Dynamic Response of Unreinforced Masonry Building

비보강 조적조의 동적 거동

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국문요약

본 연구는 우리나라에서 저층 주거용 건물로 널리 사용되고 있는 2층 규모의 비보강조적조 의 1/3 축소 모델에 대한 진동대 실험을 수행한 것이다. 본 연구의 주목적은 내진설계가 이루어지지 않은 조적조 건물의 내진거동을 살펴보고, 실험적 자료를 확보하는데 있다. 실험대상구조물은 횡방향으로는 대칭이지만 종방향으로는 약간 비대칭이고, 비교적 강한 다이어프램을 나타내는 콘크리트 슬래브로 되어있다. 실험체에 대한 모의 지진하중은 가속도를 점차 증가시켜가면서 종방향으로 가력하였다. 실험에서 얻은 구조물의 동적 응답자료는 진동대의 입력지진과 연관지어서 분석하였다. 더욱이 성능기초설계를 위한 성능수준을 제시하였다. 실험결과 1층에서의 전단파괴가 지배적이고 상부층은 강체거동을 보여주었다. 또한 균열 발생후에도 상당한 강도와 변형능력을 보유하고 있는 것으로 나타났다.

주요어 : 진동대 실험, 비보강 조적조 구조물, 동적거동, 성능기초설계

ABSTRACT

The seismic behavior of a 1/3-scale model of a two-story unreinforced masonry (URM) structure typically used in constructing low-rise residential buildings in Korea is studied through a shaking table test. The purposes of this study are to investigate seismic behavior and damage patterns of the URM structure that was not engineered against seismic loading and to provide its experimental test results. The test structure was symmetric about the transverse axis but asymmetric to some degrees about longitudinal axis and had a relatively strong diaphragm of concrete slab. The test structure was subjected to a series of differentlevels of earthquake shakings that were applied along the longitudinal direction. The measured dynamic response of the test structure was analyzed in terms of various global parameters (i.e., floor accelerations, base shear, floor displacements and storydrift, and torsional displacements) and correlated with the input table motion. Moreover, different levels of seismic performance were suggested for performance-based design approach. The results of the shaking table test revealed that the shear failure was dominant on a weak side of the 1stfloor while the upper part of the test model remained as a rigid body. Also, it was found that substantial strength and deformation capacity existed after cracking.

Key words : shaking table test, unreinforced masonry structure, dynamic behavior, performance-based design

1. 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.

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 de-

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본 논문에 대한 토의를 2004년 12월 31일까지 학회로 보내 주시면 그 결과를 게재하겠습니다. (논문접수일 : 2004. 4. 21 / 심사종료일 : 2004. 10. 1)

sign code and development of performance criteria.

- Observe how the structural characteristics change during shaking.

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 6 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.

2. Test Model and Testing Method

2.1 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 1^{st}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 load 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 load against that the structure must resist. Fig. 1 shows a basic configuration of the prototype structure and these dimensions were reduced in 1/3 scaled size considering the similitude laws (Harris and Sabnis 1999) which required additional masses on both floors of the test structure. This study conducted unit compression tests, prism compression tests, and diagonal tension tests for the prototype and 1/3 scale model units and their average values are summarized in Table 1. The test structure, one-bay by two-bay (in the shaking direction), was built ac-

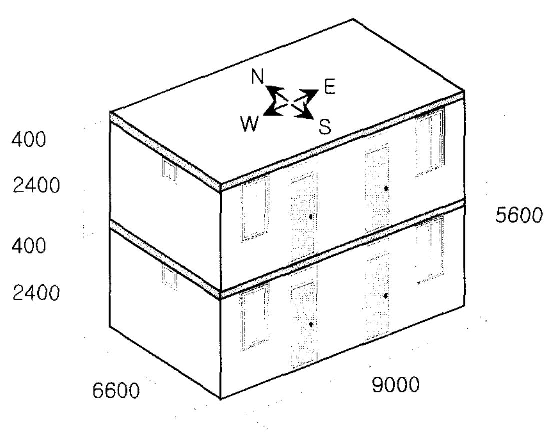


Fig 1 Typical URM Structure (unit:mm)

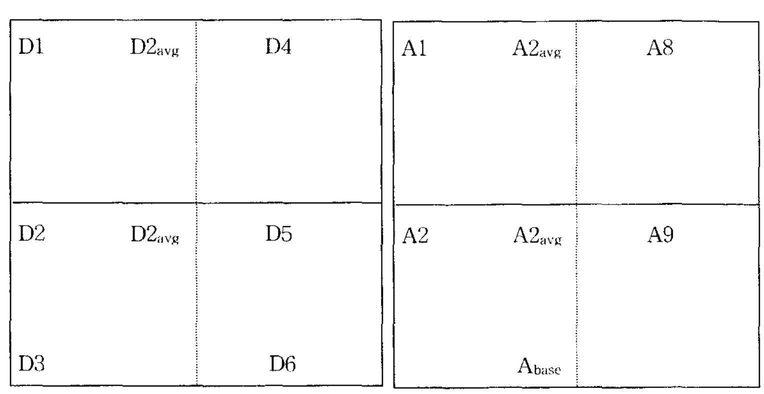


Fig 2 Locations of the signal data on West side wall

cording 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.

2.2 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 D1~D6 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. Since the only data measured consisted of floor accelerations and displacements, the other data needed to represent structural responses must be derived from these measured data. The data collection was run at exactly 50 Hz with 0.02sec. The primary dynamic test motion appliedat 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 axis about which the test structure was asymmetric.

