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An Integrated Framework for Seismic Risk Assessment of Reinforced Concrete Buildings Based on Structural Health Monitoring

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1 Abstract

In recent years, several locations in the United States have been experiencing a significant increase in seismicity that has been attributed to oil and gas production. As oil and natural gas production in the United States continues to increase, it is expected that the seismic hazard in these locations will continue to experience a corresponding upsurge. However, many urban structures in these locations are not designed to withstand these increasing levels of seismicity. Accordingly, it is crucial to develop methodologies that can help us quantify the seismic performance of these structures, establish their risk levels, and identify optimal retrofit strategies that will enhance the seismic resilience of these structures. In this context, structural health monitoring (SHM) plays an important role in understanding the seismic performance of structures. SHM can be used, in conjunction with finite element modelling, to provide a realistic representation of the structural performance during a seismic event. In this paper, a framework for seismic risk assessment of reinforced concrete buildings based on SHM is presented. The framework combines nonlinear finite element modeling and SHM data to establish the seismic fragility profile of the structure. The approach is illustrated on a multi-story reinforced concrete structure located on the Oklahoma State University Campus.

Keywords: Induced seismicity; seismic fragility curves; nonlinear finite element analysis; structural health monitoring.

2 Introduction

A significant increase has been observed in the seismic activity in Oklahoma, United States (US) since 2009. The number of earthquake events with magnitudes greater than 3 significantly increased from 41 events in in 2010 to 903 events in 2015 [1]. Several research studies aiming at identifying the cause of this upsurge in earthquakes have been conducted. Several of these studies showed that the wastewater injection due to oil and gas production was the primary cause of the increased seismic activity; hence it is termed *induced seismicity* [2, 3].

Due to the low occurrence probability of major natural earthquakes in the region, most of the structures in Oklahoma have not been designed to withstand this level of seismic activity. Besides, as the oil and gas production in the US continues to increase [4], it is expected that the induced seismic hazard in Oklahoma will also increase in future. With this increased induced seismic hazard, it is crucial to develop methodologies that can help evaluate the real-time performance of buildings, quantify their risk levels, and identify optimal retrofit strategies that will enhance their seismic resilience.

In this context, seismic risk assessment (SRA) can play a vital role in quantifying the seismic performance of structures. SRA provides the expected loss by incorporating the seismic hazard, structural vulnerability, and exposure at a given location [5]. In addition, it can also help decision-makers and disaster management authorities towards their preparedness and post-disaster relief activities. Therefore, SRA serves as a useful tool for mitigating the probable loss due to an earthquake and helps in developing the course of emergency response after the event [6].

Recently, performance-based earthquake engineering (PBEE) has been shown to be a robust tool for evaluating the seismic risk of reinforced concrete (RC) buildings through the application of incremental dynamic analysis [7]. PBEE can estimate the level of the structural damage expected after a seismic event of a specific intensity. While failure criteria are well understood for designing a new structure, SRA of existing buildings not designed for earthquake loads represent a significant challenge. PBEE can play a crucial role to enable the assessment in these situations [8,9]. In addition, PBEE can also assist in developing fragility curves to quantify the seismic resilience of a building, estimate losses, and plan optimal retrofit strategies [10]. Fragility curves provide the probability of reaching or exceeding a specific damage level for a component or the entire building when subjected to a certain level of seismic loads [8,10,11].

Structural health monitoring (SHM) can help in quantifying the seismic performance of structures. SHM can be used to quantify the structural performance measures (e.g., accelerations and rotations) during a seismic event. Moreover, with the help of system identification (SI) methods, SHM can also help in estimating the changes in structural properties [12] and detect the occurrence of structural damage during the service life [13]. Thus, it can be used, in conjunction with finite element modelling (FEM) and PBEE, to provide a realistic representation of the structural performance during seismic events.

This paper presents a framework for seismic risk assessment of RC buildings based on SHM. The framework employs SHM data to calibrate a finite element (FE) model that can simulate the dynamic response of a reinforced concrete building subjected to ground motions. The calibrated FE model is then used to develop fragility curves given various damage states and several ground motion records. This approach is illustrated on a multi-story RC structure located on the Oklahoma State University, Stillwater campus.

3 Case Study

3.1 Selected Structure

In this paper, the seismic risk assessment framework is applied to Kerr Hall building located in Oklahoma State University, Stillwater campus, OK. Kerr hall, shown in Figure 1, is a 12-story residential hall with RC frame structure built in the 1960s. In order to monitor the building response under earthquake loads, a Trimble NET-RS receiver in conjunction with two tri-axial strong motion accelerometers are installed on this building. To capture the earthquake-induced ground motion and the acceleration response of the structure, one accelerometer is installed at the ground level while the other one is installed at the roof level of the building. Figures 2 and 3 show the installed Trimble NET-RS Receiver and strong motion accelerometer, respectively.

