

Article

Seismic Performance of a New Structural Design Solution for First-Story Isolated RC Buildings with Coupled Beam-Column Connections

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Abstract: This study proposes a new structural design of the first-story isolation system in reinforced concrete (RC) structures. Compared to the conditional buildings with independent columns, this new design integrates the independent columns with beams to increase the seismic capacity of the building by increasing the integrated stiffness of the coupled columns and the stability of the isolation system. The seismic responses of the proposed structure and the corresponding isolation effect were investigated by performing a series of numerical simulation and shaking table tests on a typical 7-story RC frame structure. The structure models were subjected to four earthquake waves with two PGAs (peak ground acceleration) of 0.30 g and 0.40 g for seismic analysis regarding the peak acceleration and inter-story displacement. Both simulation and testing results showed that the story acceleration and inter-story displacement of the superstructure in the isolated model decreased significantly. While the substructure below the isolation layer had a negligible decrease of acceleration. The connection of beams with concrete columns significantly increases the seismic capacity of the RC frame buildings compared to non-isolated frame buildings. The coupled beam-column connections could thus be potentially adopted in the practical first-story isolation system to avoid the requirements of large column stiffness and large column size.

Keywords: seismic design; first-story isolation structure; coupled beam-column connections; shaking table test

1. Introduction

The seismic performance of frame structures has been continually investigated by researchers, mainly through developing structures with maximum energy dissipation capacity [1–9]. In the 1950s, a new type of frame structure with a soft first story was designed by pioneers such as Le Corbusier [10] who applied the soft-first-story idea by lifting the structure off the ground. This kind of building is functionally desirable considering it can provide parking spaces or grand entrances in hotels. The ‘soft story’ structure is classified if it has less lateral stiffness than the story above it according to design standards such as in reference [11].

Initially, designers and researchers believed that the soft first story is like a soft spring that has a positive isolation effect on the whole structure and it thus can alleviate the seismic response of the superstructure above the soft first story [12]. However, a series of earthquake incidents showed that the soft first story was severely damaged after the earthquake, though the superstructures above the first story were only slightly damaged [13,14]. This result could be caused as the lateral stiffness of the soft first story was abruptly decreased from that of the superstructures, resulting in a weak connection between the first story and its above stories. The low bearing capacity could then damage the boundary

conditions when the weak interface moves from an elastic to a plastic state under earthquake conditions. Consequently, a large structure deformation occurred, and the energy generated by earthquakes were accumulated in the weak interface [13–15]. To illustrate the typical earthquake-caused damage in the frame structure with the soft first story, two examples of severe damage in the soft first story of two buildings in Turkey (Figure 1a) and China (Figure 1b) are shown in Figure 1. Although the structures with the soft first story are inherently vulnerable to collapse during earthquakes, it is still in demand especially in urban areas [16]. It is of great importance to improve the seismic design of this type of structures.



Figure 1. Typical earthquake damage to buildings with weak substructure: (a) Strong earthquake in Turkey, 1999 [11]; (b) strong earthquake in Wenchuan, China, 2008 [17].

To improve the seismic performance of soft first story structures, the lateral stiffness of the soft first story can be strengthened. For example, according to the Code for seismic design of buildings [18] the section stiffness of the column in the first soft story can be increased to avoid the building collapsing under strong earthquake excitations; this method (named as the ‘hard-resistant’ method), however, still fails to maintain a good structure safety of the weak interface between the first story and the above superstructure [19]. Although infill panels [4], metal shear panels [5], or curtain walls [20,21] can be used to improve the seismic protection in frame buildings, these panels are normally constructed above the first floor to maintain general space in the first floor for the type of frame structure with a soft first story; thus, the seismic performance of a soft first story is still required for improvement. Montuori and Muscati [6,7] proposed a new design strategy for failure mechanism control of moment resisting frames structures; this new design is under development. Alternatively, the first-story isolation system can be implemented to enhance the seismic performance of frame structures in practice. Generally, flexible laminated rubber bearings [22,23] are inserted between the first floor and the columns that support the first floor [19,24,25]. To the best of our knowledge, the current studies such as reference [26] mainly focus on the steel frame buildings. The experimental study on reinforced concrete (RC) frame buildings with a first-story isolation system were ignored.

The columns in the first story of the story-isolation structure are generally independent columns, which independently support the floor as shown in Figure 1. Several studies have shown that this type of frame structure can significantly increase the seismic capacity by reducing the lateral acceleration and inter-story displacement response of the superstructure above the isolation layer while having no impact in increasing the seismic capacity of the substructure below the isolation layer [16,26]. Thus, the isolation layer could have a large displacement under a strong earthquake excitation, causing a P-Delta effect [27] on the column and producing extra damage to the building. Therefore, the lateral stiffness of the first story columns are expected to be large enough to limit the displacement and stress of the isolation rubber bearing and thus to avoid damage to the rubber bearings when they have a large deformation. This stiffness, however, should be limited to a relatively moderate value as too large a column section with high stiffness would decrease the space size on the first floor and increase the engineering cost. In addition, the stability of the isolation system could be another concern as it may deteriorate when the building is subjected to strong earthquakes in multiple directions.

This study proposes a new structural design of the first-story isolation system in an RC frame building, where the RC columns connecting with isolation rubber bearings are inter-connected by beams. The coupled beam-column connections are potentially used to increase the seismic capacity of the whole RC building by increasing the integrated stiffness of the coupled columns and the stability of the isolation system. A 7-floor RC frame building with the coupled beam-column connections was designed according to a typical prototype structure and was then modeled by using the Finite Element Analysis (FEA). The inter-story displacement and the acceleration of each floor were analyzed. A scaled model test was then carried out in the lab to verify the simulated results. This study aims to establish a new structural design of the first-story isolation system with the coupled beam-column connections in an RC frame building, and to verify its good seismic performance.

2. Numerical Analysis

An FEA model was first established and analyzed regarding its seismic performance under four earthquake excitations. Section 2.1 describes the prototype structure that was referred to for designing the structural geometry with the first-story isolation layer or without the isolation system. The design of these two geometry models is detailed in Section 2.2, followed by the description of the structural model and boundary conditions for FEA in Section 2.3. The loading conditions are then shown in Section 2.4. The simulated results are then presented in Section 2.5.

2.1. Prototype Structure

The numerical model used in this study is established based on a prototype building to obtain a good representation of the practical RC frame building. A typical 7-story RC frame structure was selected as the prototype building (Figure 2). The first floor has a story height of 4.8 m, and the height of each story from the second floor to the seventh floor is 3.6 m. The infilled walls in the prototype structure are composed of hollow concrete block. The longitudinal length and crosswise direction of the building are 72.0 m and 13.4 m, respectively. There are ten spans (span length of 7.2 m) and two spans (span length: 5.7 m and 7.7 m) in the longitudinal direction and crosswise direction, respectively. As the crosswise direction of the building is the most disadvantageous direction of earthquake force, the longitudinal direction of the model (X direction) is set to the crosswise direction of the prototype, the crosswise direction of the model (Y direction) is set to the longitudinal direction of the prototype. Table 1 shows other main design parameters.

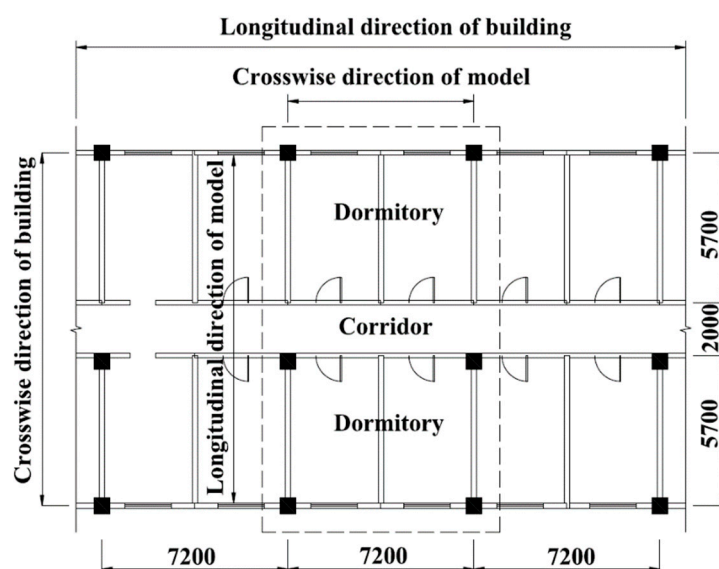


Figure 2. Top view of the middle part of the prototype building (unit: mm).

