

Experimental study on seismic retrofit of a RC frame using viscoelastic dampers

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ABSTRACT

The deformation capacity of a viscoelastic damper (VED) is generally limited by the thickness of the VE pad. In this study, a VED with fail-safe mechanism was developed and its seismic performance was evaluated by a series of experiments of full-scale two-story reinforced concrete (RC) frame before and after seismic retrofit. The VED is composed of high damping rubber viscoelastic pads and is installed vertically between stories. One bare frame and two retrofitted frames with different damper capacities were tested under cyclic loads to validate the effectiveness of the newly developed dampers. The Bouc Wen model was developed to simulate the seismic behavior of the damper, which turned out to represent the behavior of the VED accurately. As a case study, the proposed VED was applied to seismic retrofit design of a 4-story RC moment framed structure, and it was found that the proposed retrofit scheme could adequately improve the seismic performance of the structure by reducing both inter-story and residual displacements.

1. Introduction

2016 Gyeongju and 2017 Pohang earthquakes, which were the largest in Korea in the past 400 years, have proven that non-seismically designed old buildings are vulnerable for seismic actions, and there is an urgent need for applying effective retrofit strategies to these structures. One of the common methods for rehabilitation of existing buildings is the use of passive energy dissipation devices [1–3]. Recently many seismic retrofit strategies were suggested [4–7], but limited studies have been performed in a full scale test [8,9].

Among the many passive seismic energy dissipation devices, viscoelastic dampers (VEDs) have long been used to reduce structural responses induced by earthquakes and wind loads. They are usually installed in a bracing configuration, which is less effective in terms of seismic performance. Along with recent advances in VED technologies, new installation schemes of VEDs were investigated to increase the efficiency of retrofit [10]. Kasai et al. [11] combined a velocity-dependent device and a displacement-dependent device in series, which showed superior performance as compared to using a single device. Marko et al. [12] compared performance of five different damping systems, and showed that the hybrid configuration of horizontal friction damper and diagonal VED has the highest reduction in structural responses. Marshall and Charney [13] studied analytically different combinations of a

buckling restrained brace and either a viscoelastic or viscous fluid devices in a series arrangement. They showed that each combination improved some part of structural responses. Eskandari and Kim [14] developed a hybrid steel-slit viscoelastic damper which is effective both for medium and large earthquakes. Xu et al. [15,16] developed a base isolation system combined with viscoelastic devices for seismic vibration mitigation of a platform structure. Christopoulos et al. [17] developed a viscoelastic coupling damper system for high-rise buildings. They incorporated viscoelastic coupling damper in lieu of coupling beams in conventional structural configurations. Recently the effectiveness of VED on the seismic retrofit of a soft-first story structures was evaluated considering soil-structure interaction [18].

In this study, a new viscoelastic damper with a fail-safe mechanism was developed by introducing a lateral gap between cover and main steel plates of the damper. When a specified shear deformation occurs in the damper due to seismic load, the gap is closed, and the force is transferred from the cover plate to the main steel plate directly preventing further shear deformation of the viscoelastic pad. This load transfer mechanism ensures stable behavior of the VED even during extreme earthquakes. In addition the damper was developed to be installed vertically, not diagonally, between upper and lower floor beams with enhanced seismic performance and occupation of less space compared to the conventional bracing systems. A series of experiments

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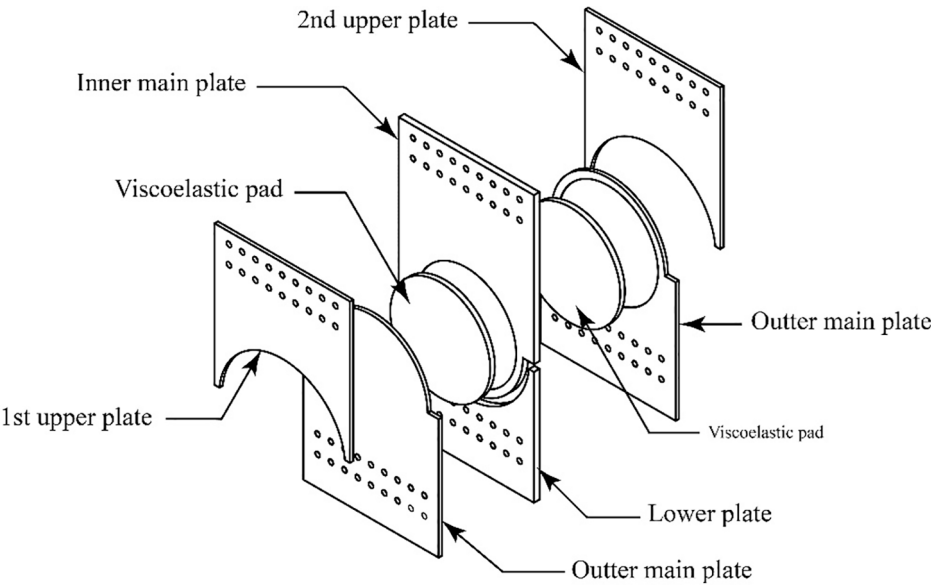


Fig. 1. Conceptual figure of the proposed viscoelastic damper.

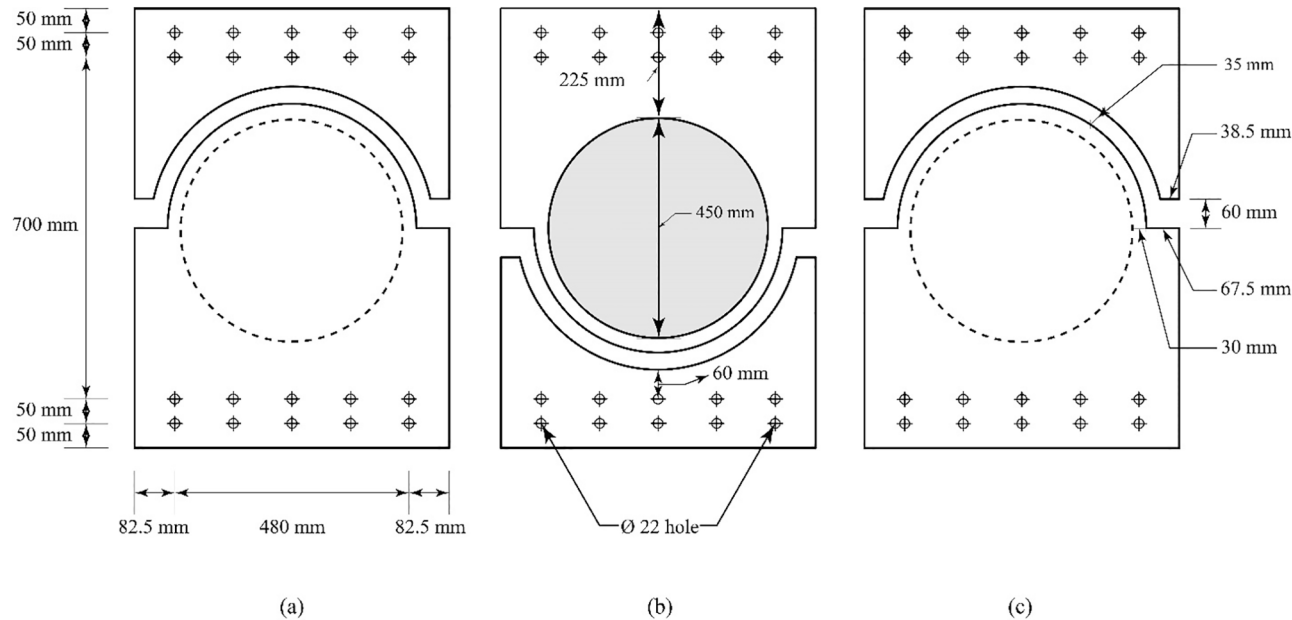


Fig. 2. Dimensions of the proposed viscoelastic damper.

Table 1
Storage modulus of the VE material for different excitation conditions.

Amplitude (mm)	Frequency (Hz)				
	0.1	0.5	1.0	1.5	3.0
5	1.0366	1.0973	1.1346	1.1766	1.2731
10	0.76344	0.82045	0.77188	0.84738	–
15	0.57249	0.60448	0.58999	–	–
20	0.46785	0.53801	–	–	–
25	0.452	0.48801	–	–	–
35	0.44014	0.43705	–	–	–
45	0.48056	0.37094	–	–	–
60	0.42836	–	–	–	–

Table 2
Loss modulus of the VE material for different excitation conditions.

Amplitude (mm)	Frequency (Hz)				
	0.1	0.5	1.0	1.5	3.0
5	0.38562	0.4245	0.50965	0.43841	0.42834
10	0.23643	0.24883	0.26687	0.2679	–
15	0.1899	0.17271	0.19691	–	–
20	0.14897	0.12648	–	–	–
25	0.12483	0.10444	–	–	–
35	0.093094	0.074356	–	–	–
45	0.075268	0.061548	–	–	–
60	0.057581	–	–	–	–