Figure 1. Kerr Hall (investigated building)

Figure 2. Trimble NET-RS receiver

Figure 3. Strong motion accelerometer

3.2 Finite Element Modeling and Calibration

A three-dimensional finite element (FE) model of the building, shown in Figure 4, is constructed using CSi SAP2000 [14] based on the as-built structural drawings obtained from the university administration. Reinforced concrete is modeled using Mander's nonlinear stress-strain model for confined concrete [15] as an isotropic material. Following the construction drawings, a compressive strength (f'_C) of 35 MPa for columns and 30 MPa for all other elements is used. Steel reinforcement with yield strength (f_y) of 345 MPa and 138 MPa are used for flexural and shear rebars, respectively. Conventional RC framing with a 127 mm (5 in.) slab is used for first two stories, while 178 mm (7 in.) flat slabs are used for the third to the twelfth stories.

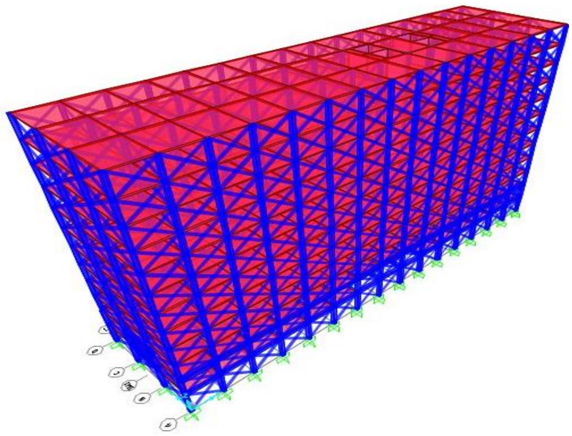

Figure 4. FE model of the building

In order to incorporate the effect of infill walls, FEMA 356 diagonal compression struts are used in each bay in both directions. Following FEMA 356 [16], the width of the equivalent strut *a* has been calculated as

$$a = 0.175\,(\lambda_1 h_{col})^{-0.4} r_{inf} \tag{1}$$

where

$$\lambda_1 = \left[\frac{E_{me} t_{inf} \sin 2\theta}{4 E_{fe} I_{col} h_{inf}}\right]^{\frac{1}{4}} \tag{2}$$

and λ_1 is the coefficient representing the equivalent width of infill struts, h_{col} is column height, h_{inf} is the height of infill panel, E_{fe} is the Young's modulus of frame material, E_{me} is the Young's modulus of infill material, I_{col} is the moment of inertia of column, r_{inf} is diagonal length of infill panel, t_{inf} is thickness of infill panel and equivalent strut, and θ is angle whose tangent is the infill height-to-length aspect ratio.

The model is then calibrated based on earthquake acceleration records obtained from the installed SHM system. The acceleration time-history associated with a 4.6 magnitude earthquake recorded on April 07, 2018 using the ground level accelerometer is applied to the FE model. The corresponding response of the structure recorded using the roof accelerometer is then compared to the results of FE model. The stiffness of the columns and infill walls and the damping characteristics were chosen as the calibration parameters based on sensitivity analysis. An iterative procedure was conducted to establish the optimum values of the calibration parameters. These optimum values minimize the difference between the modal response parameters and the corresponding power spectral ratios obtained from the FE model and the accelerometer records. The frequencies associated with the first three modes and the corresponding power spectral density ratios of FE model are compared with the respective output responses obtained from the SHM system.

The frequencies of first three modes of the building obtained from the roof accelerometer records are 1.3, 2.1 and 2.2 Hz, respectively. After the FE model calibration, the frequencies obtained from the model are 1.3, 1.7 and 2.2 Hz, respectively. The difference in the frequency of second mode can be attributed to the torsional effects associated with the second mode of the building.

Figure 5 shows the power spectral ratio of the roof to the base of the investigated structure. As shown, the calibrated model is capable of capturing the building response with reasonable accuracy.