Table 1. Main design parameters for the prototype structure.

Design Parameters	Values	Design Parameters	Values
Fortification intensity	8 degrees	Peak acceleration (PGA)	0.20 g
Characteristic period	0.40 s		
Column section in the first story	750 mm × 750 mm ^{*1}	Column section of the 2nd to 7th story	550 mm × 550 mm ^{*3}
Section of the coupled beam and the isolation layer frame beam	400 mm × 700 mm ^{*2}	Section of beams in other layers	300 mm × 550 mm ^{*4}
Isolation layer thickness	170 mm	Other story thicknesses	120 mm
Steel type and its strength	HRB400 with yield strength = 400 MPa	Concrete strength grade	C30

Note: ^{*1} Section reinforcement ratio of 2%; ^{*2} the reinforcement ratios of upper and lower section are both 1.2%; ^{*3} (section reinforcement ratio of 1.5%); ^{*4} the reinforcement ratios of upper and lower section are both 1.5%.

The isolation layer is composed of isolation rubber bearings with a diameter of 600 mm, and it is located at the top of the columns that connect to beams. These rubber bearings have a maximum surface compressive pressure of 9.4–13.0 MPa in the vertical direction and shear modulus of 0.392 MPa. Lead rubber bearings (LRB) and linear natural rubber bearings (LNR) are used in the isolation layer on the top of the four columns in the corner, and on the top of the two middle columns, respectively.

2.2. Design of Structural Model for Simulation

This study simulates one of the horizontal frames in the middle position of the above prototype structure. The dynamic characteristics and response of the longitudinal direction (X-direction) of the model under earthquake excitations are investigated. The model has a geometric similarity ratio of 1:5. The detailed similarity ratios of model parameters are shown in Table 2. The model has two spans in the X direction and one span in the Y direction; its total height is 4.92 m. The first story and its above story height are 0.96 m and 0.66 m, respectively. The isolation system is inserted on the top of columns at the position of 0.23 m below the first floor. There are no infilled walls in the model frames. A non-isolated model without an isolation layer and the coupled beams was also simulated as a reference model for comparison. Figure 3 shows the top view and the front view of the isolation model and non-isolated model.

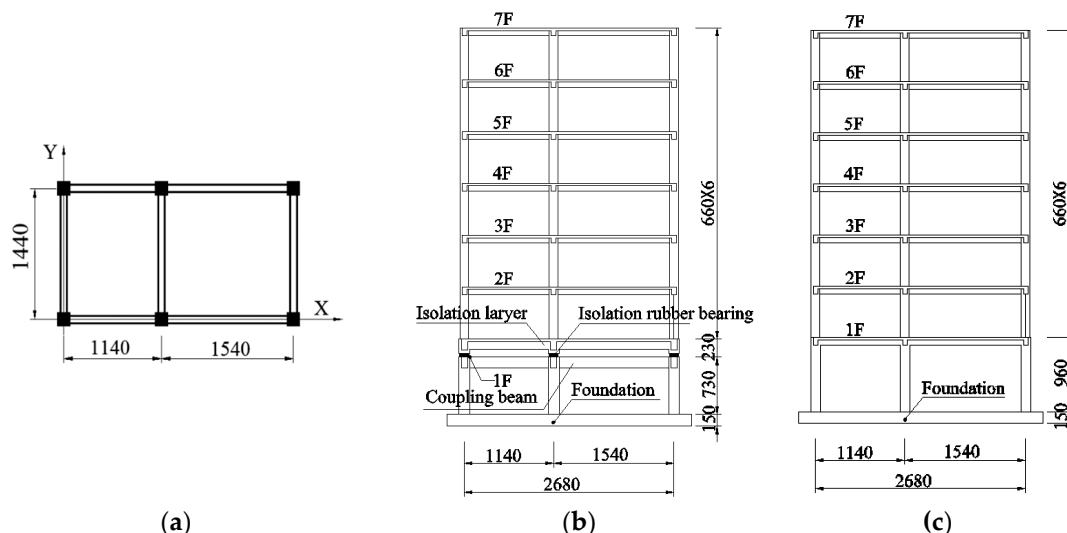


Figure 3. Top view and front view of the isolation model and non-isolated model (unit: mm): (a) Top view of the simulated model; (b) isolation model (front view); (c) non-isolated model (front view).

Table 2. Similarity ratios of the model structure.

Length	Elastic Modulus	Stiffness	Acceleration	Time	Displacement	Mass
(mm)	MPa	N/mm	gal	s	(mm)	Kg
1/5	1	1/5	2	1/3	1/5	1/98

The first story of the prototype is composed of six columns that connect to beams, the section of which was then scaled to be 150 mm × 150 mm (Figure 4a). The column height of the first-story isolated model reduces to 730 mm from 960 mm. The column slenderness ratio is calculated as 4.87 after the isolation layer with the slab thickness of 35 mm was inserted on the top of the column. While in the non-isolated model, the section of the column is 125 mm × 125 mm with a column height of 960 mm and slenderness ratio of 7.68; the slab thickness is 25 mm. Figure 4 presents the frame beam, the column section size, and the reinforcement distribution of the structural model.

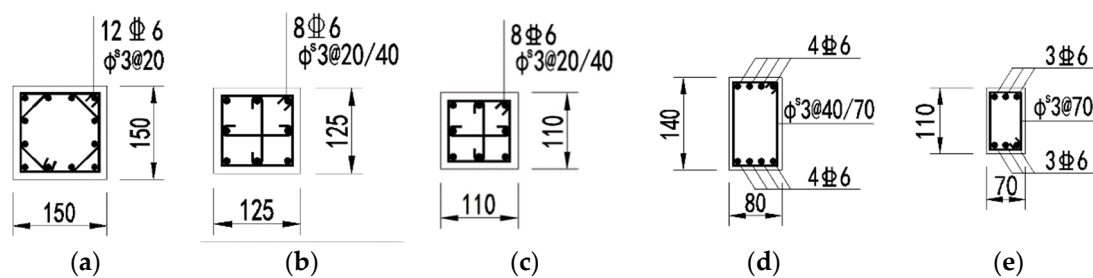


Figure 4. Cross-section and reinforcements of columns and beams of the isolation and non-isolated models (unit: mm): (a) Bottom column of isolation model; (b) bottom column of non-isolated model; (c) columns in the 2–7 layers; (d) The frame beam of the isolation layer and coupled beam; (e) beams in the 2–7 layers.

The concrete cubic (strength grade C30) used in the prototype structure has the compressive strength of 38.4 MPa and the elastic modulus of 3.81×10^4 MPa. Table 3 shows the coupon test data of the steel bar. The high-strength steel wires are used as steel slab bars and stirrup bars.

Table 3. Coupon test data of steel bar.

Diameter d (mm)	Grade of Steel bar	Yield Stress f_y (MPa)	Ultimate Strength f_u (MPa)	Elongation	Elastic Modulus E (MPa)
C6	HRB 400	501.9	619.3	19.1%	1.95×10^5
ØS3	High-strength steel wire	423.0	790.8	16.1%	2.09×10^5

2.3. Modelling of Structures

The FEA software PERFORM-3D (2018, ACE-Hellas, Athens, Greece) was used to simulate the testing models (the isolation one and the non-isolated one) considering the elasto-plastic mechanics [28]. The structural model was first established with boundary conditions, followed by the employment of elasto-plastic concrete and steel material model. Subsequently, the seismic waves were inputted into the model for analyzing the mechanical performance of both isolation structure and non-isolated one. The details of each part are described as follows.

Figure 5 shows the structural model of the simulated isolation structure and a non-isolated one. The dimensions of the buildings are the same to those shown in Figure 3. The bar elements are selected in the FEA model. The fiber model (column, inelastic fiber section) and plastic hinge model (moment hinge, curvature type) are respectively used as the material model for simulating the plastic area in the columns and beams. The plastic hinge adopts the moment-curvature model to calculate the component yield stress, moment and curvature when the component reaches its maximum bearing

capacity. The 3D fiber elements with eight steel fiber elements and 36 concrete fiber elements are used in the columns; this meshing is sufficient to cause the stable stress-strain analysis results.

