Vibration Characteristics of Traditional Masonry Buildings in the Kingdom of Bhutan

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Abstract. Ensuring the seismic resilience of traditional rammed earth and stone masonry buildings in the Kingdom of Bhutan is essential for their preservation. This study aims to clarify the vibration characteristics of traditional masonry buildings during an earthquake, based on shaking table tests and seismic response analyses. Shaking table tests were conducted on four 1/6 scaled specimens of the same design and geometry as the full-scale specimens in previous studies. The test results clarified the relationship between the nominal PGA and acceleration response factor and the change in the vibration characteristics due to damage. For the numerical modelling, two-mass system models for each specimen were constructed based on the microtremor measurements conducted for each specimen before the shaking table tests and static full-scale lateral loading tests. Seismic response analysis, using two-mass system models, was conducted to simulate the dynamic behavior observed and recorded during the shaking table tests. The results showed that the numerical analysis produced a similar output trend until rocking or large horizontal cracks occurred.

Keywords: Composite masonry building · Shaking table test · Seismic response analyses

Y. Endo and T. Hanazato (Eds.): SAHC 2023, RILEM Bookseries 46, pp. 75–82, 2024.
https://doi.org/10.1007/978-3-031-39450-8_7

1 Introduction

In the Kingdom of Bhutan, 66% of households, mostly in rural areas, live in traditional rammed earth and stone masonry buildings. These traditional buildings are particularly vulnerable to earthquakes, as evident after the earthquakes in the eastern region of Bhutan (September 21, 2009, M6.1) and the India-Nepal border (September 18, 2011, M6.9). Therefore, ensuring the seismic resilience of traditional buildings is essential for their preservation. In our previous studies, static loading tests were conducted on full-scale specimens with the same design and geometry as typical two-story traditional rammed earth and stone masonry buildings [1, 2]. In these studies, we examined the seismic performance of traditional masonry buildings and proposed a seismic retrofitting solution. However, studies that consider the vibration characteristics of traditional masonry buildings are scarce. Therefore, the present study aims to clarify the vibration characteristics of traditional masonry buildings during an earthquake, based on shaking table tests and seismic response analyses. The shaking table tests were conducted on four 1/6 scaled specimens of the same design and geometry as the full-scale specimens in our previous studies. In addition to unreinforced rammed earth and stone masonry structures, reduced-scale mesh-wrap retrofitted specimens of these structures were also investigated because of their effectiveness, as shown in our previous studies. For numerical modelling, two-mass system models for each specimen were constructed based on the microtremor measurements conducted for each specimen before the shaking table tests and static full-scale lateral loading tests. Seismic response analysis, using two-mass system models, was conducted to simulate the dynamic behavior observed and recorded during the shaking table tests. From the test results, the changes in vibration characteristics were clarified.

2 Shaking Table Tests

2.1 Specimen Configuration

The specimens were a reduced-scale prototype of a traditional Bhutanese masonry building, considering the limitation of the shaking table capacity. Eight specimens were constructed: four rammed earth and four stone masonry specimens, and two of each specimen were retrofitted. That is, there were two identical specimens, each with a different excitation direction in the short and long directions. The same materials were used in both the prototype and the models. The specimens were constructed by local craft men following the standard construction procedures in the Kingdom of Bhutan. All specimens were built on a steel base plate with a thickness of 10 mm and later fixed on a shaking table using bolts. The specimens were cured for 30 days after the construction was completed in the open air.

The specimens were geometrically reduced to a scale of 1:6. As shown in Fig. 1, the two-storied prototype building had a floor area of 1350 mm $\times$ 900 mm, with a height of 580 mm for each floor. The wall thickness was 100 mm. After completing the construction, two of each specimen types were retrofitted with a mesh wrapping technique same as in the previous studies [1, 2]. As shown in Fig. 2, a hexagonal shaped chicken wire mesh having 0.4 mm diameter was used as the main material for retrofitting and anchored using nails. 10 mm thick cement plasters with a 1:3 cement–sand ratio were used to provide better bonding between the walls, mesh, and plasters.

