THE STRUCTURAL DESIGN OF TALL AND SPECIAL BUILDINGS
Struct. Design Tall Spec. Build. **20**, 314–326 (2011)
Published online 25 November 2009 in Wiley Online Library (wileyonlinelibrary.com/journal/tal). DOI: 10.1002/tal.558

Seismic performance evaluation of tall and nonseismic-designed wall-type structures by shaking table tests

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SUMMARY

Two 1/5-scaled models of a nonseismic-designed wall-type structures were constructed and tested on a shaking table to evaluate their seismic performances. The prototype structure had shear walls only along the short side of the structure, which was a typical structural plan of apartment buildings constructed by tunnel forms before the seismic design code was enforced in Korea in 1989. Of the two models, one model was reinforced by steel angle sections placed on the walls and under the slabs for seismic retrofit. They were tested on a shaking table to investigate performance for earthquake ground excitations with various intensities. The experimental results showed that the nonseismic-designed wall-type structure without seismic retrofit failed to satisfy the life-safety and collapse-prevention performance objectives, whereas the retrofitted structure satisfied all the performance objectives.

1. INTRODUCTION

In Korea, the seismic load has been considered in the structural design of building structures since 1989; however, there are still many structures in which seismic design was not implemented. Among these, reinforced concrete (RC) wall-type apartment buildings are considered to be particularly vulnerable to earthquakes because lateral-load resisting walls are placed mainly along the transverse direction and there are few shear walls along the longitudinal direction. Before the seismic load was implemented into structural design, the application of such a structure system was expedited by the wide use of the so-called tunnel forms, which significantly reduced the construction cost of apartment buildings.

For seismic retrofit of such nonseismic-designed structures, diverse schemes have been proposed. For example, the use of precast panels for frame infills has been investigated (Frosch *et al*., 1996), and a range of steel bracing systems have been proposed for upgrading existing concrete frames (Badoux and Jirsa 1990; Bush *et al*., 1991; Masri and Goel 1996).

The effectiveness of retrofit schemes can be evaluated by various methods; in particular, the shaking table tests utilizing real earthquake records is considered the most realistic to verify the seismic capacity of retrofitted structures. The validity of the shaking table tests with scaled models was verified by Gulkan and Sozen (1971), who showed that the performance of the 1/4-scaled and 1/8-scaled models were similar to that of the prototype structure. Kwan and Xia (1995) carried out shaking table tests of 1/3-scaled model structures retrofitted by reinforced concrete shear walls and in-filled masonry walls, and observed that seismic performance of the model structures were greatly enhanced as a result of the seismic retrofit. Dolce *et al*. (2005), using shaking table tests, investigated the effect of passive control system on the retrofit of existing structures. They showed that the addition of passive

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control braces in the reinforced concrete frame resulted in significant benefits to the overall seismic behaviour.

In the present study, the seismic performance of a typical nonseismic-designed RC apartment building structure was investigated through shaking table test. The validity of a simple seismic retrofit scheme using added steel sections was also evaluated. Two 1/5-scaled models of a five-storey wall-type structure were constructed using tunnel forms with and without retrofitting steel channel sections for seismic retrofit. Shaking table tests of the model structures were carried out using the El Centro earthquake record as an input ground motion with its peak ground acceleration (PGA) varying from 0.06 to 0.5 *g*.

2. DESIGN AND CONSTRUCTION OF MODEL STRUCTURES

2.1. Prototype structure

The prototype structure is an apartment building built in Korea before 1989, and thus seismic load was not considered in the structural design. As the structure was constructed by tunnel forms, shear walls were placed only along the transverse direction, and no lateral load-resisting system exists along the longitudinal direction. Figure 1 shows the schematic view of the typical nonseismic-designed apartment building constructed using tunnel forms. Preliminary analysis of the prototype structure by capacity spectrum method (Applied Technology Council, 1996) showed that the structure lacked the strength to resist the code-specified seismic hazard of 10% probability of not being exceeded in 50 years. The strengths of concrete and reinforcing steel of the prototype structure were assumed to be 21 and 400 MPa, respectively.

2.2. Scaled models

Two 1/5-scaled nonseismic-designed RC wall-type structures were constructed for shaking table test. Of the two scaled models, one was used as a reference structure, and the other, reinforced by steel channel sections, was used to observe the effectiveness of the seismic retrofit scheme. Table 1 shows the dimensions of the prototype and the model structures scaled based on the similitude law (Bertero and Aktan, 1983). According to the similitude law, the mass of a 1/5-scaled model structure needs to be reduced to $1/5^2$; however if the dimension is reduced to 1/5 following the similitude law, the mass is reduced to $1/5^3$. In this research, the similitude law for mass was met by adding lumps of lead to the walls and slabs of the model structures (Table 2). Steel angle sections were added to the inside of the external walls and under the slabs in the reinforced model. Figure 2 shows the dimension of