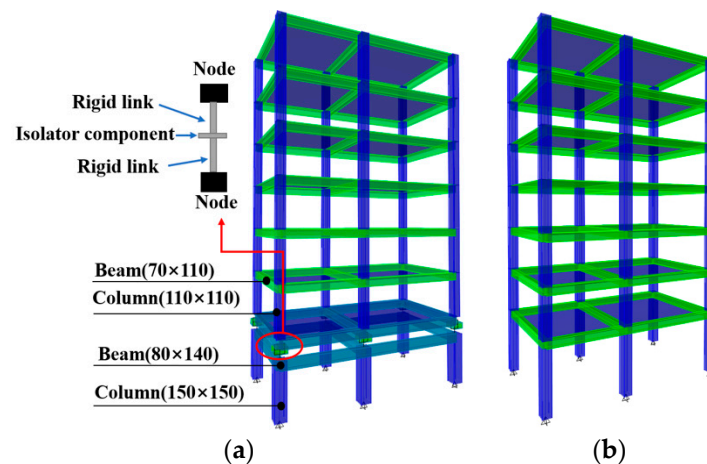


Figure 5. Numerical model of isolation model and non-isolated model: (a) Isolation model; (b) non-isolated model.

Only the elastic deformation of the isolation rubber bearing without considering its plastic deformation in the vertical direction is considered. This is reasonable as the earthquake excitations inputted to the model in this study only have the horizontal component. The strength loss and stiffness hardening caused by the shear deformation in the horizontal direction are ignored. The horizontal stress-strain property is not influenced by the stress and strain in the vertical direction. The LNR adopts a linear elastic model with an equivalent stiffness as the horizontal control parameter. This equivalent stiffness was calculated from Table 1. The LRB uses a bilinear restoring force model with stiffnesses before yielding and after yielding as well as yield force as the horizontal control parameters. Both LNR and LRB adopt the same tension and compression stiffness in the vertical direction. The above assumption is made to properly simulate the stress–strain relationship of the isolation rubber bearing.

The theoretical mechanical property values of LNR and LRB were obtained according to the specifications and model type, as shown in Table 4. These parameters were then calibrated by experimental work, and then the updated values were inputted into the numerical model for FEA. Note that the tested shearing force vs. horizontal deformation of LNR and LRB are shown in Figure 16. It should be mentioned that studies [29–31] show that the constitutive law of seismic devices is sensitive to static or dynamic loading conditions. Thus, we strictly follow the seismic standard [18] to ensure the constitutive law of our isolators is reasonable. Under the static condition (i.e., the shear strain of isolator equals 0), the stiffness before yielding is used; under the dynamic condition (i.e., the shear strain of isolator equals 100% and 250%, corresponding to design earthquake and rare earthquake), the horizontal equivalent stiffness, equivalent damping ratio, and post-yield stiffness were applied in the material model.

The trilinear kinematic model of steel material with the post-yield stiffness ratio of 0.01 and a yield strength of 400 MPa is used [28]. This model can well describe the stress–strain relationship to accurately simulate the mechanical properties of the steel. The Kent-Park model was adopted as the concrete constitutive model considering the degradation of the concrete hysteresis energy but not considering its tensile strength. The consideration of no tensile strength is because the floor slab is mainly simulated as the additional mass, causing no influence on the modelling results. The concrete strength grade is C30 with a compressive strength of 20.1 MPa. The material model parameters were calibrated then they were input into the FEA model for numerical analysis of the RC frame structure subjected to the earthquake excitations presented in Section 2.4.

The bottom of the models is fixed with the base to simulate that the substructure is rigidly connected with the base in the field. The inter-story stiffness of the simulated model is then calculated

before the model is subjected to the earthquake excitations in Section 2.4. The influence of mesh size on the structural and mechanical performance was carried out, and then sufficient mesh sizes at different locations, considering a low computational cost, were used in the structural model.

Table 4. Specification parameters and mechanical properties of the isolation rubber bearings.

Model	LNR120 *	LRB120 *
The shear modulus/MPa	0.392	0.392
The effective diameter/mm	120	120
Effective height/mm	60	60
rubber layer thickness	24	24
Initial shear stiffness(kN/mm)	/	2.21
Horizontal equivalent stiffness k_h ($\gamma = 100\%/250\%$)/(kN/mm)	0.198/0.134	0.276/0.188
Post-yield stiffness K_d /(kN/mm)	/	0.22
Yield force Q_d /kN	/	3.02
Equivalent damping ratio ($\gamma = 100\%/250\%$)	0.04/0.04	0.25/0.16
Horizontal displacement limits U_d (mm)	66	66

Note: * LNR120 or LRB120 means the diameter value of linear natural rubber bearings (LNR) or lead rubber bearings (LRB) is 120 mm.

2.4. Earthquake Excitations

Three earthquake waves were selected from the strong earthquake database of the Pacific Earthquake Engineering Research (PEER) Center; they are Taft, El Centro, and Northridge earthquake waves. In addition, another artificial earthquake wave (Rgbtongan) that is commonly used for testing buildings in China was selected. The peak accelerations (PGAs) of each seismic wave were adjusted to 0.30 g and 0.40 g to consider two different earthquake levels [32,33]. The maximum PGA was set as 0.40 g to be a relatively safe value, above which the structure could be damaged. It ensures that all the test scenarios can be carried out with the structures having good mechanical performance. The four earthquakes in the sequence of Taft, El Centro, Northridge, and Rgbtongan were continuously inputted in the X-direction from PGA level of 0.30 g to 0.40 g during the simulation. This arrangement is to minimize the damage accumulation of consecutive earthquakes. Note that the earthquake responses of structures subjected to the Taft, El Centro, and Northridge waves are similar but moderately less than that under the excitation of the Rgbtongan wave.

Both numerical analysis and the test (Section 3) were then performed using the above seismic excitation data. Figures 6 and 7, respectively, show the time history data and the acceleration response spectra for these four earthquake waves, and Table 5 presents the characteristics of the four seismic waves.

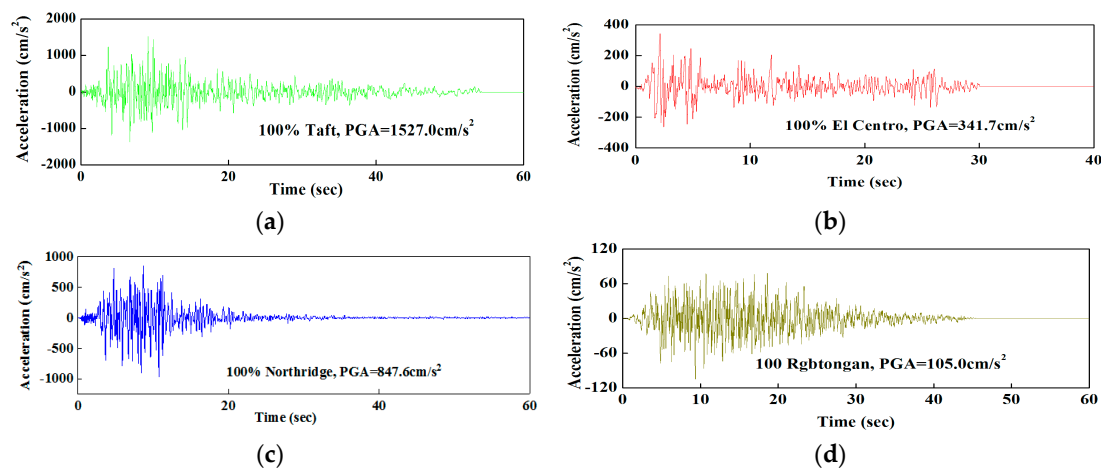


Figure 6. Time history data for the four earthquakes: (a) Taft earthquake; (b) El Centro earthquake; (c) Northridge earthquake; (d) Rgbtongan earthquake.

