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Copyright Year	2018	
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Author	Family Name	Mohammed
	Particle	
	Given Name	Mohammed A.
	Prefix	
	Suffix	
	Division	School of Civil and Construction Engineering
	Organization	Oregon State University
	Address	Corvallis, USA
	Email	
Author	Family Name	Yu
	Particle	
	Given Name	Hanshun
	Prefix	
	Suffix	
	Division	Department of Civil and Environmental Engineering
	Organization	Tufts University
	Address	Boston, USA
	Email	
Author	Family Name	Furtado
	Particle	
	Given Name	Andre
	Prefix	
	Suffix	
	Division	CONSTRUCT-LESE, Faculty of Engineering (FEUP)
	Organization	University of Porto
	Address	Porto, Portugal
	Email	
Corresponding Author	Family Name	Barbosa
	Particle	
	Given Name	Andre R.
	Prefix	
	Suffix	
	Division	School of Civil and Construction Engineering
	Organization	Oregon State University
	Address	Corvallis, USA

	Email	andre.barbosa@oregonstate.edu
Author	Family Name	Moaveni
	Particle	
	Given Name	Babak
	Prefix	
	Suffix	
	Division	Department of Civil and Environmental Engineering
	Organization	Tufts University
	Address	Boston, USA
	Email	
Author	Family Name	Varum
	Particle	
	Given Name	Humberto
	Prefix	
	Suffix	
	Division	CONSTRUCT-LESE, Faculty of Engineering (FEUP)
	Organization	University of Porto
	Address	Porto, Portugal
	Email	
Author	Family Name	Rodrigues
	Particle	
	Given Name	Hugo
	Prefix	
	Suffix	
	Division	RISCO, School of Technology and Management
	Organization	Polytechnic Institute of Leiria
	Address	Leiria, Portugal
	Email	
Author	Family Name	Vila-Pouca
	Particle	
	Given Name	Nelson
	Prefix	
	Suffix	
	Division	CONSTRUCT-LESE, Faculty of Engineering (FEUP)
	Organization	University of Porto
	Address	Porto, Portugal
	Email	
Author	Family Name	Wood
	Particle	
	Given Name	Richard L.
	Prefix	
	Suffix	
	Division	Department of Civil Engineering
	Organization	University of Nebraska-Lincoln
	Address	Lincoln, USA

Abstract

This paper describes the system identification and finite element modeling of an 18-story unreinforced masonry infilled reinforced concrete building that was damaged following the 2015 M7.8 Gorkha Earthquake, Nepal, and subsequent aftershocks. The identification is performed using the ambient vibration data collected using broadband seismometers and accelerometers. The new apartment building was seismically designed using Indian Codes and Standards. At the time of the earthquake mainshock, its construction was complete as the building was being finalized for handover to the clients. Following the earthquake mainshock and aftershocks, most of the damage to the building was observed in the non-structural masonry infill walls, while minor damage was observed in the diaphragms and elevator core link beams on lower stories. The ambient vibration response of the damaged building was recorded using either: (1) fifteen accelerometers at different setups, or (2) three seismometers. The dynamic characteristics of interest are the modal parameters of the first few lateral and torsional vibration modes. The modal identification results from the two measurement systems are compared to provide insight into the uncertainty of the results. In addition, two finite element models of the structure are developed from the information provided in the design documentation of the building. The models are tuned to match the identified modal parameters and compared with each other. Special attention is placed on the stiffness of structural and non-structural elements.

