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Shake-table testing of 2-story steel framed building with self-centering modular panels and slit steel plate walls

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A R T I C L E I N F O

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A B S T R A C T

Recent numerical study and cyclic testing results have suggested that the self-centering modular panels (SCMP) offer a promising prefabricated structural panel technology for enhanced seismic performance of beam-through steel frames (BTSFs). Several types of seismic fuse devices, including tension-only braces and slit steel plate shear walls (SWs) for energy dissipation, have been studied for their viability in adoption by SCMP. To further investigate the dynamic response and resilience behavior of this seismic-force-resisting system, shaking table tests have been conducted on a 2-story SCMP-BTSF building model with SWs as seismic fuse device. Two far-field ground motion records with various intensity levels were used in the shake table tests. The presented experimental testing study is the first shaking table test of SCMP-BTSF, which is a newly proposed pre-fabricated structure system with self-centering features. Nonlinear finite element (FE) model was also established to perform time history analysis of the test structure and assist with specimen design. The shake test results show that the structure response under frequent ground motions meets the inter-story drift limit given in Chinese code - GB 50011–2010 (Code for Seismic Design of Buildings, abbreviated as CSDB hereafter). When the hazard level was greater than that of frequent ground motions, the system exhibited flag-shaped hysteresis curves of the story shear as expected. It is observed that the SWs dissipated seismic energy through yielding and buckling of steel slats between slits, while other structural members and PTFs remained elastic. No cracks were seen in the concrete floor slabs after all shake table tests. After replacing SWs, the test structure showed nearly identical dynamic properties and seismic responses to the original test structure.

1. Introduction

While structures designed to modern seismic codes are anticipated to survive severe earthquakes without collapse, they intend to experience extensive structural and non-structural damage under design level earthquakes [1]. For conventional seismic force resisting systems, ductile damage is expected to be the source of seismic energy dissipation during strong earthquakes. For example, steel moment resisting frames (MRFs) and steel concentrically braced frames (CBFs) are designed to dissipate energy through yielding near the beam ends and yielding of braces, respectively [2–4]. In general, these energy dissipation mechanisms are related to damage mechanisms of main structural members, which may cause large residual drifts of buildings. In damage-controlled structures, the damage would be concentrated in some specified replaceable elements which yield prior to the other structural members [5]. After strong earthquakes, only damaged structural fuse members need to be repaired, which eases the repair process and thus cuts the repair cost if replaceable fuse members are used. Recently, replaceable seismic fuse members have drawn interests and metal hysteretic fuse dampers have been investigated, such as steel angles [6,7], U-shaped steel plates [8], shear links [9] and others.

Another key measure in lowering the seismic loss is to minimize the residual inter-story drifts of the structure, since the residual inter-story drift is regarded as a critical factor in economic feasibility of repairing a damaged structure after earthquakes [10]. Considering the indirect loss-of-function cost due to building closure, repairing a building with large residual inter-story drifts may be much more expensive than reconstruction. In recent years, several high-performance seismic lateral-force-resisting systems, which can return to the pre-earthquake position and confine damage to replaceable seismic fuse members,

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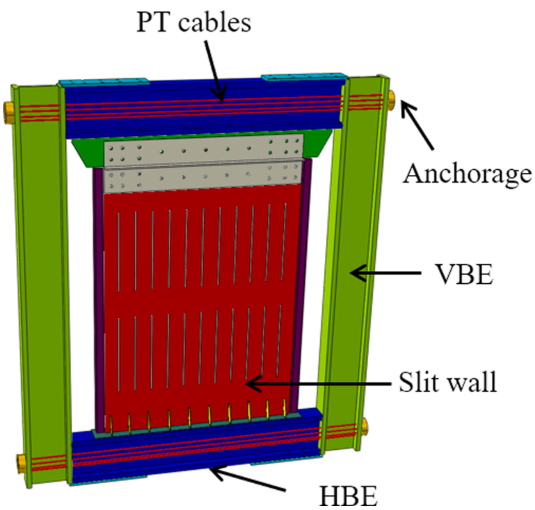


Fig. 1. Schematic diagram of SCMP (PTF-SW system).

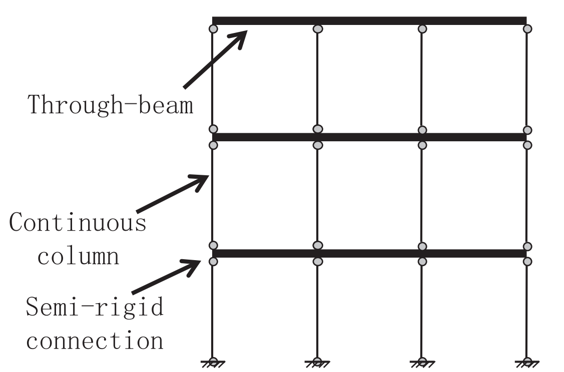


Fig. 2. Schematic diagram of a beam-through steel frame.

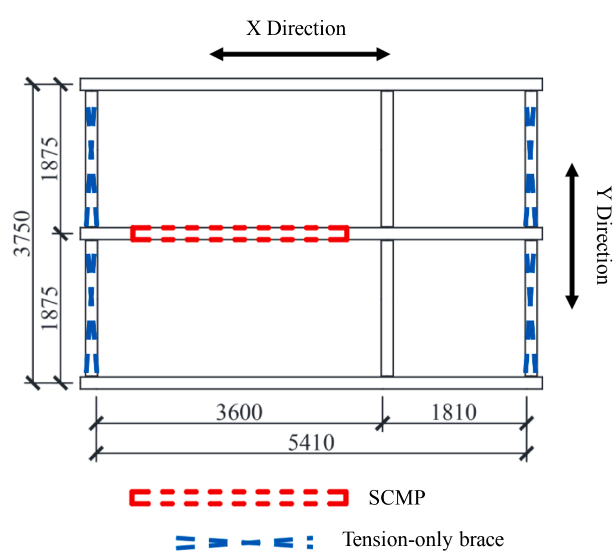


Fig. 3. Floor plan of the test structure.

have been investigated [6,11–18]. One of such self-centering seismic systems is the self-centering modular panel (SCMP) [19,20], which is comprised of two primary components - a post-tensioned frame (PTF) and energy dissipation devices. The PTF is composed of horizontal boundary elements (HBEs), vertical boundary elements (VBEs) and post-tension (PT) cables. Fig. 1 illustrates a PTF-SW system. The SCMP can be shop fabricated and then installed into a frame through bolted connections at the construction site. The PTF-SW exhibits a typical flag-shaped hysteresis loop [21], since the bilinear elastic behavior of PTF [22] and the pinching hysteretic behavior of slit walls [23,24] are combined. In this study, the SCMP is installed in a beam-through steel frame (BTSF) system (as shown in Fig. 2), which is a rapid-construction frame mainly used for low-rise fabricated building [25–27]. In the system, the BTSF functions as gravity load carrying frame composed of strong through-beams and small-section columns, while the PTF-SWs act as seismic lateral-force-resisting system. While several self-centering structure systems have been investigated through experimental shake-table testing [28–35], the current experimental investigation reported here is the first shaking table tests conducted on SCMP-BTSF.