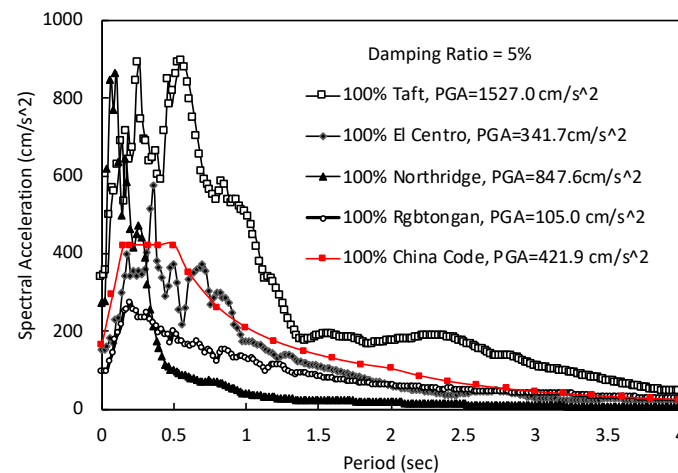


Figure 7. Acceleration response spectra.

Table 5. Details of the earthquakes.

Seismic Wave	PGA (gal)	Duration (s)	Frequency (Hz)	Direction
Taft	1527.0	54	50	North-south component
El Centro	341.7	30	50	North-south component
Northridge	847.6	60	50	West-east component
Rgbtongan	105.0	46	50	—

The model test was conducted first on the story isolation model and then on the non-isolated model. The acceleration response of the mass center of each story and the inter-story displacement between two stories were measured after all the earthquake excitations were input. Note that a total of 16 simulation conditions (four seismic waves $\times$ two PGAs $\times$ two models) were carried out in this simulation. Modal analysis was also performed to understand the first natural period of the simulated model. This is to compare the modal analysis result from the simulated model with that from the fabricated model. The comparison result can be found in Section 3.4.

2.5. Results of Numerical Analysis

2.5.1. Inter-Story Stiffness and First Natural Period

Table 6 shows the values of the inter-story stiffness in the simulation models. The results show that the first floor of the isolation structure has a rather large stiffness value of 157.61 kN/mm, which is 2.61 times the stiffness value of the first floor of the non-isolated model due to the shorter calculated column value after inserting the isolation layer in the first-story column (Figure 3b). With the addition of the isolation layer, the first floor of the isolation structure (157.61 kN/mm) is much larger than its above floors (81.62 kN/mm). While the first floor of the non-isolated structure (60.28 kN/mm) is smaller than its above floors (81.62 kN/mm) when the non-isolated structure has no isolation layer.

Table 6. Inter-story stiffness of the model structure.

Story	Inter-Story Stiffness (kN/mm)
2–7 story of the model	81.62
Isolation layer	1.53
First floor of the isolation structure	157.61
First floor of the non-isolated model	60.28

Through the modal analysis, the first natural periods of the whole non-isolated model and story isolation model were obtained as 0.61 s and 0.71 s, respectively. This result is expected because the stiffness of the story isolation model is smaller than that of the non-isolated model. They are then compared with the results obtained from the fabricated structure model in Section 3.4.

2.5.2. Acceleration and Inter-Story Displacement

Tables 7 and 8 respectively present the average absolute accelerations and inter-story displacements of both story isolation FEA model and non-isolated FEA model under earthquake excitations with different PGAs. The response's decreasing ratios of story isolation model compared to the non-isolated model are also presented. Note: The decreasing ratio = (response of the story isolation model – response of the non-isolated model)/response of non-isolated model. The response means acceleration or displacement in this study.

Table 7. Acceleration of different stories (numerical).

Story	0.30 g		0.40 g	
	Story Isolation Model (g)	Non-Isolated Model (g)	Story Isolation Model (g)	Non-Isolated Model (g)
7	0.170	0.702	0.291	1.088
6	0.165	0.597	0.255	1.011
5	0.166	0.531	0.229	0.885
4	0.167	0.512	0.220	0.794
3	0.164	0.461	0.209	0.712
2	0.158	0.412	0.179	0.562
Isolation layer	0.149	/	0.184	/
1	0.296	0.362	0.418	0.495

Table 8. Inter-story displacement values of both models (numerical).

Story	0.30 g				0.40 g			
	Isolation Model		Non-Isolated Model		Isolation Model		Non-Isolated Model	
	Displacement (mm)	Displacement Angle	Displacement (mm)	Displacement Angle	Displacement (mm)	Displacement Angle	Displacement (mm)	Displacement Angle
6–7	0.69	1/956	3.20	1/206	1.44	1/458	5.57	1/118
4–5	1.46	1/452	5.00	1/132	2.46	1/268	8.36	1/79
2–3	2.02	1/327	6.80	1/97	3.43	1/192	9.82	1/67
Isolation layer	22.21	/	/	/	34.51	/	/	/
Base-1	0.72	1/1014	8.50	1/113	1.01	1/723	13.51	1/71

The results show that the insert of the isolation layer significantly decreases the acceleration values of different stories above the first story. The acceleration values of the story isolation model range from 0.149 to 0.418 g, while those of the non-isolated model increase from 0.362 to 1.088 g. The inserting of the isolation layer produces a significant seismic damping effect on the acceleration response of stories above the isolation layer as the decreasing ratios of acceleration values are generally more than 61% regardless of PGAs (Figure 8). The above results also can be illustrated in Figure 9 where the variations of story acceleration from base to the top floor are shown when the PGAs are 0.30 g and 0.40 g.

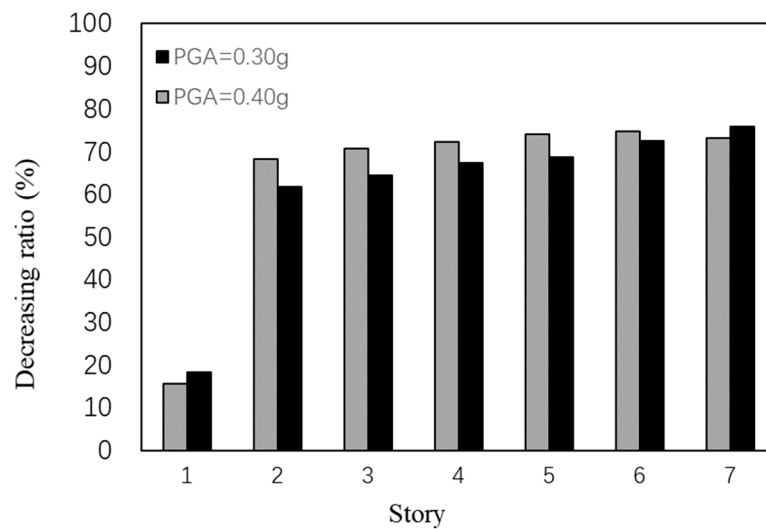


Figure 8. The decreasing ratio of story acceleration values in the story-isolated structure after being subjected to earthquakes at peak acceleration (PGA) of 0.30 g or 0.40 g compared to the non-isolated structure (numerical).

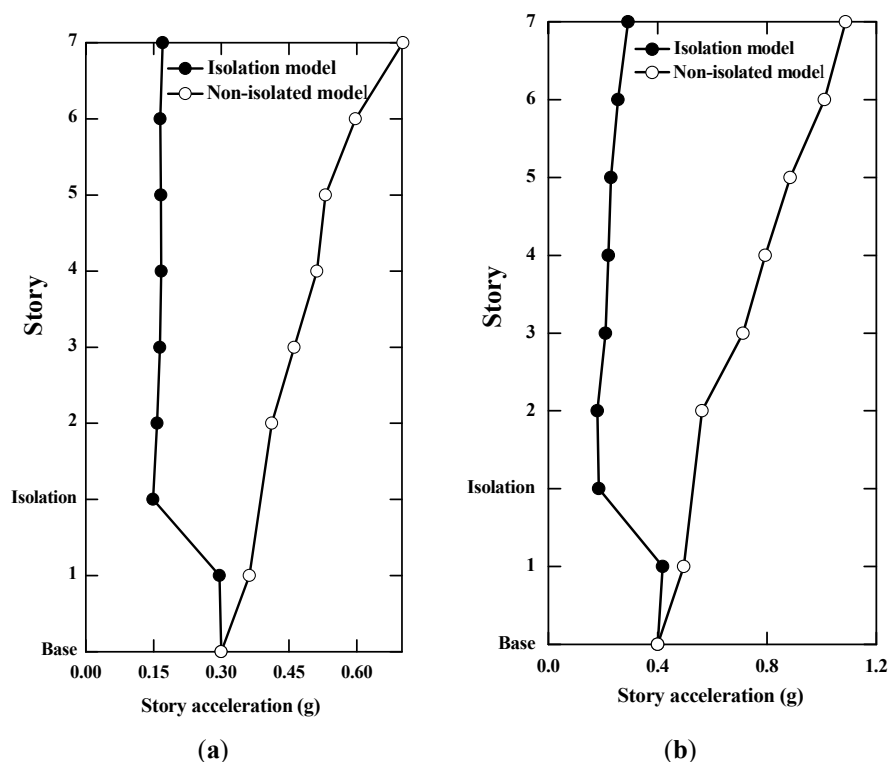


Figure 9. Comparison of story acceleration between isolation model and non-isolated model at PGAs of (a) 0.30 g and (b) 0.40 g.

