershock environment will be useful for deciding if continued occupancy of the building is permitted or if evacuation of the building occupants is necessary. Typically, after a mainshock, there is a need to inspect mainshock-damaged buildings to decide if it is necessary to evacuate their occupants based on the damage sustained by the buildings. The decision is denoted by a tag, green, yellow or red, on