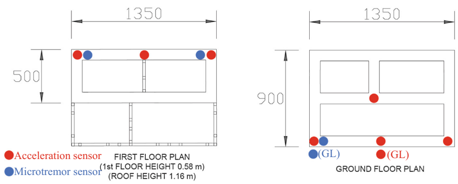


Fig. 1. Plan of specimen.

Fig. 2. Reinforcement with wire mesh.

2.2 Outline of Shaking Table Tests

The shaking table tests were performed on two specimens at the same time: one was unreinforced, and the other was retrofitted. Comparisons were made on the spot, as shown in Fig. 3. The response of the structure was measured using an accelerometer (STP-300S), and data logging was conducted using the National Instrument System (Signal Express). A total of 16 accelerometers were used to measure the response of the two specimens: eight on the unreinforced specimen and eight on the retrofitted specimen. For each specimen, one sensor was installed at the base, four at the second-floor level, and three at the roof level, as shown in Fig. 1. The sampling frequency was set to 200 Hz.

The test was performed only in one direction of the specimens and was subjected to two types of dynamic excitations: sweep sine waves and real earthquake inputs. A sweep test was carried out with a low intensity (0.03 g by gradually increasing the frequency from 1 Hz to 25 Hz to obtain the vibration characteristics of a model in the elastic range. Following the sweep test, a series of earthquake motions with increasing intensities were used for testing in the nonlinear range. The earthquake ground motion recorded in Thimphu, Bhutan, on September 12, 2018, was used as the input motion. The original wave was scaled following the similitude rule to suit the reduced-scale specimen [3], that is, the time axis of the original wave was reduced by a factor of $6^{-3/4}$ times. The test was performed with increasing the maximum acceleration of a ground motion from 0.2 g to1.4 g in 0.2 g increments. When the specimen was about to collapse, the test was stopped halfway through. Figure 4 and Fig. 5 show the time history and acceleration response spectra of the input wave of 0.2 g.

2.3 Results and Discussion

The acceleration response factors at the floor and roof levels, crack patterns after tests in the short and long directions, and changes in natural frequencies are shown in Fig. 6, 7, 8 and 9, respectively. The acceleration response factors were obtained from the average absolute peak acceleration response at each level by the absolute peak acceleration at

Fig. 3. Installation of specimen.

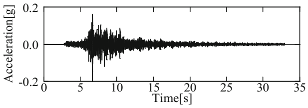


Fig. 4. Example of time history.

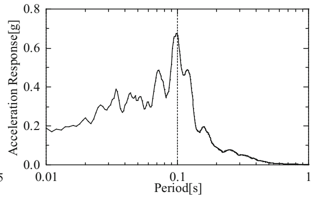
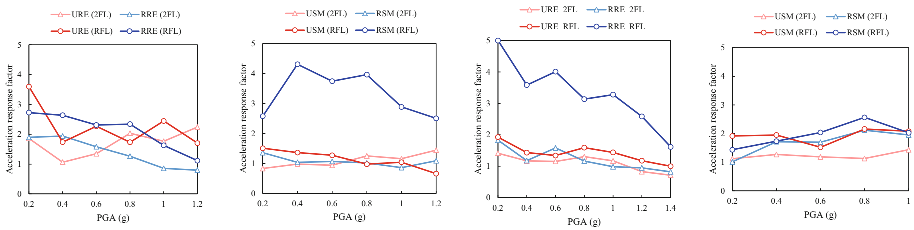


Fig. 5. Example of acceleration spectrum.