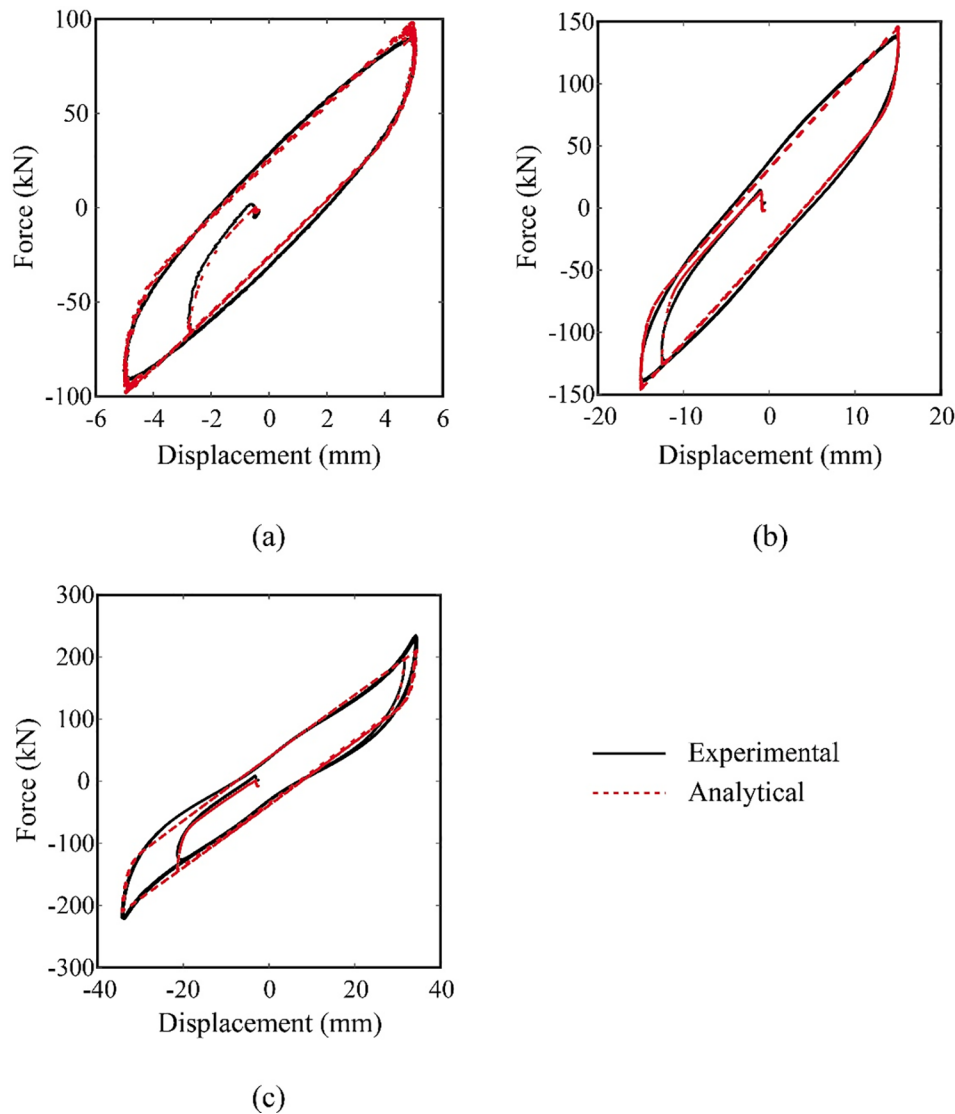


Fig. 3. Comparison of the experimental and analytical results of the VED: (a) Amplitude = 5 mm; (b) Amplitude = 15 mm; (c) Amplitude = 35 mm.

were carried out on a full-scale two-story reinforced concrete (RC) frame before and after seismic retrofit to evaluate the effectiveness of the proposed VED. An analytical model was developed based on the test results, and was applied to seismic retrofit design of a 4-story RC moment frame building as a case study to investigate the applicability of the developed energy dissipation device.

2. Proposed viscoelastic damper

2.1. Configuration of the viscoelastic damper

A schematic configuration of the viscoelastic damper developed in this research is shown in Fig. 1, and the dimensions of the viscoelastic dampers manufactured for the tests are shown in Fig. 2. The damper consists of three main steel plates and two layers of viscoelastic material pad with the diameter of 450 mm, which are placed between the three steel plates. Two upper and one lower steel plates are added as fillers, which also provide fail-safe gaps for the viscoelastic pads. The VED pads are produced in two different thicknesses of 22 mm and 17 mm. The thicknesses of the internal and external steel plates are 15 mm and 25 mm, respectively. The viscoelastic pad is made of high damping rubber, which has the following chemical composition: natural rubber (34.0%), antioxidant and vulcanizing agent (8.0%), damping agents (58.0%) such

as synthetic rubber and liquid rubber, and carbon black. The pads are strongly bonded to the main steel plates under high temperature and pressure. To prevent damage to the VE pads, a 70 mm gap between the upper and lower plates is adjusted. This gap ensures that even in large displacements caused by extreme earthquakes the viscoelastic dampers remain undamaged and work stably throughout the earthquake.

2.2. Analytical model

In practice, conventional analytical models such as *Maxwell* and *Kelvin-Voigt* models are often used to simulate the behavior of viscoelastic materials. These models consist of springs and dashpots, which represent the characteristics of ideal elastic solids and *Newtonian* liquid, respectively. Specifically, the *Kelvin-Voigt* model is defined by an elastic stiffness, K_{ve} , and a viscous damping, C_{ve} , which are connected in parallel. Therefore, the damper force F_{VE} is the sum of spring and dashpot forces:

$$F_{VE} = F_D + F_R = C_{ve}(\dot{u})^\theta + K_{ve}u \quad (1)$$

where F_D is a damping force, F_R is a restoring force, u is the relative shear deformation of VE material, and θ is exponent of deformation rate. For vibrations associated with small deformations, the *Kelvin-Voigt* model is

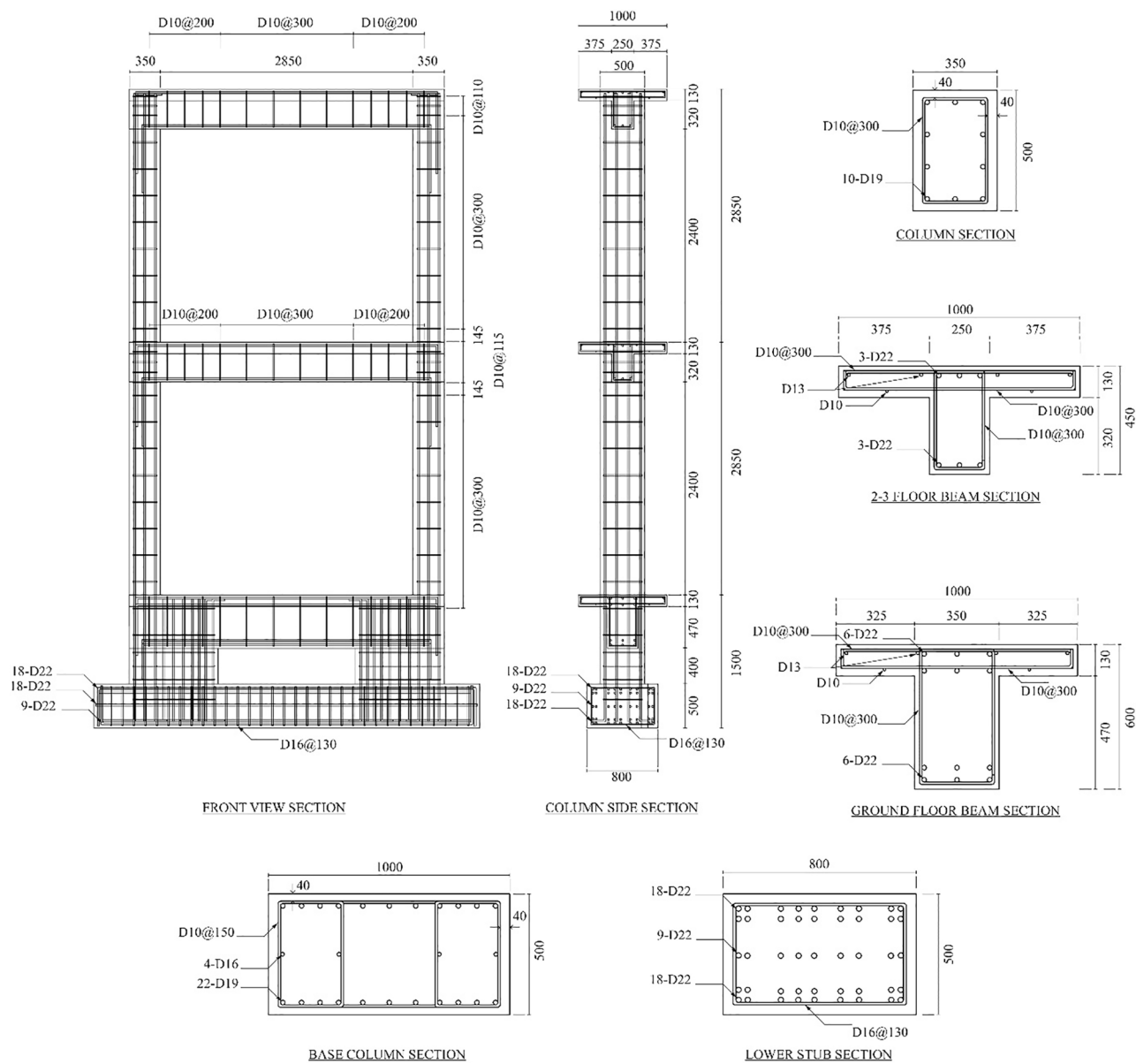


Fig. 4. Details of two-story RC frame test specimen.



Fig. 5. Strength tests of concrete and rebar.

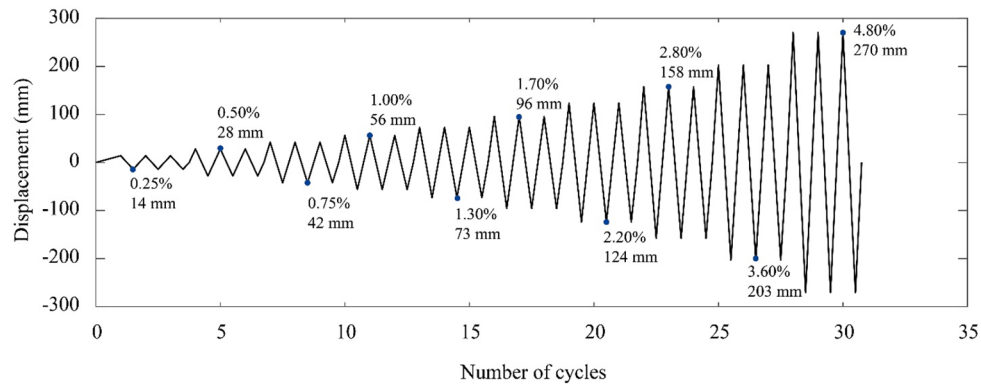


Fig. 6. Loading protocol used in the experiments on RC frames.

