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SEISMIC PERFORMANCE OF UNREINFORCED MASONRY BUILDING IN LOW SEISMICITY REGION

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SUMMARY

The shaking table test results of a 1/3-scale model of a two-story unreinforced masonry structure typically used in constructing low-rise residential buildings is discussed in this paper. This test was performed to provide a better understanding of the seismic behavior of URM structures in Korea of low seismicity region. The test structure was symmetric about the transverse axis but asymmetric to some degree about longitudinal axis and had a relatively strong diaphragm of concrete slab. The test structure was subjected to a series of different levels of earthquake shakings that were applied along the longitudinal direction. The results of the shaking table test revealed that the shear failure was dominant on a weak side of the 1st floor while the upper part of the test model remained as a rigid body. The measured dynamic response of the test structure has been analyzed in terms of various global parameters (i.e., floor accelerations, base shear, floor displacements and story drift, and torsional displacements and torsional moment) and correlated with the input table motion. Moreover, representative dynamic characteristics are developed toward an equivalent single degree of freedom model. The results of this study will provide useful information in constructing fragility curves for URM structures in low seismicity regions.

INTRODUCTION

Most URM residential buildings in Korea were not engineered but were built mainly depending on past experience. Moreover, the construction of URM structures vary from country to country in many aspects including material properties, layering methods, and floor systems. Therefore, despite a considerable amount of research in the behavior of masonry structures subjected to seismic actions carried out in many countries during last decades (Abrams 1995; Calvi 1994; Costley and Abrams 1996), it seems difficult to extrapolate the previous research results to predict seismic behavior of URM structures unique in Korea. Most URM structures in Korea are characterized by masonry shear walls with more than two sides and many openings on exterior walls and rigid diaphragms of concrete floor slabs.

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The main objective of this study was to investigate seismic behavior and damage patterns of URM buildings constructed according to the traditional construction procedure in Korea. Specific objectives include the followings:

- Provide experimental data on the seismic response of URM structure subjected to earthquake loading for use in refinement of design code and development of performance criteria.
- Observe how the structural characteristics change during shaking.
- Develop a load-displacement curve for use in the development of an equivalent single degree of freedom (SDOF) system.

To achieve the objectives of this study, a two-story typical URM building was selected as a prototype and reduced to 1/3-scale so that shaking table test was possible. A series of tests was performed using the Taft 1952 N21E ground acceleration component as an appropriate base acceleration for the test structure. The chosen earthquakes range from minor-moderate, to moderate-severe, and to severe ground motions, in terms of structural damage state. The instruments including 9 accelerometers and 16 displacement measuring differential transformers were used to monitor structural response quantitatively.

This paper discusses the seismic behavior of URM structure of rigid diaphragm based on the experimental findings and the implications of these observations to design consideration.

TEST MODEL AND TESTING METHOD

Test Model

A two-story URM structure of one-bay by two-bay that is typically used in constructing low-rise residential buildings was selected as prototype of a building considering simplicity and completeness of the model. The structure consisted of masonry shear walls with double wythe cement bricks and concrete floor slabs of 12cm in thickness. Many openings are made on exterior walls which may cause soft story failure and consequently lead to a 1st story failure when such weakness starts in the first floor where more shear forces are induced. The plan of the prototype structure was symmetric about the transverse axis but asymmetric to some degree about longitudinal axis and this asymmetry may cause torsional behavior which would add undesired force due to the torsion. The concrete slabs considered as rigid diaphragms provide large friction force proportional to their gravitational loads against lateral loading but yield large acceleration force during excitation of the structure. While the former help to reduce the effects of crack openings in some ways, the latter increase the lateral loading against that the structure must resist.

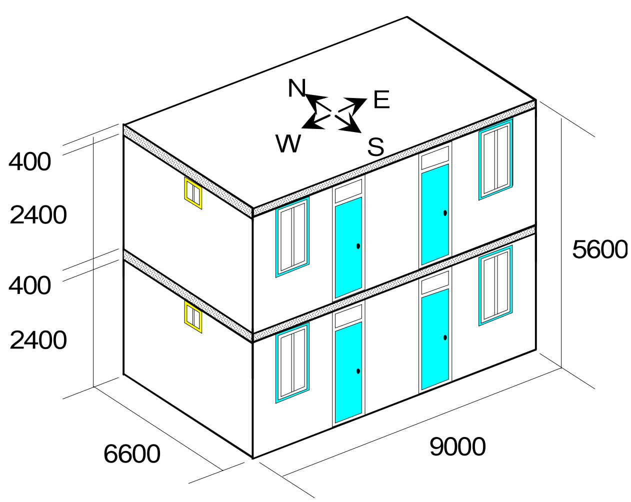


Fig. 1 Typical URM Structure Considered (unit:mm)

