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Estimating aftershock collapse vulnerability using mainshock intensity, structural response and physical damage indicators

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ABSTRACT

This paper describes statistical models for estimating aftershock collapse vulnerability of buildings using mainshock intensity, structural response and physical damage indicators. The performance of mainshock-damaged buildings is assessed by performing Incremental Dynamic Analyses to collapse using sequential ground motions. Alternative dependent variables are suggested including the ratio of the conditional collapse probability for the damaged and intact buildings. Challenges arising from strong correlation among predictors are addressed using more advanced methods, including Best Subset Regression, Least Absolute Shrinkage and Selection Operator, Principal Components Analysis and Gaussian Kernel Ridge Regression. The models are evaluated based on their accuracy and stability while dealing with issues stemming from high dimensionality. Overall, Gaussian Kernel Ridge Regression is the most favorable model based on the accuracy and stability of its predictions. Of the three types of predictors, those related to observable physical damage to key structural components produced the most accurate and stable estimates of aftershock collapse vulnerability.

1. Introduction

When a building is subjected to a sequence of earthquakes, damage resulting the mainshock event can increase its vulnerability to collapse in subsequent events that occur prior to performing the necessary repairs. This has been demonstrated in recent earthquake sequences in Chi-Chi (1999), Wenchuan (2008), Christchurch (2010–2011), Tohoku (2011) and Central Italy (2016). Quantifying the collapse safety of mainshock-damaged buildings can provide decision-makers with useful information regarding whether they are safe enough to occupy during repairs (shelter-in-place). This is particularly important immediately following a large magnitude event when the aftershock hazard is highest.

The earliest studies on aftershock building performance used single-degree-of-freedom systems subjected to sequential ground motions to quantify the impact of structural damage on response demands [1–4]. Since then, many investigators have used sequential response history analysis of multi-degree-of-freedom system models to assess the aftershock performance of steel, reinforced concrete and woodframe structures. Luco et al. [5] used probabilistic seismic demand analysis of partially-inspected, damaged steel moment frame buildings, to compute the mean annual frequency of exceeding a limit state (e.g. a pre-defined story drift level). Lee and Foutch [6] estimated the structural response of a damaged steel moment frame using a linear static model with simulated brittle fractured connection. Li and Ellingwood [7] used sequential response history analysis to formulate probabilistic descriptions of performance limit states in damaged steel moment frame buildings. Ruiz-Garcia and Negrete-Manriquez [8] used fourteen as-recorded sequences from the 1994 Northridge and 1980 Mammoth Lake earthquakes to evaluate the effect of aftershocks on steel moment frame buildings. Ribeiro et al. [9] developed a reliability-based framework for quantifying the robustness of steel moment frame structures subjected to sequential seismic events.

One of the earliest studies on aftershock performance of reinforced concrete structures was performed by Hatzigeorgiou and Liolios [10]. They subjected eight concrete frame structures to real and artificial ground motion sequences and found that the response demands were higher than those obtained when the same structures were subjected to a single ground motion. Jeon et al. [11] developed aftershock fragility curves for low-, mid- and high-rise non-ductile reinforced concrete moment frame buildings. The cloud approach (unscaled ground motion), which was first implemented by Li [12], was used to assess the aftershock damage state of the structure. Pre- and post-earthquake fragilities were compared to assess the effect of various types and levels of mainshock damage on aftershock vulnerability. Han et al. [13]

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compared the response demands and fragility for two non-ductile reinforced concrete buildings subjected to as-recorded and "synthetic" mainshock-aftershock sequences. The as-recorded ground motion sequences were obtained from earthquakes occurring in the western United States. The synthetic sequences were generated by combining as-recorded mainshock ground motions with simulated aftershock ground motions with properties that are consistent with that of the mainshock. The authors demonstrated that the response demands obtained from the two types of sequences were statistically similar. Tesfamariam et al. [14] subjected reinforced concrete frame buildings with infills to mainshock-aftershock sequences to quantify the effect of mainshock damage on the fundamental period and drift demands. Compared to steel and concrete, fewer studies have been done to assess the aftershock performance of woodframe structures. Nazari et al. [15] evaluated the effect of mainshock-aftershock sequences on woodframe damage fragilities. In a follow-up study [16], the same authors examined the structural design changes needed in woodframe building to account for aftershock hazard.

The aforementioned studies did not explicitly consider the collapse limit state in their assessment of aftershock performance. However, advancements in nonlinear structural modeling and response simulation have led to a few recent studies on this topic. Li et al. [17] evaluated the collapse performance of a 4-story steel moment frame building designed to modern code provisions. Fragility functions described the probability of collapse conditioned on the aftershock ground motion intensity and the state of damage from the mainshock. Raghunandan et al. [18] used nonlinear response history analyses to evaluate how the extent of mainshock damage in reinforced concrete frame structures affected their aftershock collapse vulnerability. The study also investigated the relationship between various types of physical damage recorded during the mainshock, and the reduction in aftershock collapse capacity. Ruiz-García and Aguilar [19] subjected a 4-story steel moment frame building to sequential ground motion pairs from the 1994 Northridge earthquake. Five levels of residual story drift ratios were targeted under the mainshock records followed by Incremental Dynamic Analyses to collapse under the aftershock ground motions. Burton and Sharma [20] assessed the reduced collapse safety of mainshock-damaged reinforced concrete frame buildings with infills. The results were used to develop fragility functions that describe the probability of building closure triggered by structural damage, conditioned on the mainshock intensity.