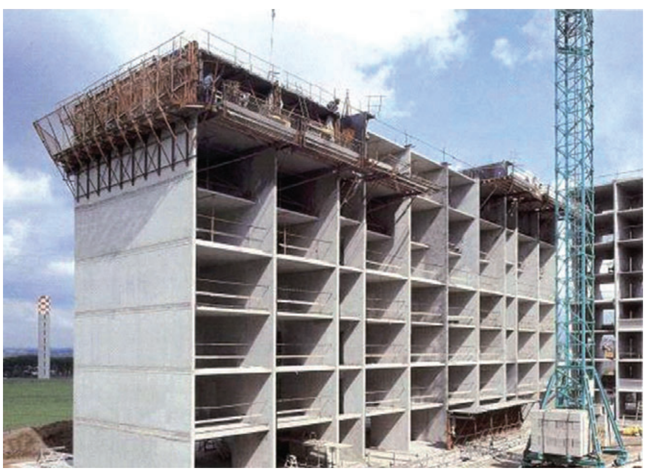


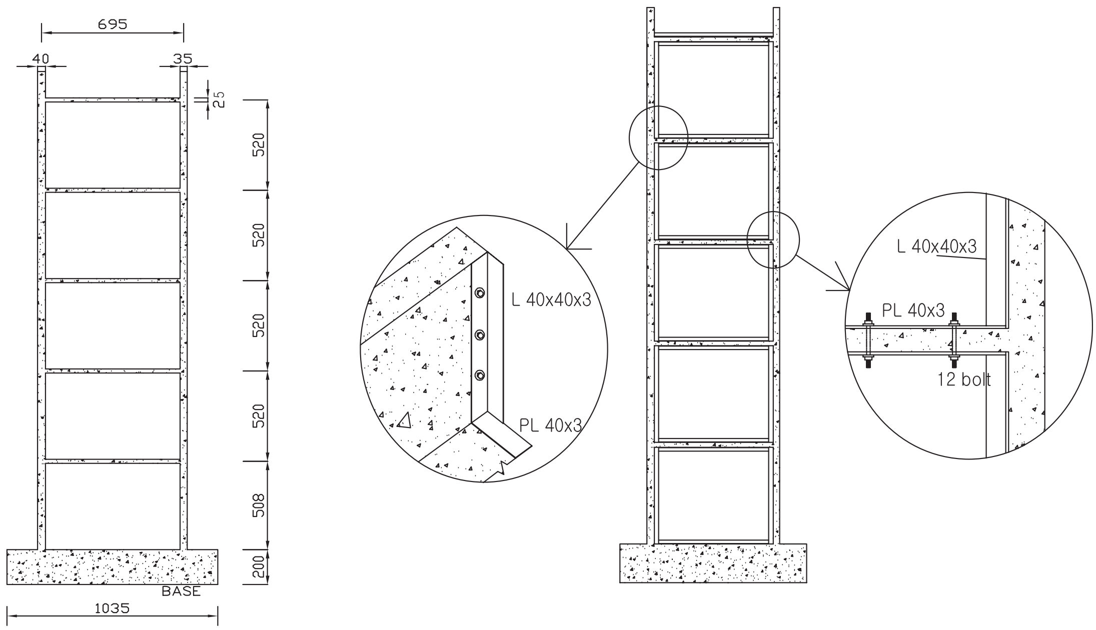
Figure 1. Wall-type apartment building constructed by tunnel forms.

Table 1. Dimension and properties of the prototype and the scaled model.

Element	Prototype model	1/5-Scaled model
Left wall (mm)	200 × 1500	40 × 300
Right wall (mm)	170 × 1500	35 × 300
Slab thickness (mm)	120	30
Storey height (mm)	2600	520
Reinforcing bar	D10 + D13	D2
F_y (MPa)	300	350
f'_c (MPa)	21	24
Maximum aggregate size (mm)	25	5

Table 2. Attachment of additional mass to meet similarity law.

	Weight of prototype structure (kg)	Weight of scaled model (kg)	Weight of additional mass (kg)	Difference (%)
Left wall	1872	74.88	74.98	0.2
Right wall	1591	63.64	63.1	2.7
Slab	1782	71.28	69.26	3.6



(a) Reference model (b) Retrofitted model

Figure 2. Model structures without and with seismic reinforcement (unit : mm): (a) reference model; (b) retrofitted model.

the model structures and the location of the retrofitting steel. This simple scheme turned out to be easily applicable and economical for the seismic retrofit of nonseismic-designed wall-type structures.

2.3. Manufacture of scaled deformed bars

In the test models the reinforcing bars were constructed following the similitude law. The cross-sectional area of the reinforcing bar used in the prototype structure is 0.71 cm^2 (HD10), which is

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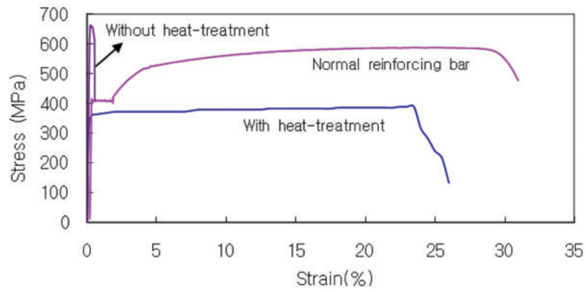

Figure 3. Stress–strain relationship of reinforcing bars.

Table 3. Fineness of coarse aggregate.