To investigate the nonlinear seismic response and its damage pattern of the concerned seismic-force-resisting system with SCMPs subjected to real ground motion records, shaking table test of two-story BTSF buildings with SCMPs (PTF-SWs) was conducted and specific test results are discussed here. The test specimen was conducted to subject to two earthquake ground motion records selected from FEMA P695 far-field ground motion records [36] with various intensity levels. Details of the specimen and test setup are first presented. To probably predict the structural response in testing, a FE model is developed in OpenSees [37], and numerical simulation results of modal properties and nonlinear responses are presented. The experimental test results presented here include fundamental modal properties, typical acceleration and inter-story drift response time histories, internal force response and description of observed behavior and damage patterns. The experimental results show that the structure can re-center after strong earthquakes and damage is only concentrated to the fuse member (i.e., steel plate wall - SWs in this study) while other structural members remained elastic. Furthermore, minimal PT stress loss measured using a PT force sensor was observed over multiple tests up to very strong intensity ground motions. The shaking table test results further confirm the promising seismic performance of the proposed self-centering system.

2. Test specimen, setup and instrumentation

2.1. Test structure

The test structure is designed referencing to the current Chinese seismic code - Code for Seismic Design of Buildings (CSDB, 2010) [38]. The soil condition is assumed to be Group 2 and Type II with characteristic period of 0.4 s. The peak ground acceleration (PGA) for frequent ground motion (63% in 50 years probability of exceedance), which is used in elastic design in CSDB [38], is 0.2 g. The corresponding maximum value of seismic influence coefficient is 0.448 (equivalent to the design basis earthquake's response spectral value at short periods, S_{DS}, of 0.448 g). The PTFs of the SCMP specimens in this shake-table testing were reused from previous quasi-static cyclic tests and their hysteretic behavior can be found in reference [21]. To work within the operation limit of the shaking table and existing SCMPs' lateral force resistance, the mass of test structure is set as 33.0 tons (7.5 tons for the 1st-floor and 25.5 tons for the 2nd-floor). This floor mass distribution intends to match the experimental measured lateral force capacity of two SCMPs installed in the 1st-floor and 2nd-floor to avoid potential soft story issue that might arise if uniform floor mass distribution was adopted.

The main frame (gravity load carrying frame) of the test structure is a two-story two-bay BTSF in the test load direction. Fig. 3 shows the test structure's horizontal layout and has been designed to fit the steel mounting setup on the shaking table available in the lab. The total height of the test structure is 6,880 mm and its elevation views in the X and Y direction are shown in Fig. 4 and Fig. 5 respectively. The steel through-beams and columns in the BTSF are I-sections: H300 × 150 × 10 × 12 and H125 × 125 × 7 × 9, respectively. Both beams and columns

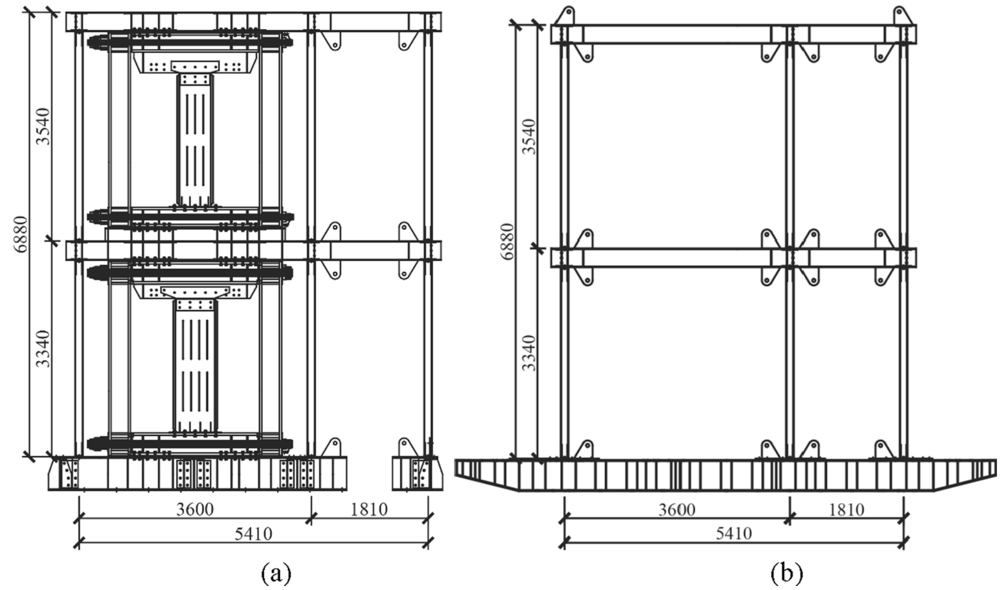


Fig. 4. Elevation view of the test structure in the X direction: (a) interior frame equipped with SCMPs; (b) bare exterior frames.