Keywords
(separated by '-')

Ambient vibration testing - Finite element modeling - Gorkha earthquake - Structural health monitoring - System identification

Post-earthquake Field Measurement-Based System Identification and Finite Element Modeling of an 18-Story Masonry-Infilled RC Building

Mohammed A. Mohammed¹, Hanshun Yu², Andre Furtado³,
Andre R. Barbosa^{1(✉)}, Babak Moaveni², Humberto Varum³,
Hugo Rodrigues⁴, Nelson Vila-Pouca³, and Richard L. Wood⁵

¹ School of Civil and Construction Engineering,
Oregon State University, Corvallis, USA
andre.barbosa@oregonstate.edu

² Department of Civil and Environmental Engineering,
Tufts University, Boston, USA

³ CONSTRUCT-LESE, Faculty of Engineering (FEUP),
University of Porto, Porto, Portugal

⁴ RISCO, School of Technology and Management,
Polytechnic Institute of Leiria, Leiria, Portugal

⁵ Department of Civil Engineering,
University of Nebraska-Lincoln, Lincoln, USA

Abstract. This paper describes the system identification and finite element modeling of an 18-story unreinforced masonry infilled reinforced concrete building that was damaged following the 2015 M7.8 Gorkha Earthquake, Nepal, and subsequent aftershocks. The identification is performed using the ambient vibration data collected using broadband seismometers and accelerometers. The new apartment building was seismically designed using Indian Codes and Standards. At the time of the earthquake mainshock, its construction was complete as the building was being finalized for handover to the clients. Following the earthquake mainshock and aftershocks, most of the damage to the building was observed in the non-structural masonry infill walls, while minor damage was observed in the diaphragms and elevator core link beams on lower stories. The ambient vibration response of the damaged building was recorded using either: (1) fifteen accelerometers at different setups, or (2) three seismometers. The dynamic characteristics of interest are the modal parameters of the first few lateral and torsional vibration modes. The modal identification results from the two measurement systems are compared to provide insight into the uncertainty of the results. In addition, two finite element models of the structure are developed from the information provided in the design documentation of the building. The models are tuned to match the identified modal parameters and compared with each other. Special attention is placed on the stiffness of structural and non-structural elements.

Keywords: Ambient vibration testing · Finite element modeling · Gorkha earthquake · Structural health monitoring · System identification

1 Introduction

A M7.8 shallow earthquake hit Nepal on April 25, 2015 with the epicenter located 80 km Northwest of Kathmandu, which is the capital of Nepal. More than hundred aftershocks occurred after the mainshock (Fig. 1), including M7.3 and M6.3 earthquakes on May 12, 2015 [1]. The mainshock ground motion accelerograms (Fig. 2a, b) were recorded at USGS station in Kathmandu (KATNP, 27.71N, 85.31E), and the recorded horizontal peak ground acceleration was 0.164 g in the North–South direction. Figure 2c shows the response spectrum of the two horizontal components of the recorded ground motion. As it can be seen in the figure, the spectral acceleration of the horizontal component is more than 0.6 g in the short period range, and there is a large spectral magnification in the long period range. This earthquake killed more than 8000 people and destroyed more than 500,000 buildings and many more were damaged to different levels of damage [1, 2].

Fig. 1. Nepal 25 April 2015 earthquake and aftershock earthquakes distribution.

Structural identification is an important tool to estimate the dynamic properties of structures and to predict the changes that can occur over time in aging structures. Using multiple recorded responses of full scale structures that have been damaged by real natural disasters is an essential and effective way to validate the different methods of structural identification [3]. System identification by using measured vibration data is a method used with increased frequency in the recent years to evaluate the performance of existing structures and as part of the basic information needed for finite element model updating. The ambient vibration test is considered as one of the most attractive techniques to carry out the system identification of structures, this mainly because of the low economical cost and fast installation of the system that can be used to perform the test in the field.

Fig. 2. Horizontal acceleration time histories and response spectra of the KATNP (27.71°N, 85.31°E) during the M7.8 Gorkha 2015 Nepal earthquake.

The main purpose of this paper is to describe the system identification that was performed on an 18-story building that was damaged during the Gorkha 2015 Nepal earthquake. The system identification for the building is carried out using two groups of ambient vibration data that were collected using two systems independently: (1) accelerometers, or (2) seismometers. A comparison between the results of the modal parameters (natural frequencies and damping ratios) by the two measurements is presented to improve our understanding on the differences in the results of the two systems. Moreover, two linear elastic finite element (FE) models are developed to predict the dynamic characteristics of the building. The dynamic characteristics are then compared with the ones identified from the ambient vibration testing. Lastly, the numerical models are calibrated by assigning different reduction factors for the masonry infill walls to account for the damage that was observed following the earthquake.

