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FULL SCALE STEEL PLATE SHEAR WALL: NCREE/MCEER PHASE I TESTS

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ABSTRACT

Measuring eight meters tall and four meters wide, two 2-story SPSW specimens were constructed in NCREE. The thickness of SS400 grade steel plate for the first story wall is 3 mm and for the second story it is 2mm. All the boundary beam and column elements are A572 GR 50 steel. In the Phase I tests, each of the SPSWs have horizontal tube restrainers on both sides to minimize the out-of-plane displacement and the buckling sound. In the Phase 2 tests, damaged steel plates were removed and replaced with new plates without the use of any restrainer. In both phases, the specimens was pseudo-dynamically tested using three ground accelerations, which were recorded in the 1999 Chi-Chi earthquake and scaled up to represent seismic hazards of 2%, 10%, and 50% probabilities of exceedance in 50 years. Results of the Phase 1 tests show that; 1) the SPSWF specimen sustained three earthquakes without any significant wall fracture or overall strength degradation, 2) the horizontal restrainers were very effective in improving the serviceability of SPSWs, 3) the responses of the SPSWF can be simulated accurately using the strip model and the tension-only material property implemented in PISA3D computer program.

Introduction

A typical SPSW frame structure is shown in Fig. 1. Because of the high stiffness and strength of the SPSW frame system, thin steel plates are often used. The thin plate is very easy to buckle in shear. After the infill plate is buckled in shear, diagonal tension field action can be developed as shown in Fig. 2. The SPSW system can then dissipate energy through the yield of the tension struts. In recent years, several researchers have confirmed that the steel plate shear wall (SPSW) can be a viable seismic force resisting system for building structures (Berman 2002, Berman and Bruneau 2003, Vian and Bruneau 2003, Driver et al. 1998, Lubell 2000). Although the SPSW can cost-effectively satisfy the lateral stiffness, strength and ductility requirements for seismic buildings, experimental research on large-scale SPSW structures is rather limited. Considering the small-scale structure test results could not satisfactorily represent the

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seismic performance of real buildings, a full scale 2-story SPSW specimen was constructed and tested recently in the National Center for Research on Earthquake Engineering (NCREE). This study was a collaborative research (Tsai et al. 2006) among National Taiwan University (NTU), NCREE, University at Buffalo (UB) and Multidisciplinary Center for Earthquake Engineering Research (MCEER). The specimen measures eight meters tall and four meters wide. The tests included two phases. In the Phase I tests, each of the SPSWs has horizontal tube restrainers on both sides of the steel plate to minimize the out-of-plane displacement and the buckling sound (Photo 1). Before the Phase 2 tests, damaged steel plates were removed and replaced with new plates without the use of any restrainer. This paper focuses on the design method and procedures, experimental setup and phase I test result. A companion paper discusses the test results associated with the Phase 2 tests (Qu et al. 2007).

Cyclic Responses of Single-story SPSWS

The strip model (Fig. 3) proposed by Kulak (1983) is often used for the analysis of SPSW frame. In his model, a series of inclined, pin-ended, tension members are used to represent the tension field action in the steel plate. Since the thin steel plate in the SPSW can be buckled easily before tension field actions developed, these tension members should possess very little compressive capacity. In this research, a tension-only material model was implemented (Tsai et al. 2006) for the PISA3D computer program (Lin and Tsai, 2003). The stress versus strain relationships for the tension-only material are shown in Fig. 4. This material property could represent the responses of a typical tension strip developed in the thin steel plate subjected to cyclic strains. Figure 5 shows the elevation of a SPSW test specimen (Lin and Tsai, 2004) and the analytical tension-only strip model. In Fig. 6, it can be found that the analytical results well agree with the test. In addition, the analytical results can be conveniently used to study the deformation demands imposed in the center and corner of the steel shear wall. It can be found that the tension field action is much more pronounced in the center (Strip+8) than that in the corner (Strip+1) of a SPSW.