Prototype structure		1/5-Scaled model	
Sieve (mm)	Per cent retained	Sieve (mm)	Per cent retained
25	25	6	25
10	65	4	65
5	95	2	95
2.5	98	1	98
1.2	99	0.5	100
0.6	100	–	–

reduced to 0.0284 cm^2 when the similarity law is applied. To produce the deformed reinforcing bar with a 2-mm diameter, round steel wire with a 2.1-mm diameter was compressed in a press which was especially manufactured in the laboratory. After this process, the maximum and the minimum diameter became 2.13 and 1.95 mm, respectively. However, as plotted in Figure 3, the scaled deformed bar showed quite brittle behaviour after the deforming process. To provide more ductility and toughness, the scaled bars were heated to 550°C for 12 minutes and were slowly cooled in room temperature. Figure 3 shows that the maximum strength of the heat-treated bars was much reduced compared with those before the heat treatment, but the ductility was significantly increased. The strength was around 350 MPa, which is close to the nominal strength of HD10 bars.

2.4. Casting of model structures

In the construction of the scaled model structures, normal Portland cement was used with the size of coarse aggregates scaled down to meet the similitude law. The maximum size of coarse aggregate used in the prototype structure was assumed to be 25 mm, and therefore the coarse aggregates smaller than 5 mm were prepared for scaled models. Table 3 presents the sieve analysis of the aggregates used in the prototype and the scaled model structures.

The slump of the mixed concrete turned out to be 16 cm, and the air contents was measured to be 2.9%. For strength test, moulds of 10 cm × 20 cm cylinders were prepared following the Korea Agency for Technology and Standard (1992) KS F 2403. The structures were exposed in the air for 28 days before experiments. Table 4 shows the material properties of the concrete and the reinforcing bar used in the construction of model structures.

2.4. Retrofit of the model structure

To enhance the seismic-load resisting capacity of the reference model structure, rolled steel sections of L-40 × 40 × 3 and L-40 × 3 were attached to inside of the vertical walls and both sides of the slabs, respectively, using high-tension bolts. The gap between the steel sections and the concrete structure was filled by epoxy resin. The increment of the mass was 13.33 kg, which corresponded to 1.3% of the structure mass of 1090 kg. This reinforcing scheme is simple and economical, and if proved effec-

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Table 4. Material properties of the model structures.

	Concrete	Reinforcing bar	
	f'_c (MPa)	F_y (MPa)	Extension ratio (%)
Specimen 1	25.61	372.86	21
Specimen 2	27.45	373.47	19
Specimen 3	26.94	359.59	17
Average	26.67	368.64	19

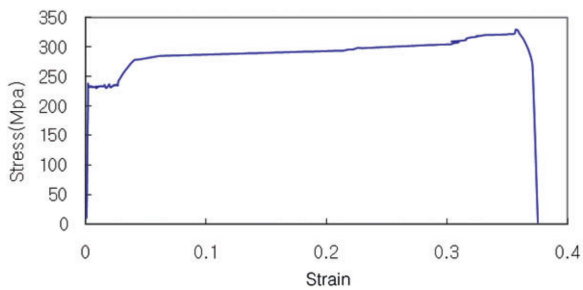

Figure 4. Stress–strain relationship of steel angle section used for seismic retrofit.

Table 5. Capacity of the uniaxial shaking table used in the test.

Size	5 m × 3 m
Maximum specimen weight	30 tons
Table mass	10 tons
Control mode	Uniaxial horizontally
Maximum stroke	±100 mm
Maximum velocity	500 mm/s
Maximum acceleration	1.0 *g*
Frequency range	30 Hz

tive in increasing lateral-load resisting capacity, it may be readily applied in seismic retrofit of non-seismic-designed wall-type structures. Figure 4 shows the stress–strain relationship of the angle sections used in the retrofit scheme.

3. SHAKING TABLE TESTS

3.1. *Test setup*

The shaking table tests were performed on the earthquake simulator at the Research Institute of Hyundai Engineering and Construction Co. The shaking table is 5 m by 3 m in size and can realize maximum ground acceleration of 1.0 *g* at 10 tons and maximum frequency of 30 Hz. Table 5 presents the properties of the shaking table. The test results were obtained using a data logger with 32 channels. Shaking table tests were carried out with El Centro earthquake record as an input ground motion with its PGA varying from 0.06 to 0.5 *g*. Figure 5 shows the test setup and location of measuring devices; accelerometers and LVDT's were installed at each storey and on the shaking table to measure both the input and the responses. The photograph of the reference model structure mounted on a shaking table is presented in Figure 6.

3.2. *Dynamic characteristics of model structures*

As a preliminary test, the model structures were subjected to free vibration and white noise ground motion to identify the natural frequency. To induce free vibration, the structure was impacted laterally

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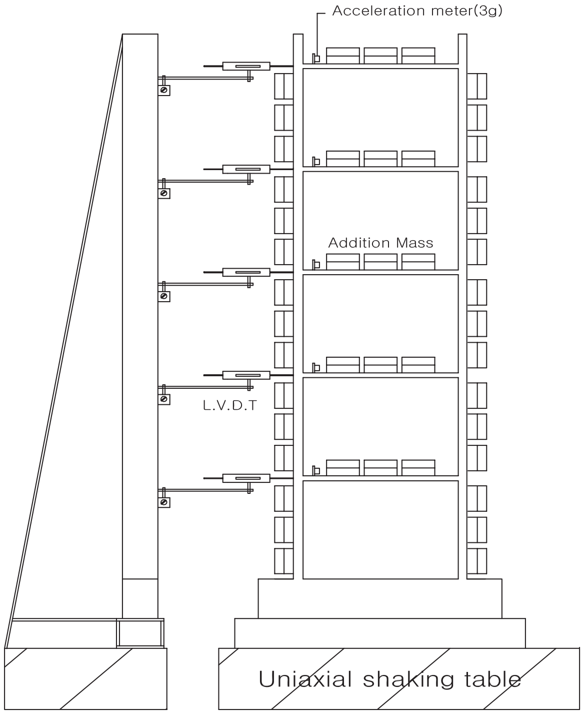

Figure 5 Test set-up for shaking table test.