To demonstrate the story displacement response, Table 8 shows the inter-story displacements of the non-isolated model and the story isolation model, and Figure 10 presents the decreasing ratios of inter-story displacement values when the isolation layer was applied. The results show that both the displacement and its angle in the story isolation model (either above or below the isolation layer) were significantly less than those in the non-isolated model regardless of PGA. The decreasing ratio ranges from 70.3% up to 91.5% (PGA = 0.3 g) and from 65.1% up to 92.5% (PGA = 0.40 g). In addition, the inter-story displacement value decreases with increasing story height, regardless of the structure model type or PGA value. For the isolation model, the increase of the story displacement is less

compared to the non-isolated model. The stories above the isolation layer almost horizontally move, while the stories in the non-isolated model increase significantly in their horizontal displacements, as shown in Figure 11.

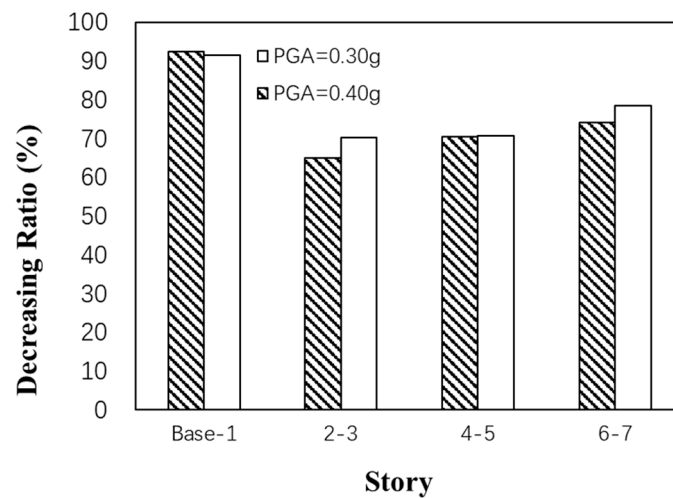


Figure 10. The decreasing ratio of inter-story displacement values in the story-isolated structure after being subjected to earthquakes at PGA of 0.30 g and 0.40 g compared to the non-isolated structure (numerical).

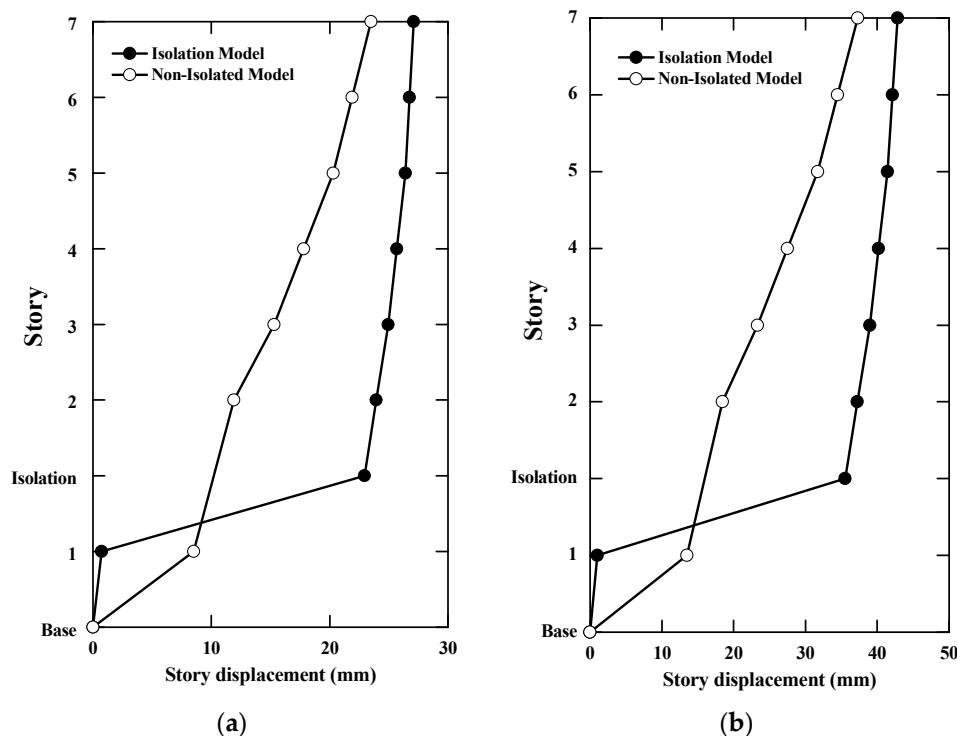


Figure 11. Comparison of story displacement between isolation model and non-isolated model at PGA of (a) 0.30 g and (b) 0.40 g.

3. Shaking Table Tests

A series of shaking table tests were conducted to verify the above simulation results, and the results were analyzed for comparison. The physical structural model was designed and fabricated referring to the prototype structure (Section 2.1), and it is presented in Section 3.1. The key components are the rubber bearings in the isolation system; these rubber bearings and their properties are

then described in Section 3.2. The testing system with the data acquisition was then shown in Section 3.3, and the testing results and their comparison with the numerical results are then presented in Sections 3.4 and 3.5, respectively.

3.1. Fabrication of Model Structure

Figure 12 shows the six fabricated columns, on the top of which the isolation layer was inserted. The top of the isolation layer was then connected with the above six stories by the pre-embedded steel plates and bolts. The isolation layer is composed of rubber bearings that are described in Section 3.2. The whole story isolation model was then fabricated as shown in Figure 13a. The non-isolated model (Figure 13b) was completed by fabricating the first story and then directly welding it with the above six stories without inserting an isolation layer.

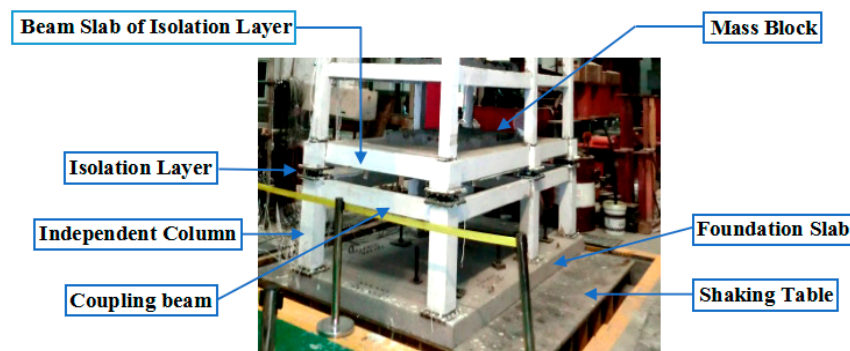


Figure 12. Configuration of the substructure of the story isolation model.

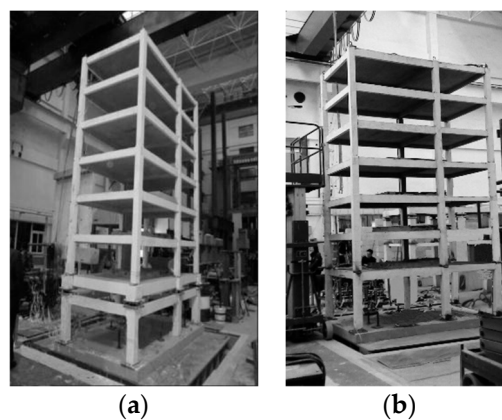


Figure 13. The fabricated story isolated model and non-isolated model: (a) Story isolation model; (b) non-isolated model.