Fig. 1 shows a basic configuration of the prototype structure and its dimensions in 1/3 scaled size. This study conducted unit compression tests, prism compression tests, and diagonal tension tests for the prototype and 1/3 scale model units. The test structure, one-bay by two-bay (in the shaking direction), was built according to the construction procedure recommended in structural design part of Korean Standard code (KBC 2000). The KS code limits the height of layered bricks each construction time which can not exceed the half of story height each day.

Testing Method

The instruments including accelerometers and displacement measuring differential transformers were used to monitor structural response quantitatively in terms of time-signal data. Fig. 2 shows the location of measuring instruments placed at the floor level: A1~A9 represent the accelerometers and L1~L6 represent the linear variable differential transformers (LVDT). It should be noted that the locations of LVDTs were designed to obtain both translational and torsional responses of the test structure due to asymmetry about one axis while the number of instrumentation was kept not to exceed the limited capacity of data acquisition system. The data collection was run at exactly 50 Hz with 0.02sec. The primary dynamic test motion applied at various amplitudes through the shaking table was an acceleration record of the Taft N21E component of the 1952 Kern County, California, earthquake. This earthquake was selected because of its broad frequency spectrum and long duration of strong amplitude motion. The shaking table's amplitude of motion was continuously increased by varying its EPGA from 0.05g up to a maximum 0.35g in terms of a scaling factor. The loading direction was West-East about which axis the test structure was asymmetric.

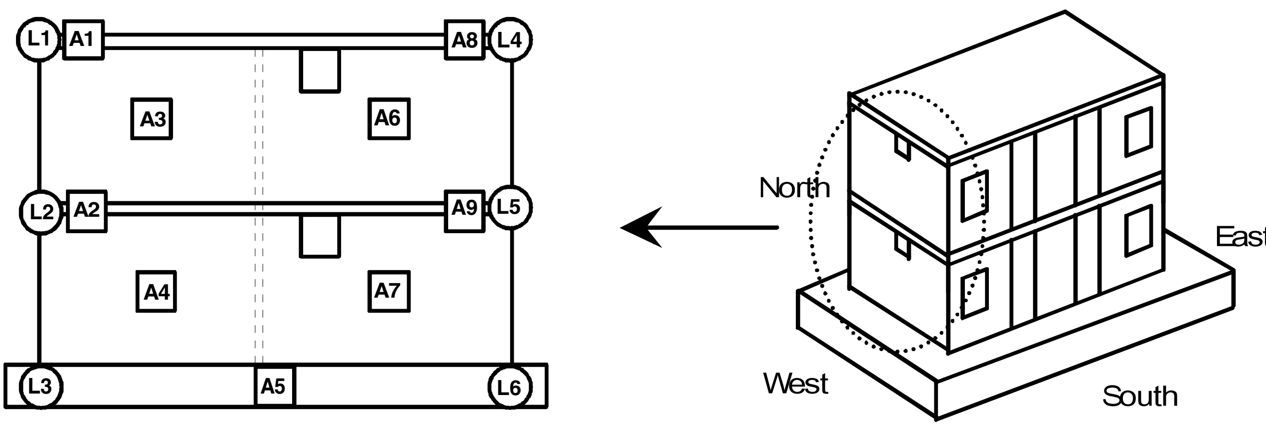


Fig. 2 Test Instrumentation

DYNAMIC TEST RESULTS

The experimental results presented here are based on observations made during the experimental phase and measured data. To help understand the dynamic behavior of the test structure the following figures are prepared: Fig. 3 for displacement time histories; Fig. 4 for acceleration time histories; Fig. 5 for torsional time histories and Fig. 6 for hysteretic curves. In order to simplify the prediction of the seismic resistance of the unreinforced masonry building, the actual hysteretic behavior is represented by a dynamic resistance envelope as shown in Fig. 7. The envelopes consist of representative pairs of base acceleration and top displacement for several time segments. Note that each pair was not concurrent in time but was peak values for each time segment. The peak values for structural response parameters are summarized in Table 1 for comparison of the motion during the prime tests.