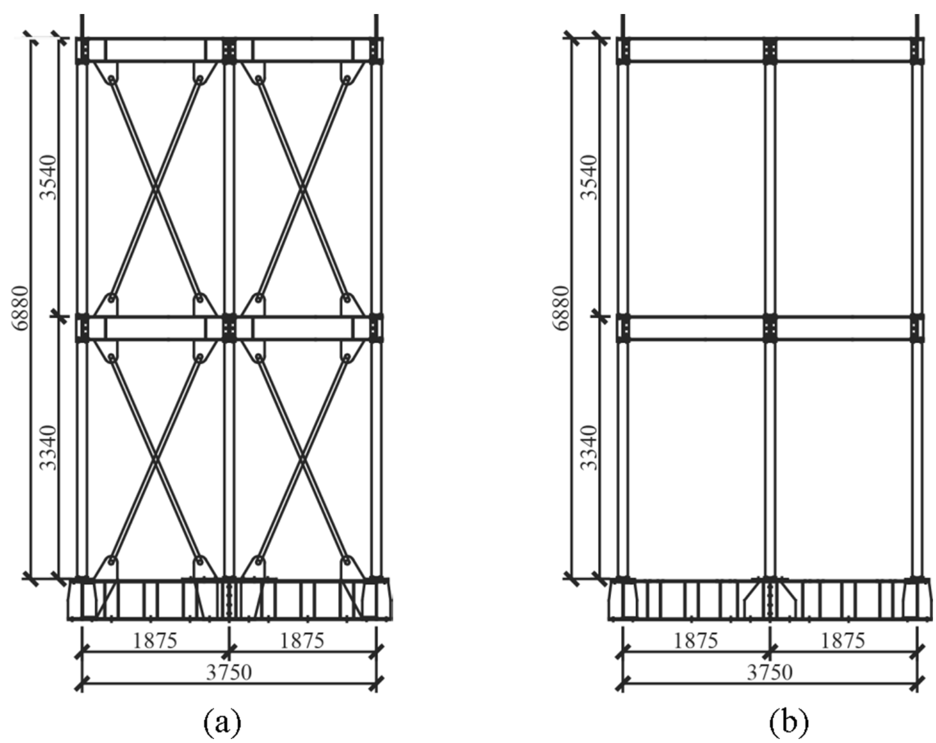


Fig. 5. Elevation view of the test structure in the Y direction: (a) braced perimeter bay; (b) unbraced interior bay.

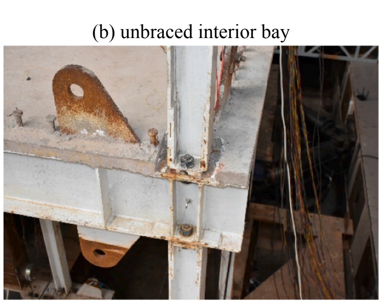


Fig. 6. Close-up view of PTF's beam-to-column connection.

are made of Grade Q345 steel (nominal yield strength is 345 MPa in China code). High-strength bolt slip-critical connection are used in the beam-to-column joints. To eliminate slab composite effect, the reinforced concrete floor slab does not enclose the beam-to-column connections through pre-arranged gap space around the connections. A typical beam-column connection is illustrated in Fig. 6. Because the structure is only loaded in the X direction (i.e., seismic excitation direction), tension-only concentrically braces are installed in the perimeter bays in the Y direction to control Y-direction drift and accidental torsional response of the test structure. All tension-only braces are made of Grade Q235 steel strap with a rectangular cross section of 80 × 7.

The SCMPs are installed in the 3600-mm-wide bay of the interior

Table 1
Dimensions and section sizes of post-tensioned steel frames.

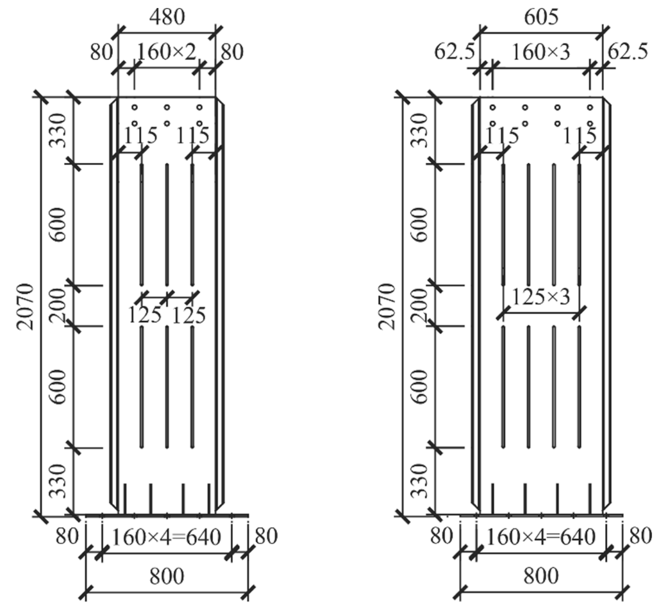
Story	HBE (mm)		VBE (mm)		PT cables area (mm^2)	Gap opening force (kN)
	Section	Length	Section	Length		
1	H350 × 150 × 20 × 40	2,000	H340 × 150 × 30 × 35	3,000	1,120	151 (−99)
2	H300 × 150 × 20 × 35	2,000	H340 × 150 × 30 × 35	3,000	840	114 (−117)

frame in the X direction. HBEs and VBEs of the PTFs are made of Grade Q345 steel. PT cables use 1 × 7–15.2–1860 type, whose cross-section area is 140 mm^2 and ultimate tensile strength is 1,860 MPa, as defined in Steel Strand for Prestressed Concrete (SSPC) [39]. Yield strength of the PT cables is stipulated as 90 percent of the ultimate tensile strength. The initial prestress of the 1st-story PTF's PT cables is measured as 484.8 MPa (at lower HBE) and 516.9 MPa (at upper HBE), while that of the 2nd-story PTF's PT cables is 603.2 MPa (at lower HBE) and 421.6 MPa (at upper HBE). More details about the dimension of PTFs, including the section of PTF elements and the total cross-section area of PT cables, are given in Table 1. The SWs are installed through high-strength bolt slip-critical connections to the PTFs. Fig. 7 shows pictures of connections between SW and PTF. The main slit steel plates are made of Grade Q235 steel, while the other parts of SWs, such as edge stiffeners and bottom end plates, are made of Grade Q345 steel. The main slit steel plates are 4 mm thick plate and the slits are 10 mm wide. More details about dimension of SWs are shown in Fig. 8.

Payload mass are secured to the reinforced concrete floor slabs in which the steel reinforcements are Grade Q345 steel and the concrete are Grade C30 (the design value of axial compressive strength is 14.3 MPa in China code, Code for Design of Concrete Structures [40]). The two floor slabs are designed to be 40 mm and 120 mm thick, respectively, in consideration of the floor mass requirement and corresponding payload acting on them. The steel reinforcement of the 1st-floor is weld steel mesh with a diameter of 4 mm for each plain steel rebar and spacing of 40 mm between rebars. The 2nd-floor uses steel ribbed rebars with a diameter of 8 mm in both X and Y direction and the distance between rebars is 150 mm. In final, the actual seismic floor mass (including payload mass blocks) of the test structure is 7,550 kg and 25,580 kg for the 1st-floor and the 2nd-floor respectively.