the shaking table level. The measured values were used for the analysis after baseline correction and filtering using the band-pass filtering technique with cut-off frequencies of 1–50 Hz. Microtremor measurements of the specimens were performed before and after each test to understand the change in the vibration characteristics. The data loggers and sensors used for the measurement were GEODAS 15 h and CR 4.5-2S velocity sensors, respectively. Microtremor waves were measured over 3 min by setting four sensors at each level in both directions, as shown in Fig. 1, at a sampling frequency of 200 Hz. Subsequently, the measured values were divided by 20.48 s after filtering using the band-pass filtering technique with cut-off frequencies of 1–50 Hz. Those portions with less noise, such as those caused by traffic vibrations, were subjected to ensemble averaging and smoothing (Hanning Window:30). The obtained records were Fourier-transformed and the natural frequency of the specimen was estimated from the ratio of the Fourier spectra at each measurement point to that at the roof level.

The test results for the short direction are as follows: After a nominal PGA of 0.8 g for an unreinforced rammed earth specimen (URE), the acceleration factor became almost the constant between the floor and roof level, and the natural frequency drastically dropped because of the occurrence of large horizontal cracks at the floor level and huge vertical cracks in the middle of the back wall (Fig. 7a). In a retrofitted rammed earth specimen (RRE), the acceleration factors gradually decreased at both the floor and roof levels and approached 1.0, owing to the effect of rocking. However, the natural frequency was almost constant, and there were few cracks although the nominal PGA increased in steps (Fig. 7b). In an unreinforced stone masonry specimen (USM), the acceleration factor was close to 1.0, at the roof level from the beginning owing to initial cracks, and the natural frequency was almost constant despite the gradual increase in the number of cracks. After a nominal PGA of 0.8 g, the natural frequency drastically decreased because of the occurrence of large horizontal cracks on both sides at the floor level (Fig. 7c). In a retrofitted stone masonry specimen (RSM), the acceleration factors at the roof level and the natural frequency gradually decreased, although almost no cracks appeared on the surface of the exterior walls. (Fig. 7d).

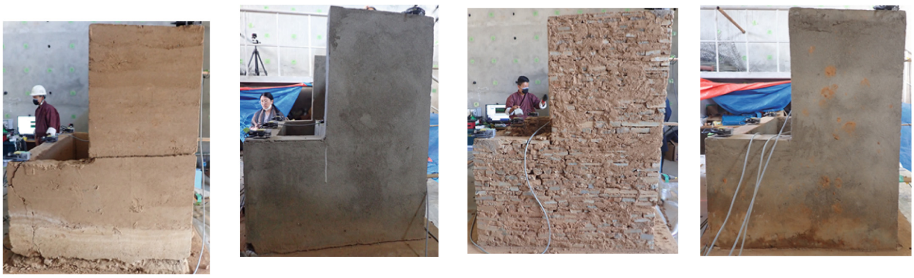
The test results for the long direction are as follows: In an unreinforced rammed earth specimen (URE), the acceleration factors at both the floor and roof levels were almost constant, and almost no cracks appeared on the surface of the exterior walls (Fig. 8a), although the natural frequency gradually decreased. In a retrofitted rammed earth specimen (RRE), the acceleration factors gradually decreased at the roof level owing to the effect of rocking, especially after a nominal PGA of 1.0 g. On the other



(a) Rammed earth (Short direction)
(b) Stone masonry (Short direction)
(c) Rammed earth (Long direction)
(d) Stone masonry (Long direction)

Fig. 6. Acceleration response factor.

hand, the natural frequency was almost constant, and there were few cracks on the surface of the exterior walls (Fig. 8b). In an unreinforced stone masonry specimen (USM), the acceleration factors at the roof level and the natural frequency gradually decreased until a nominal PGA of 0.6 g. However, after a nominal PGA of 0.8 g, these values began to increase owing to the prominence of wall vibration in the out-of-plane direction at the first floor (Fig. 8c). In a retrofitted stone masonry specimen (RSM), the natural frequency gradually decreased and there were few cracks on the surface of the exterior walls, even though the nominal PGA increased in steps (Fig. 8d).