Experimental Program

Design of A Two-story SPSW Specimen

Figures 7 through 9 indicate the floor framing plan, elevation of the SPSW, and the 3D perspective of the prototype structure. It is assumed that the 2-story prototype building has a perimeter steel moment resisting frame (MRF) and two SPSWs in the transverse direction. There are two assumption of this 2-story prototype building: 1) this structure is located in East District in Chiayi City of Taiwan, 2) floor weight of this building is 700kg/m^2. The fundamental vibration periods are 0.52 and 0.72 seconds in the transverse and longitudinal directions, respectively. According to the latest seismic force requirements for new buildings in Taiwan (ABRI 2002), the design base shear for both directions is about 22% weight of the structure. Considering the availability of the thin steel plate, the SS400 grade steel was chosen for the steel shear wall. All the boundary beam and column elements are A572 GR. 50 steel. First assuming that two SPSW frames (steel plate and boundary frame) resist 75% of the total lateral force, but the boundary columns resist 30% of the SPSW frame lateral force. The plate thickness and boundary frame member sizes were decided based on recommendations provided by Berman and Bruneau (2003). In order to insure the tension field action can develop when SPSW frame is subjected to the lateral loads, the capacity design method is used in selecting the size of the boundary elements. First assuming all boundary beams and columns were fixed in both end while subjected to the distributed load due to tension field action as shown in Fig. 10. The distributed loads acting on the beams and columns is given by the following equation:

$$\text{Beam: } W_b = F_y t \cos^2 \alpha \tag{1}$$

$$\text{Column: } W_c = F_y t \sin^2 \alpha \tag{2}$$

where F_y is the infill plate yield stress in tension; t is the thickness of infill plate; α is the angle of inclination of the strips.

Then considering the effects of material overstrength, the demands of axial force and shear force for boundary beams are calculated from Eqs. 3 and 4:

$$P_{\max} = \frac{1}{2} \times \Omega_s \times W_b \times \tan\alpha \times L \qquad (3)$$

$$V_{\max} = \frac{1}{2} \times \Omega_s \times W_b \times L \qquad (4)$$

where Ω_s is the material overstrength factor of steel plate (assume Ω_s=1.5 here); α is the angle of inclination of the strips; *L* is the beam span. However, the flexural demand for boundary beams includes two parts ($M_{boundary\ frame}$ and $M_{tension\ field}$). $M_{tension\ field}$ is the maximum moment induced by the distributed load at boundary beams as shown in Fig. 11. The maximum shear in the panel due to tension field action can be calculated from Eq. 4. And assume the boundary frame resist 30% of the SPSW frame lateral force, the shear force carried by boundary frame can then be computed. It is assumed that the boundary frame remain elastic, therefore the $M_{boundary\ frame}$ can be obtained from a elastic analysis using the shear force carried by the boundary frame as shown in Fig. 12. Similarly, the column axial, shear and flexural demands are computed from both the MRF action and tension field action. The demand to capacity ratio (DCR) of boundary elements was kept under 0.9 using the force demands noted above. The thickness of steel plate for the first story wall is 3mm and for the second story is 2mm. The actual yield strengths for the steel plates are 335MPa (1F) and 338MPa (2F). Detailed specimen member sizes are shown in Fig. 8. In order to reduce the buckling sounds and minimize the out-of-plane buckling of the steel panels, this specimen is restrained by three horizontal restrainers on both sides of the infill plate (Lin and Tsai, 2004). The restrainer is designed by considering a uniformly distributed out-of-plane tributary load equal to 3% of the SPSW maximum shear. The sizes of the tube restrainers are: Tube-125x75x4 mm for the first story and Tube-125x75x2.3 mm for the second.