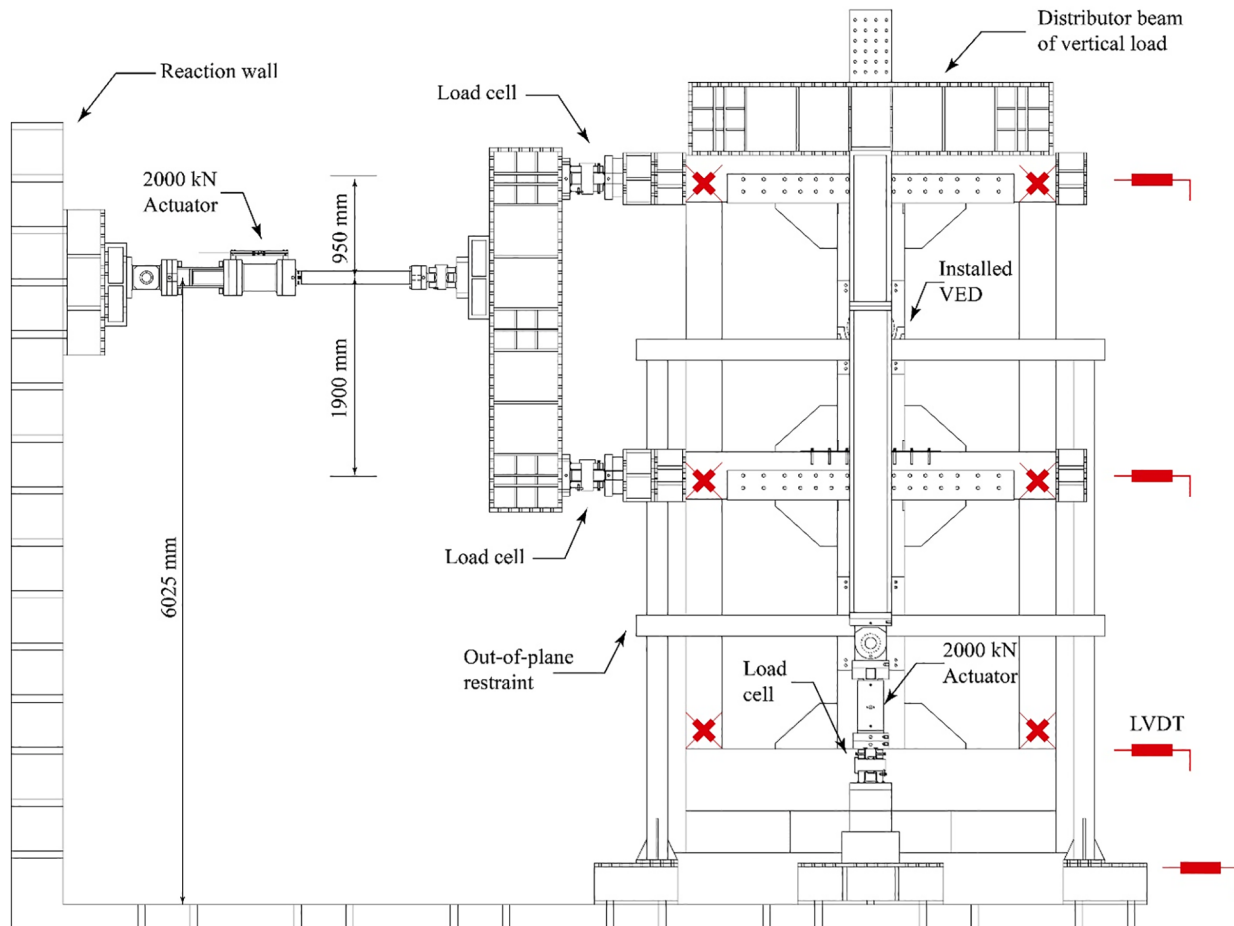


Fig. 7. Test setup for the 2-story RC frame installed with VEDs.

generally good at simulating the behavior of viscoelastic materials. However, it has been reported that the model shows considerable deviation from reality in the simulation of viscoelastic materials under large deformations [14].

In this study, the *Bouc-Wen* model with a plastic Wen element was used to represent the restoring force instead of an elastic spring element used in the *Kelvin-Voigt* model. The equation of nonlinear motion of a single degree of freedom oscillator and the forcing function $f(t)$ can be defined as

$$f(t) = F_I + F_D + F_R \quad (2)$$

where F_I is inertia force, and F_R is restoring force expressed as

$$F_R = F_e + F_h = \alpha k_0 \gamma + (1 - \alpha) k_0 z \quad (3)$$

where F_e and F_h are the elastic and hysteretic components, respectively. The parameter α is the ratio of final tangent stiffness k_t and the initial stiffness k_0 , and z is the hysteretic displacement which is calculated using the first order nonlinear differential equation:

$$\dot{z} = A\dot{u} - [\gamma\dot{u}|z|^n + \beta|\dot{u}||z|^{n-1}z] \quad (4)$$

where A , β , γ and n are the parameters controlling the shape of the hysteretic curve. More details about the plastic Wen element can be found elsewhere [14,19].

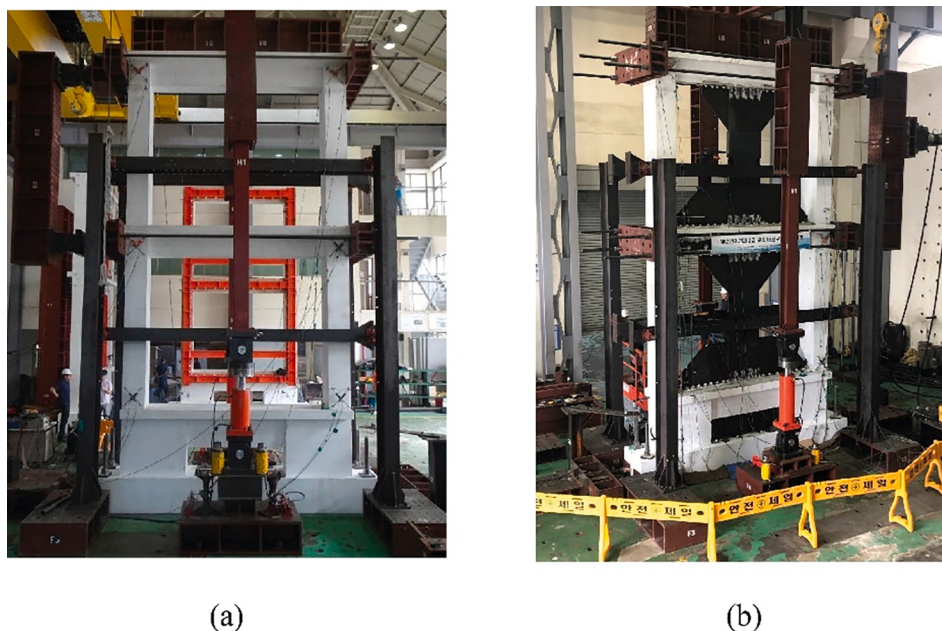


Fig. 8. Photographs of the test frames: (a) Bare frame; (b) Retrofitted frame.

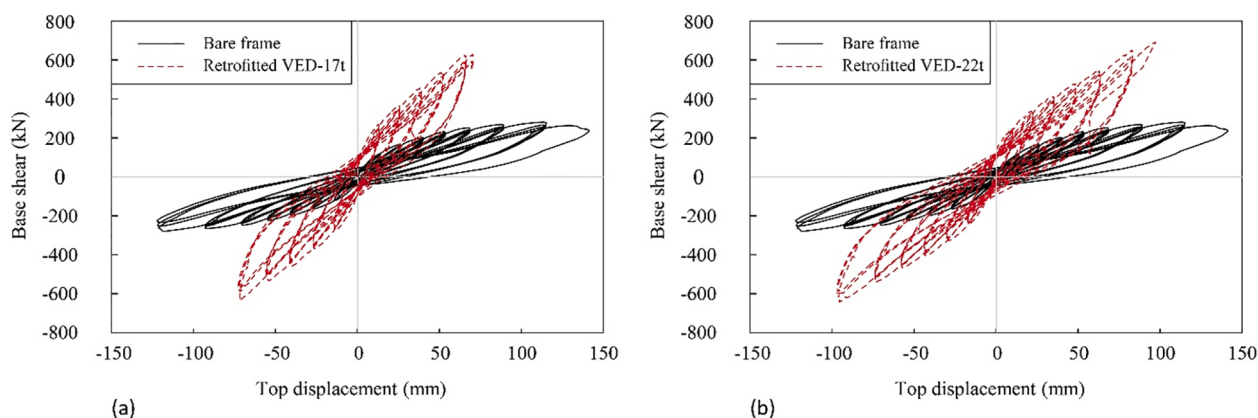


Fig. 9. Comparison of the base shear-top displacement relationships of the bare frame and the retrofitted frames; (a) Retrofitted with 17 mm VEDs, (b) Retrofitted with 22 mm VEDs.

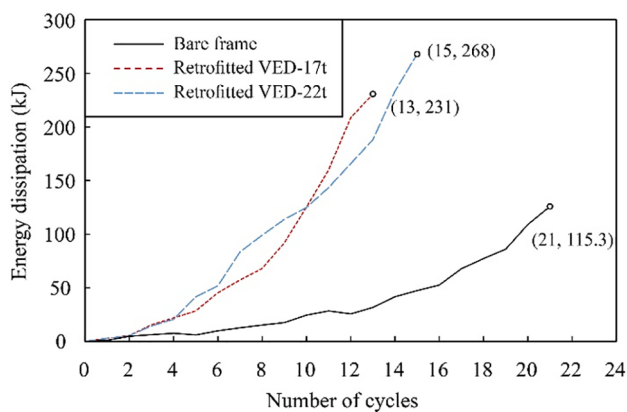


Fig. 10. Comparison of dissipated energy in the bare and the retrofitted frames.

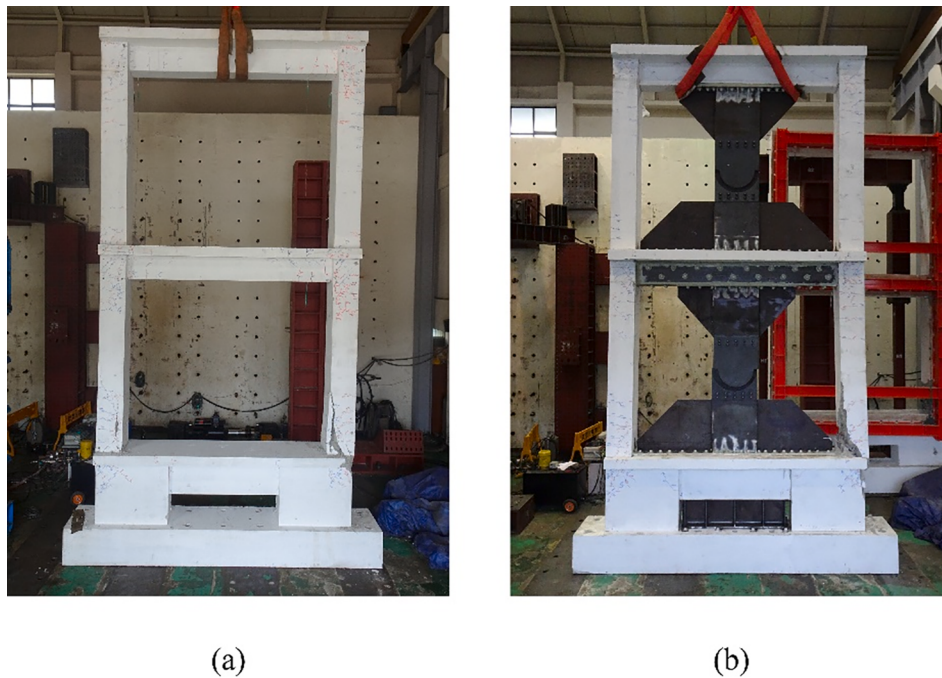


Fig. 11. State of damage in the test specimens: (a) Bare frame; (b) Retrofitted frame with 22 mm VED.