2 Description of the 18-Story Masonry Infilled RC Building

2.1 Structural Details

The structural system of the 18-story reinforced concrete (RC) building (Fig. 3) (17-stories above grade and a basement level) consists of RC frames with shear walls located in the core area of the building around the elevators. There are two staircases located to the north and south of the middle part of the building. Unreinforced masonry walls with different size of openings, thicknesses and heights are used as partitions along the height of the building.

Fig. 3. 18-story RC building.

2.2 Seismic Damage

The infill walls were moderately damaged after the earthquake. Minor to no damage was observed in the structural elements of the building. Crushed corners, diagonal and horizontal cracks are some of the damage that were observed in the masonry infill walls, and most of this damage was heavily concentrated in the lower stories of the building. Shear and flexural cracks can be seen clearly in the coupling beams that connect the elevator cores. Figure 4 shows different types of damage that was observed in the infill walls during a site visit 6 weeks after the mainshock.

Fig. 4. Observed infill wall damage in the 18-story building after the earthquake.

3 Instrumentation for the Ambient Vibration Test

Two systems were used to collect ambient vibration data. The first was a system consisting of fifteen accelerometers and the second was a system that made use of three seismometers. Details of the two systems are provided below.

3.1 Accelerometers

A network of 15 accelerometers with two different setups of sensors were used to collect the ambient vibration data. For setup A1, the sensors were installed on floor 9, 12, and 15. For setup A2, the sensors were installed on floor 3, 6, and 9. For both setups A1 and A2, the accelerometers were installed in the center, as well as the northwest and southeast corners of each specific floor to measure the acceleration in two perpendicular directions. The accelerometers were fixed on brackets that were distributed over the locations illustrated in Fig. 5a. Then, the sensors were wired to a National Instruments compact DAQ by BNC cables of upto 50 m in length. Figure 5b shows the setup for the installed accelerometers. Table 1 shows the sensors layout for setup A1. Approximately 1 h of ambient vibrations were recorded for each setup.

Fig. 5. Ambient vibration testing setup.

3.2 Seismometers

In the present work, the ambient vibration test was performed using three portable triaxial macro seismographs GSR from GeoSIG [4] (see Fig. 6). Each macro-seismograph allows for the recording of acceleration signals in three orthogonal directions under specific conditions of trigger, reading, and sampling rates. Series of 900 s duration were considered with a sampling rate of 250 Hz. Time series lengths adopted for each setup were essentially limited by restrictions for the tests' duration, and counting the expected relevant natural frequencies. Each macro-seismograph was set to work independently, avoiding the use of cables and minimizing labor associated with test preparation. This was made possible resorting to internal clock synchronization. The tests consisted in successive vibration measurements in building locations, corresponding to several locations on the building plan (at the top story) and along the building height. The vibration data are collected with the macro-seismographs in four

Table 1. Sensors layout for setup A1.

Sensor no.	Floor	Corner	Dir.
1	9	SE	N
2	9	SE	E
3	12	SE	N
4	12	SE	E
5	9	M	N
6	9	NW	N
7	9	NW	E
8	15	SE	S
9	15	SE	E
10	15	M	N
11	15	NW	W
12	15	NW	S
13	12	NW	N
14	12	M	N
15	12	M	E

setups. A reference seismograph (S03) was placed in the center close to the elevators core on level 15 for all setups, which was not moved for the duration of testing for all setups. In the first setup S1, seismographs S01 and S02 were placed on floor 15 in the South–West side of the building to capture the local torsional mode. In the second setup S2, two sensors were placed at two opposite corners on floor level 15 to capture the global torsional mode of the building. Finally, in the third and fourth setup, all seismographs were installed in the center of the building along the same vertical alignment as the reference seismograph S03. For setups S3 and S4, seismograph S01 and S02 were placed on floor levels 9 and 12, and floor levels 3 and 6, respectively. Figure 7 shows the floors layout for all setups.