2.2. Design performance

In these shaking table tests, the SCMP-BTSF structure was designed to meet the seismic code requirement of CSDB (2010) [38]. The expected performance of the test model is summarized as follows:



(a) (b)

Fig. 8. Dimensions and configuration of slit steel plate walls: (a) 1st-story SW; (b) 2nd-story SW.

Fig. 9. Photo of the test structure on the shake table.

(a) (b)

Fig. 7. Photo of bolted connection between SW and PTF: (a) upper connection; (b) lower connection.

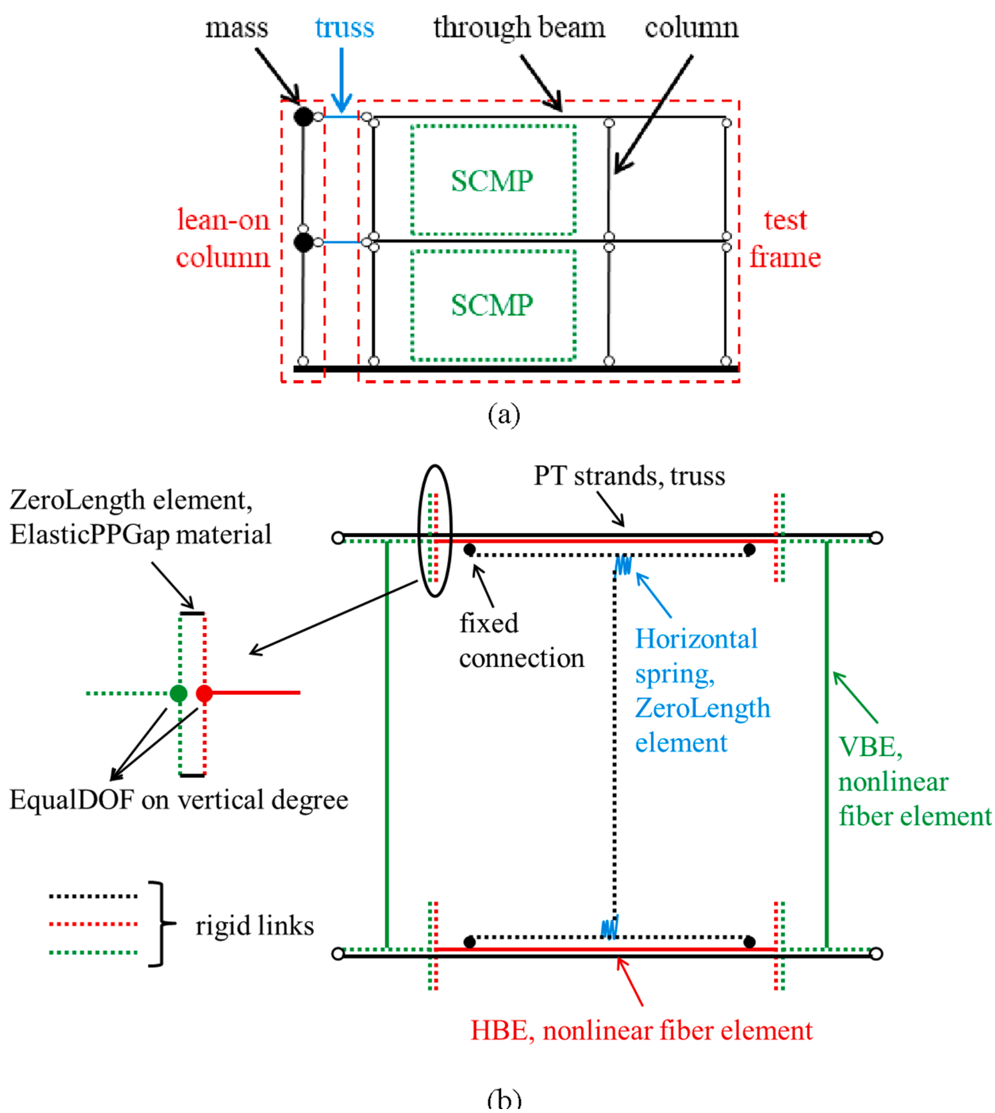


Fig. 10. Schematics of FE model (not to scale): (a) test structure; (b) SCMP modeling details.

(1) The test model would remain linear elastic and the inter-story drift response would meet the peak inter-story drift limit in Chinese seismic code (CSDB, 2010), when the test model is subjected to frequent ground motions corresponding to 63% probability of exceedance in 50 years.
(2) The structural damage would be mostly concentrated to the SW elements, while other structural components would remain elastic, including structural beams, columns and PTFs.
(3) The test model is able to re-center itself after tests with design level ground motions, meaning the residual inter-story drift should be less than 0.2%.

2.3. Test setup

The two-story test structure was tested on a shake table at the Multi-functional Shaking Tables Lab of Tongji University in Shanghai, China. The shake table has an overall table dimension of 6 m × 4 m, maximum payload capacity of 700 kN, operational frequency range 0.1–50 Hz, PGA of 1.5 g and displacement range of +/-500 mm. The overall view of the test structure on the shaking table is shown in Fig. 9.

2.4. Finite element analysis and response prediction

A FE model has been developed using a nonlinear structural analysis software - OpenSees [37] for specimen design, nonlinear response prediction and test planning. In this FE model, only the seismic-force-resisting system including the SCMPs and associated steel frames are modeled while other gravity load carrying frames are not explicitly modeled. Fig. 10 shows schematics of the FE model for the test structure. A hysteretic uniaxial material model, Steel01, available in OpenSees has been used for most steel members including through beams, columns, HBEs and VBEs. For Grade Q345 steel used for these components, its material properties are set as follows: modulus of elasticity, E_0, of 2.06 $\times$ 10^5 MPa, yield stress, F_y, of 345 MPa and a strain-hardening ratio, b, of 1%. Here b is the ratio of post-yield modulus to initial elastic modulus. Steel02 material has been adopted for steel PT cables, since one of its parameters, *sigInit*, can be used to model the initial PT stress. Other material properties of Steel02 for steel PT cables are set as follows: $E_0 =$ 2.06 × 105 MPa, F_y = 1720 MPa, b = 2%. Nonlinear fiber element is used to model through beams, columns, HBEs and VBEs, while truss element is used for PT cables. The beam-to-column connections and column bases in the gravity load carrying steel frame are modelled as hinged connections, because their rotational stiffness is fairly small due to their simple connection configuration. A lean-on column is also