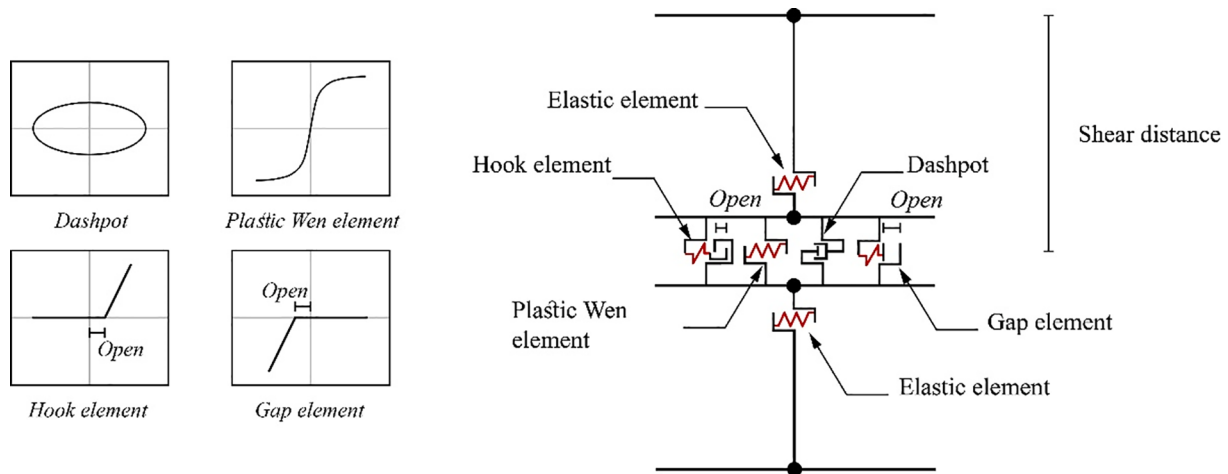


Fig. 12. Analytical model of the VED and the installation plates.

The initial storage modulus G'_0 and the loss modulus G'' of a VE material panel can be obtained using the following relationships:

$$G'_0 = \frac{k_0 t}{A_b} \quad (5)$$

$$G'' = \frac{C_{ve} \omega t}{A_b} \quad (6)$$

where A_b , t , and ω are the bonded area, thickness of the viscoelastic pad, and the natural frequency, respectively. The behavior of the VED was modeled by the Bouc-Wen model using the storage and loss moduli obtained from the cyclic loading tests of the damper, which are summarized in Tables 1 and 2. The test results of the developed damper subjected to various loading frequencies and amplitudes under three different temperatures are reported in other paper [21]. To estimate the parameters of the Bouc-Wen model based on the test results, the nonlinear least-squares optimization method was implemented.

The developed numerical model of the VED was validated by

comparison of the analytical and the experimental results obtained at different amplitudes as shown in Fig. 3, where it can be noticed that the simulation results are in good agreements with the experimental data.

3. Experiments of two-story RC frame

To investigate the effectiveness of the proposed viscoelastic damper in the seismic retrofit of reinforced concrete structures, an experimental study was conducted on three two-story RC frames: one bare frame and two frames retrofitted using the VED with 22 mm and 17 mm-thick viscoelastic pads. In the retrofitted frames, one VED was located in each story in the mid-bay, and the beams were reinforced with triangular rib plates and 6 mm thick steel plates on their side and bottom faces to prevent local failure. Fig. 4 shows the details of the bare frame used in the experiment. The member size and rebar details were obtained from guidelines for structural design of school buildings published in the 1980s. As a seismic design code was first applied in the 1990's in Korea, the design did not consider earthquake loads. The frame has an overall

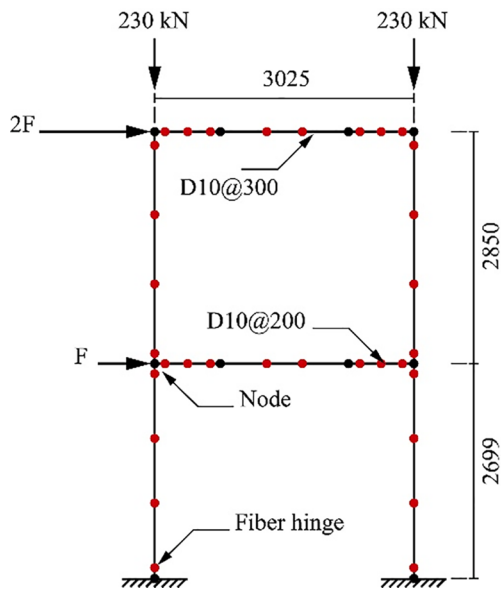


Fig. 13. Analysis model of the two-story test frame.

height of 7.2 m, a story height of 2.85 m, and an overall width of 3.55 m. Two short stub columns were added to the bottom of the frames. Even though the height and member sizes are the same as those recommended in the guidelines, the width of the frame was reduced to 2/3 of the regular size due to the limitations of the test facilities. The average 28-day compressive strength of the concrete is 34 MPa, which was obtained from 3 cylindrical specimens. The mean yield and ultimate strength of the steel rebars, obtained from tensile tests of 12 rebar specimens with four different sizes, were 496 MPa and 588 MPa, respectively. Fig. 5 shows the strength tests of the concrete and the rebar used in the construction of the test specimens.

Cyclic load was applied horizontally using a servo actuator with a maximum load capacity of 2000 kN and a maximum displacement of 400 mm. The loading protocol used in the displacement controlled tests was prepared following the guidelines of FEMA 461 [22] and is shown in Fig. 6. The minimum displacement imposed on the frames was 14 mm, which corresponds to an inter-story drift ratio of 0.25%. After three cyclic loadings at the same displacement amplitude, the displacement amplitude was further increased by 14 mm at the next three cycle loading tests. The maximum imposed displacement is 270 mm, which corresponds to an inter-story drift of 4.8% of the story height.

Fig. 7 shows the test setup of the 2-story RC frame installed with VEDs, which depicts the location of the 2000 kN actuator used to apply horizontal cyclic loads at a height of 6025 mm from the ground. The

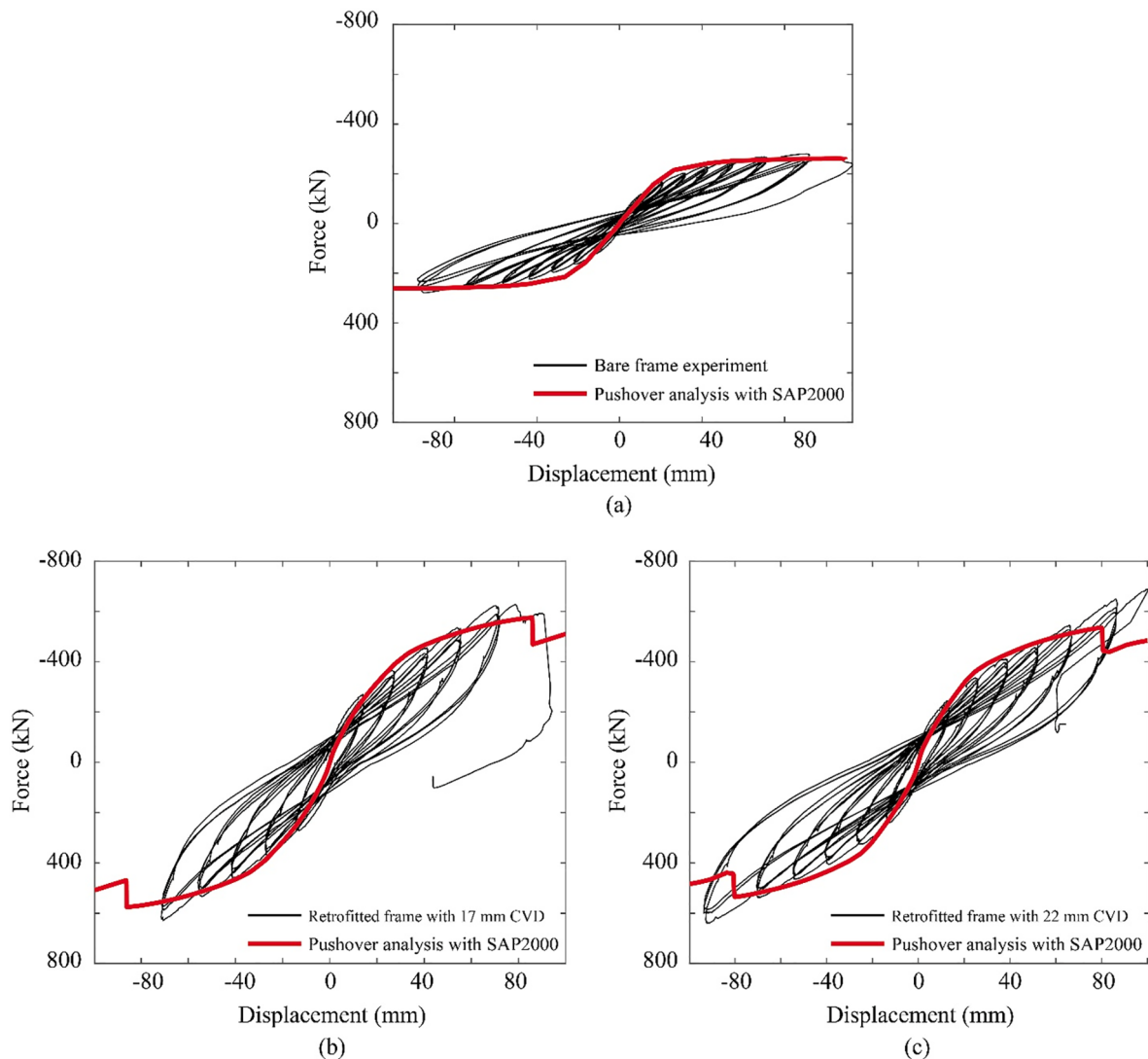


Fig. 14. Comparison of the pushover curves obtained from the analysis model and envelopes of the hysteresis curves.