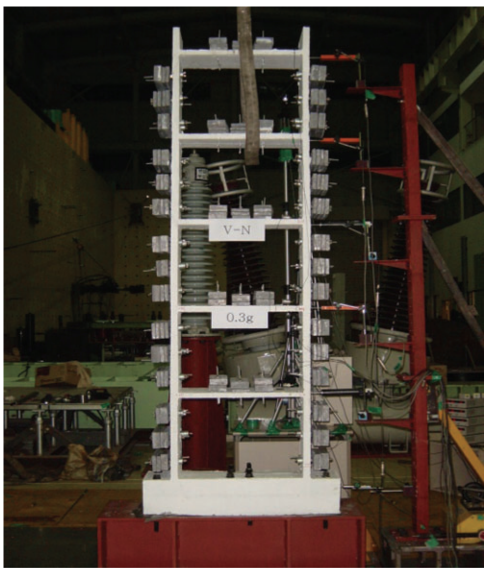

Figure 6. Reference model structure placed on the shaking table.

at the centre of the roof-storey slab with a rubber hammer. Figure 7 plots the time history of the white noise input acceleration. Figures 8 and 9 depict the acceleration frequency response of the reference and the retrofitted model structures obtained from the free vibration test and the white noise vibration test, respectively. The fundamental natural frequencies are shown in Table 6. The fundamental natural frequencies of the reference structure, obtained from the free vibration and the white noise tests, were measured to be 4.37 and 4.27 Hz, respectively. For the retrofitted structure, the natural frequencies were 4.98 and 4.84 Hz, respectively. For both structures, the difference between the natural frequen-

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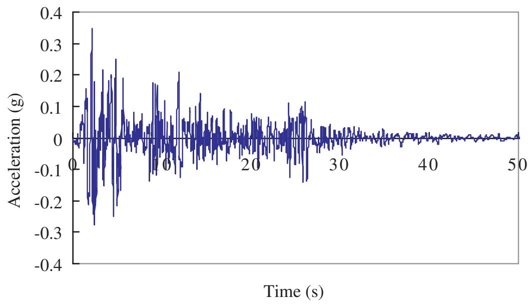
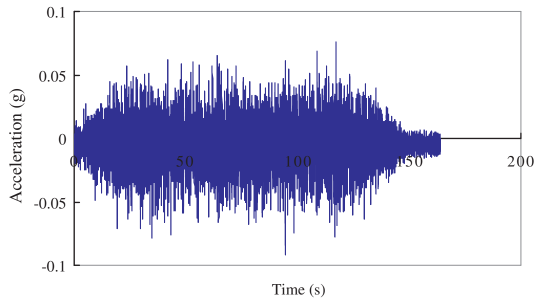
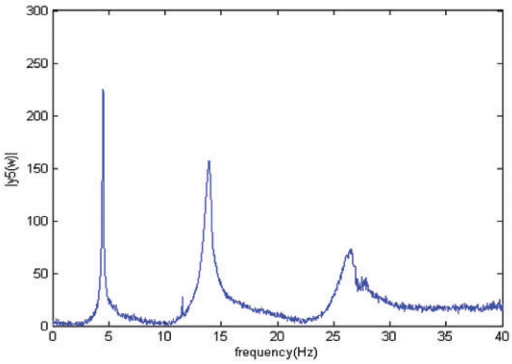


(a) El Centro earthquake

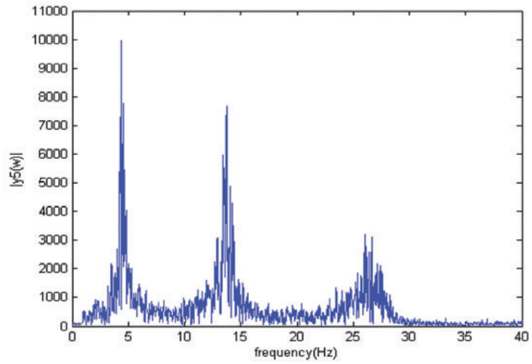


(b) White noise

Figure 7. Input ground accelerations used in the shaking table tests: (a) El Centro earthquake; (b) white noise.

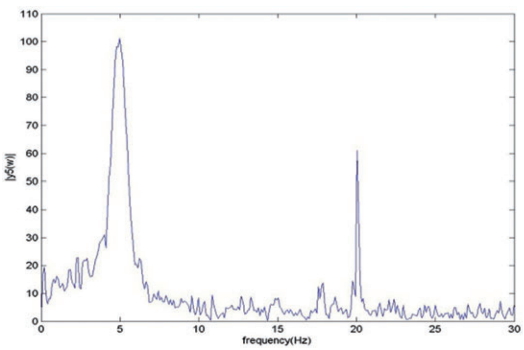


(a) Response for free vibration

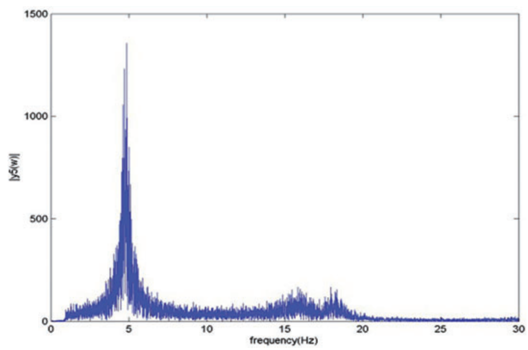


(b) Response for white-noise ground motion

Figure 8. Frequency response of the reference model structure: (a) response for free vibration; (b) response for white-noise ground motion.