Analytical Predictions

Before the actual testing, analytical predictions were performed on the complete 2-story PISA3D structure model including the parameter MRF and the SPSW. For each SPSW frame, two series of strips with inclined angles of ±41 degrees were constructed (as that shown in Fig. 9). For the parameter MRF, all the beam and column members in MRF and boundary frame of SPSW adopt the bi-linear beam-column element. The tension coupon strengths (Table 1) of the steel plates, the beams and columns were incorporated into the analytical model. Based to the analytical results, it was decided to use three 100-ton actuators for each floor. Another two actuators were installed in the out-of-plane direction of the specimen. The test setup is given in Fig. 13. During the actual hybrid experiments, the mechanical properties of the entire MRF was analytically simulated while the two SPSWs were assumed identical and experimentally tested. After the Phase I tests, the steel panels were removed and a new set of steel panels were installed for the Phase II tests in order to study the seismic performance of the SPSWs without the steel tube restrainers. Details of the test results have been documented (Qu et al. 2007,Tsai and Lin 2006).

Substructure Pseudo Dynamic Tests

Test Procedures

In phase I tests, it was planed to test the specimen using pseudo-dynamic test procedures and a Chi-Chi earthquake record scaled up to represent seismic hazards of 2%, 10%, and 50% probabilities of exceedance in 50 years. The original ground acceleration record is TCU082EW as shown in Fig. 14. Test schedule and excitation information is shown in Table 2. However, premature crack of concrete slab occurred in the second floor slab in the very first test. Thus, the test had to be stopped at the time step of 9.5 sec. Then four H300mm floor beam were added below the concrete slab in order to allow the lateral force to transfer from the actuators into the SPSW frame. Afterwards, Test 1 was restarted after the strengthening. However, unexpected failure occurred again at south column base at the time step of 24.0 sec. In this case, it was found that two anchor bolts in the column base plate were fractured. The test was stopped again. Welds were then added to attach the column base plate to the strong floor tie-down plate. Test 1 resumed and hybrid test was finally successfully completed. It was found in the specimen that significant buckling and a number of small cracks had occurred in the steel plate in both floors as exampled in photo 2. It was also found evident yielding of various boundary members (Photos 3 to 7). After the Test 1, Test 2 and 3 were successfully completed in the Phase I study. After the Phase I tests, steel plate had seriously buckled and several cracks were observed. However, no fracture was found in the boundary frame. During the tests, all the key analytical predictions and experimental responses were broadcasted from a website (http://exp.ncree.org/spsw).

Key Test Results

Figures 15 and 16 present the roof experimental displacement and base shear time histories in both the 2/50 and 10/50 events. The peak story drifts for 2/50 and 10/50 events are 0.025 and 0.02 radians, respectively. It is evident that the peak roof displacement and the base shear responses can be satisfactorily predicted by PISA3D as shown in Figs. 15 and 16. Figure 17 shows the inter-story drift verses story shear relationships. It appears in Fig. 17 that the energy dissipation of the SPSW in 2/50 is evident. In the 10/50 event, the energy dissipation of the SPSW is less pronounced as that found in the 2/50 event. During the Phase I tests, it appears that the specimen's strength or stiffness degradation is not significant.

Conclusions

Base on the test results and analytic study, conclusions and recommendations are made as follows:

- The responses of the SPSWF can be accurately predicted using the strip model and the tension-only material property implemented in PISA3D computer program.
- The SPSWF specimen sustained three earthquake excitations without significant steel plate fracture or overall strength degradation. It appears that the demand of the boundary frame members computed from the propose capacity design method is adequate to sustain the MRF action and tension field action developed in the steel plate.
- After Phase1 tests, the horizontal restrainers did not show any sign of damage. It appears that the 3% of the in-plane force assumption is appropriate for sizing the restrainers.
- The strip model can be conveniently used to study the deformation demands imposed in the center and corner of the steel shear wall. It is found from the analysis and the test results that the tension field action is much more severe in the center than that in the corner of a SPSW.

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Table 1. Material coupon test results.

	coupon positions		Thickness(mm)	fy(MPa)	fu(MPa)
Steel	Panel (SS400)	1F	3	338	482
		2F	2	335	412
	Beam (A572)	Base Beam(Web)	19	285	480
		Base Beam(Flange)	28	355	487
		Middle Beam(Web)	12	505	626
		Middle Beam(Flange)	19	476	581
		Top Beam(Web)	13	305	460
		Top Beam(Flange)	22	354	517
	Column (A572)	web	25	377	505
		Flange	40	363	544
Concrete		fc'=27.5 MPa			

Table 2. Test schedule.