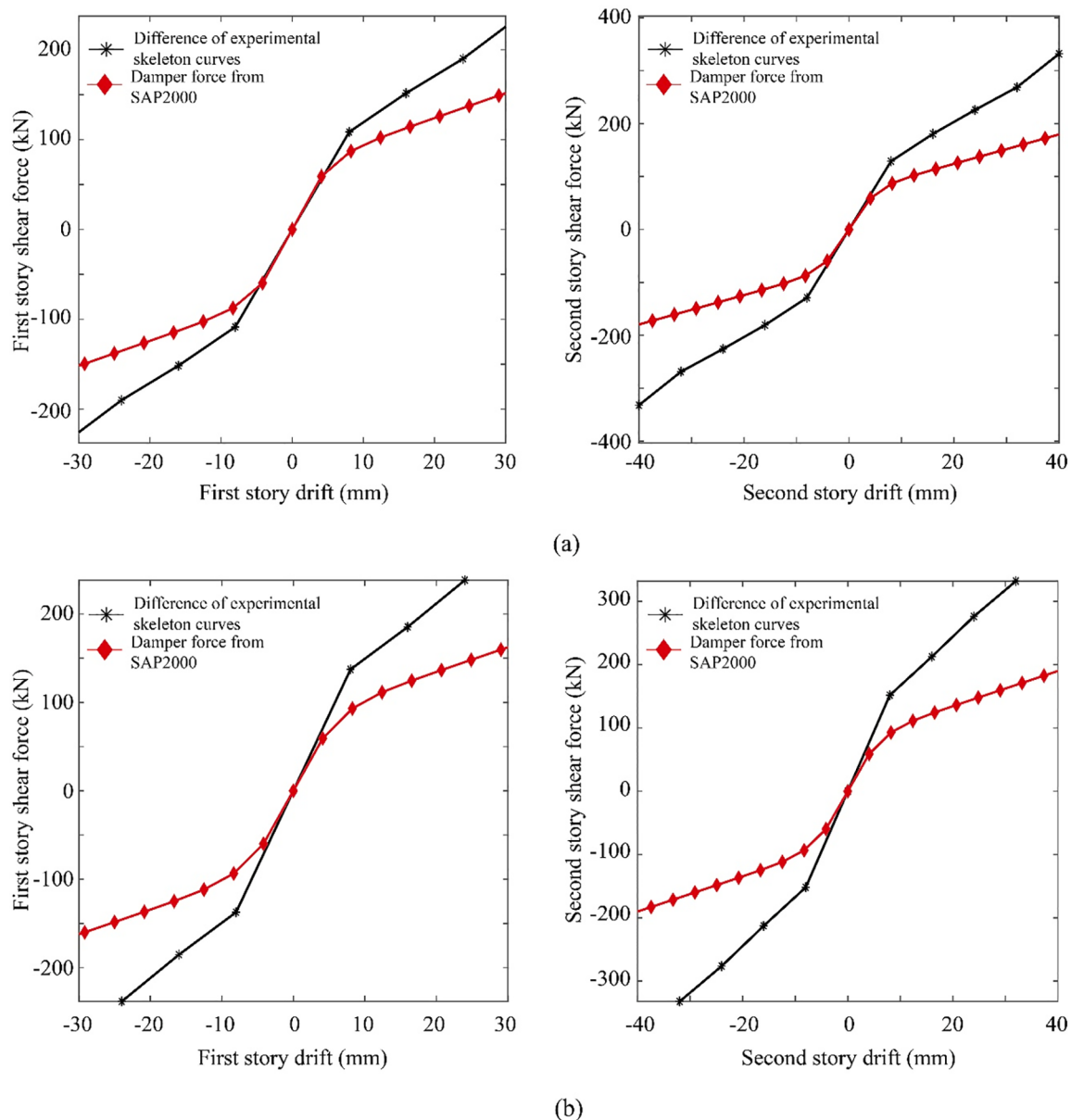


Fig. 15. Comparison of the increased story shear and the added damper force after the retrofit: (a) Retrofit with 22 mm VED; (b) Retrofit with 17 mm VED.

applied actuator force was distributed to both floors in such a way that the displacement of each story is proportional to the height of the story using a stiff H-shape section member as a distributor. An axial force of 230 kN was applied at the top of each column, which was chosen in consideration of the weight of the 130 mm thick slab within the tributary area of each column. A total of 16 LVDTs and 56 strain gages were installed to measure the displacements and strains. Fig. 8 shows photographs of the bare and retrofitted frames prepared for the tests.

The hysteresis curves of the roof displacement versus the applied actuator force are depicted in Fig. 9 for the bare frame and the frames retrofitted with 17 mm and 22 mm thick VEDs. In the bare structure, flexural cracks formed first at the lower parts of the first story columns, final failure occurred due to the propagation of shear cracks at the bottom of the first story column. The maximum inter-story drifts at final failure were 67 mm and 53 mm in the first and the second stories, respectively, which correspond to 2.3% and 1.9% of the story height, respectively. Similar crack propagation patterns were observed in the specimens with VEDs.

In the structure retrofitted with 22 mm dampers, the maximum inter-

story drifts at final failure were 60 mm and 42 mm in the first and the second stories, respectively, which corresponds to 2.1% and 1.5% of the story height, respectively. In the structure retrofitted with the 17 mm dampers, which are stiffer than 22 mm dampers, the maximum inter-story drifts at final failure were further reduced to 41 mm and 30 mm in the first and the second stories, respectively, which corresponds to 1.4% and 1.1% of the story height, respectively. The reduction in maximum inter-story drift after the retrofit can be explained by the observation that the steel plate reinforcement significantly increases the stiffness of the beams, and as a result, damage is more concentrated in the columns. This strong beam weak column state is partly because the width of the test frame is reduced to 2/3 of the regular size due to the restrictions of the test facilities. Even though the deformation capacity of the frame is somewhat reduced due to the installation of the VEDs, the ductility, i.e. the ratio of the maximum and yield displacements, has not been reduced, and the stiffness and strength of the RC frames were significantly increased after the retrofit.

The cumulative dissipated energy in the bare frame and the retrofitted frames versus the number of loading cycles is shown in Fig. 10. An

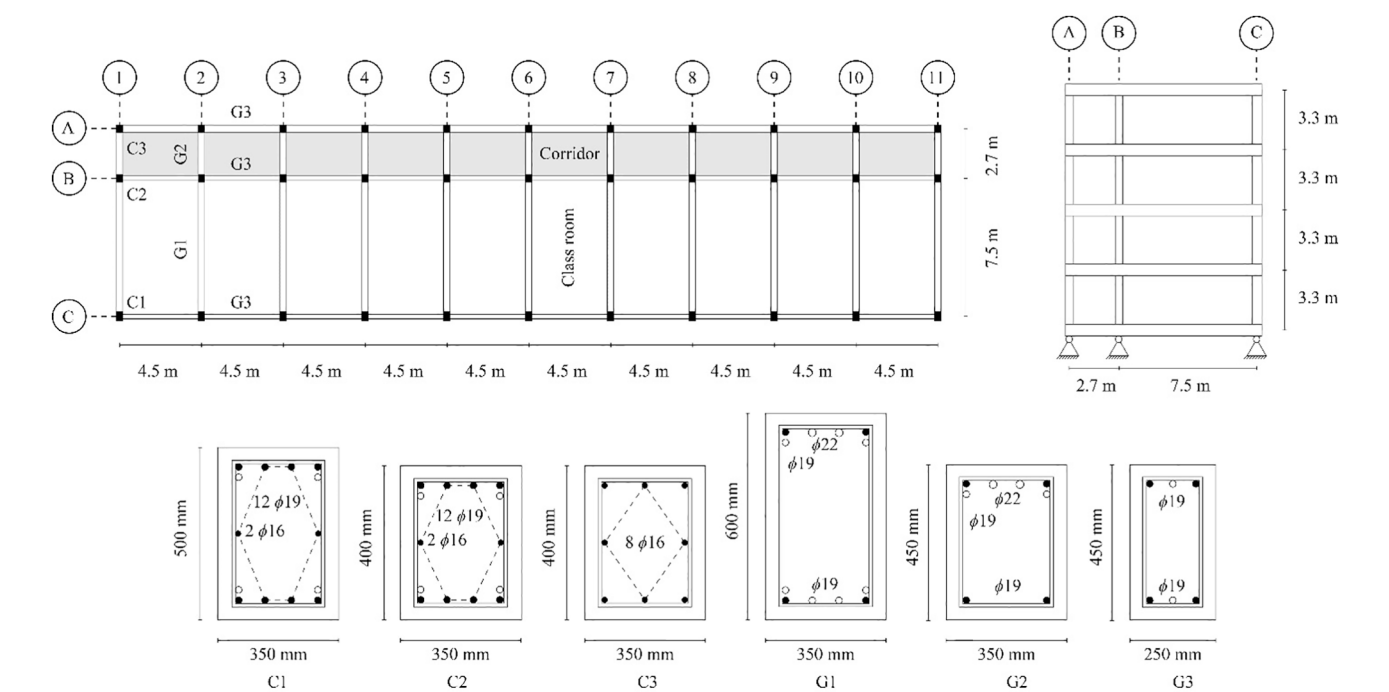


Fig. 16. Details of school building.

Table 3

Design gravity loads.

Area	Dead load (kN/m ²)	Live load (kN/m ²)
Class room	4.42	3.00
Corridor	4.42	5.00
Roof	4.82	3.00

Table 4

Material properties used in the analysis.

Material	Elastic modulus	Over strength factor	f_c	f_y
Concrete	1.9×10^4 MPa	1.2	18 MPa	–
Steel	2.1×10^5 MPa	1.25	–	240 MPa

Table 5

Earthquake records used in the analysis.

Earthquake	Station	Magnitude	Rupture distance (km)	PGMD record no.
Kern County	Pasadena	7.36	125.59	13
Kern County	Santa Barbara Courthouse	7.36	82.19	14
Tabas - Iran	Tabas	7.35	2.05	143
Imperial Valley	Coachella Canal #4	6.53	50.1	166
Superstition Hills	Brawley Airport	6.54	17.03	719
Loma Prieta	San Francisco	6.93	63.15	804
Landers	Mission Creek Fault	7.28	26.96	880

energy of 115.3 kJ is dissipated in the bare frame after 21 cycles of loading while an energy of 268 kJ and 231 kJ, respectively, are dissipated in the retrofitted frame with 22 mm and 17 mm VEDs. Fig. 11 depicts the damage states of the test specimens, where it can be observed that in both specimens severe shear failure occurred at the bottom of the first story columns.