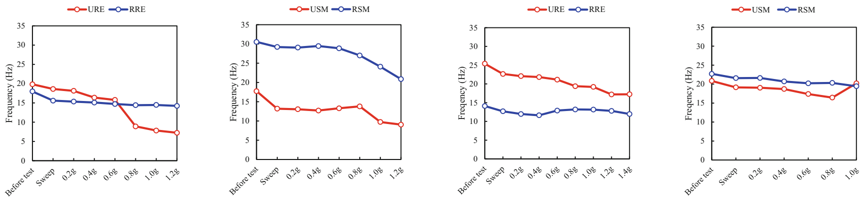


(a) URE
(b) RRE
(c) USM
(d) RSM

Fig. 7. Crack patterns after tests (Short direction).

(a) URE
(b) RRE
(c) USM
(d) RSM

Fig. 8. Crack patterns after tests (Long direction).



(a) Rammed earth (Short direction) (b) Stone masonry (Short direction) (c) Rammed earth (Long direction) (d) Stone masonry (Long direction)

Fig. 9. Change of natural frequencies.

3 Seismic Response Analyses

3.1 Analysis Model

In this study, a simple numerical analysis was performed using a two-mass system model to simulate the behavior during the shaking table tests. For each specimen, an analysis model was constructed based on the results of the microtremor measurements for each specimen before the shaking table tests and static full-scale lateral loading tests [1, 2].

For the mass values m_1 and m_2, unit weights of 1700 kg/m^3 and 2600 kg/m^3 were assumed for the rammed earth and stone masonry walls, respectively. The values of stiffness k_1 and k_2 were calculated based on the natural frequencies of the reduced-scale specimens immediately before the shaking table tests. The stiffness ratios k_1 and k_2 were assumed to be $k_2 = 1/2k_1$ and $k_2 = 1/3k_1$ in the short and long directions, respectively, based on the length of the in-plane walls. Rayleigh damping was applied with the first and second damping ratios assumed to be 0.02. The damping value was determined from the results of microtremor measurements before the shaking table tests based on the random decrement technique [4].

The skeletal curves for the rammed earth and stone masonry specimens, both unreinforced and retrofitted in the short and long directions, were assumed to be perfect elastic-plasticity models. Based on the results of static full-scale lateral loading tests [1, 2], the maximum story shear coefficient was used as the yield shear coefficient of a perfect elastic-plasticity model. To perform the analysis on the reduced-scale specimen, the yield shear coefficients of the full-scale tests were transformed to obtain corresponding values [3], that is, the maximum story shear coefficient of the full-scale specimen was multiplied six times. Because there is no common hysteresis model for nonlinear analysis of composite masonry structures, the Takeda model was used for this analysis.

