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Risk-based assessment of aftershock and mainshock-aftershock seismic performance of reinforced concrete frames

Mehrdad Shokrabadi*, Henry V. Burton

Department of Civil and Environmental Engineering, University of California Los Angeles, Los Angeles, CA 90095, United States

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ABSTRACT

The increase in seismic activity after a large-magnitude earthquake coupled with the reduction in the lateral load-carrying capacity of the affected structures presents a significant human and financial risk to communities. The focus of this paper is placed on quantifying the impact of both the elevated post-mainshock seismic hazard as well as the mainshock-induced structural damage on the seismic risk of three reinforced concrete moment frame structures. The seismic hazard due to sequential earthquakes is examined in both pre- and post-mainshock environments. The time-dependent nature of seismic hazard in the post-mainshock environment is accounted for through the adoption of a Markov risk assessment framework. In the post-mainshock environment, the seismic risk is examined as a function of the time elapsed since the mainshock's occurrence while in the pre-mainshock environment, the risk is investigated during an assumed lifespan of 50 years for the studied structures. For the buildings and the high-seismicity site used in this study, both the increased post-mainshock seismic hazard as well as the reduction in the structural capacity are found to have a great influence on the seismic risk. The substantial contribution of aftershocks to the collapse risk in the pre-mainshock environment highlights the need for a design procedure that accounts for the additional seismic risk from aftershocks.

1. Introduction

Aftershock events have been shown to exacerbate the damage caused by mainshocks and in some cases, have led to collapse [1,2]. While aftershocks are generally smaller in magnitude than their preceding mainshock, structures can be particularly vulnerable to aftershocks due to their high rate of occurrence and the reduction in the lateral load-carrying capacity caused by damage induced by the mainshock. In the 1999 Kocaeli earthquake, several buildings that survived the mainshock, which had a moment magnitude (M_w) of 7.4, collapsed during a M_w 5.9 aftershock, which occurred one month later, killing seven people and injuring more than two hundred [3]. The aftershocks that followed the 2008 Wenchuan earthquake damaged 196 dams and claimed more lives. The 2010 M_w 7.1 Darfield earthquake was followed by two M_w 6.2 and M_w 6.0 aftershocks, which resulted in 185 fatalities and damaged approximately 100,000 buildings in the city of Christchurch [4]. The five aftershocks with M_w over 7 that followed the 2011 Tohoku earthquake caused additional damage to infrastructure, liquefaction and loss of lives [5].

The risk of aftershock collapse also influences the post-mainshock decisions of owners and occupiers regarding reoccupancy of damaged buildings, thereby affecting the recovery process [2]. A large portion of the central business district in Christchurch was evacuated due to the perceived aftershock collapse risk posed by several damaged buildings [6], resulting in widespread business disruption and substantial indirect losses [7].

In recent years, significant advances have been made in classifying mainshock-aftershock (MS-AS) sequences (e.g., Wooddell and Abrahamson [8]), characterizing their spatial and temporal distribution, and quantifying the time-dependent hazard associated with aftershock events [9–11]. Moreover, recent databases developed as part of the Next Generation Attenuation (NGA) ground motion projects (particularly NGA-West2 [12], and NGA-Subduction) have increased the availability of recorded ground motions from MS-AS sequences. However, the structural engineering community is still in the early stages of leveraging these developments. While the threat posed by aftershocks is now well recognized, research to quantify the associated risk is still in its infancy, particularly with regards to integrating the time-dependent aftershock hazard with the increased vulnerability to collapse of damaged buildings.

Most studies to date on aftershock performance have focused on quantifying the change in vulnerability experienced by buildings subjected to mainshock damage. Li and Ellingwood [13] examined the damage in the lateral force resisting elements caused by sequential

* Corresponding author.
E-mail address: mshokrabadi@g.ucla.edu (M. Shokrabadi).

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ground motions in 9- and 20-story steel moment frames. Two ground motion suites were selected to represent seismic events with 10% and 2% probability of exceedance in 50 years. They found that the extent of damage under the aftershocks had a higher correlation with the extent of damage under the mainshocks rather than the intensity of the mainshock. Ruiz-García and Aguilar [14] subjected a 4-story steel moment frame building to 14 ground motion pairs from the 1994 Northridge earthquake. Five levels of residual story drift ratios (SDR) were targeted under the mainshock records followed by collapse analysis under the aftershock ground motions. A reduction of up to 20% in the collapse capacity was observed as the residual story drift ratio under the mainshocks increased to up to 2%.

The performance of reinforced concrete (RC) moment frames under sequential ground motions has also been investigated in several studies. Hatzigeorgiou and Liolios [15] evaluated the response of a set of ductile and non-ductile RC moment frame structures to as-recorded and artificially-generated ground motion sequences. An increase in the displacement demand and a change in the distribution of plastic hinges under the sequential ground motions in comparison with the single record analyses were reported. More recently, Raghunandan et al. [1] evaluated the post-mainshock collapse capacity of code-conforming RC frame buildings. Each ground motion pair in their study consisted of one single record as both the mainshock and the aftershock. For each pair, the buildings were first subjected to the mainshock ground motions to target a specific maximum SDR. The median collapse capacity was reduced by up to 46% when the buildings sustained an SDR of 4% under the mainshocks. The relationship between the reduction in the collapse capacity and eight damage measures was examined and story drift ratios (both maximum and residual) were found to be the best indicators of the reduction in the collapse capacity when the buildings were subjected to a mainshock seismic event. In another study, Jeon et al. [16] proposed a framework for developing aftershock damage fragility curves for RC frames. The methodology was applied to a set of three non-ductile RC frames and fragility curves were developed for five different damage states, ranging from cosmetic repair measures to replacing structural members. A direct relationship between the damage sustained during mainshocks and the extent of damage under aftershocks was observed. The vulnerability to aftershock damage was shown to increase with building height. Other noteworthy studies on aftershock performance assessment include those by Luco et al. [17], Bazzurro et al. [18], Maffei et al. [19], Ryu et al. [20], Nazari et al. [21], Han et al. [22] and Tesfamariam et al. [23].