This paper describes the application of advanced statistical techniques to estimate aftershock collapse vulnerability. The median collapse capacity and conditional probability of collapse of a mainshock-damaged building (normalized by that of the intact case) are suggested as alternative dependent variables. Multiple predictor variables related to the mainshock ground motion intensity, structural response and visual damage indicators are incorporated. Five statistical procedures are investigated including (i) Ordinary Least Squares (OLS), (ii) Best Subset Regression (BSR), (iii) Least Absolute Shrinkage and Selector Operator (LASSO), (iv) Principal Component Regression (PCR), and (v) Gaussian Kernel Ridge Regression. The methods are evaluated based on their efficiency, stability and the ability to provide reasonable estimates of reduced collapse capacity as reflected in their prediction errors.

The proposed statistical models have several useful applications. Estimates of reduced collapse safety can be used to augment the judgement of structural engineers carrying out onsite investigations of the post-earthquake structural safety of damaged buildings. Field observations and assessments of physical damage can be used as input parameters for the proposed models, which in turn can provide estimates of reduced collapse safety. In the case of instrumented buildings, structural response and ground motion intensity parameters can be recorded and used to facilitate rapid assessments of the effect of mainshock damage on aftershock collapse vulnerability. The proposed statistical models can be embedded into tools and frameworks that enable automated post-earthquake damage assessment of instrumented buildings [21–23], which are critical to realizing the vision of smart [24] and resilient [25] cities.

2. Estimating the reduction in collapse safety of mainshock-damaged buildings

2.1. Building description and structural modeling

Statistical models of aftershock collapse vulnerability are developed for a reinforced concrete frame building with unreinforced masonry infills (infill frames). The building is a 4-story multi-family residential building designed in accordance with the Indian Standard Criteria for Earthquake Resistant Design of Structures, IS 1893 [26]. A regular rectangular geometry is used with 4 bays in the longitudinal direction and 3 bays in the transverse direction. The typical bay width is 3.7 m in both directions and typical story heights are 3.7 m. The building is constructed with full height infill panels with large window and door openings. The building site is located in Noida, India, which is a region of moderate seismicity. More details on the design are provided in Burton, 2014 [27], where the building is identified as 4-MF-EFS-SS. This particular construction type was chosen because the current effort is as part of a larger study on improving the seismic resilience of communities (such as Noida) with large populations of infill frame buildings [27]. The effect of infill panels on the behavior of concrete frames has been the subject of many experimental and numerical investigations (e.g. [28–31]). A multitude of numerical (micro and macro) modeling strategies for infill frames can be found in the existing literature (e.g. [32–34]). More recently, there have been efforts to develop tools and methods to quantify the seismic collapse safety of infill frame buildings (e.g. [35–37]). In the current study, a two-dimensional model is constructed to represent the exterior longitudinal frame of the prototype building in *OpenSees* [38] and a leaning column is used to incorporate P-Δ effects. Beams and columns are idealized using elastic elements with concentrated flexural plastic hinges with the Ibarra-Krawinkler [40] material to capture nonlinear behavior. The parameters for the plastic hinges of the reinforced concrete beams and columns are obtained using the predictive equations developed by Haselton and Deierlein [39]. The infill panels are modeled using a pair of inelastic compression-only struts in each direction that incorporate strength and stiffness deterioration. The dual compression struts [35] capture the column-infill interaction that can cause shear failure of the columns. A peak-oriented hysteretic model is used to capture the nonlinear behavior of the infill compression struts. For this study, 25% of the total strut strength and stiffness is assigned to the off-diagonal strut and 75% to the central strut [35,41]. The analytical relationships developed by Saneinejad and Hobbs [42] are used to compute the capping strength, F_c, and the initial stiffness, K_e, of the infill struts. The guidelines by Burton and Deierlein [35] are used to obtain the other strut model parameters including the ratio of capping strength to yield strength, F_c/F_y, the ratio of capping displacement to yield displacement, Δ_c/Δ_y, and the ratio of post-capping stiffness to yield stiffness, K_c/K_e.

A zero-length shear spring is placed in series with the flexural hinges at the ends of the columns and assigned a rigid-softening shear force versus deformation material-model [35]. Shear failure is followed by a negative post-peak slope that captures the shear strength degradation. A high initial stiffness is used for the elastic region, meaning, negligible deformation is assumed in the spring up to the shear strength of the column. The deformation