(a) Response for free vibration



(b) Response for white-noise ground motion

Figure 9. Frequency response of the retrofitted model structure: (a) response for free vibration; (b) response for white-noise ground motion.

Table 6. Fundamental natural frequency of the scaled models.

Specimen	Input vibration	Fundamental natural frequency (Hz)	Difference (%)
V-N (reference)	Free vibration	4.37	2.3
	White-noise	4.27	
V-SC (retrofitted)	Free vibration	4.98	2.8
	White-noise	4.84	

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cies obtained from two different methods was less than 3%. Based on the measured natural frequencies, the retrofit of the model structure increased stiffness of the structure by about 4%.

3.3. *Shaking table test results*

Shaking table tests were conducted using the El Centro earthquake (North-South component) as the input ground motion. The PGA of the input motion was scaled to 0.06, 0.12, 0.2, 0.3, 0.4 and 0.5 g. To meet the similitude law, the time scale of the acceleration records was reduced to the square root of the scale of the model. Figure 10 presents the frequency response of the structures computed from the frequency analysis of the acceleration response time histories. In the experiment of the reference structure, it was observed that no crack occurred for ground motion with PGA = 0.06 g and the structure behaved elastically. Although Figure 10(a) indicates that the natural frequency slightly decreased, no noticeable damage was observed at PGA = 0.12 g. When PGA was increased to 0.2 g, cracks started to occur at the second-storey slab–shear wall joint and at the first-storey shear wall. At PGA = 0.3 g and 0.4 g, no additional crack was observed; however the existing cracks at the joint and the wall enlarged significantly. Therefore, it would be expected that at the design earthquake level of Korea, which corresponds approximately to PGA = 0.2 g, the reference structure might experience some damage. The same tests were also carried out with the retrofitted structure. In this case, it was observed that no visible crack occurred at El Centro ground vibration up to PGA = 0.2 g. Figure 10(b) shows that the fundamental natural frequency did not change up to that level of earthquake, which indicates that the structure remained elastic. At the ground vibration of PGA = 0.3 g, narrow cracks occurred around the bolt holes used to connect lead blocks to the structure. When the PGA was increased to 0.4 g, although cracks occurred at the slabs of the second and third storeys. No crack was observed at the shear walls. When the ground acceleration reached PGA = 0.5 g, additional cracks formed at the first-storey walls and slabs. Table 7 shows the fundamental natural frequency of the model structures obtained from frequency analysis of the acceleration responses. It can be observed

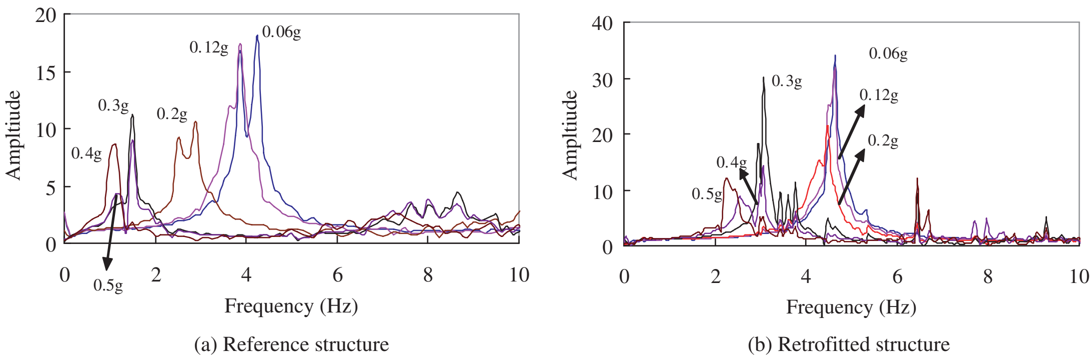


Figure 10. Frequency response of the model structures for El Centro earthquake with various intensities: (a) reference structure; (b) retrofitted structure.

Table 7. Change in the natural frequency for various intensities of earthquake ground motion.