Fig. 6. Macro-seismograph GeoSIG, model GSR-18.

Fig. 7. Layout for the ambient vibration test using seismographs.

4 System Identification

4.1 Using Ambient Vibration Data Collected by Accelerometers

The modal parameters of the building are estimated using the NExT-ERA method [5, 6]. For both setups A1 and A2, the reported ambient vibration data is divided into 6 sets depending on the length of the available data. Each data set is approximately a 9 min long ambient vibration accelerogram. However, the last set of data of setup A1 are only 6 min long. A cleaning process was used to eliminate any noise from the recorded data before using it for system identification. The data cleaning process is carried out as follow: (1) filter the data in the frequency range of interest (0.2–10 Hz), (2) down-sample from 2048 Hz to 256 Hz for computational efficiency, and (3) remove voltage spikes in the time domain [7]. After examining the acceleration time history and power spectral density plots, measurements from some specific sensors were removed from the system identification process due to the high level of noise in the recorded data. Figure 8 shows the power spectral densities of the cleaned data for the first dataset in setup A1. Figure 9 plots the sample stabilization diagrams of the natural frequencies for modes 1–3. In the stabilization diagram of the natural frequency, the frequencies that are repeatedly identified at increasing orders are considered to be “stable”. Vertical thick (red) lines indicate the stable frequencies of interest in this study. The statistics (mean and standard deviation) of the identified natural frequencies and damping ratios for

Fig. 8. Sample power spectral densities after data cleaning (setup A1 and dataset 1).

Fig. 9. Sample stabilization diagram for the first three modes.

Setups A1 and A2 are reported in Table 2. Comparing the mean values of natural frequencies for the two setups, there is an excellent agreement between the two setups with a maximum error of 0.8% for mode 3. The damping ratios show larger variability across different datasets, but they are still reasonable.

4.2 Using Ambient Vibration Data Collected by Seismometers

The determination of the frequencies and vibration mode shapes of the building in this section is based on the acquisition of the acceleration measurements using the seismographs. For this purpose the ARTEMIS [8] software for analysis and signal processing is used. In the ARTEMIS software, the natural frequencies and the vibration mode shapes can be found by using the peak peaking and the frequency domain decomposition (FDD) techniques. Concerning the peak peaking technique, natural

Fig. 10. Spectral density and vibration modes.

frequencies are identified from the peaks of the normalized average power spectra of the measured accelerations in each section, assuming that the dynamic output in resonance is due only to one vibration mode shape. The modal identification is performed through the application of the enhanced frequency domain composition method (EFDD). From the spectral density information illustrated in Fig. 10a, it can be observed that the natural frequencies of the building are 0.61 Hz, 0.70 Hz, and 0.75 Hz with the corresponding identified modes shown in Fig. 10b. It is possible to observe that the next level of natural frequencies of the building are between 2.3–3 Hz.

4.3 Comparison Between the Modal Parameters Obtained Using the Two Different Systems

Table 2 lists the identified natural frequencies and the damping ratios of the building that were obtained from the system identification when using ambient vibration data collected using accelerometers and seismometers. It can be seen that the natural frequency of the first mode is same by both measurements. In addition, there is a reasonable match between the values of the natural frequency of the third mode with a difference of 2.7%. However, for the second mode, the difference is 4.4% between the two measurements. Considering the different equipment and setups, the authors consider that there is a very good match overall between the first three identified natural frequencies. For the damping ratios, however, the difference between the two measurements is approximately 30% for the first three modes.

Table 2. Comparison of identified frequencies by two measurements.