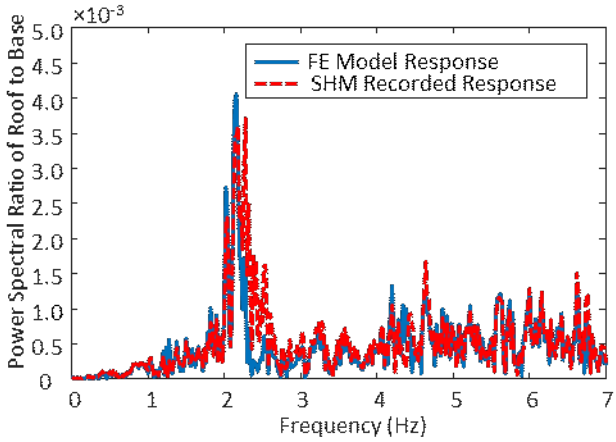

Figure 5. Power Spectral ratio matching

3.3 Earthquakes Selection

After calibrating the model, 20 ground motion time-histories are used for analyzing the structure to estimate the probability of failure given different earthquake excitations. These ground motions are a combination of actual recorded earthquakes and synthetically generated waveforms based on the approach provided by Melgar et. al [17]. The recorded ground motions are adopted from the PEER database [18]. This waveform generation approach offers 1-D velocity model and is based on a discretized fault model [19]. In this model, the target magnitude can be set, and ruptures limits can be chosen using empirical scaling relationships and target magnitudes. In this way, stochastic slip distribution is generated within the rupture region corresponding to the target magnitude and the hypocenter is randomly chosen within the rupture region. Afterwards, rupture propagation and slip duration are chosen using the velocity structure and slip amount. Green's functions for synthetic waveforms are computed using frequency-wavenumber method [20] and a prescribed source time function. The April 07, 2018 M4.6 Oklahoma earthquake has been chosen to verify the synthetic waveform generation process.

3.4 Dynamic Analysis and Fragility Curves

The adopted ground motions are applied to the calibrated FE model. Monte Carlo simulation with 1,000 samples is adopted in this paper to account for uncertainties associated with input parameters. The Modulus of elasticity of the structural elements is considered as random variable. A MATLAB [21] script is prepared to iteratively execute the FE model and obtain the displacement responses associated with each ground motion and random sample. The inter-story drift ratio (IDR) is selected as the engineering demand parameter (EDP) while peak ground acceleration (PGA) is selected as intensity measure (IM) for conducting the fragility analysis. The probability of exceeding a certain limit state given the applied loads is calculated as:

$$P_i(PGA) = P[any\ G_i(PGA, X)] < 0 \tag{3}$$

where

$$G_i(PGA, X) = D_{FE}(PGA, X) - D_{limit,i} \tag{4}$$

G_i (*PGA*,**X**) represents the performance function associated with i^{th} limit state, corresponding vector of random variables **X**, and a given PGA value. D_{FE} (*PGA*,**X**) is the maximum inter-story drift ratio calculated from FE analysis associated with vector of random variables **X**, and a given PGA value. $D_{limit,I}$ is maximum allowable drift associated with

Four common performance levels known as immediate occupancy, life safety, damage control, and collapse prevention with mean of 1%, 2%, 2.5%, and 4% drift ratios are considered, respectively [11]. In addition to these ratios, a 3% drift ratio is also included and considered as the onset of structural instability [8]. It is assumed that each of these drift limits follow a normal distribution with the defined mean and 0.1 coefficient of variation. The resulting fragility curve for the investigated building is presented in Figure 6.

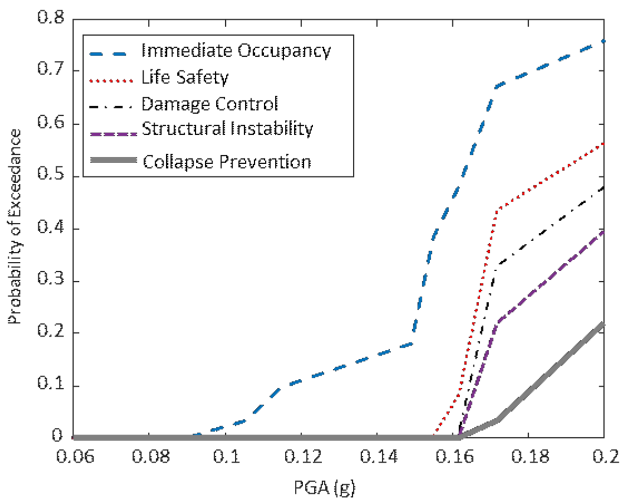

Figure 6. Fragility curve for Kerr Building

4 Conclusions

This paper presents an approach to establish the collapse fragility curves for existing reinforced concrete buildings by incorporating structural health monitoring (SHM) and finite element modelling. The SHM data is used to calibrate the FE model. The approach is illustrated on a multi-story reinforced concrete structure located at Oklahoma State University, Stillwater campus. Results indicate that the probability of exceeding the structural instability limit state is 40% when subjected to an earthquake of 0.2g PGA, 55% for life safety and 75% for immediate occupancy performance levels. This shows that the investigated structure can experience significant damage due to future possible earthquakes.

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