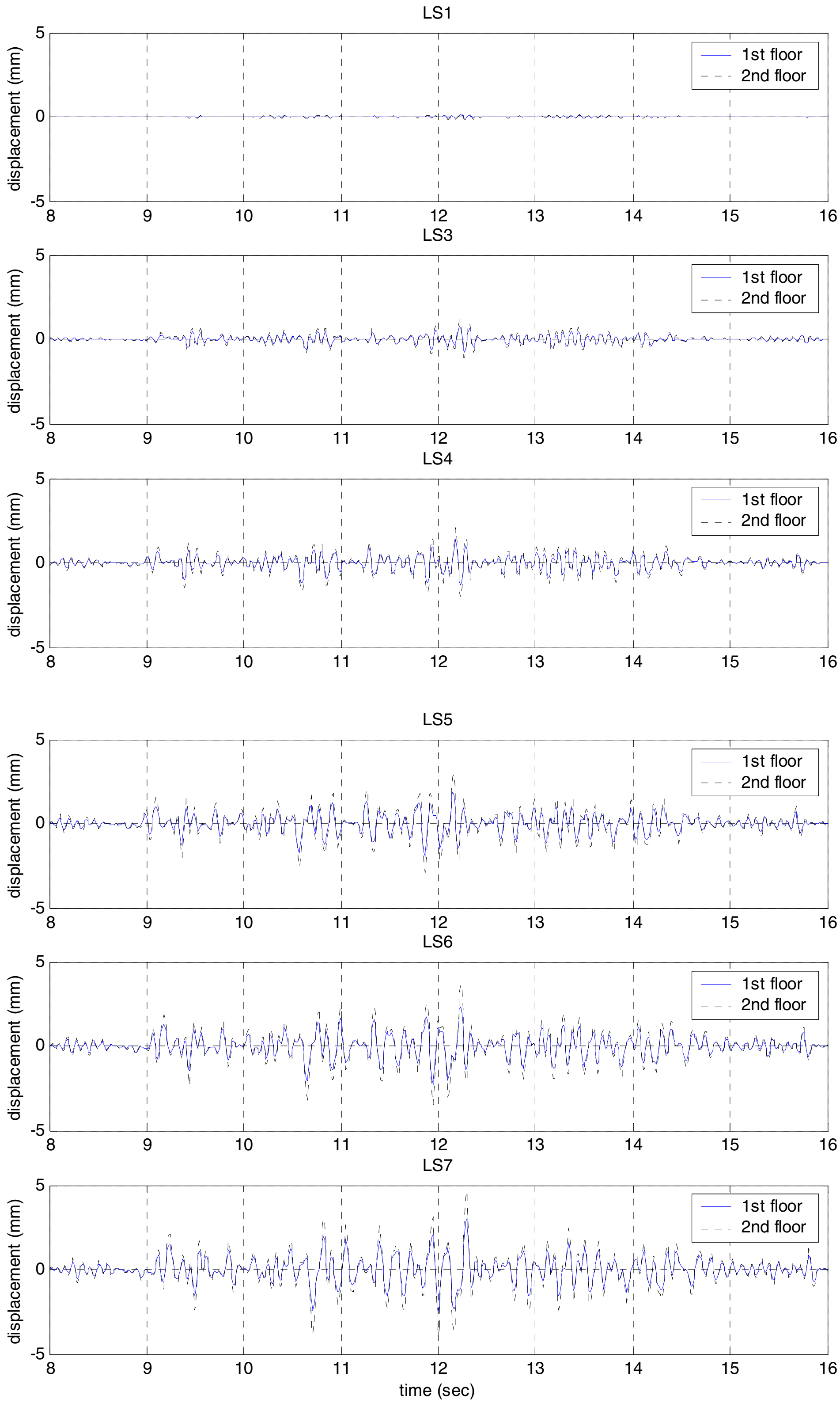


Fig. 3 Story Displacement Time Histories

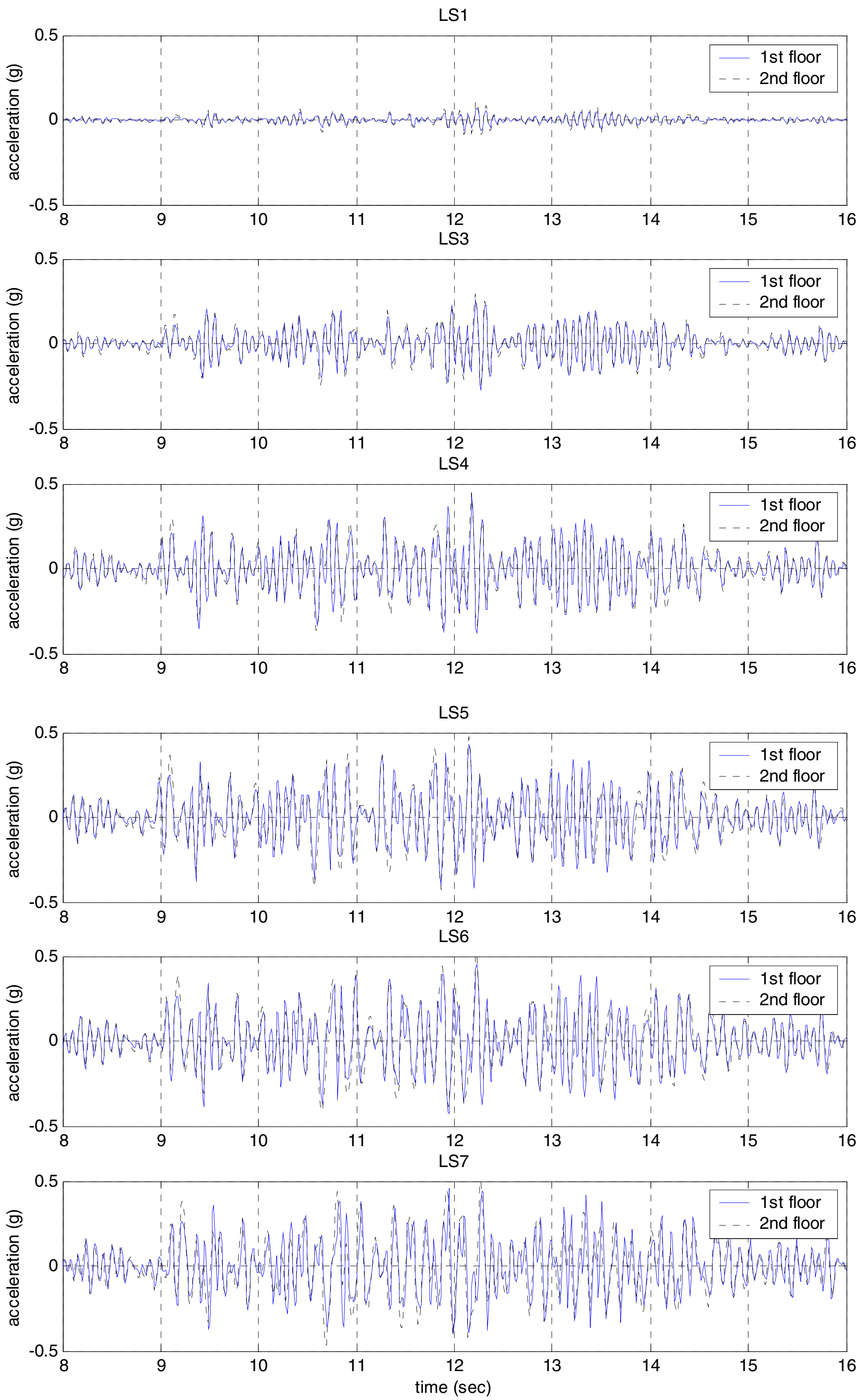


Fig. 4 Story Acceleration Time Histories

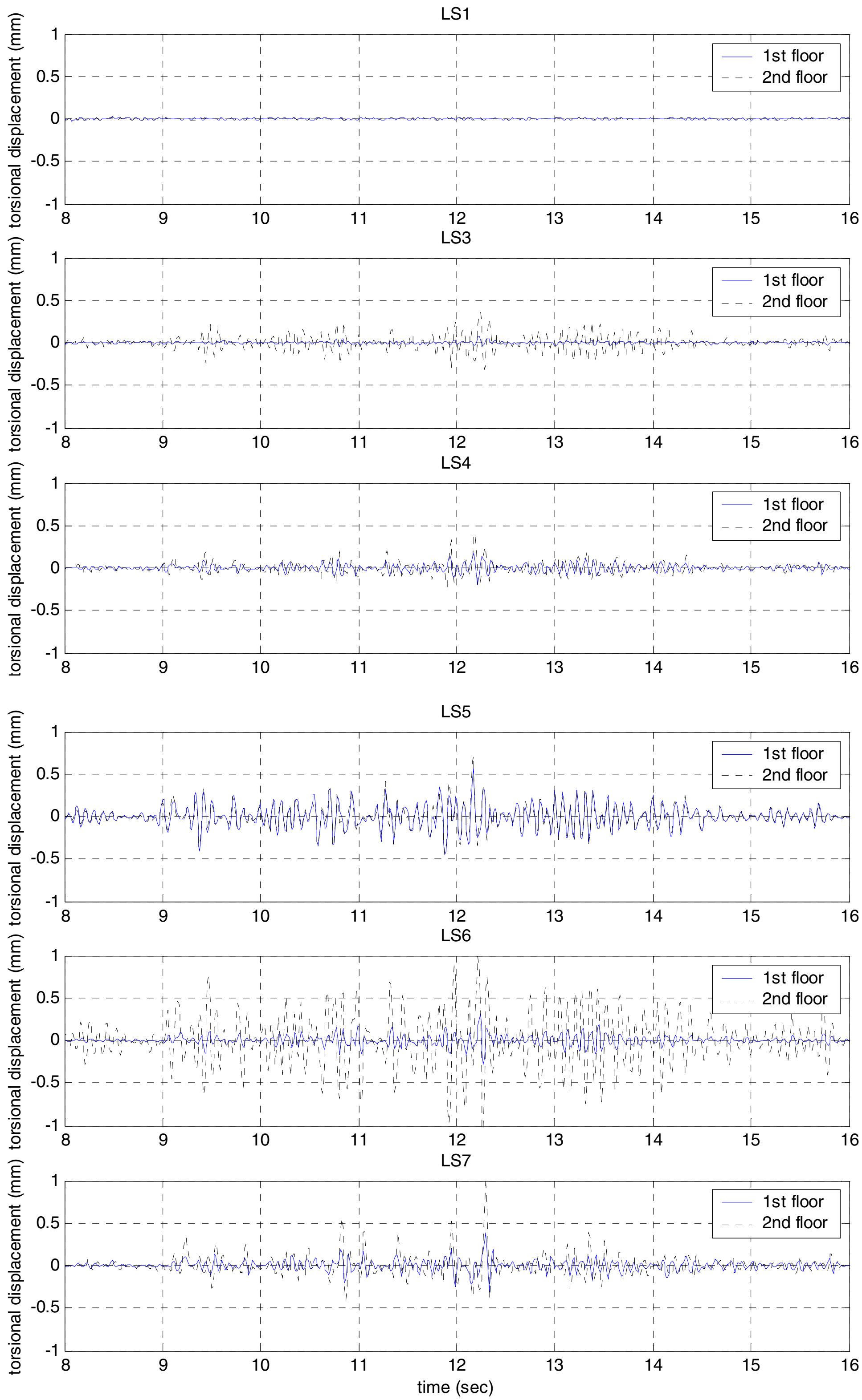


Fig. 5 Torsional Deformation Time Histories

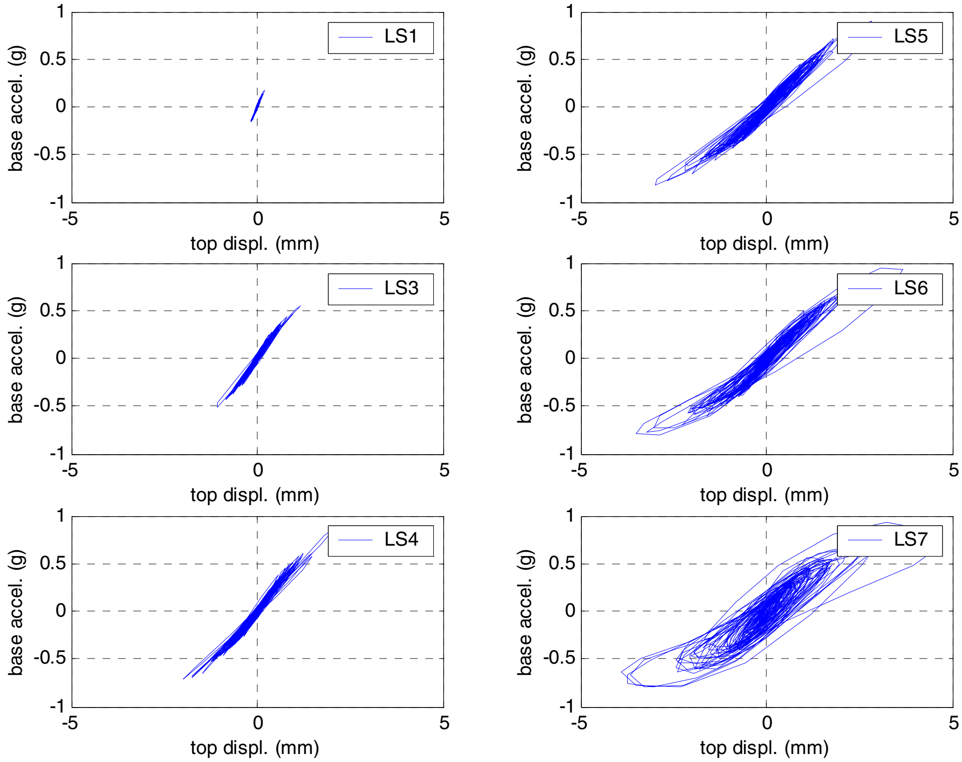


Fig. 6 Base Acceleration vs. Top Displacement

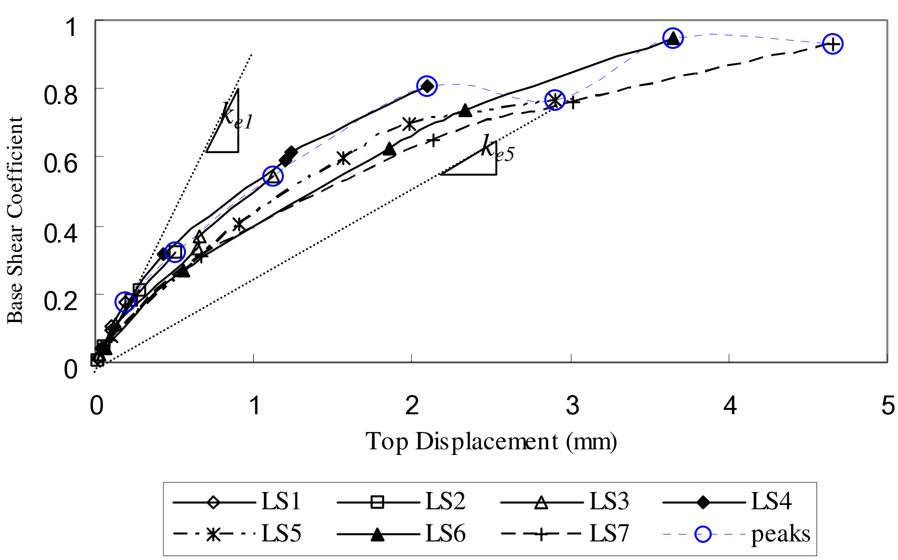


Fig. 7 Dynamic Resistance Envelope