To compare these experimental results with numerical simulations, analytical models of the bare frame and the retrofitted frames were created using commercial software, SAP2000 [20]. The VEDs were modeled using the analytical model shown in Fig. 12. The viscoelasticity of the damper was modeled using the developed Bouc-Wen model consisting of a dashpot and a plastic Wen element to account for the viscosity and stiffness of the VED. In order to consider the elastic deformation of the steel plates above and below the viscoelastic pads, two elastic links were used in series with the VED. The gap between the upper or lower plate and the main damper steel plate was modeled using a hook and a gap elements. The shear distance was applied in consideration of the bending moment on the beams caused by the damper force.

The columns were modeled using 8 unconfined and 9 confined concrete fiber elements and a fiber element for a rebar. The beams were modeled by 5 concrete fiber elements and rebar fiber elements, which were automatically created by SAP 2000. As the fiber elements are more computationally intensive requiring more computer storage and execution time, the SAP2000 manual [23] recommends to find proper number of fibers considering balance between accuracy and computational efficiency. In this study the above numbers of fiber elements were chosen since the analysis model could sufficiently account for the nonlinear behavior of the test specimens. The analysis model of the two-story frame is shown in Fig. 13. The stress strain relations of the fibers were constructed based on the strength tests of concrete and steel rebars. The expected concrete and rebar strengths were obtained based on the test results. The elastic modulus of concrete was obtained as [24]

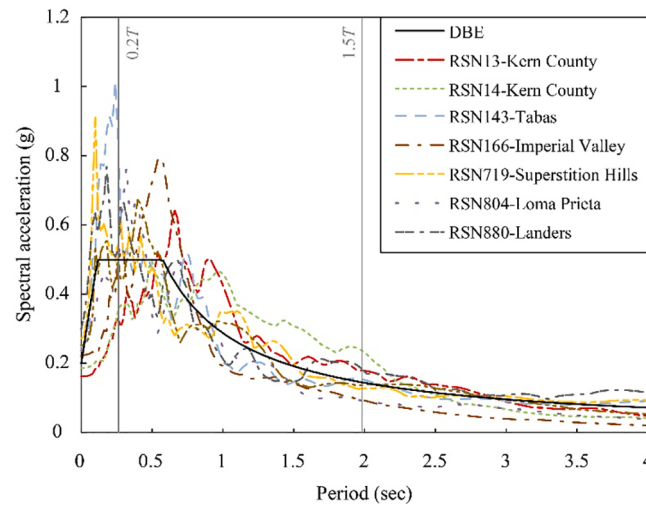


Fig. 17. Response spectrum of the scaled ground motion records.

$$E_c = 4700\sqrt{f'_c} = 2.7 \times 10^4 \text{ MPa} \quad (7)$$

where f'_c is the specified concrete strength.

The elastic modulus of steel was assumed to be 2.0×10^5 MPa. The nonlinear behaviours of the beams and columns were obtained from ASCE 41-13 [26]. Quasi-static pushover analysis was carried out in SAP 2000 to simulate the envelop curves of the test results. The pushover analysis results of the bare and the retrofitted frames are plotted in Fig. 14 along with the hysteresis curves obtained from the cyclic loading tests. It can be observed that the analytical model can properly predict the envelop curves of the VEDs that were obtained from experiments.

As mentioned earlier, the rib plates and the steel plates were added to reinforce the beams of the RC frames, which makes the frames much stiffer. Fig. 15 depicts the beam reinforcement effects on each story using the damper force and the difference between skeleton curves of shear force-drift before and after the retrofit. The damper force was determined using the validated analysis model of the damper, and the difference in skeleton curves on each story was obtained from the experimental data. The fact that the increase in maximum force after the retrofit is larger than the strength of the damper itself indicates the contribution of strengthening the beams.

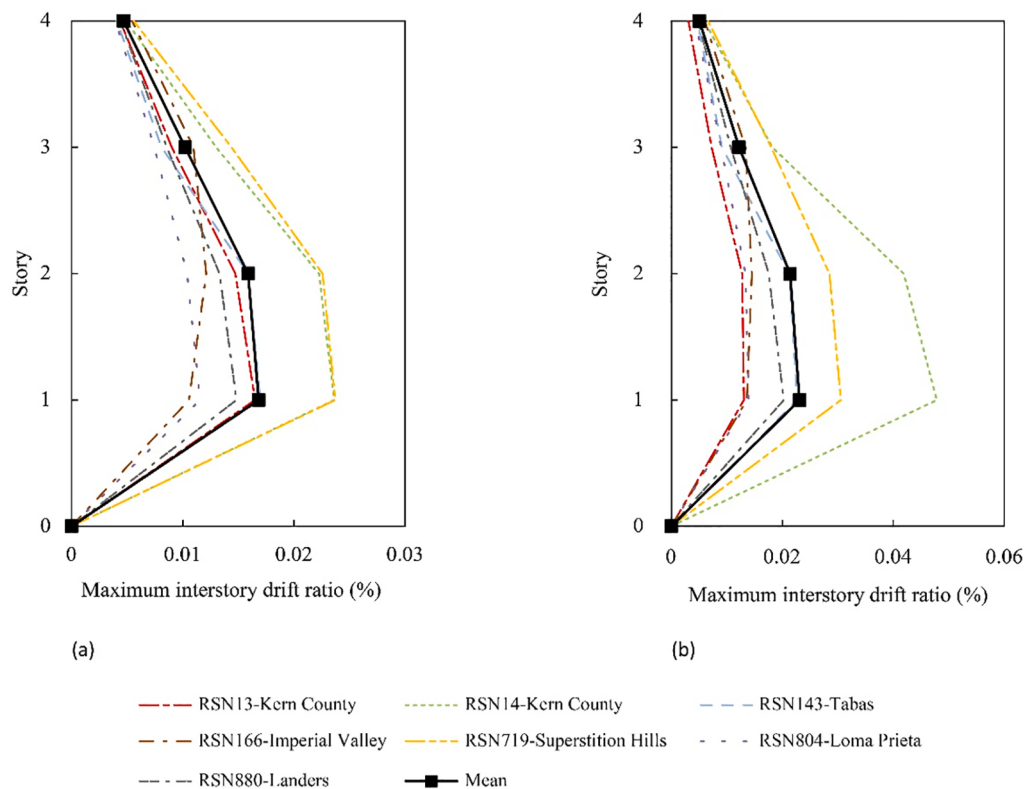


Fig. 18. Maximum inter-story drift ratios of the model structure subjected to the seven earthquakes: (a) DBE level; (b) MCE level.

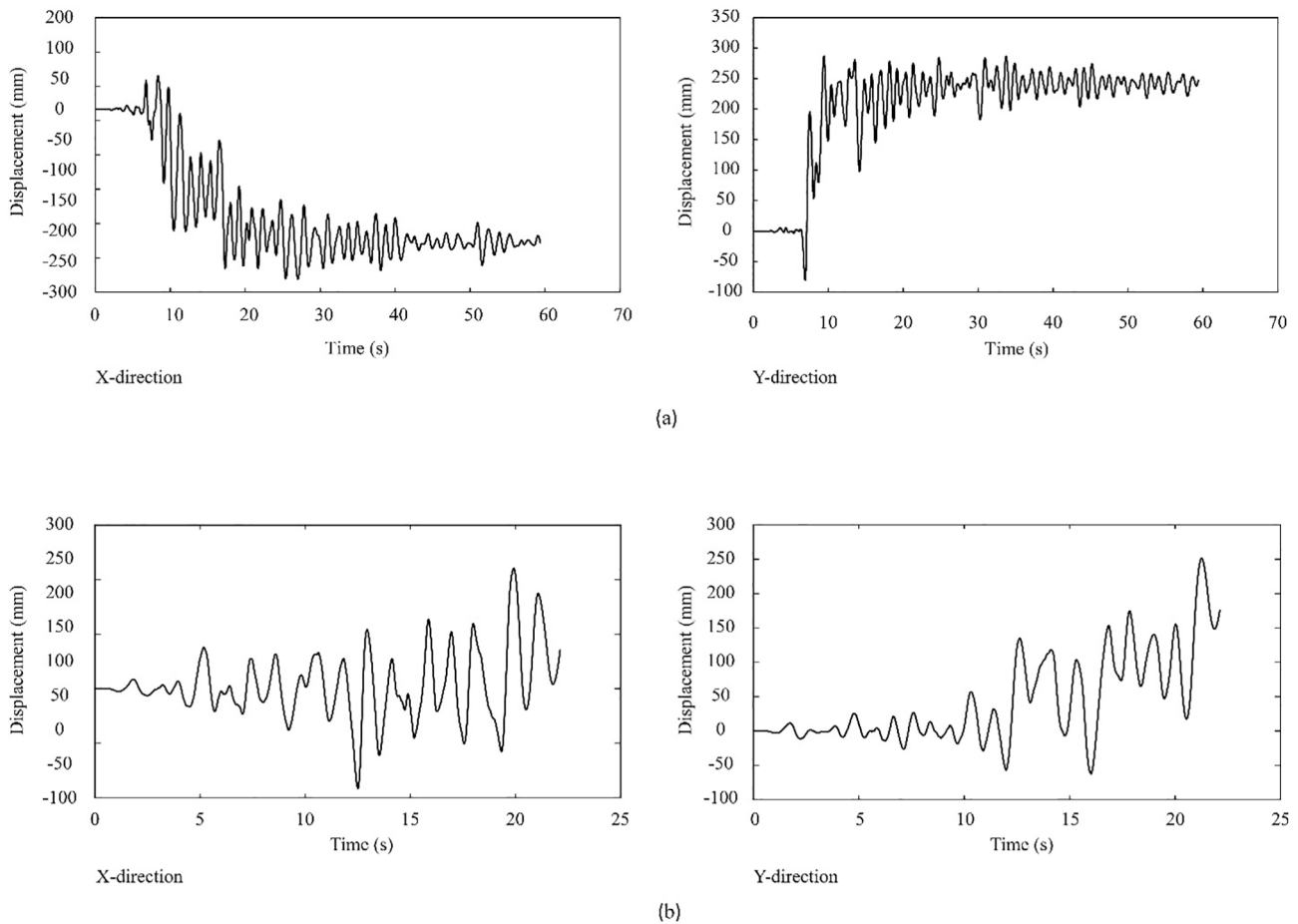


Fig. 19. Displacement time history of the example building subjected to MCE level earthquakes: (a) RSN 14; (b) RSN 719.