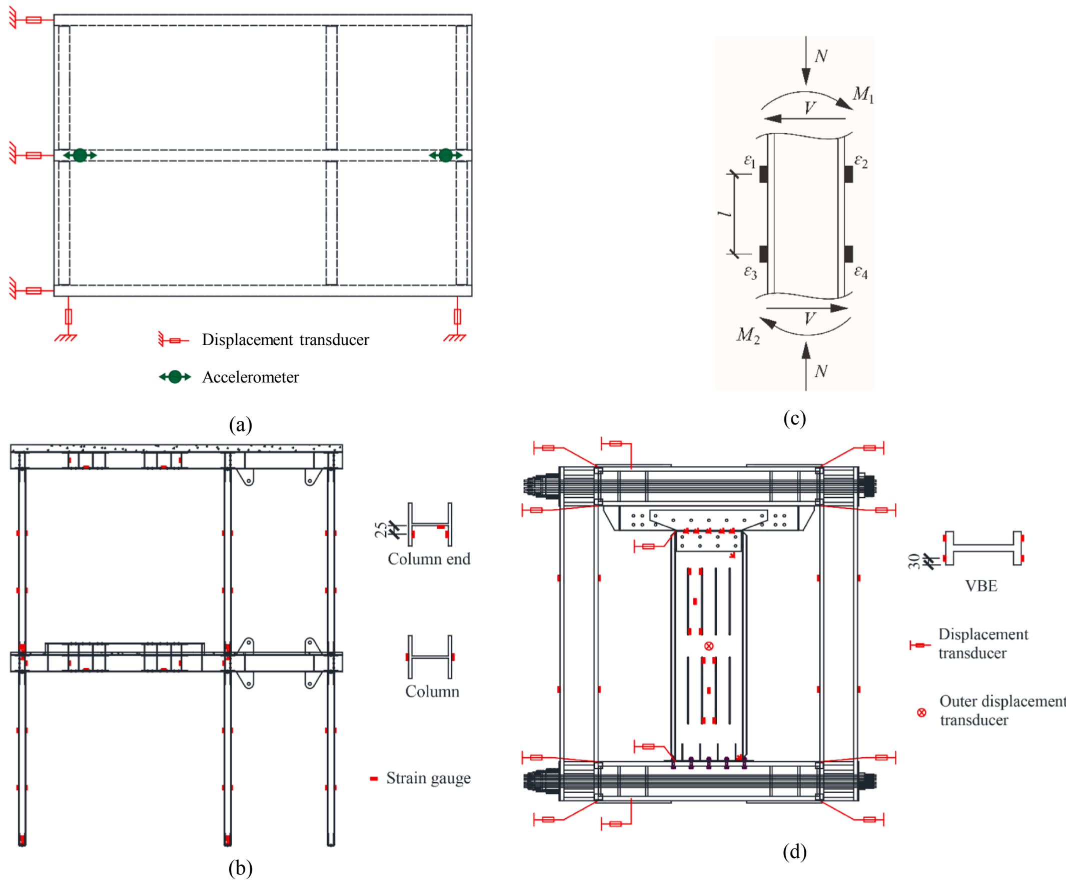


Fig. 11. Arrangement of instrumentation: (a) accelerometers and displacement transducers in BTSF; (b) strain gauges in BTSF; (c) location of strain gauges for calculating column internal forces; (d) strain gauges and displacement transducers in the 1st-story SCMP.

Table 2
Summary of earthquake ground motions information.

Name	M	Year	Recording station	Records name	Vs_30 (m/s)	PGA_{max} (g)	PGV_{max} (cm/s)	Duration (s)
Northridge	6.7	1994	Canyon Country-WLC	NORTHR/LOS270	309	0.48	45	20.0
Superstition Hills	6.5	1987	Poe Road (temp)	SUPERST/B-POE360	208	0.30	36	22.3

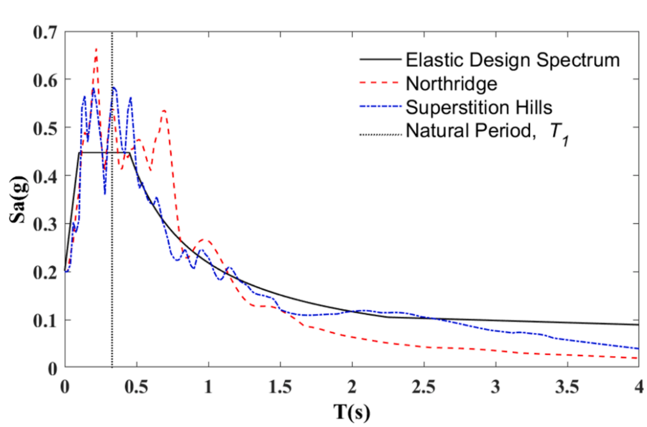


Fig. 12. Response spectrum of the selected ground motions.

included in the FE model to consider P-Δ effects.

SCMP model elements are connected to BTSF through rigid links between HBE and corresponding locations in the through beam. Fig. 10 (b) shows a SCMP model, in which the PT connection model similar to the approach by Ricles et al [41] has also been created to simulate the gap opening and closing mechanisms of rocking members. For the SWs, horizontal zero-length springs were used to simulate its nonlinear macro hysteretic behavior. The Pinching4 material in the OpenSees software [42] is uasually used to simulate wall with pinching characteristic [43]. In this study, the nonlinear springs simulating SWs are specified with the Pinching4 material, whose parameters are calibrated with test result from both numerical and experimental studies involving cyclic loading of SWs.

Considering the reasonable arrangement of damping ratio of a bare steel frame without non-structural components, a 0.50% damping ratio is assigned to the FE model in the form of Rayleigh damping. Based on modal analysis results, the natural period of calibrated FE model is 0.343 s.

2.5. Instrumentation

2.5.1. Structural frame

Two uniaxial accelerometers are installed near the beam-to-column joints of interior frame on each floor to measure the dynamic response and derive the story shear in the X direction based on inertial force. Displacement transducers are installed to measure the drift at the ends of

Table 3
Shaking table test cases.