The total mass of the story isolation model and the non-isolated model is 10,319 kg and 10,223 kg including the additional counterweight of 5215 kg in each model. This additional mass was added by placing a steel block in the middle of the second story top surface. The foundation slab was connected with the shaking table by the reserved bolts. The columns were connected with the foundation slab by the pre-embedded steel plates.

3.2. Properties of the RUBBER Bearings

To ensure the selected proper rubber bearings with good working conditions, horizontal performance tests were performed on a series of rubber bearings using a computer-controlled electro-hydraulic servo-pressure shear testing machine before the shaking table tests. Two rubber bearings, considered as one group, were placed above and below the shear plate, as shown in Figure 14. The shear deformation was caused by the horizontal movement of the shear plate. The shear plate tension was collected by the acquisition system of the machine. The displacement of the rubber bearing was measured using the GWC150 displacement device on the top of the shear plate. The vertical pressure during the test was set to the surface pressure of the rubber bearing as 6 MPa, and the axial compressive stress was maintained constant during the test. A total of 10 LNRs and 5 LNRs were tested.

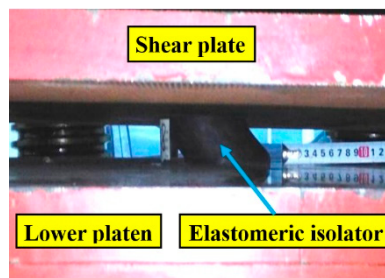


Figure 14. Elastomeric isolator.

After analyzing the testing results, four LRBs and two LNRs with good working conditions were selected for a series of shaking table tests. The cross-section diagram and a photograph of an isolation rubber bearing used in our experiment are shown in Figure 15. The average horizontal equivalent stiffness (rubber shear strain $\gamma = 100\%$) of the LRBs and LNRs was 0.296 kN/mm and 0.173 kN/mm, respectively. Typical hysteresis curves of the rubber bearings (LRB and LNR) are shown in Figure 16. The lateral stiffness of the seismic isolation was then obtained as the combined stiffness of the four LRBs and two LNRs, equal to 1.53 kN/mm (i.e., $0.296 \times 4 + 0.173 \times 2$) which is 13.07% lower than the theoretical value (1.73 kN/mm) obtained through similarity theory by multiplying the value of 8.64 kN/mm in the prototype structure with the geometric similarity ratio of 1:5. The detailed specification parameters and mechanical properties of the isolation rubber bearings can be found in Table 4, and these parameters are in line with the requirements of the GB 50011-2010 Code for seismic design of buildings [18]. Considering this minor difference between the practical and theoretical value, these rubber bearings could be used in the shaking table tests.

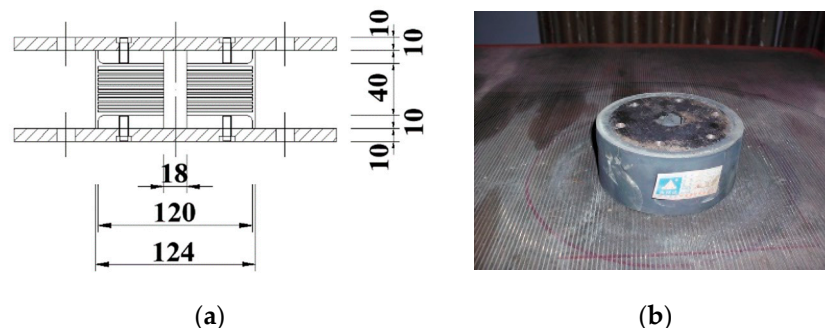


Figure 15. Isolation rubber bearing: (a) Sectional view of the LRB (unit: mm); (b) isolation bearings.

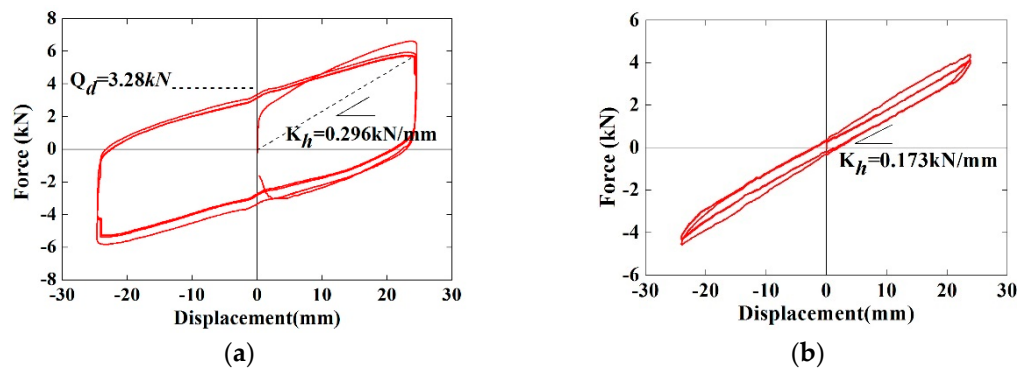


Figure 16. Hysteresis curve of rubber bearings: (a) LRB; (b) LNR.

3.3. Shaking Table System and Data Acquisition

Following the property test of the rubber bearings and the fabrication of structural models, a series of shaking table tests were then carried out. The shaking table systems have a table size of $3\text{ m} \times 3\text{ m}$, the load capacity of 10 t, the maximum acceleration of 1.0 g, the maximum displacement $\pm 127\text{ mm}$, and the maximum velocity of 600 mm/s. With these parameters and the corresponding capacity, the shaking table system meets the requirements of a typical shaking table test.

The measurement equipment for the dynamic test in the data acquisition system mainly includes the IMC (Instrumentation and controls) dynamic data acquisition system and linear displacement sensors (941B sensor) that are placed on every two stories of the substructure below the isolation layer, the isolation layer and the superstructure above the isolation system to measure the story displacement. The accelerometers (horizontal IMC vibration pickup) in the X direction are placed on each story. For the substructure, the accelerometer is placed on the steel plate that connects the middle column and the isolation rubber bearings. For the stories from the isolation layer to the seventh layer, the accelerometers are placed in the middle of the steel plate surface.

The structural model was first subjected to a white noise frequency sweep to test if the model fabrication work causes a negligible error. Note that this white noise with such a small PGA (0.10 g) would produce no damage to the structural model. The natural vibration characteristics of the isolation model and non-isolated model were then measured by modal analysis. The results show that the isolation model and non-isolated model have the first natural periods of 0.77 s and 0.64 s, which are respectively larger only by 8% and 5% than those of the simulated models, indicating that the test model has a negligible fabrication error and thus can be used for the subsequent shaking table test. The first natural period of the non-isolated model is smaller than that of the isolation model. This is expected as the mass of the non-isolated model is 10,223 kg that is less than that of the isolation model.

3.4. Test Results and Analysis

The damage of RC models was first assessed by their natural vibration characteristics after the structural models were subjected to all the earthquakes. Table 9 shows the first natural period values before and after the earthquakes for both models. For the story isolation model and non-isolated model, the first natural period respectively increased by 10.4% and 15.6% after the earthquakes.

Table 9. Analysis results of first natural periods of the models.

Model	The First Natural Period (s)	
	Before Earthquake	After Earthquake
Story isolation Model	0.77	0.85
Non-isolated Model	0.64	0.74

Table 10 shows the average peak acceleration values of each story at its mass center measured after both models were subjected to four consecutive earthquakes with PGAs of 0.30 g and 0.40 g. The two models present different acceleration values due to the influence of the isolation layer. Compared to the non-isolated model, the isolation model has much lower acceleration values regardless of PGA level (Figure 17). As expected, the first floor below the isolation layer has a negligible variation of acceleration, while the floors above the isolation layer have significant decreases in accelerations by more than 68%. It was also found that the PGA had a minor influence on the decreasing ratio of accelerations.

Table 10. Acceleration values of both models.