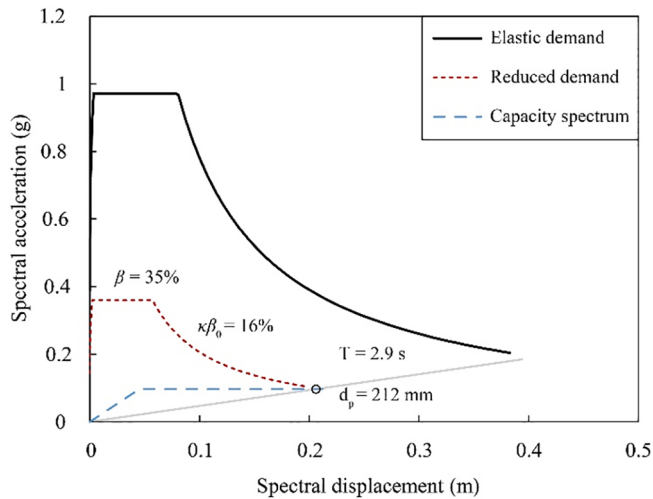


Fig. 20. Determination of effective damping ratio to satisfy the limit states.

4. Application to seismic retrofit of a case study building

4.1. Structural details

In this section, a RC framed building designed in the 1980s without consideration of seismic loads was selected as the case study structure in order to evaluate the efficiency and applicability of the developed VEDs. Fig. 16 shows the plan, elevation, and sectional dimensions as well as reinforcement details of the model structure. Table 3 lists the design gravity loads applied in the design of the structure. The structure was modeled in SAP 2000 according to ASCE 41-13 [26]. The fundamental natural period of the model structure is 1.31 s, and a fundamental modal damping ratio of 5% of critical damping was utilized in the analyses. Frame elements in SAP2000 were applied to model the RC beams and columns, considering the longitudinal and shear reinforcement. Plastic hinges were assigned at the ends of the RC beams and columns to consider the non-linear behavior of the members. The parameters of the plastic hinges were defined based on the suggestions of ASCE 41-13 [26]. In consideration of cracked sections, the flexural stiffness of the beams and columns was decreased to 50% of the uncracked values, and the shear stiffness was decreased to 40% of the original value.

The nominal compressive strength of the concrete used in the model

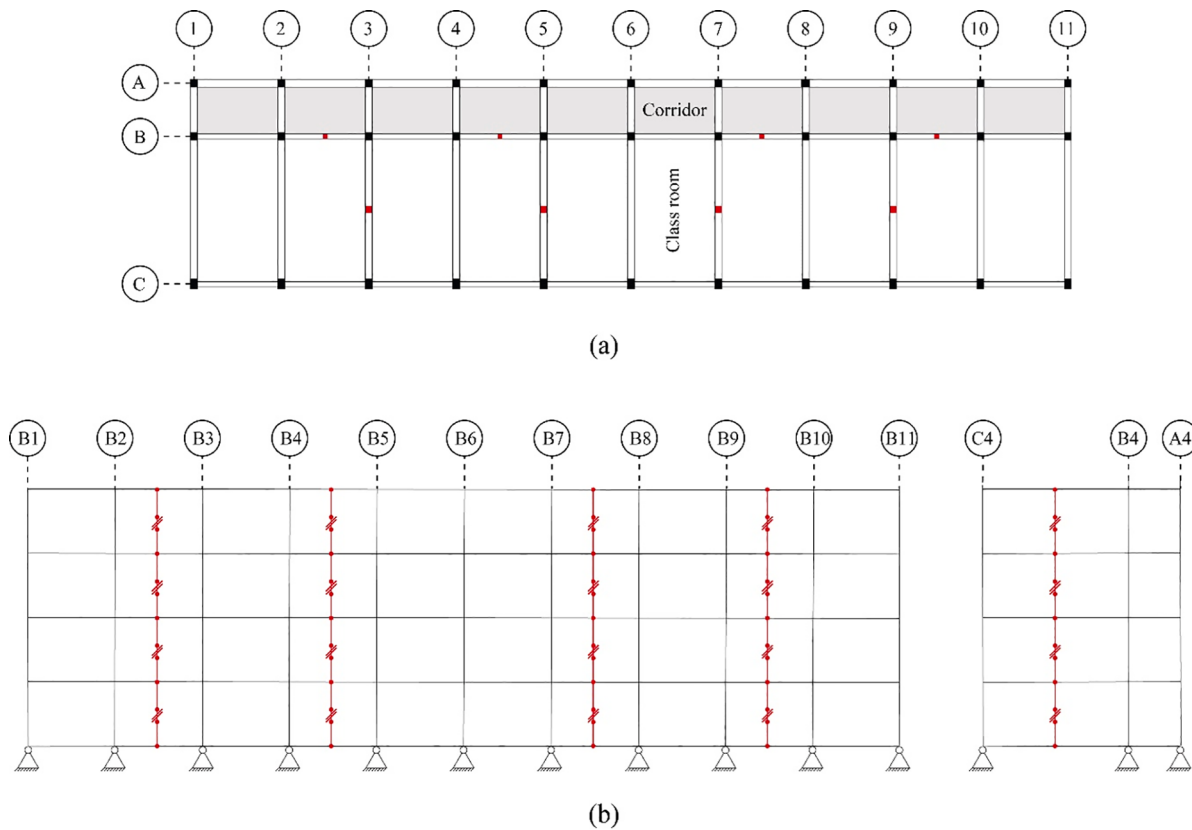


Fig. 21. Location of VEDs: (a) Plan view; (b) Elevation view.

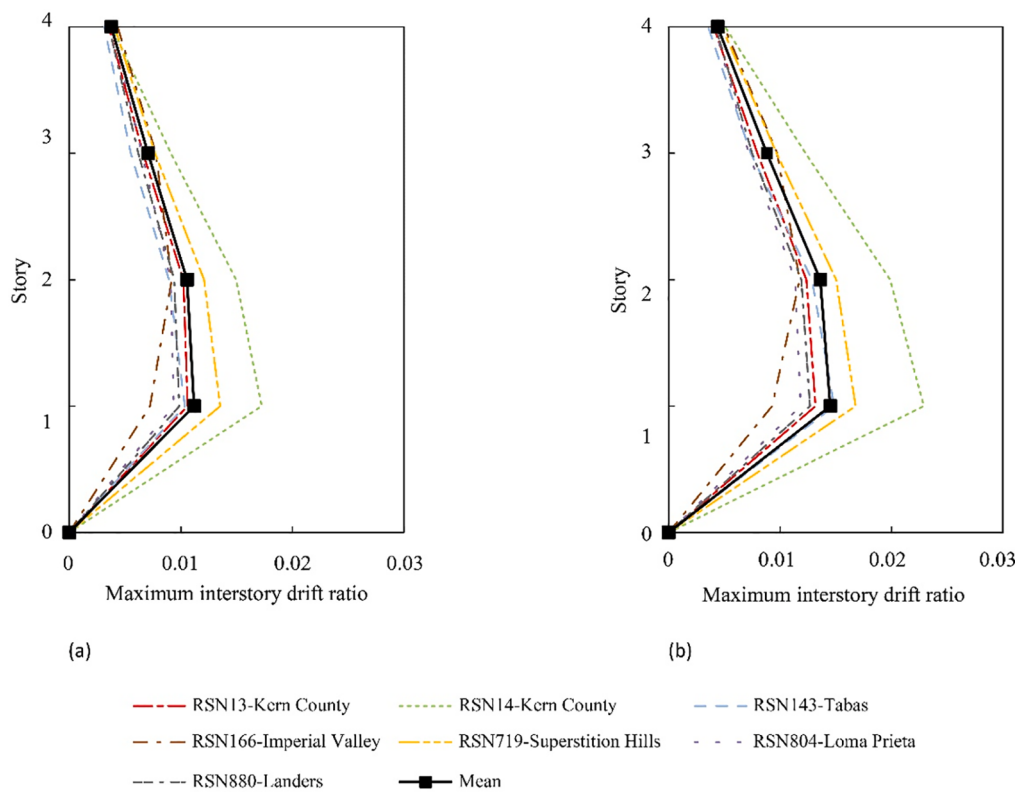


Fig. 22. Maximum inter-story drift ratio of the retrofitted structure: (a) DBE level; (b) MCE level.

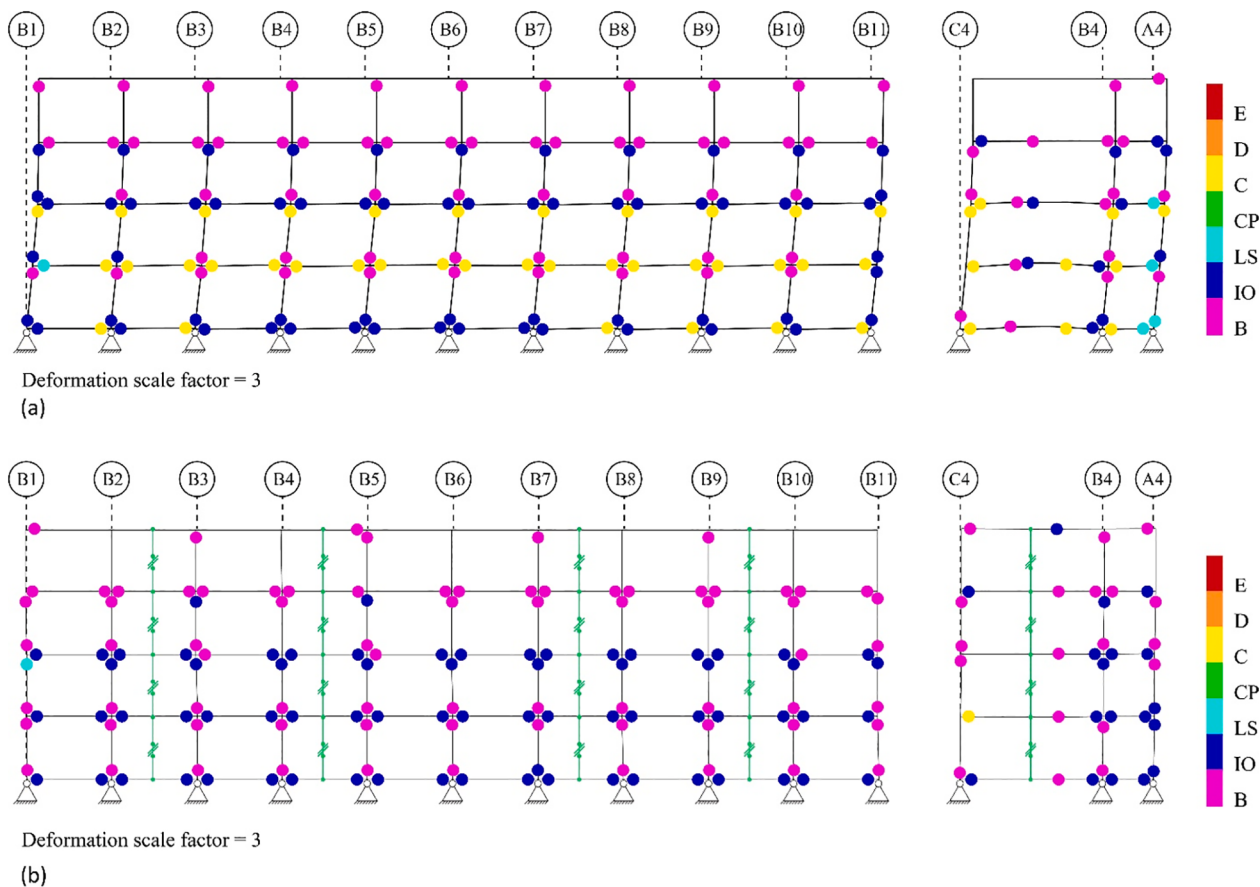


Fig. 23. Plastic hinge formation for the RSN14-Kern County ground motion (MCE): (a) Bare structure; (b) Retrofitted structure.

structure was assumed to be 18 MPa and the yield strength of steel was 240 MPa. The formula of Eq. (7) was used to obtain the elastic modulus of the concrete. The expected compressive strength of concrete and the yield strength of steel were determined by multiplying factors of 1.2 and 1.25, respectively, according to the Korea Building Code. Table 4 shows the material properties used in the analysis.