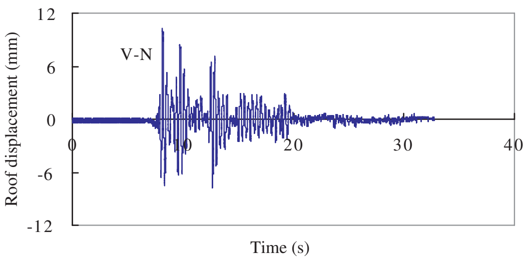
	Natural frequency	
PGA (g)	Reference model (Hz)	Retrofitted model (Hz)
0.06	4.25	4.64
0.12	3.88	4.64
0.2	2.88	4.49
0.3	1.5	3.08
0.4	1.5	3.07
0.5	1.25	2.25

DOI: 10.1002/tal

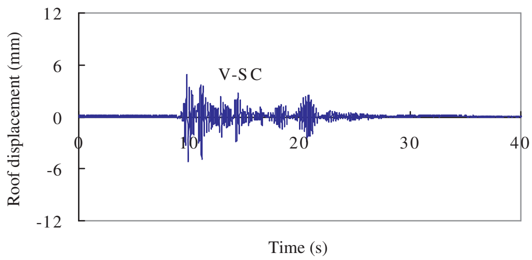
that at the ground excitation with PGA = 0.6 g, the natural frequency of the reference structure was reduced to less than a third of the undamaged structure due to the formation of cracks. The natural frequency of the retrofitted structure, however, was reduced only to about half of the original value, which implies that damage was reduced as a result of the retrofit.

Figure 11 plots the time history of the roof-storey displacement for ground motion with PGA = 0.2 g. Table 8 presents the maximum roof-storey displacements of the structures for input ground motions with various PGAs. It can be observed that for PGA = 0.2 g, the maximum displacement of the retrofitted structure (V-SC) was only 51.7% of that of the original structure (V-N). For ground motion with PGA = 0.4 g, the maximum displacement of the retrofitted structure was 89.0% of that of the original structure.

Figure 12 depicts the maximum drift of each storey of the model structures obtained from the shaking table test for various amplitudes of input ground acceleration. It can be observed that the



(a) Reference structure

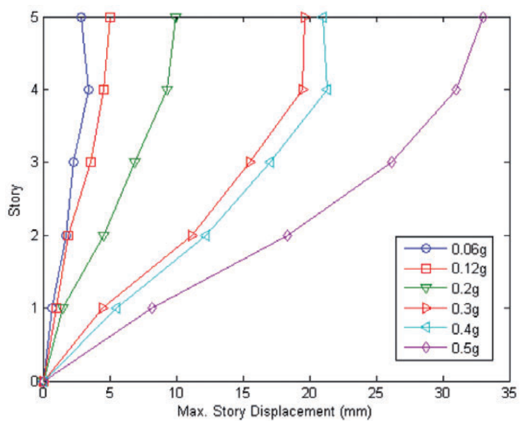


(b) Retrofitted structure

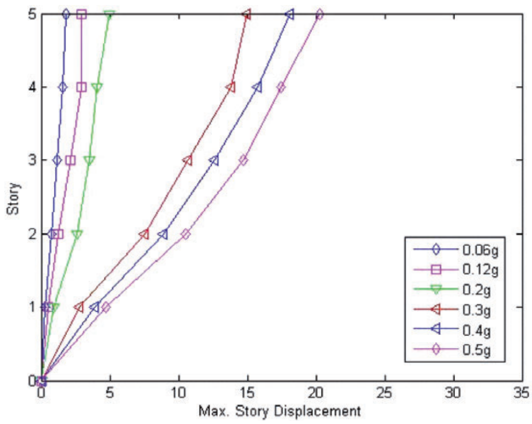
Figure 11. Roof-storey drift of the model structures subjected to earthquake ground motion of PGA = 0.2 g: (a) reference structure; (b) retrofitted structure.

Table 8. Maximum displacement at the top storey.

PGA (g)	Reference model (mm)	Retrofitted model (mm)	Reduction (%)
0.06	3.46	1.91	44.8
0.12	5.22	3.53	32.4
0.2	10.24	4.94	51.7
0.3	19.94	15.12	24.2
0.4	21.13	18.81	11.0
0.5	34.09	22.70	33.4



(a) Reference structure



(b) Retrofitted structure

Figure 12. Maximum storey drift of the model structures for various amplitudes of the input El Centro earthquake: (a) reference structure; (b) retrofitted structure.

maximum drift increased in large scale at the input ground motion with PGA = 0.3 *g* and 0.5 *g* due to the damage in structures.

Figure 13 plots the ratio of the maximum acceleration response to the input ground acceleration. It can be observed that in the reference structure the amplification factor decreased as PGA of the ground motion increased, which implies that the amount of damage increased. The amplification factor for the retrofitted structure, on the other hand, remained constant up to PGA = 0.2 *g*, which means that the structure was undamaged. It decreased from PGA = 0.3 *g*; however, the decrease in the amplification factor was less than that of the original structure, which confirms that damage was less significant.

Figure 14 depicts the maximum storey acceleration responses of model structures for each level of input ground acceleration. In the reference model the acceleration response increased almost linearly as the height of the floor increased when the input ground acceleration was small (PGA = 0.06–0.2 *g*).