3. Dynamic Test Results

The experimental results presented here are based on observations made during the experimental phase and measured data. The deformation of the test model was measured on the south and north sides at two points on each side at three levels (i.e., at table, at first floor and at second level) including records of D1, D2, D3, D4, D5 and D6. Therefore, the values measured are total displacements and floor displacements with respect to the table, so called relative displacements, were calculated by subtracting the table displacements from the corresponding total displacements. The acceleration of the test model was measured at the corners and middles of south and north sides at two levels (i.e., at first floor and at second level) and at the base, which were 9 points altogether. However, the measured values at the middles of panels were corrupted and thereby only 5 accelerations of the instrumentation of A1, A2, A8, A9 and A_{base}were considered in this study. Also, the accelerations at each floor were averaged out to provide representative story values. Fig. 3, Fig. 4 and Fig. 5 show the time histories of displacements and accelerations for test runs of LS1, LS4 and LS7, which oscillated in phase following similar patterns. The response was thus considered as primarily in the first vibration mode in the north-

Table 1 Measured Mechanical Properties of Test Specimens (kgf/cm^2)

	Brick Compressive Strength	Prism Compressive Strength	Diagonal Tension Strength
Prototype	303.6	116.0	13.9
1/3 scaled model	274.5	138.7	3.0

south direction. The time history response of story acceleration also proves that the first vibration mode is dominant.

The differences between the two translational displacements of the corners at each level showed some torsion. If the test building did not experience some torsion, the two motions would have been identical, because the two points where the translation accelerations recorded were apart about equally from the center of the mass.In this study, the displacement of D4 is larger than D1 this is because that the south side wall where there are more openings is weaker than the north side wall. The torsional deformation at both floors was in similar pattern. After the test structure had experienced serious deformation, the torsional behavior at the 2nd floor became manifest in comparison with that of the 1st floor (Fig. 6: LS6). Because the torsional resistance could not be provided from the 1st floor after more pronounced structural damage was experienced on a weak side of the 1st floor in LS5.

The story shear versus story drift relationships were estimated at four locations where both accelerations and displacements were measured to provide hysteretic characters of the test building. The story shears were computed from the accelerometer data and on the mass blocks multiplied by the proportion of structural mass associated with each accelerometer. Moreover, the base shear was computed by summing up the story shears. Fig. 7 shows the story shear versus story drift relationships. Note that the shear results were normalized as percentage of the total weight of the building. Most of the damage was confined to the bottom story walls, in a "soft-story"type of mechanism. The complexity of the response of the test structures had been increased under larger seismic excitation. Fig. 8 shows the base shear coefficients against top displacements to provide a comprehensive understanding about the whole structure.

The natural frequencies of the time history data obtained during the excitation were computed using the Fast Fourier transform (FFT) analysis. Fig. 9 shows that natural frequencies were reduced as structural damage increased. The reduction of the fundamental vibration frequency was first made during the run of LS4 especially 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 was not occurred 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 structuresuffered another noticeable degradation in its lateral stiffness. Then, the same pattern of the frequency change of LS6 was appeared during the run of LS7, which indicated that in this case the stiffness degradation was not seriously changed.

With regard to the damage patterns, Fig. 10 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. 10a. For moderate earthquake, some significant damage was occurred on the exterior wall of the North side in run of LS5, as shown in Fig. 10b. For severe earthquake, the cracks were propagated far more and the interior walls were damaged seriously in run of LS7, as shown in Fig. 10c. After the completion of the series of test, it was evident that the shear failure was dominant for the 1stfloor, and then the upper part of the model behaved as a rigid body.