4.2. Seismic performance of the model structure

To evaluate the performance of the VEDs, seven earthquake records were chosen from the PEER NGA database [27] as shown in Table 5, and were scaled to meet the design spectrum of Korea. The design spectral response acceleration parameters are $S_{DS} = 0.498$ g for short periods and $S_{D1} = 0.287$ g for the period of 1 s. Each ground motion record was scaled so that its square root of the sum of the squares (SRSS) spectrum does not fall below the design spectrum between the period range from 0.2 to 1.5 times the fundamental natural period [28]. The design spectrum for the site class D and the response spectra of the scaled ground motion records are shown in Fig. 17.

The seismic performance of the bare structure subjected to the scaled earthquake ground motions are shown in Figs. 18 and 19. Fig. 18 shows the maximum inter-story drift ratio at each level, in which the mean values of the maximum inter-story drift ratios are 1.7% and 2.3% for the design basis earthquakes (DBE) and maximum considered earthquake (MCE), respectively. These values turned out to exceed the 1% and 2% limit stated for the DBE and MCE, respectively. Fig. 19 shows the roof displacement time

history of the structure subjected to the RSN 14 and 719 earthquakes scaled to the MCE level, where it can be observed that at the end of the ground motions, the structure shows significant residual displacements.

4.3. Seismic retrofit of the example structure

For seismic retrofit design of the example building, the capacity spectrum method presented in ATC-40 [29] was used to find out the first trial value for the required number of viscoelastic dampers. To this end, pushover analysis was carried out using a lateral load proportional to the fundamental mode shape of the structure to obtain the capacity spectrum, and then this was bi-linearized and transferred into the acceleration-displacement response spectrum as described by Fig. 20. The elastic demand spectra with a 5% inherent damping ratio were also defined for maximum considered earthquake (MCE) level earthquake. When the structure deforms into a nonlinear region, the demand spectra is reduced based on the effective damping ratio β_{eff} . According to ATC-40, the provided damping ratio for the performance point d_p is $\kappa\beta_0$ in which hysteresis loops, degradation, etc. are all considered depending on the type of structure and ground shaking duration.

The case study structure was assumed to show type C structural behavior as prescribed in ATC-40, in which the structure consists of noncomplying primary elements with poor or unreliable hysteretic behavior. To calculate the effective damping required to reach the given limit states, the performance points d_p for DBE and MCE levels are calculated so that the average maximum inter-story drift ratios for each

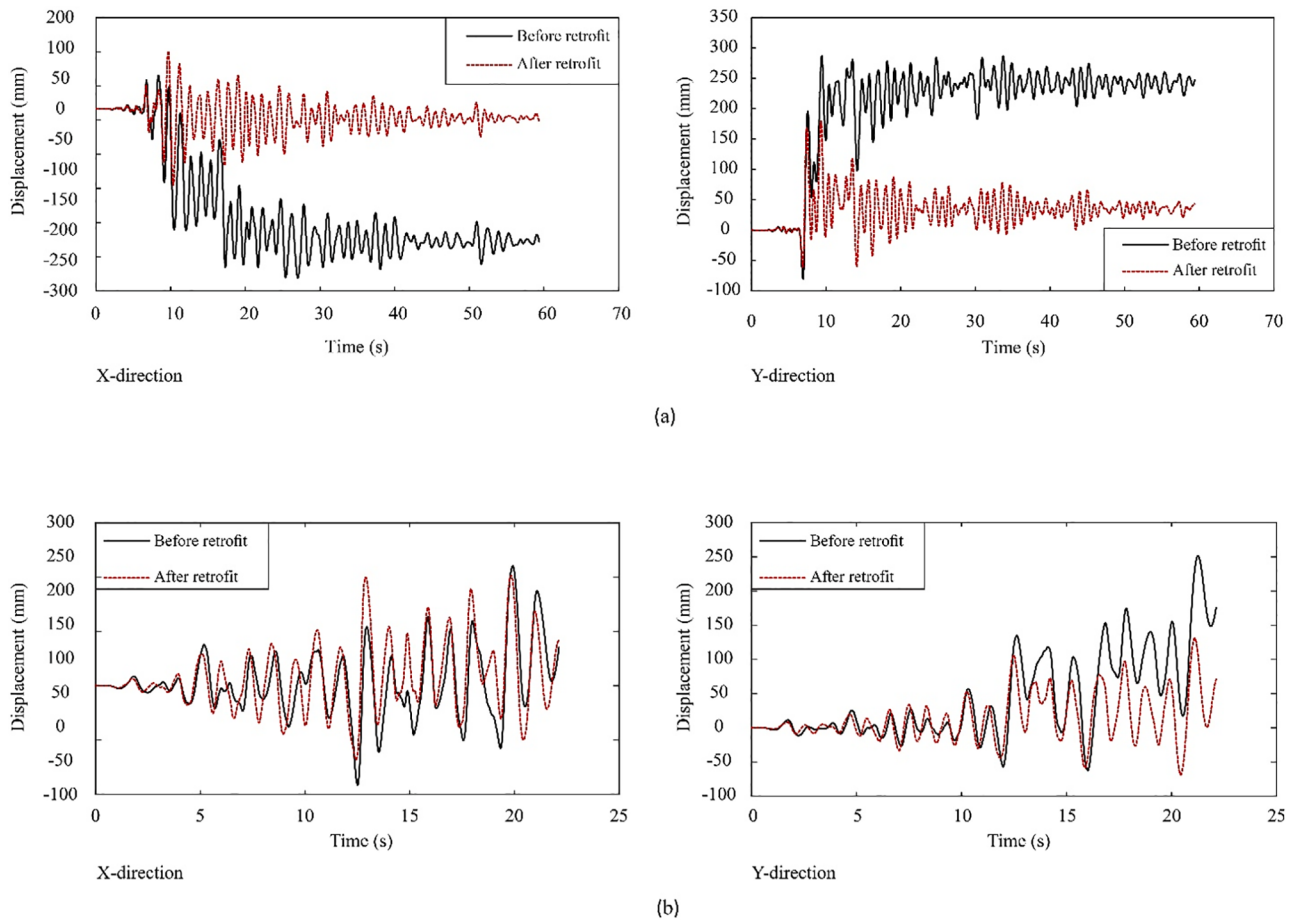


Fig. 24. Comparison of roof displacement time history before and after retrofit: (a) RSN 14; (b) RSN 719.

story are respectively 1% and 2%. Considering the story height of 3.3 m, these drift ratios correspond to 132 mm and 264 mm, respectively. The damping ratios provided by the structure, $5\% + \kappa\beta_0$, for these performance points are 17% and 21%, whereas the required effective damping ratios, β_{eff} , to meet these limit states are 45% and 35%, respectively. The extra required damping is provided by the dampers and is calculated according to ASCE 41-13 [26] by,

$$\beta_{eff} = 5\% + \kappa\beta_0 + \frac{\sum_j W_j}{4\pi W_k} \quad (9)$$

where W_j is the work performed by the linear viscous device j in one complete cycle, and W_k is the maximum strain energy in the frame.

The aforementioned procedure estimated the required number of dampers on each story as four in both directions for the MCE level earthquake. Based on the result, 4 dampers were installed in each story in each direction as depicted in Fig. 21, and the seismic performance of the retrofitted structure was further evaluated using nonlinear time history analysis. Fig. 22 shows the maximum inter-story drift ratios of the model structure for both sets of the seven DBE and MCE ground motions. It can be seen that the mean values of the maximum inter-story drift ratios are 1.1% and 1.5% for DBE and MCE motions, respectively, which are considered to marginally satisfy the 1% and 2% limit states for DBE and MCE motions, respectively.

Plastic hinge formations in the structure before and after the retrofit when subjected to the RSN14-Kern County ground motion are compared in Fig. 23. It can be noticed that the implementation of the VEDs shifted the plastic hinges from collapse state to immediate occupancy range.

Fig. 24 compares the time history displacements of the roof story subjected to the RSN 14 and RSN 719 earthquakes before and after the retrofit. It can be seen that for both the RSN 14 and RSN 719 earthquakes the residual displacements are significantly reduced in both directions.

5. Conclusions

New viscoelastic dampers with fail-safe mechanism were developed to improve the seismic performance of existing buildings against earthquake ground motions especially at large displacement. To assess the seismic performance of the VEDs, cyclic loading tests were conducted on a 2 story RC frame with and without seismic retrofit. The VEDs were also applied to a seismic retrofit design of a 4-story RC framed case study structure designed without consideration of seismic load.

The results of the cyclic loading tests verified that the VEDs were effective in increasing the stiffness, strength, and energy dissipation capacity of the test structure. The analysis model of the VEDs turned out to predict the envelop curve obtained from the cyclic load test quite well. In the case study structure that was subjected to seven ground motions, the mean maximum inter-story drift ratios were reduced from 1.7% and 2.3% for DBE and MCE motions to 1.1% and 1.5%, respectively, after the retrofit using the proposed VEDs. It was also observed that the residual displacements were significantly reduced in both directions after the seismic retrofit. Based on the experimental results and analytical simulations, it could be concluded that the proposed viscoelastic dampers with fail-safe mechanism are effective in enhancing the seismic safety of structures.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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