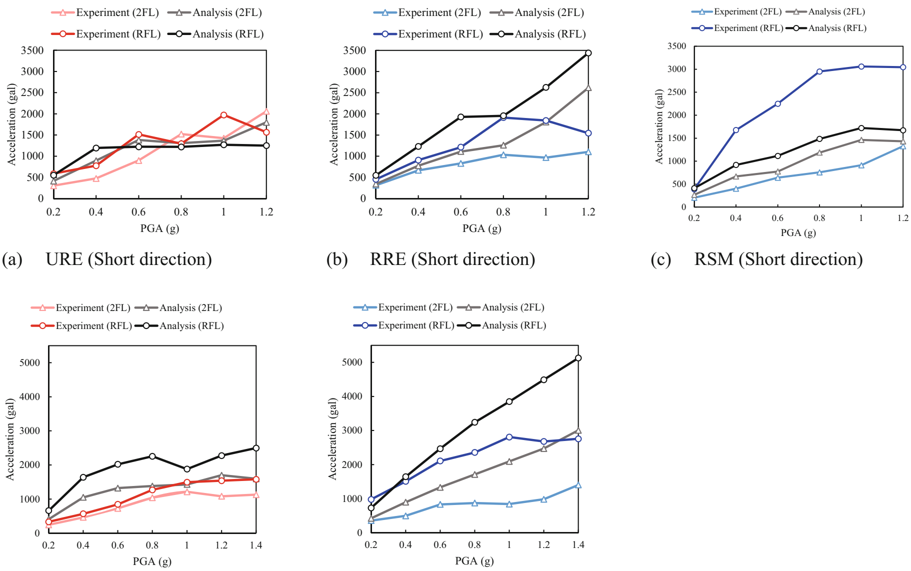
3.2 Analysis Method

In this study, the ground motion from shaking table tests was used as an input earthquake. All input ground motions were simultaneously used in this analysis to consider the effects of residual deformation. Five nonlinear models were subjected to time history analysis, except for the unreinforced stone masonry specimens in the short direction and unreinforced and reinforced stone masonry specimens in the long directions without the results of full-scale static loading tests. The Newmark-β method was used to compute the acceleration responses, and the maximum response acceleration for each test case was calculated.

3.3 Results and Discussion

The experimental results were compared with the numerical analysis results for the acceleration response at each floor level. The maximum acceleration response of the experimental result is the average absolute maximum acceleration accounting response of all sensors on each floor. Furthermore, the numerical results were correlated with the damage occurrence on the specimens during the experiment. The comparison results for each input of the nominal PGA for each specimen are shown in Fig. 10.

The comparison results for the short direction are as follows: At nominal PGA from 0.2 g to 0.8 g for the unreinforced rammed earth specimen (URE), all the acceleration response is in a close range and during the experiment there was no significant damage detected. From the experiment, at a nominal PGA of 0.8 g, large horizontal cracks developed at the floor level, and the analysis results were smaller than the experimental results at the floor level. At nominal PGA from 0.2 g to 0.8 g for the reinforced rammed earth specimen (RRE), all the acceleration response is in a close range. However, for a nominal PGA of 0.8 g, the analysis results are larger than the experimental results at the floor level owing to the effect of rocking. At each nominal PGA for the reinforced stone masonry specimen (RSM), the analysis results were smaller than the experimental results at the roof level, even though almost no cracks appeared on the surface of the exterior walls. It is possible that the initial cracks inside the wall were affected.



(a) URE (Short direction) (b) RRE (Short direction) (c) RSM (Short direction)

(d) URE (Long direction) (e) RRE (Long direction)

Fig. 10. Comparison of maximum acceleration.

The comparison results for the long direction are as follows: At each nominal PGA for the unreinforced rammed earth specimen (URE), the analysis results were larger than the experimental results at the roof level, although almost no cracks appeared on the surface of the exterior walls. There is a possibility that the prominence of the wall vibration in the out-of-plane direction on the first floor is affected. At low nominal PGA from 0.2 g to 0.6 g for the reinforced rammed earth specimen (RRE), all the acceleration response is in a close range. However, as the nominal PGA increased, the difference between the experimental and analytical results increased at each level owing to the rocking effect.

4 Conclusions

To understand the behavior of structures during earthquakes, this study aimed to study the dynamic characteristics of composite masonry buildings through shaking table tests and numerical analysis.

The natural frequency of the specimen showed a decreasing trend with an increase in the nominal PGA, reflecting the degradation of the stiffness due to damage. The acceleration factor also reflected damage propagation, where a decrease in factors occurred when the damage was significant. However, there is no clear tendency of the relationship between the nominal PGA and acceleration response factor owing to the variation in measurement results or the effect of rocking.

A comparative analysis was performed between the numerical analysis and the experiments in terms of the maximum acceleration response at each floor level relative to the nominal PGA. The results showed that the numerical analysis produced a similar output trend until rocking or large horizontal cracks occurred.

Acknowledgement. This research was supported by the JST/JICA, SATREPS (Science and Technology Research Partnership for Sustainable Development) project. We would also like to acknowledge the support of the Department of Culture, Ministry of Home and Cultural Affairs, and the Royal Government of Bhutan.

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