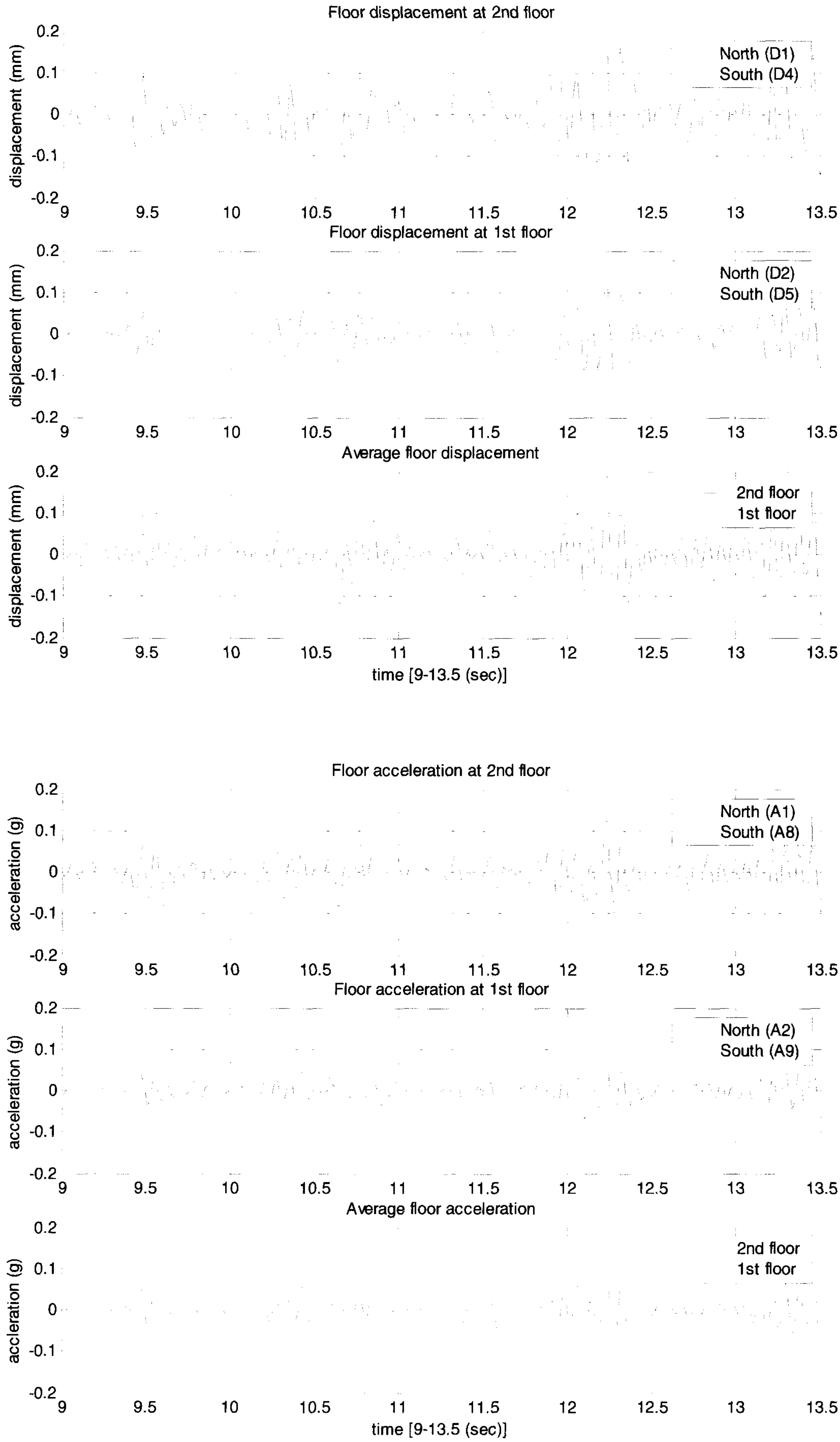


Fig 3 Floor Displacement and Acceleration Time Histories (LS1)

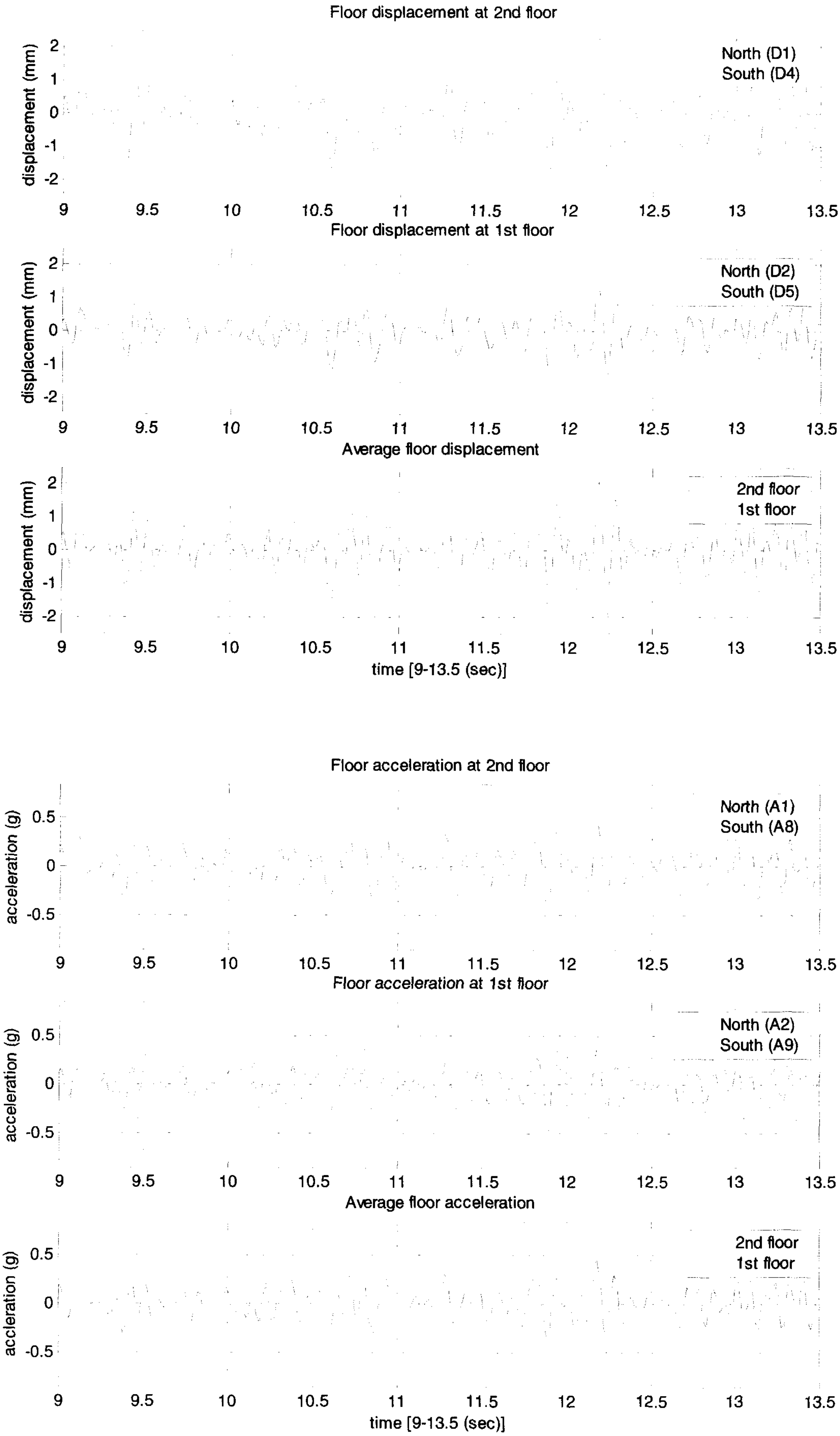


Fig 4 Floor Displacement and Acceleration Time Histories (LS4)