A common theme in above-mentioned studies is that they all involved vulnerability-based assessments and did not consider time-dependent aftershock hazard and subsequent risk. It is well established that the rate of seismic events increases significantly after the occurrence of a large-magnitude mainshock [24]. This elevated seismic hazard, combined with the reduction in structural capacity caused by the mainshock damage, can increase the seismic risk posed to buildings and communities already affected by a major mainshock seismic event.

A necessary step in performing risk-based assessment is the characterization of seismic hazard. Yeo and Cornell [11] used the empirical relationship that describes the exponential decay of aftershocks [25] to formulate a probabilistic seismic hazard analysis (PSHA) methodology for quantifying the aftershock seismic hazard at a site given the occurrence of a mainshock. Other aftershock PSHA methodologies developed by Boyd [9] and Iervolino et al. [10] are aimed at predicting the combined seismic hazard due to mainshocks and aftershocks before the occurrence of a causative mainshock. Such methodologies would prove useful when there is a need to account for the aftershock seismic hazard in the design or retrofit of a structure.

Several studies have taken the necessary steps to address the time-dependent nature of seismic risk in the aftershock environment. Yeo and Cornell [26] proposed a methodology for estimating life-time financial losses due to sequential seismic events. Ebrahimian et al. [27] formulated a framework to account for the fact that the transition between discrete structural limit states in the post-mainshock environment is a function of the number of aftershock events that are likely to happen during the time window that the seismic performance is being evaluated. Iervolino et al. [28] examined the application of a state-dependent Markov approach in evaluating time-dependent limit state exceedance probabilities for SDOF systems. Nazari et al. [29] developed a methodology to assess the necessary changes in structural design to account for aftershock hazards. The methodology was applied to a 2-story woodframe building and the authors found that an approximately 10% increase in strength and stiffness was needed when aftershock collapse risk was considered.

The current study seeks to perform a risk-based assessment of the aftershock and mainshock-aftershock (MS-AS) seismic performance of code-conforming RC frame buildings. Section 2 describes the steps involved in formulating a time-dependent seismic risk evaluation approach using non-stationary Markov transition probability matrices. The need for such a time-dependent framework for seismic risk assessment is also discussed in Section 2. The mainshock and aftershock seismic hazard at a high-seismicity site in Southern California are assessed using the conventional seismic hazard analysis as well as the method proposed by Yeo and Cornell [11] in Section 3.1. The latter seismic hazard assessment method allows for the explicit consideration of the time dependency of seismic hazard in the post-mainshock environment. Hazard curves are developed using the spectral acceleration at the initial first mode period of the structures ($S_a(T_1)$) as the intensity measure. Interested readers are referred to Luco and Cornell [30], Tothong and Luco [31] and Eads et al. [32] for alternative intensity measures that could reduce the computational cost of response history analysis and improve the accuracy of seismic risk assessment. A description of the reinforced concrete moment-frame buildings including the structural modeling is presented in Section 4. Nonlinear response history analyses are performed using a set of carefully-selected MS-AS sequences (Section 3.2). All the selected ground motion pairs are from the as-recorded ground motions that are designated as MS-AS sequences based on time and distance windowing algorithms [8]. The outcomes of the seismic hazard assessment and response history analyses are combined using the methodology formulated in Section 2 to estimate the aftershock and MS-AS seismic risk for the RC frame buildings (Section 5).

2. Formulating limit state exceedance probability under sequential seismic events

The performance-based earthquake engineering (PBEE) framework comprehensively addresses the limit state exceedance probability (P_{LS}) calculation. The PBEE methodology assumes that the state of the structure prior to the occurrence of the seismic event is known (usually the structure is assumed to be in the intact state) and the structure will return to its pre-damaged state before the next earthquake occurs. These assumptions are valid when the seismic performance is evaluated under major mainshock events. The time interval between major mainshock events is usually long enough to allow for affected buildings to be restored to their pre-mainshock state. However, buildings that have been subjected to a major mainshock event and sustained a level of structural damage are usually not repaired during the short time period immediately following the mainshock when the rate of occurrence of aftershocks is at its highest. This, coupled with the accumulation of structural damage under successive post-mainshock events, adds to the uncertainty in determining the state of a structure before being subjected to each of the damaging aftershocks. As such, evaluating the seismic performance of structures subjected to sequential seismic events requires explicit consideration of the uncertainty in the state of the structure after being subjected to any of the events in a cluster of a mainshock and subsequent aftershocks.

Assuming that the state of the structure under event i in the sequence only depends on its state under the event $i-1$ and is independent

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