PhaseI Test: Restrained Steel Plate Shear Wall		
	Excitation	Hazard Level
Test 1	Chi-Chi(TCU082EW)	2% in 50 Years (PGA=0.67g)
Test 2	Chi-Chi(TCU082EW)	10% in 50 Years (PGA=0.53g)
Test3	Chi-Chi(TCU082EW)	50% in 50 Years (PGA=0.22g)

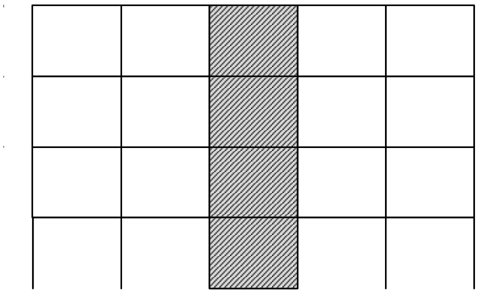


Figure 1. SPSW frame system.

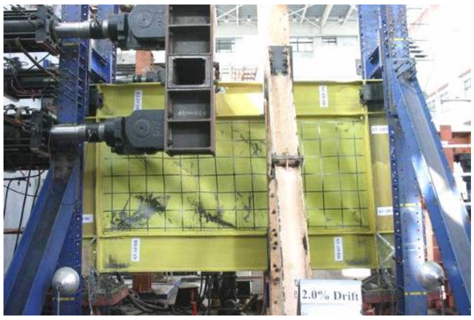


Figure 2. Tension field action.

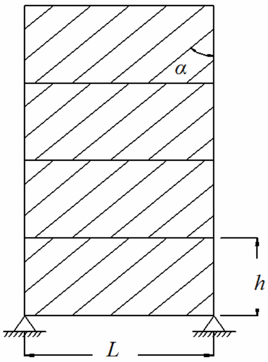


Figure 3. Strip model.

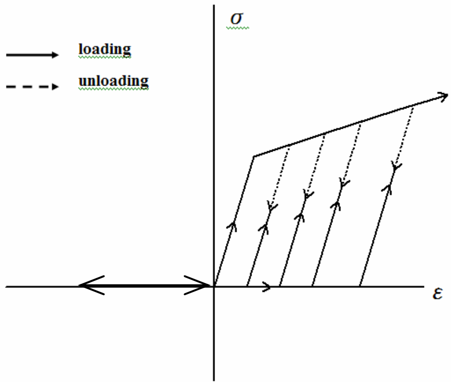


Figure 4. Tension-only material.

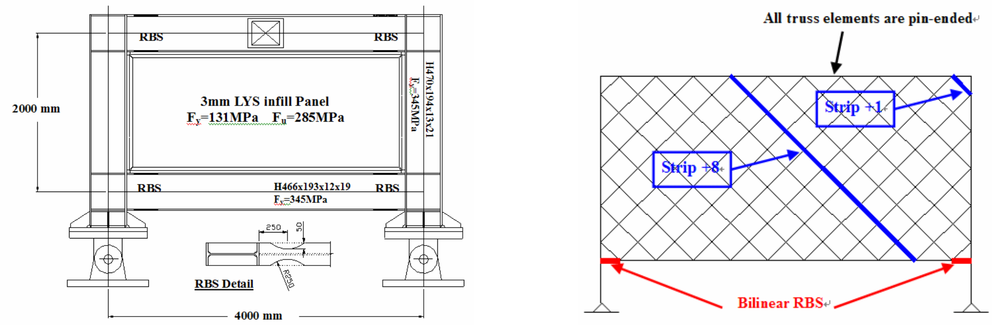


Figure 5. The SPSW specimen and PISA3D analytical model (Lin and Tsai, 2004).