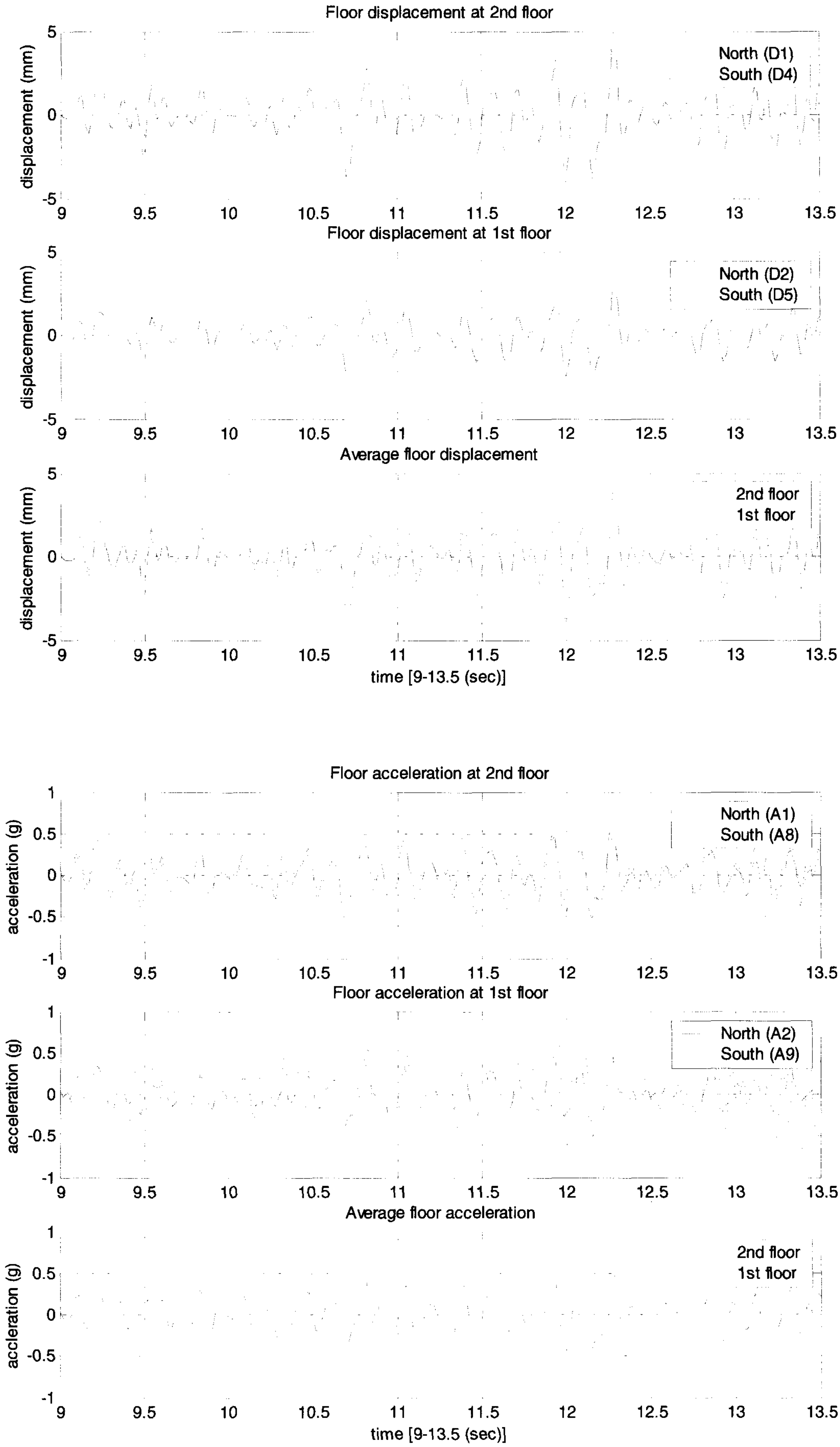


Fig 5 Floor Displacement and Acceleration Time Histories (LS7)