Mode	Natural frequencies (Hz)		Difference (%)	Damping ratios (%)		Difference (%)
	Accelerometers	Seismometers		Accelerometers	Seismometers	
1	0.61	0.61	0.0	2.4	1.6	33.3
2	0.67	0.70	4.4	1.9	1.3	31.5
3	0.73	0.75	2.7	1.3	1.7	30.7

5 Finite Element Models

Two linear elastic finite element models of the 18-story building (Fig. 11) were developed using the SAP2000 software [9]. The two models were developed by two teams independently, one at Oregon State University (model 1) and the other at the University of Porto (model 2) without any interaction between teams, not to bias the modeling approaches and assumptions used by the two teams. For the both models, linear elastic shell elements are used to model the shear walls, without any reduction factor for the flexural rigidity. The RC beams and RC columns are modeled using linear elastic frame element with flexural rigidity as follows: (1) for model 1, $EI_{\text{effective}} = 0.7 EI_{\text{gross}}$ for columns, and $EI_{\text{effective}} = 0.3 EI_{\text{gross}}$ for beams, and (2) for model 2, $EI_{\text{effective}} = 0.4 EI_{\text{gross}}$ for columns, and $EI_{\text{effective}} = 0.6 EI_{\text{gross}}$ for beams. The dimensions of the RC columns are obtained from the design drawings. The building has a typical 3 m story height with 125 mm thickness for the slabs. For all RC elements in the model, a compressive strength of 30 MPa is used as a typical value based on Schimdt hammer testing performed on the site. A compressive strength of 4.5 MPa and elastic modulus equal to 2400 MPa are used as material properties for the infill walls [10]. Two types of infill walls are used: (1) 120 mm thick for interior walls, and (2) 240 mm thick for exterior walls. Stiffness reduction factors with values ranging from 0.3 to 0.7 are used to account for openings with different sizes and locations [11]. Additional reduction factors are used to simulate the observed damage following the earthquake. For model 1, the equivalent diagonal strut method is used to model the masonry infill walls. In this method, the infill wall is replaced with two compression struts. The thickness of the struts is equivalent to the thickness of the infill walls. FEMA 356-2000 procedures [12], which are based on the work of Stafford-Smith and Carter [13], are used to calculate the width of the compression struts. The equivalent diagonal strut method requires the infill walls to be confined with columns on both sides. Because of this requirement, only the

Fig. 11. FE models

Table 3. Summary of obtained natural frequencies from FE models.

Mode	Natural frequencies (Hz)		Difference (%)
	Model 1	Model 2	
1	0.64	0.70	9.3
2	0.72	0.77	6.9
3	0.85	00.84	1.1

walls with columns at both ends are modeled and the walls with only one column at one side are not modeled in model 1. For model 2, the equivalent diagonal strut method is also used to model the infill walls. However, the width of the diagonal strut is based on the work of Priestley et al. [14]. Then, the dynamic parameters of the two models are obtained by performing linear modal analysis. Table 3 shows the natural frequencies obtained from the two FE models. It can be observed that the natural frequencies of model 1 are very close to the ones obtained from model 2. The maximum difference between the obtained natural frequencies by the two FE models is less than 10%. Comparing results in Tables 2 and 3, it can be inferred that the errors between the natural frequencies estimated from the ambient vibration tests and the ones obtained from the FE models range between 4.5% and 15%, even with simple FE models.

6 Conclusions

The system identification and modeling of an 18-story masonry-infilled RC building that was damaged during the 2015 Gorkha earthquake are described in this paper. The ambient vibration test of the building is executed using two different measurement systems, including: (1) 15 accelerometers, or (2) 3 seismometers. The system identification of the building is performed by using the collected ambient vibration to extract the modal parameters of the building. The natural frequencies are accurately identified using the two sets of collected vibration data. Moreover, there is a good agreement between the first three natural frequencies that are identified using the two measurements, even though differences were observed for the identified modes 2 and 3. Two linear elastic FE models are developed adopting the equivalent strut method. A stiffness reduction factor with different values is used to simulate the observed damage in the masonry infill walls of the building after the earthquake. There is a reasonable match between the obtained natural frequencies from the two models and the identified frequencies for the first three modes, suggesting that these models with minor modifications, can be used for post-earthquake response prediction. Nonetheless, in the experimental systems used, it is worth noting that little to no variation was observed in the estimation of Mode 1, while Modes 2 and 3 showed differences ranging from 5% to 10%, which was not expected. Further investigation is sought to clarify reasons that can explain these differences and how they can be avoided with appropriate system setups.

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