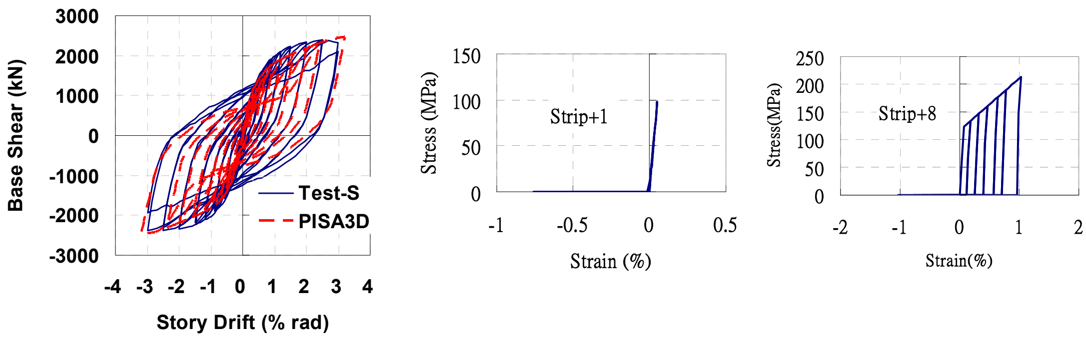


Figure 6. PISA3D analytical results.

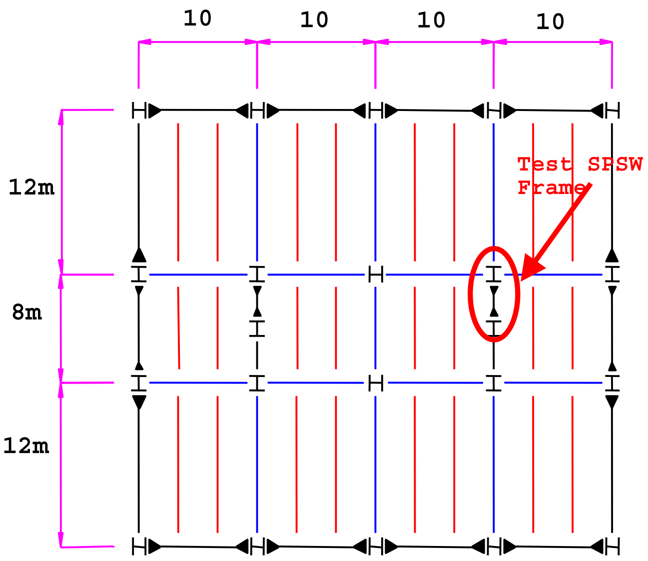


Figure 7. Floor framing plan of the protype building.

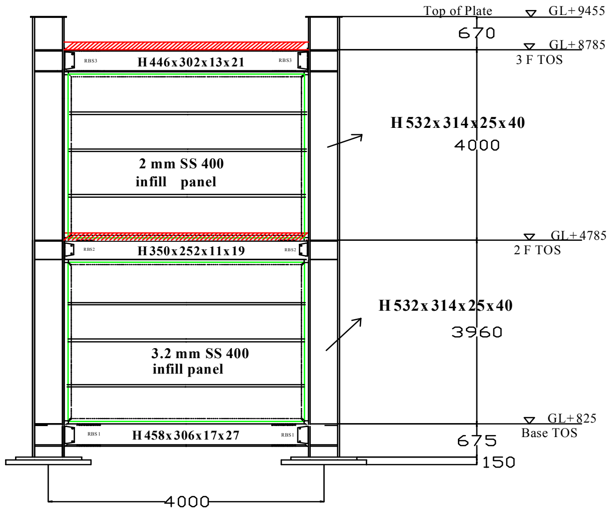


Figure 8. Specimen elevation.