Case	Input excitation	PGA (g)	Test duration (s)
1–1	White Noise	0.05	62.40
1–2	Northridge	0.11	29.49
1–3	White Noise	0.05	62.52
1–4	Northridge	0.21	29.04
1–5	White Noise	0.05	61.65
1–6	Northridge	0.31	29.57
1–7	White Noise	0.05	61.52
1–8	Northridge	0.54	29.30
1–9	White Noise	0.05	61.89
1–10	Northridge	0.78	29.40
1–11	White Noise	0.05	62.30
1–11 s	Northridge	1.16	29.12
1-11ss	White Noise	0.05	61.28
At completion of 1-11ss case, both seismic fuse members – SWs were replaced			
1–12	White Noise	0.05	61.79
1–13	Northridge	0.11	28.89
1–14	White Noise	0.05	62.33
1–14 s	Northridge	0.11	29.05
2–1	Superstition Hills	0.10	54.17
2–2	White Noise	0.05	61.25
2–3	Superstition Hills	0.22	54.03
2–4	White Noise	0.05	61.85
2–5	Superstition Hills	0.41	52.93
2–6	White Noise	0.05	61.11
2–7	Superstition Hills	0.48	53.74
2–7 s	Superstition Hills	0.59	54.14
2–8	White Noise	0.05	61.92

Table 4
Summary of structure response from shake table tests.

Test Case	PGA (g)	Peak acceleration (amplification factor, β_i)		Peak inter-story drift (%)		Residual inter-story drift (%)	
		1st-floor	2nd-floor	1st-floor	2nd-floor	1st-floor	2nd-floor
Ground motion: Northridge – Canyon Country-WLC							
1–2	0.11	1.96	2.67	0.11	0.16	0.006	0.007
1–4	0.21	1.81	2.52	0.23	0.31	0.001	0.019
1–6	0.31	1.55	2.24	0.36	0.42	0.010	0.005
1–8	0.54	2.00	1.78	0.59	0.68	0.012	0.011
1–10	0.78	1.86	1.51	1.05	1.17	0.006	0.012
1–11 s	1.16	1.79	1.10	1.50	1.61	0.002	0.013
1–13	0.11	1.91	2.56	0.09	0.17	0.001	0.004
1–14 s	0.11	1.89	2.59	0.09	0.16	0.001	0.001
Ground motion: Superstition Hills – Poe Road (temp)							
2–1	0.10	2.59	3.02	0.12	0.19	0.004	0.002
2–3	0.22	2.23	2.35	0.26	0.35	0.005	0.008
2–5	0.41	2.01	2.00	0.45	0.59	0.005	0.013
2–7	0.48	2.19	1.93	0.59	0.72	0.003	0.006
2–7 s	0.59	3.58	2.02	1.20	1.40	0.010	0.017

the beams in both X and Y directions. Fig. 11 (a) shows the locations of these accelerometers and displacement transducers.

Fig. 11 (b) shows the locations of strain gauges in the main frame. Strain gauges have been used to monitor the strain and yielding occurrence of structural members. Also, they can be used to calculate the shear force in the columns, which can be calculated using Eq. (1). Fig. 11 (c) shows the column internal forces and strain gauge location.

$$V = \frac{M_2 + M_1}{l} \tag{1}$$

$$M_1 = \frac{EW_c(\varepsilon_1 - \varepsilon_2)}{2} \tag{2}$$

$$M_2 = \frac{EW_c(\varepsilon_3 - \varepsilon_4)}{2} \tag{3}$$

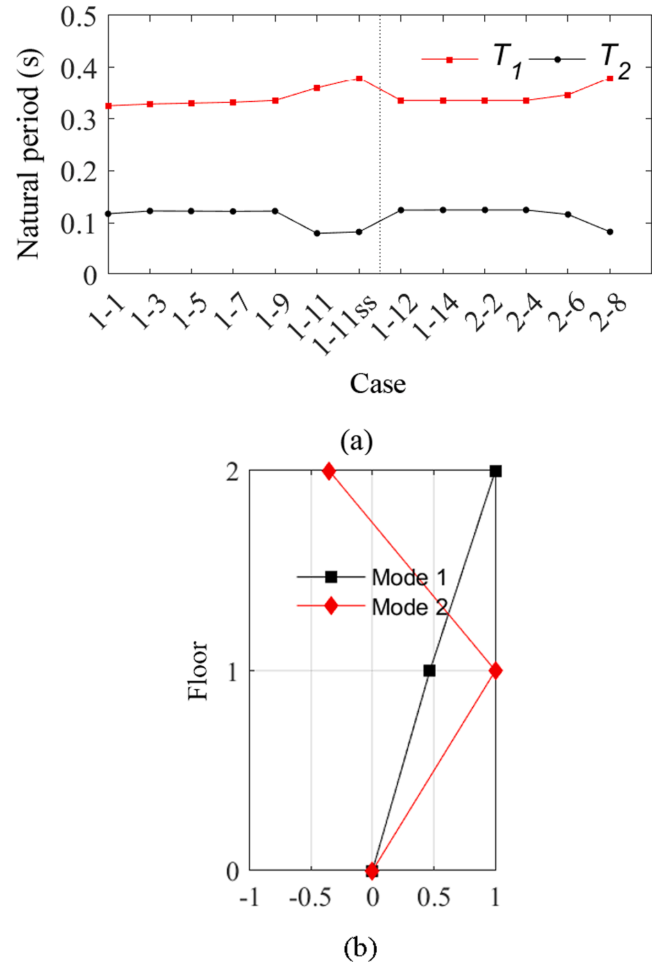


Fig. 13. Modal properties in white noise cases: (a) natural period; (b) mode shapes of the initial test structure.

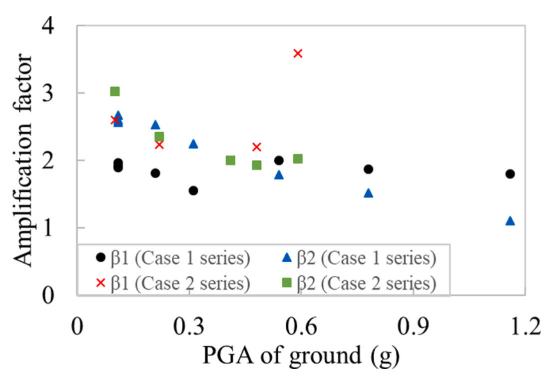


Fig. 14. Acceleration response amplification factor vs. base excitation PGA.