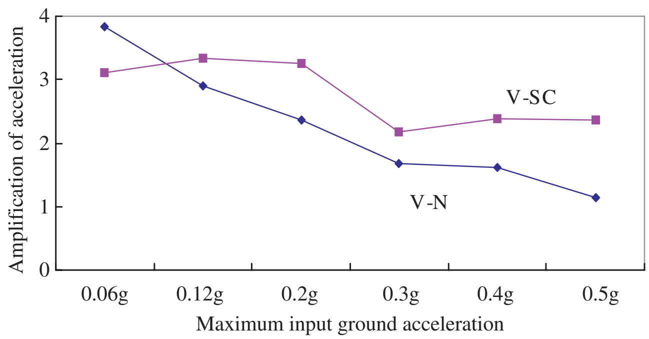
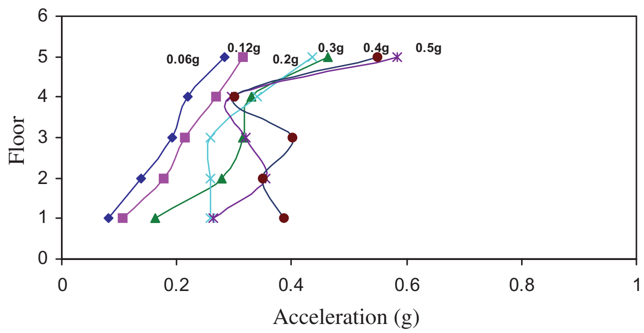
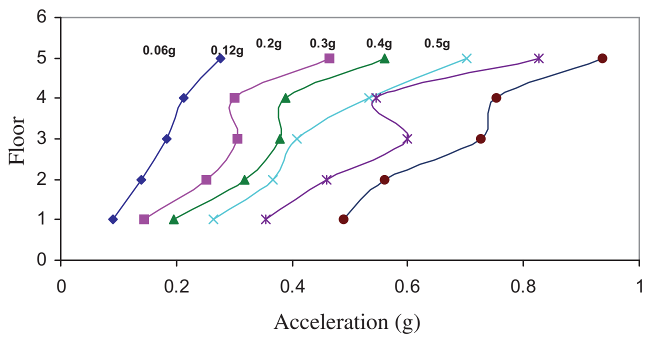


Figure 13. Maximum acceleration response for various input ground accelerations.



(a) Reference model



(b) Retrofitted model

Figure 14. Maximum storey acceleration response for various input ground accelerations: (a) reference model; (b) retrofitted model.

Table 9. Change in the damping ratio for various intensities of earthquake ground motion.

PGA (g)	Reference model (%)	Retrofitted model (%)
0.06	3.01	1.05
0.12	3	1.02
0.2	3.85	1.43
0.3	3.82	2.52
0.4	8.2	6.47
0.5	10.68	7.8

Table 10. Performance levels and limit states for various intensities of earthquake ground motion.

Earthquake level (PGA, g)	Performance level	Limit state (Inter-storey drift, %)
0.06–0.12	Immediate occupancy	0.5
0.2–0.3	Life safety	1.0
0.4–0.5	Collapse prevention	2.0

As the input acceleration increased to larger than 0.3 g, the maximum acceleration at the fourth floor rather decreased slightly. This implies that a larger part of the input acceleration was not transferred because of the damage that occurred in the fourth floor, which matched the visual observation of the crack pattern. In the retrofitted structure, however, the input ground accelerations were transferred to the higher storeys more smoothly. This implies that less damage occurred in the retrofitted specimen.

The half-power band-width method was employed to estimate the damping ratio of the model structures. Table 9 shows that as the intensity of the input ground motion increased the damping ratio also increased. In both specimens, the damping ratio increased significantly at PGA = 0.4 g. The damping ratio of the reference structure turned out to be more than twice as high as that of the retrofitted structure, which implies that a significant amount of energy was dissipated by the newly formed cracks.

4. SEISMIC PERFORMANCE LEVELS

The International Building Code IBC-2000 (International Code Council, 2003) requires that for design-level earthquakes (two-thirds of the earthquake with 2400-year return period) the maximum inter-storey drift of a building structure should be less than 1.0% of the storey height in case of Seismic Design Category D, and 1.5% of the storey height for structures belonging to Seismic Design Category C. The Federal Emergency Management Agency (FEMA)-356 (2000) specifies inter-storey drift limits for various seismic performance levels: for reinforced concrete shear wall-type structures, 2.0% of the storey height for collapse prevention; 1.0% for life safety; and 0.5% for immediate occupancy performance level. Table 10 shows the seismic performance levels and the corresponding limit states, presented in the FEMA-356 (Building Seismic Safety Council, 2000), for the earthquakes used as input ground accelerations in the tests.

As shown in Figure 15, the maximum inter-storey drift of the reference structure exceeded the immediate occupancy performance limit state for ground motion with PGA = 0.06 g, whereas that of the retrofitted structure was less than the limit state for the same ground motion. It also can be observed that for ground motion with PGA = 0.2 g, which corresponds approximately to the design seismic load of Korea, the maximum inter-storey drift of the reference structure exceeded the functional limit state, which is 2.6 mm; the maximum inter-storey drift of the retrofitted structure, however, turned out to be 1.8 mm, which is significantly less than the limit state. For earthquakes with PGA = 0.3 g and 0.4 g, the maximum inter-storey drifts of both the reference and the retrofitted structures exceeded the functional limit state but were less than the life-safety limit state, which is 7.8 mm. The maximum inter-storey drifts of the retrofitted structure turned out to be less than those of the reference structure