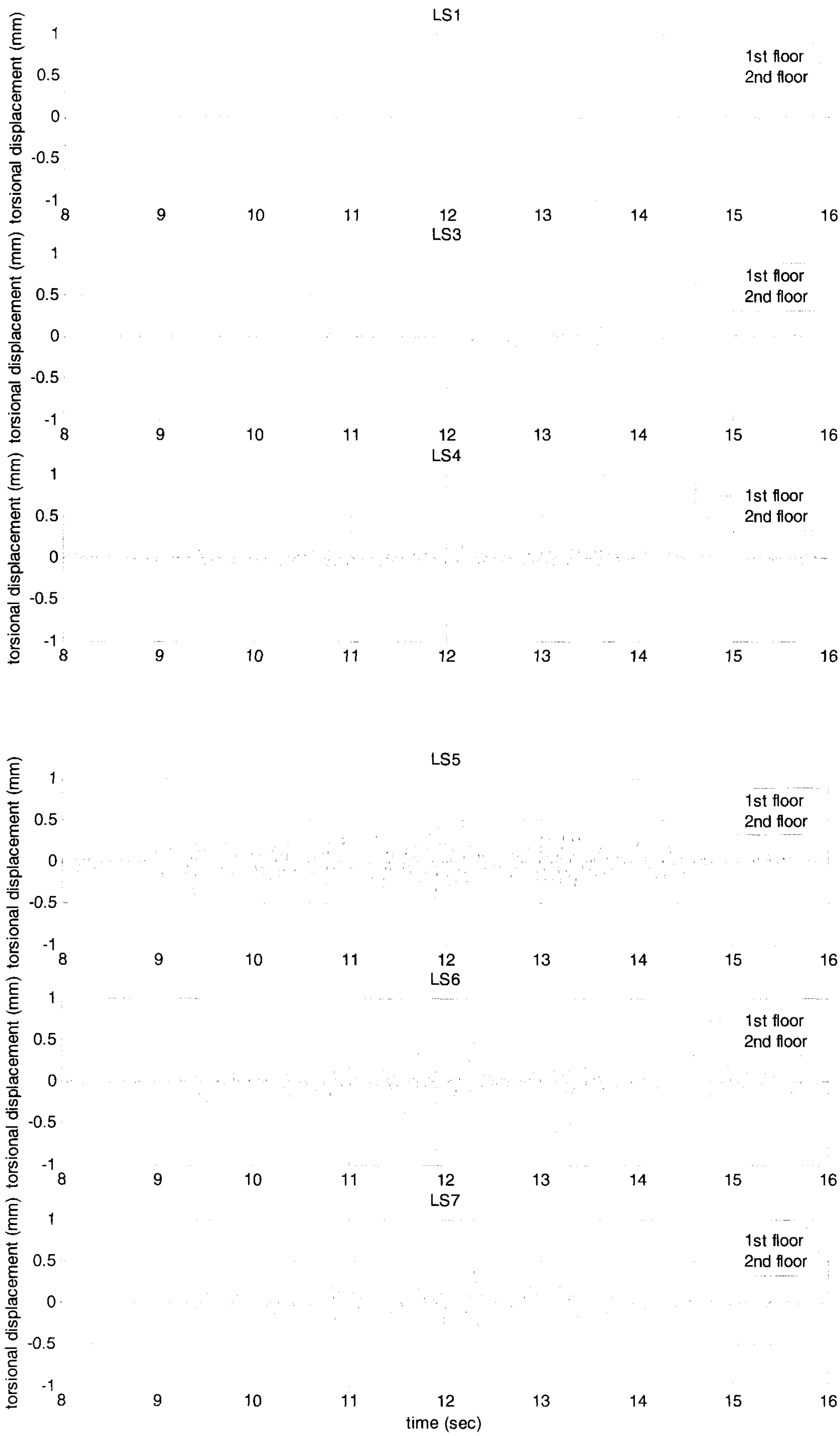


Fig 6 Torsional Deformation Time Histories

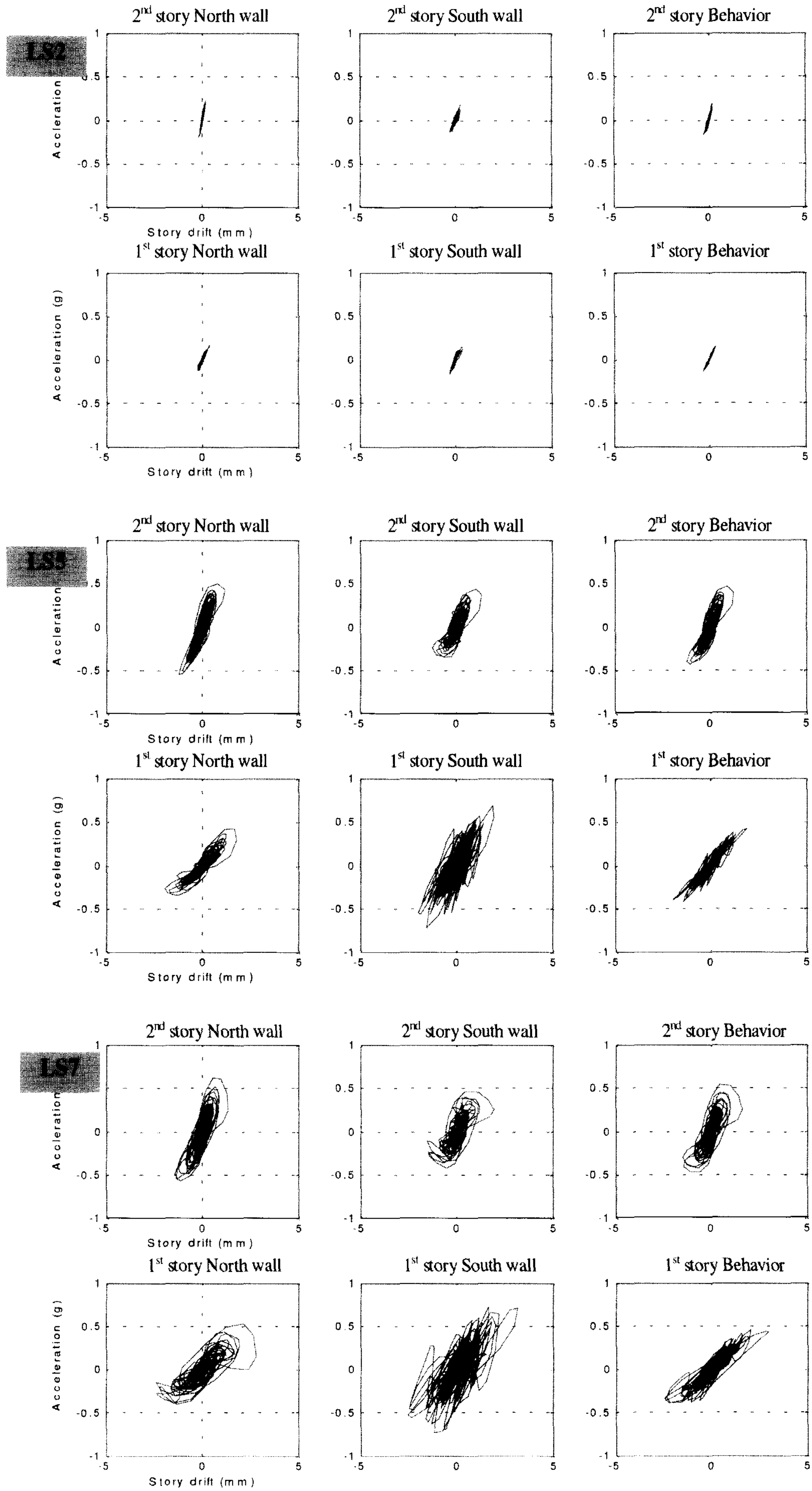


Fig 7 Story Acceleration vs. Story Drift Curves

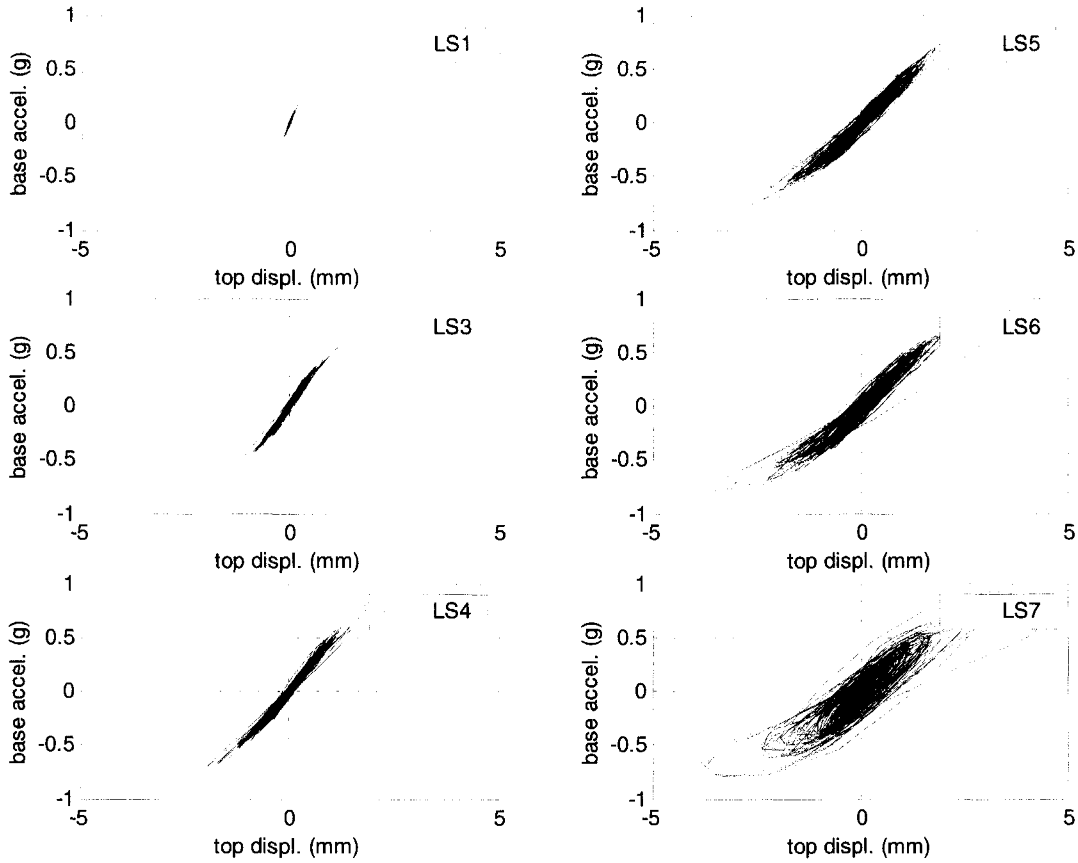


Fig 8 Base Acceleration vs. Top Displacement

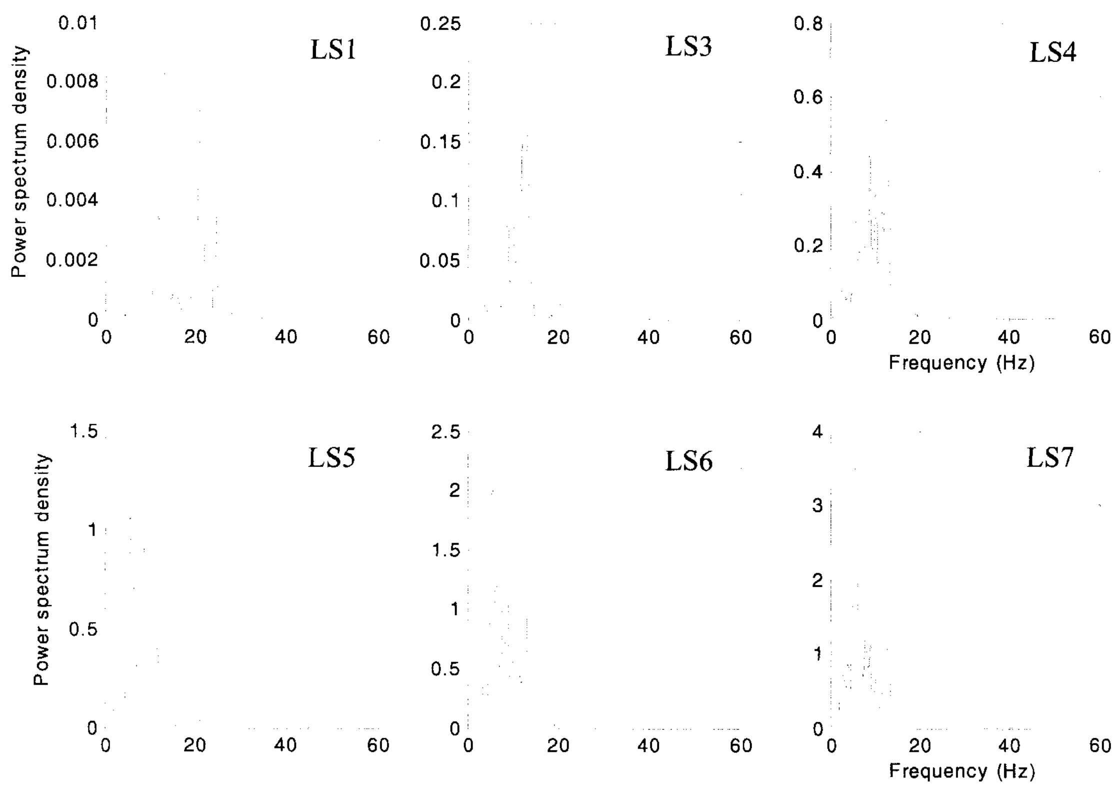
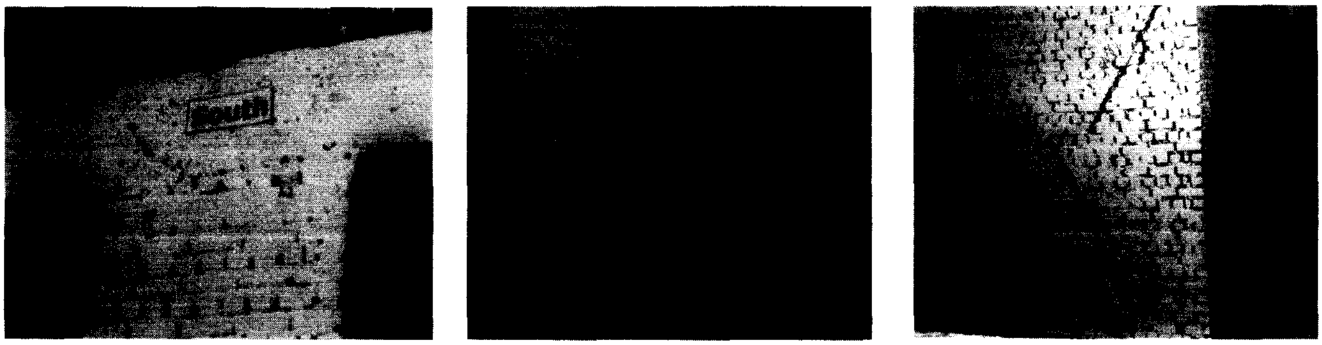


Fig 9 Frequencies obtained by FFT Analysis during the excitation



a After LS2 b After LS5 c After LS7

Fig 10 Structural Damage Patterns

4. Design Considerations

To view the test results from the point of structural design, deformed shapes, distribution of lateral forces and base shear-story drift curves were constructed from the test results. Considering the difficulty in eliciting the fore mentioned curves from the shaking tabletest results due to instrumental and time variation, these figures were prepared not to provide actual design values but to show the trend of dynamic behavior of unreinforced masonry buildings under excitation. The peak values for structural response parameters are summarized in Table 1 for comparison of the motion during the prime tests.

4.1 Deflected Shapes

Deflected shapes were produced by plotting the average of floor displacements measured at the time of peak displacement of the second floor level for test runs (Fig. 11). Note that all displacements were relative to the base of the structure. Before cracking in LS1 the drift levels for the fist and second stories are comparable in magnitude. After cracking, the first-story drifts are much greater than the second-story drifts. From LS4 the first-story drifts were approached to 1.8 times the second-story drift as the structure was damaged more severely.