Story	0.30 g		0.40 g	
	Isolation Model (g)	Non-Isolated Model (g)	Isolation Model (g)	Non-Isolated Model (g)
7	0.176	0.764	0.302	1.160
6	0.159	0.648	0.246	1.071
5	0.169	0.573	0.233	0.934
4	0.145	0.532	0.191	0.813
3	0.144	0.471	0.173	0.732
2	0.133	0.422	0.151	0.588
Isolation layer	0.144	/	0.186	/
1	0.314	0.364	0.443	0.499

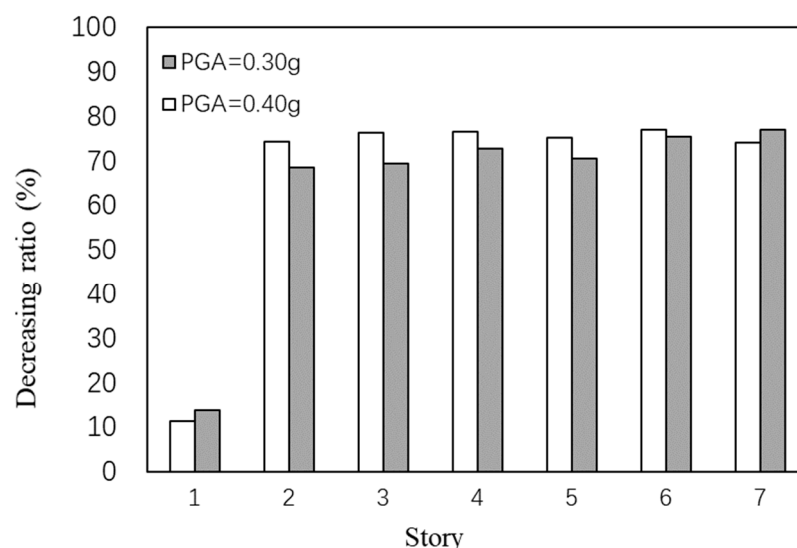


Figure 17. The decreasing ratio of peak acceleration values of each story in the story-isolated structure after being subjected to earthquakes at PGA of 0.30 g and 0.40 g compared to the non-isolated structure (testing).

Interestingly, it was found that, with the increase of PGA from 0.30 g to 0.40 g, the acceleration value of the story-isolated structure at each story generally increased much less than that of the non-isolated structure, as shown in Figure 18.

Table 11 shows the average inter-story displacement values of each two stories at its mass center measured after both models were subjected to four consecutive earthquakes with PGAs of 0.30 g and 0.40 g. The two models present different inter-story displacement values due to the influence of the isolation layer, as shown in Figure 19 where the decreasing ratios of displacement values of the story isolation model when compared to the non-isolated model are presented. The results show that the displacement values of the story-isolation structure are significantly decreased from 74.2% to 91.2% (0.30 g) or from 69.6% to 92.3% (0.40 g), indicating a good damping effect due to the isolation layer. The maximum decrease of displacement was found between the base ground and the first

floor. Similar to the case of acceleration, the PGA also has a negligible effect on the decreasing of inter-story displacement.

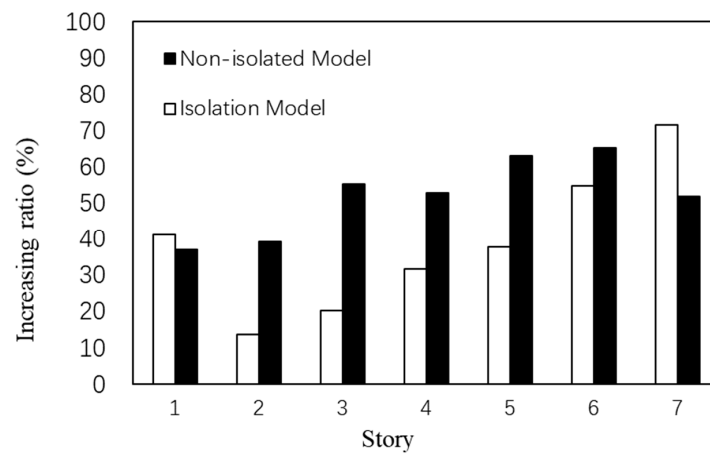


Figure 18. The increasing ratio of peak acceleration values of each story in story-isolated structure and non-isolated structure at PGA of 0.40 g compared to that at PGA of 0.30 g (testing).

Table 11. Inter-story displacement values of both models (testing).

Story	0.30 g				0.40 g			
	Isolation Model		Non-Isolated Model		Isolation Model		Non-Isolated Model	
	Displacement	Displacement Angle *	Displacement	Displacement Angle *	Displacement	Displacement Angle *	Displacement	Displacement Angle *
6–7	0.71	1/930	3.41	1/193	1.48	1/446	5.33	1/124
4–5	1.33	1/496	5.29	1/124	2.24	1/295	7.62	1/87
2–3	1.84	1/359	7.14	1/92	3.13	1/211	10.31	1/64
Isolation layer	21.30	/	/	/	31.54	/	/	/
Base-1	0.78	1/936	8.94	1/107	1.09	1/670	14.19	1/68

Note: * The small displacement angle is approximately equal to its tangent function value and thus it only presents its tangent function value. This applies to Table 11.

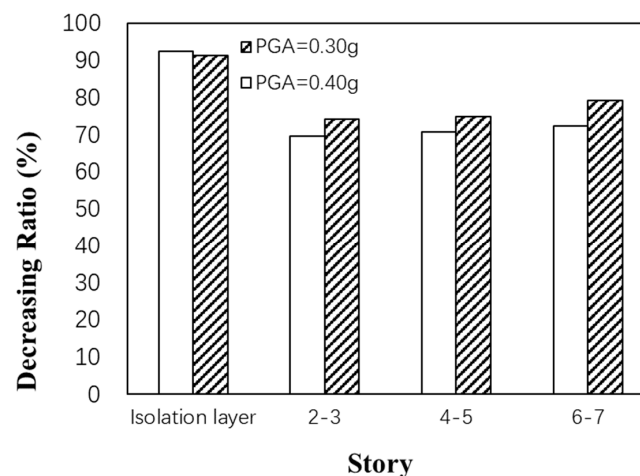


Figure 19. The decreasing ratio of inter-story displacement values in the story-isolated structure after being subjected to earthquakes at PGA of 0.30 g and 0.40 g compared to the non-isolated structure (testing).

Further analysis shows that, under the PGA of 0.30 g and 0.40 g, the displacements of the substructure in the non-isolated model are both large. The corresponding displacement angles are 1/107 (i.e., 8.94/960) and 1/68 (i.e., 14.19/960), indicating a large horizontal displacement of the first story subjected to the earthquakes. The displacement angle value of 1/68 in the first floor when the

PGA was 0.40 g is reaching the corresponding collapsing limit (1/50) of GB 50011-2010 Code for seismic design of buildings [18]. While, for the story isolation model, the displacement angle was significantly decreased, with the maximum value of 1/211 between the second story and the third story above the isolation layer when the PGA was 0.40 g. Overall, the displacement angles in the isolation model were significantly less than those in the non-isolated model.

3.5. Comparison of the Test and Numerical Analysis Results

The test results of acceleration and inter-story displacement of the isolation model were compared with the numerical results. The comparison shows a good match between the testing data and the simulation data for both acceleration values and story displacement values. For example, Figures 20 and 21 present typically good matching results of the time history curve of the absolute acceleration (displacement) in the 7th floor and of the peak story acceleration (displacement) along different stories when they are subjected to El Centro earthquake with PGA of 0.30 g, though the numerical results are slightly less than the test values at superstructures above the isolation layer.