DOI: 10.1002/tal

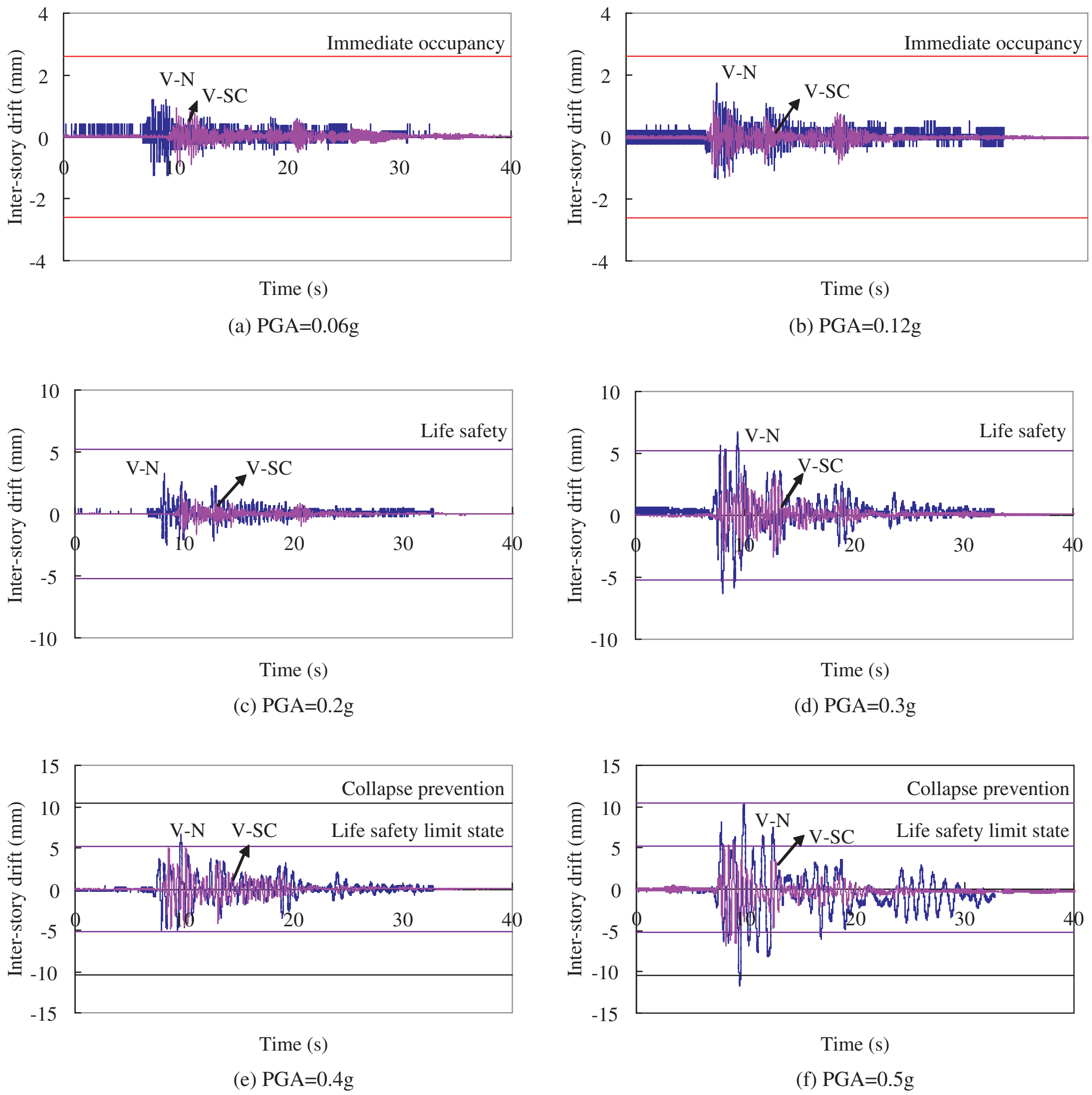


Figure 15. Time history of the second-storey inter-storey drift of the model structures: (a) PGA = 0.06 g; (b) PGA = 0.12 g; (c) PGA = 0.2 g; (d) PGA = 0.3 g; (e) PGA = 0.4 g; (f) PGA = 0.5 g.

by 24.9%–33.3%. When the shaking table was subjected to the ground motion with PGA = 0.5 g, the maximum inter-storey displacement of the reference structure exceeded the life-safety limit state, whereas that of the retrofitted structure was less than the limit state. The above observation shows that the reference structure did not satisfy the code-specified performance limit state and needs retrofit to meet the code requirement. The retrofitted structure, on the other hand, satisfied the displacement requirements for performance-based designs.

5. CONCLUSIONS

In this study, the seismic performance of a typical nonseismic-designed apartment building was evaluated and the validity of a simple seismic retrofit scheme was verified by shaking table tests. The tests were carried out with 1/5 scaled model structures subjected to ground vibrations with various intensities.

DOI: 10.1002/tal

According to the test results, the seismic response of the nonseismic-designed wall-type structure exceeded the given performance limit states for corresponding earthquake levels. It was also shown that the seismic retrofit of the reference structure by adding steel angle sections turned out to be effective in increasing stiffness and decreasing the maximum displacement of the reference structure, and the retrofitted structure responded within the given limit states. Based on the experimental results, it can be concluded that the simple retrofit scheme carried out in this study was effective in providing enough seismic-load resisting capacity to nonseismic-designed wall-type structures. It must be pointed out, however, that experiment with only two scaled models is not enough to induce generalized conclusion on the seismic performance and retrofit of all wall-type structures, and that further study on different types of wall-type structures with various design variables is necessary.

ACKNOWLEDGEMENT

This work was supported by National Research Foundation grant funded by the Korea government (Ministry of Education, Science and Technology) (2009-0076668).

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DOI: 10.1002/tal