Table 1 Maximum Response from Experimental Test Results

Test Run	Earthquake Description	Story	Max. Interstory Drift, %	Peak Story Acceleration, g
LS1	Minor shaking 0.05g	2^{nd} 1^{st}	0.008 0.014	0.107 0.071
LS2	0.1g	2^{nd} 1st	0.028 0.041	0.200 0.154
LS3	0.15g	2^{nd} 1^{st}	0.046 0.088	0.294 0.268
LS4	Moderate shaking 0.2g	2^{nd} 1^{st}	0.083 0.153	0.446 0.432
LS5	0.25g	2^{nd} 1^{st}	0.138 0.220	0.470 0.423
LS6	Severe shaking 0.3g	2^{nd} 1^{st}	0.151 0.264	0.528 0.448
LS7	0.35g	2^{nd} 1^{st}	0.186 0.338	0.540 0.457

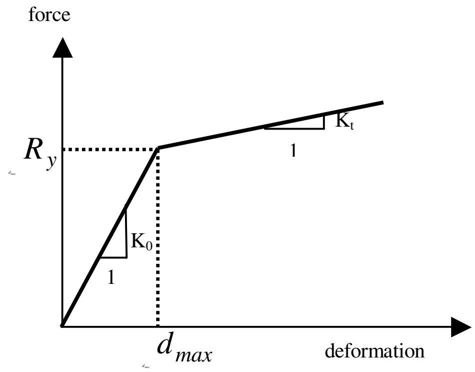
With regard to the damage patterns, Fig. 8 presents three different cases for three different levels of excitation. For minor earthquake, the separation was occurred between the floor slab and the masonry wall in run of LS2, as shown in Fig. 3a. For moderate earthquake, some significant damage was occurred on the exterior wall of the North side in run of LS5, as shown in Fig. 3b. For severe earthquake, the cracks were propagated far more and the interior walls were damaged seriously in run of LS7, as shown in Fig. 3c. After the completion of the series of test, it was evident that the shear failure was dominant for the 1^{st} floor, and then the upper part of the model behaved as a rigid body.