2.5.2. SCMP

Fig. 11 (d) shows the locations of displacement transducers and strain gauges in the 1st-story SCMP while 2nd-story SCMP has a similar arrangement. Eight displacement transducers were used for each PTF to measure the gap opening values at the PT connections. The one installed at the center of SW was used to monitor the out-of-plane displacement of SW. The others are used to measure the relative slip between the PTF and through-beams, as well as the relative slip between SW and PTF.

The strain gauges on the VBEs were used to calculate the horizontal shear force in the VBEs while the strain gauges on SW were used to monitor the strain of steel slats between slits and corners of SW.

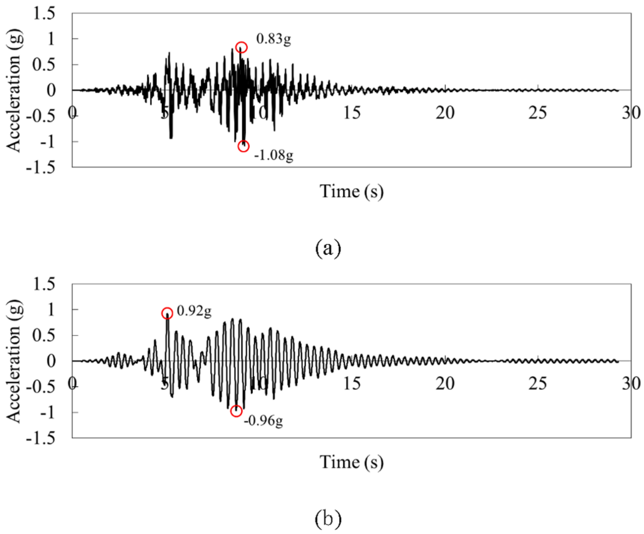


Fig. 15. Acceleration time history of test structure in Case 1–8: (a) 1st-floor; (b) roof.

Anchor cable dynamometers were used to monitor the tension force of PT cables, which can only be measured when the structure is static due to the special design of this type dynamometer device. However, the measurement done between tests has been very helpful to determine whether pre-tension force loss has happened after each test.

3. Test program

3.1. Ground motions

Two far-field earthquake ground motion records from Northridge 1994 and Superstition Hills 1987 respectively, are selected from FEMA P695 [36] and their details are listed in Table 2. The ground motions are scaled to the target PGA of 0.2 g for elastic time history analysis. Fig. 12 shows the response spectra of scaled ground motions and the elastic design spectrum with PGA of 0.2 g and a 5% damping ratio. The vertical dashed black line denotes the initial natural period of the test structure, 0.33 s.

3.2. Loading protocol

To investigate the seismic performance and resilience characteristics of BTSF with SCMP under varying seismic hazard levels, the loading protocol consisted of the selected ground motions with different scaling factors, starting with low intensity (0.1 g) and increasing towards greater amplitudes. Additionally, white noise excitation with peak acceleration value of 0.05 g was also conducted to measure the natural period and damping ratio of the test structure before and after each case (with a few exceptions). The change of properties would offer insight into the damage extents suffered by the test structure during the previous test and effect of replacing the damaged SWs. Table 3 lists the details of the adopted loading protocol.

4. Experimental results

4.1. General result

Table 4 summarizes the recorded structural response from each test case, including acceleration (amplification factor defined as maximum floor acceleration value normalized by excitation ground motion's PGA value) and displacement (peak inter-story drift and residual inter-story drift). Hysteretic response and damage pattern will be discussed later.

The input excitation to the shaking table was only applied in the X direction. It indicates that the installed tension-only braces along the Y direction are strong enough to limit the undesired displacement and rotation response. It should be also noted that the structure was subjected to series of tests and the SWs were only replaced once (see Table 3) in the tests, while in real structures the SWs would be replaced after a strong earthquake.

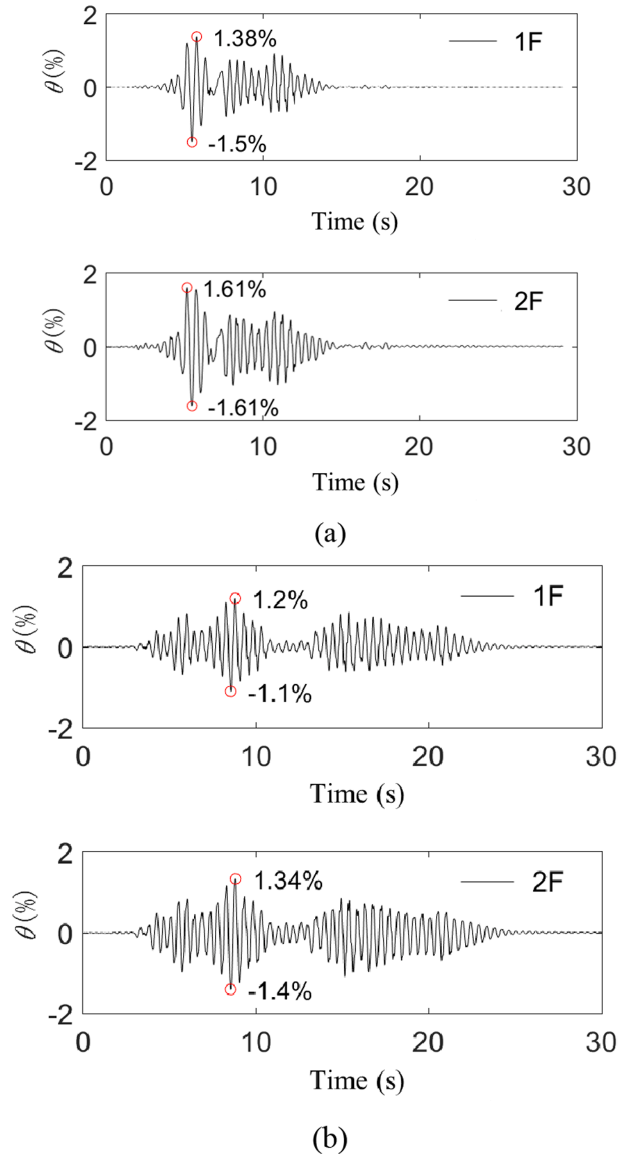


Fig. 16. Measured inter-story drift time history of test structure in two tests: (a) Case 1–11 s; (b) Case 2–7 s.