4.2 Lateral Force Distribution

Floor-level forces were determined by multiplying the average acceleration at a given level by the tributary masses. Assuming that the tributary masses are the same for each floor, the lateral force distribution can be represented in terms of

Table 1 Maximum Response from Experimental Test Results

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

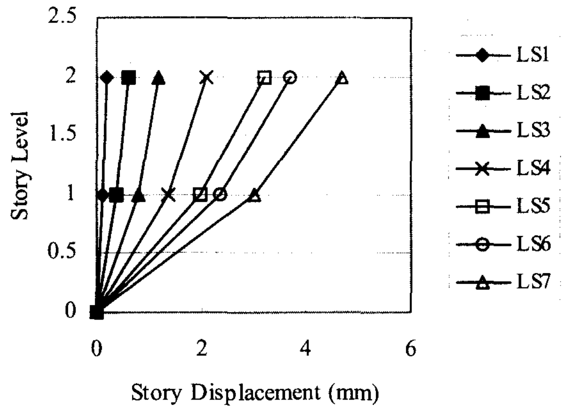


Fig 11 Measured Deflected Shape

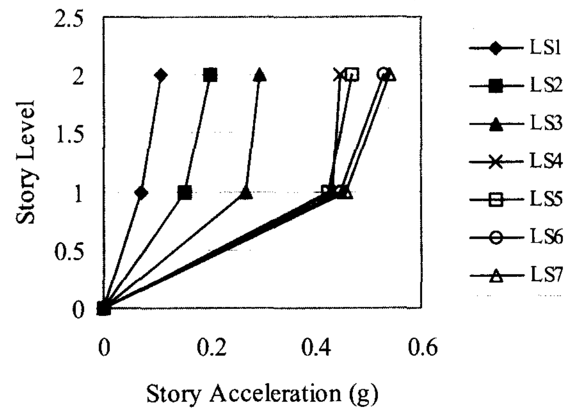


Fig 12 Distribution of Story Acceleration

story acceleration as shown in Fig. 12. Each force pair was concurrent in time from one of the largest base shear peaks of each test run. Clearly, these force pairs did not follow a linear, or inverted-triangular, force distribution. Rather, the ratio of the two floor level forces in each pair appeared nearly the same.

4.3 Dynamic Envelopes of Force and Displacement

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. 13. 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 eachtime segment. After the test run of LS4 the stiffness degradation was apparently occurred. This trend is similar to that of fundamental frequencies changed during the excitations.

4.4 Base Shear vs. Story Drift Curve

To relate the dynamic test results to the performance-based design approach different levels of seismic performance should be defined. One way to define performance levels would be to use story drifts as the bounding parameter. The base shear coefficient versus story drift curve was constructed for peak values for each test run as shown in Fig. 14. Typical damage states might include none to light damage, moderate damage, and severe damage without collapse (SEAOC 1995; SEAOC 1999). Fig. 11 shows three performance levels associated with the damage states: immediate occupancy (IO) after an earthquake; life safety (LS); and collapse prevention (CP) during an earthquake.

5. 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,

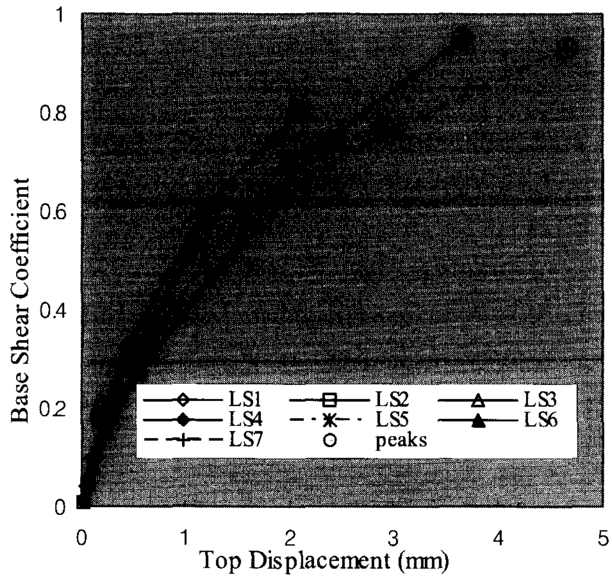


Fig 13 Dynamic Resistance Envelopes

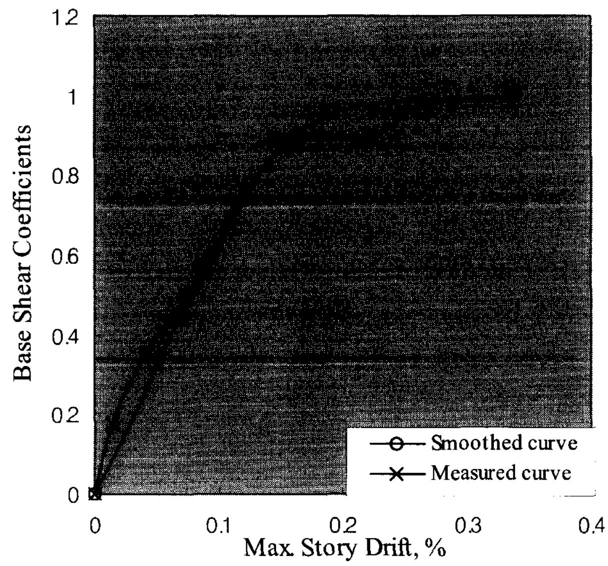


Fig 14 Base Shear vs. Story Drift

- 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.
- Substantial strength and deformation capacity still existed after cracking.
- 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.
- 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 1stfloor right after more pronounced structural damage was occurred.
- There were no out of bending failures in the walls perpendicular to the loading direction.
- Story drifts can be used to define different performance levels for rocking-controlled systems in performance-based design approaches.

This study focused mainly on the investigation on the propagation of damage and failure mechanism of the test structure using measured data. The test results represented in terms of parameters for structural behavior will be contributed as a good starting point toward the development of performance-based design approach for unreinforced masonry structures consisting of shear walls and rigid diaphragms.

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