a. After the run of LS2 b. After the run of LS5 c. After the run of LS7

Fig. 8 Structural Damage Patterns

APPROXIMATION OF BILINEAR CURVE

Toward the development of an equivalent SDOF system, a bi-linear stiffness degrading model shown in Fig.9 was adopted to characterize the test structure's force-deformation relationship. The completion of the model needs of defining its yield resistance R_y, its initial elastic stiffness K_0, the maximum displacement, d_{max}, (not necessarily the maximum displacement during the interval under consideration), and its tangential stiffness K_t.



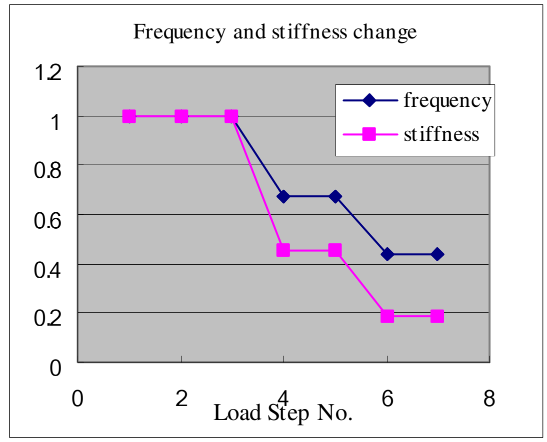


Fig. 9 Bi-linear Model for an Equivalent SDOF System Fig. 10 Frequency Changes through Load Steps

However, it is not easy to develop the bi-linear model using the experimental test results, which is still in progress in this study. This study currently focused on the use of frequency values of the test results to address such problem considering that frequency changes are very closely related to stiffness changes. Fig. 10 plots the dominant frequencies for each test run. The reduction of the fundamental vibration frequency was first made from the run of LS4 which was about 23%. It is evident that the structure suffered a noticeable degradation in its lateral stiffness and probably went through several cycles of inelastic deformation. The further reduction of the fundamental frequency did not occurr during the run of LS5, which indicated that in this case the stiffness degradation was not so dramatic. In the run of LS6 the fundamental vibration frequency was one more time reduced significantly about 20%. Again, the structure suffered another noticeable degradation in its lateral stiffness. To investigate the nonlinearity of the structure in more detail, the segmental FFT analysis was made by moving the time window along the whole duration of the test run LS7, as summarized in Table 2. Note that the linearization of the response for a single degree of freedom system was used to relate the lateral stiffness to its vibration frequency, which will be discussed later in this section.

Table 2 Frequency Changes and the Corresponding Stiffness Changes

SEG No.	Time Interval, sec	f_i (Hz)	f_i / f_0	K_i / K_0
1	0-7.5	12.909	1.000	1.000
2	7-12	11.127	0.862	0.743
3	9-14	6.784	0.526	0.276
4	11-16	5.320	0.412	0.170
5	15-20	5.539	0.429	0.184

Upon the investigation of the frequency changes structural damage patterns, this study determines the parametric values for the bi-linear curve temporarily as follows:

- The initial elastic stiffness, K_0, was determined by using the linearization of the response for a single degree of freedom system between the lateral stiffness, K, and its vibration frequency, f, as follows:

$$f = \frac{1}{2\pi}\sqrt{\frac{K}{M}} \tag{1}$$

Then, the elastic stiffness is calculated from the following equation

$$K = M(2\pi\ f)^2 \tag{2}$$

- The maximum displacement, d_{max}, takes the maximum top displacement occurred during LS3 after which the frequency value was greatly reduced.
- Given that the elastic stiffness and the maximum displacement values are known, the yield resistance R_y is then determined by multiplying the elastic stiffness by the maximum displacement.
- The slope of the tangential stiffness may be determined grounding on the frequency change, as investigated in Table 2. Finally, the parameter values for the bilinear curve for URM tested in this study are summarized in Table 3.