4.2. Modal properties

Modal properties include the test structure's natural period, T_1, viscous damping ratio, ξ and mode shapes. The change in modal properties might reflect the damage suffered by the test structure in previous tests. Natural period and mode shapes are derived using the corresponding transfer functions that relate the floor acceleration response and base excitation during the white noise tests, while the viscous damping ratio is derived from the free vibration attenuation duration after the white noise excitation inputting. Fig. 13 (a) shows the change of natural periods over the test cases. The vertical dotted line in this figure denotes the timing when the SWs were replaced in the test structure. Since the change of mode shape is very small even though the

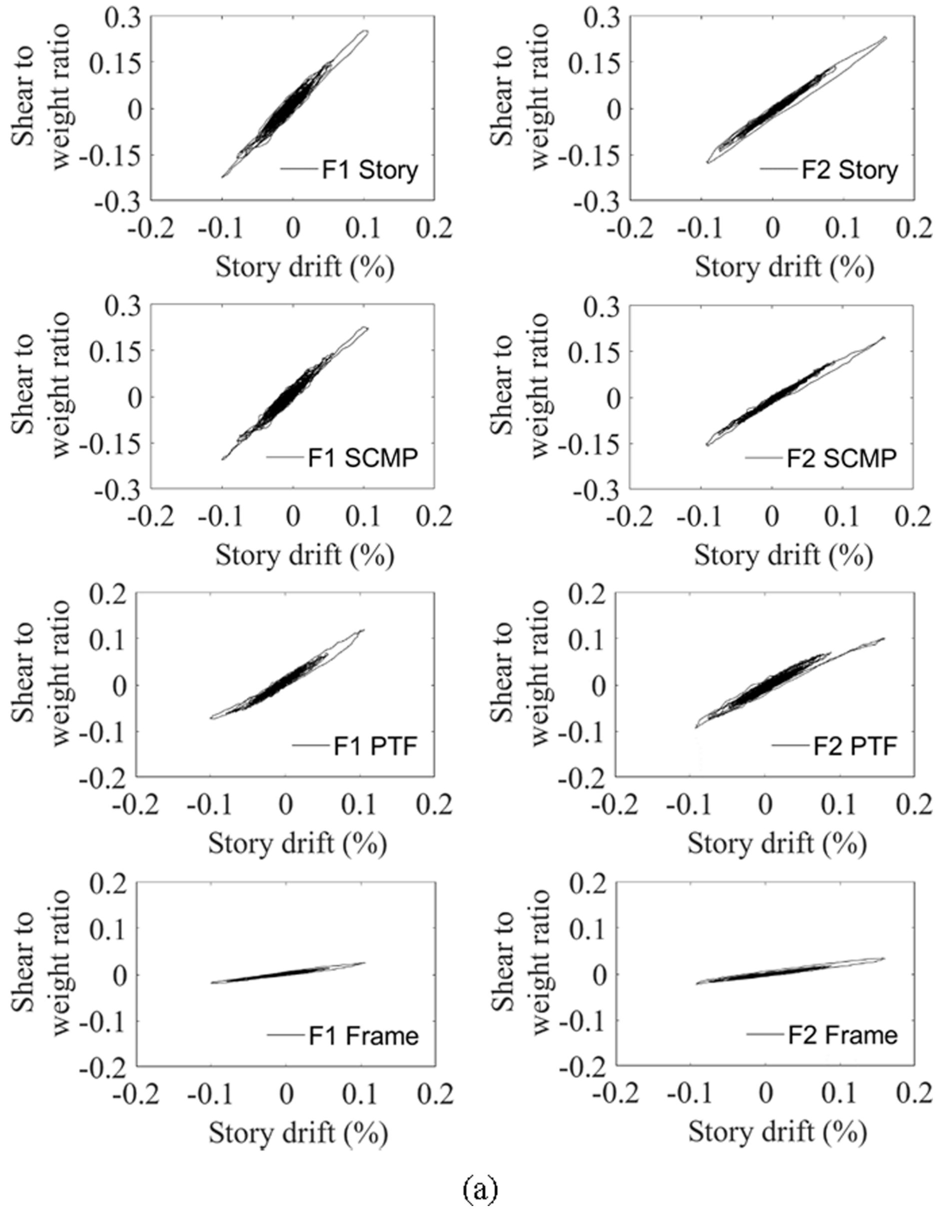


Fig. 17. Internal force response of stories, SCMPs, PTFs and gravity load carrying frame: (a) Case 1–2; (b) Case 1–11 s.

structure underwent severe earthquake loading in the tests, Fig. 13 (b) only shows the mode shapes of the initial test structure. The viscous damping ratio of the initial test structure is measured as 0.45%, and thus it is rational to adopt the damping ratio as 0.50% in the FE model for the first vibration mode.

The natural period of test structure is measured as 0.33 s. And it has barely changed after completing the tests of Case 1–2 to Case 1–6, suggesting that the test structure have no detectable damage in these cases (these are also corroborated by visual inspection of test structures after these tests and subsequent story shear hysteresis curve analysis). After Case 1–8 to Case 1–11 s, it could be found that the structure has suffered some damage, reflected by the increase in natural period, changed to 0.38 s (measured in Case 1-11ss). After Case 1-11ss, the test structure has been repaired by replacing the SWs, followed by a white noise excitation test, Case 1–12. The results show that the natural period has almost recovered. This is consistent with the observation that damage in the test structure mainly occurred in the SWs and replacing with new SWs makes the test structure return to its initial condition. Case 1–13 and Case 1–14 s are also conducted to investigate the seismic performance of the structure after SW replacement with repeated experiments.

In the test series of Case 2 (Superstition Hills earthquake), the natural period has remained stable before Case 2–5 (with PGA of 0.41 g). The test structure has been damaged again in Case 2–5 and Case 2–7, determined from the change in the measured dynamic parameters. The natural period, measured in Case 2–8, has changed to 0.38 s.

4.3. Acceleration response

The acceleration response in Table 4 is presented by the amplification factor, β_i (where i is the floor number), which is defined as the ratio of the maximum floor acceleration response value to base excitation PGA. The amplification factors vary from 1.10 to 2.67 under the Northridge cases, and the maximum amplification factor of 2.67 occurred at the second story in Case 1–2 with PGA of 0.11 g. Superstition Hills earthquake has a far greater impact on the acceleration response