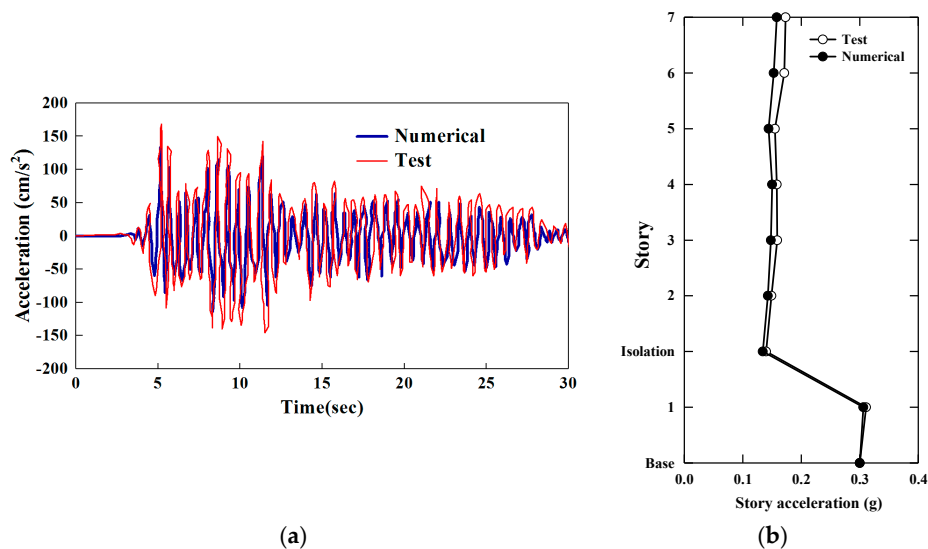


Figure 20. Comparison of (a) acceleration history at the seventh floor and (b) acceleration along the story height at the story isolation structure.

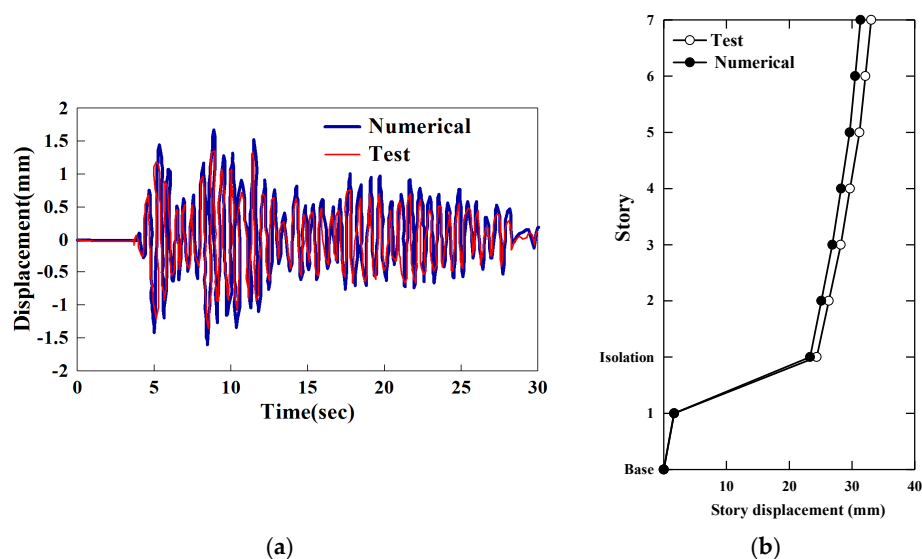


Figure 21. Comparison of (a) displacement history at the seventh floor and (b) displacement along the story height at the story isolation structure.

4. Discussion

Our study shows the new isolation system plays an important role in decreasing the peak acceleration values of each story or the inter-story displacement in the story-isolated RC structure when the structure is subjected to seismic waves at PGA of either 0.30 g or 0.40 g (Figures 8, 10, 17 and 19), and thus reducing the seismic energy that is directly applied to the building. The displacement of the isolation layer is larger by one order of magnitude than other parts of the building, indicating the majority of seismic energy was absorbed by the isolation system. This high energy dissipation capacity of the new isolation system achieves the aim of seismic design of RC frame buildings. When the earthquake wave has a higher PGA, the peak acceleration values of superstructures in the story-isolated model increases much less than those of superstructures in the non-isolated model. This seismic damping effect is more significant when the story is above the isolation layer. For the substructure, it is rigidly connected with the base, thus its acceleration response is close to the acceleration response of the shaking table, and thus a negligible damping effect was present in the story-isolated model. This was further verified by our testing results shown in Table 10 and was also consistent with results from studies such as [34,35].

The above results also suggest that the isolation performance of the superstructure could depend on the position of the isolation layer in an RC frame building, consistent with a comparison between the base isolation building and middle-story isolation building [13]. The detailed design and analysis of the inter-story isolation system can be found in references [36,37]. Our study focuses on the first-story isolated RC frame building, where the isolation-protected superstructure has a relatively large story range (e.g., the second floor to the top floor in this study). In addition, compared to the base-story isolation [38,39], the first-story isolation provides a more continuous connection between the ground and the above frame building for easier retrofitting, though the response mechanism is similar to the base-isolated structure.

The horizontal displacement angles of the base-1 floor in the first-story isolation model are much less than those in the non-isolated model, as shown in Tables 8 and 11. In addition, the displacement angles of the base-1 floor in the first-story isolation model are also less than other inter-floors. According to the seismic design code of building [18], the safe capacity of the substructure is higher than that of the superstructure. In addition, the small displacement angles indicate the elastic deformation of substructure in the first-story isolation RC building with the coupled beam-column connections.

Consistent with the acceleration results, the inter-story displacements in the isolation model significantly decreased by up to 92% compared to those in the non-isolated model, presenting good stability of structures. For inter-story between the ground and the first floor, the damping effect is the most significant as the isolation system starts to realize the damping effect on the first floor just above the position of the isolation layer. Once the isolation layer is starting to exert its effect, both the acceleration and displacement decreased sharply. Without the damping contribution to the base-first floor in the non-isolated model, the large inter-story displacement that was close to the structure collapsing limit was thus found when the earthquake level was high at a PGA of 0.40 g.

For the non-isolated model, two different earthquake intensity levels at PGAs of 0.30 g and 0.40 g cause various high inter-story displacements and displacement angles, which are approaching to the code limit of 1/50 for the first story [18]. While, for the story-isolated model, the inter-story displacements and corresponding angles are hugely decreased, indicating that the isolation system could significantly improve the seismic safety of the RC frame building. Due to the connection between columns by beams, our new design of the first-story isolation system in an RC frame building produces a good seismic safety of buildings by the integrated stiffness of the coupled columns and the stability of the isolation system. For the current first-story isolation structure with independent columns, the stiffness of column in the substructure is normally strengthened by increasing the column section size, resulting in a decrease of space size on the first floor and an increase in engineering cost. The coupled beam-column connections on the first floor would also potentially increase the stability of the isolation system when the building is subjected to strong earthquakes in multiple directions. Thus,

our design provides an alternative seismic design of RC frame buildings, and it can be potentially be considered in practice and compared with other alternative seismic design of RC frame structures such as the including of infill panels, metal shear panels, and curtain walls as well as the design based on the kinematic theorem of plastic collapse [4–7,20,21].

According to the testing verification study (Figures 20 and 21), the developed numerical model could be potentially used for a parameter analysis to optimize the seismic design regarding column section geometries and material properties in the future; this could make the seismic designing easier. For a simplified research purpose, the infilled wall is not used in this study. This may induce difference of our experimental results with the practical RC frame building [4]. As this study focuses on the isolation effect on the overall RC frame structure, we believe this simplification is reasonable within the scope of this study. Future research is being performed to examine the influence of infilled wall on the seismic performance of lab-constructed RC frame model. As the vertical components of earthquakes may cause influences on the response of structures such as axial forces and local uplift of the columns [40], and the displacement-caused P-Delta effect [27] may cause extra damage to the building, future studies could also be carried out to consider the effects of the vertical component of earthquakes and the P-Delta effects of large horizontal displacements on the seismic behavior of the first story isolated RC structure with the coupled beam-column connections.

5. Conclusions

A new structural design of the first-story isolation system in RC frame buildings was proposed and developed in this study. The numerical results were verified by the testing results regarding the seismic responses. The following conclusions were achieved:

- The first-story isolation system with the coupled beam-column connections increases the seismic capacity of the first-story isolated RC frame building under strong earthquake excitations compared to a non-isolated frame building.
- The isolation system significantly decreases the story accelerations and inter-story displacements of the superstructure in the first-story isolation model, indicating a good damping effect of the isolation system that can intensively absorb the seismic energy. For the substructure, its story acceleration increases slightly, and there is almost no damping effect as it is rigidly connected with the base.
- The displacement angels of the substructure in the first-story isolation RC building are small and corresponding column horizontal deformation is in the elastic stage. The safety capacity of the substructure is higher than that of the superstructure.
- The coupled beam-column connections could be potentially used in the practical first-story isolation system to avoid the requirements of large column stiffness and large column size on the first floor.

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