Table 3 Parameter Values for the Bi-linear Curve for URM

Parameter	Value	Remarks
K_0	7719 kN/mm	$K_0 = M(2\pi\ f_0)^2$ where, $M = 15.6\,kN - sec/mm$, $f_0 = 11\ \ Hz$
K_t	1543.8 k/mm	$K_t = 0.2\ K_0$
d_{max}	1.2 mm	Arbitrarily selected from the data of LS3
R_y	9263 kN	$R_y = K_0 \cdot d_{max}$

In order to verify the bi-linear curve discussed in this study, Fig. 11 compares the bi-linear curve defined using the values of Table 3 and the positive envelopes. This figure shows reasonable agreement between the data of the bi-linear curve and corresponding envelope data.

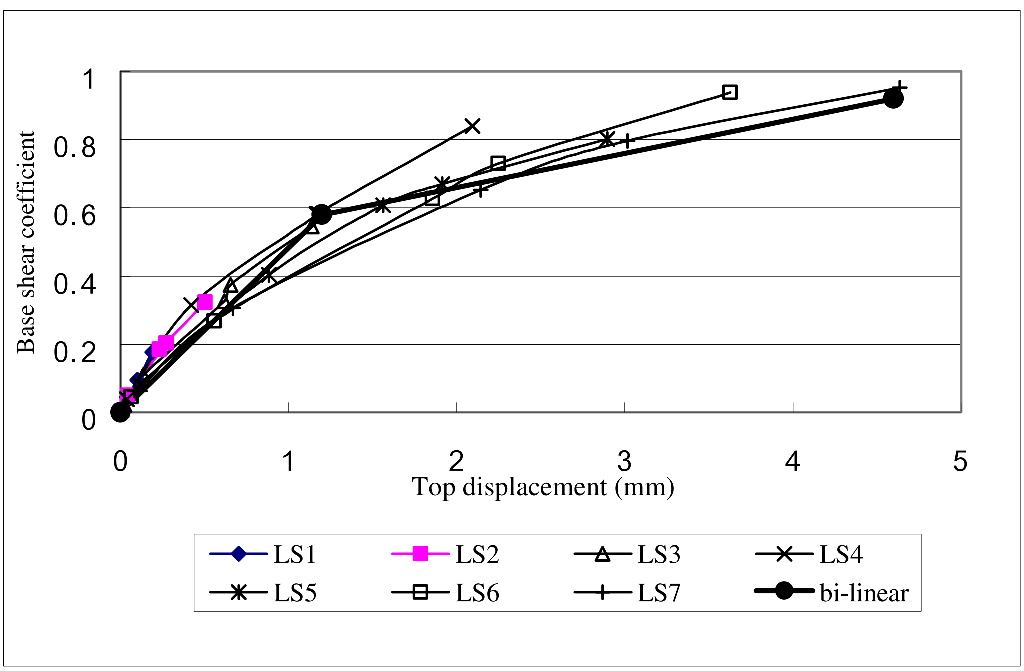


Fig. 11 Bi-linear curve on envelopes of positive peak values

CONCLUDING REMARKS

This study focused on the seismic behavior of unreinforced masonry structure typical in Korea of low seismicity. The 1/3-scaled two-story structure was tested under simulated seismic excitation using the shaking table. The investigation into the test results revealed the followings:

- The shaking table test data are useful to trace the changes of structural characteristics from elastic state to post-cracking state.
- The instrumented test data that generally contain a lot of noise could be filtered through a reasonable signal processing procedure.
- The URM behaved well after the crack was closed to the original position. When the test structure that was once damaged but restored to the original shape subjected to the next level of loading, the structure was able to resist against almost about the same capacity up to the previous load level. That is, the envelope curves in early parts of each loading step were very close to each other.
- The shear failure was dominant for the 1st floor, and then the upper part of the model behaved as a rigid body.
- It was found from the testing that measured deflected shapes were nearly in phase despite the amplitude of motion. This observation can justify the use of a single degree of freedom model for the nonlinear dynamic response analysis. With such indication, a bi-linear model has been proposed. The development of the bilinear curve for an equivalent SDOF system for the URM was possible using the natural frequency changes during the excitation of the structure.
- The overall torsional deformation was increased as the amplitude of the shaking table motion was increased. Especially, the torsional deformation at 2nd floor was much larger than that of the 1st floor right after more pronounced structural damage was occurred.
- Substantial strength and deformation capacity still existed after cracking.
- There were no out of bending failures in the walls perpendicular to the loading direction.

This study focused mainly on the investigation on the propagation of damage and failure mechanism of the test structure using measured data. The equivalent single degree of freedom system resulted from this study provides a good starting point toward the development of fragility curves for unreinforced masonry structures consisting of shear walls and rigid diaphragms.

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