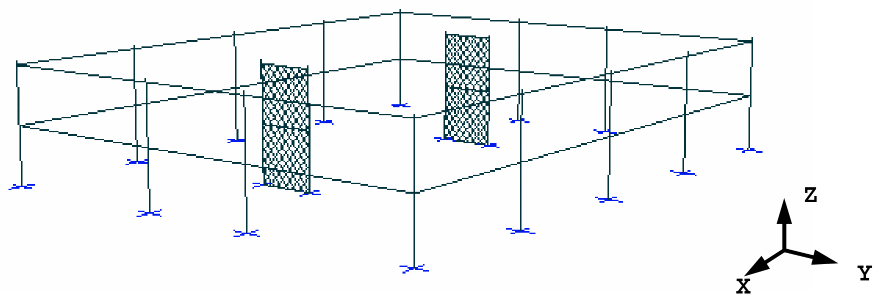
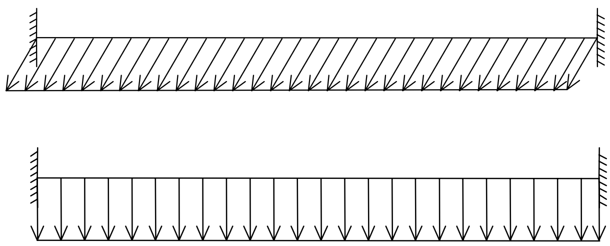


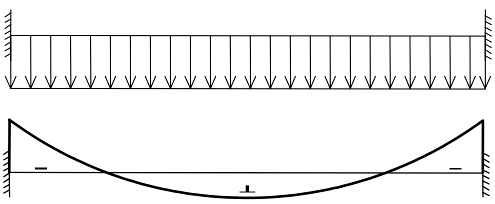
Figure 9. PISA3D analytical model.



$$w_{strip} = F_y \cdot t \cdot \cos\alpha$$

$$w_b = F_y \cdot t \cdot \cos^2 \alpha$$

Figure 10. Beam boundary condition and distributed load.



$$w_b = F_y \cdot t \cdot \cos^2 \alpha$$

Figure 11. Moment diagram due to tension field.

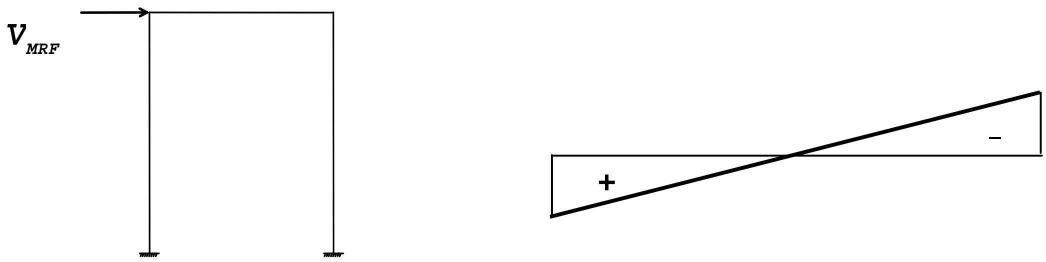


Figure 12. Moment diagram due to $V_{boundary\ frame}$.

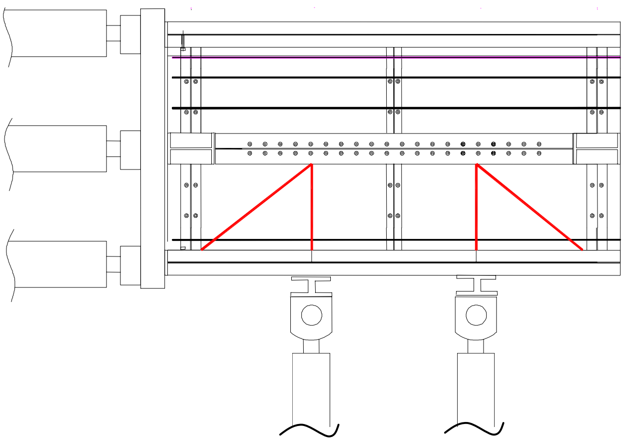


Figure 13. The arrangement of the actuators in a typical floor.

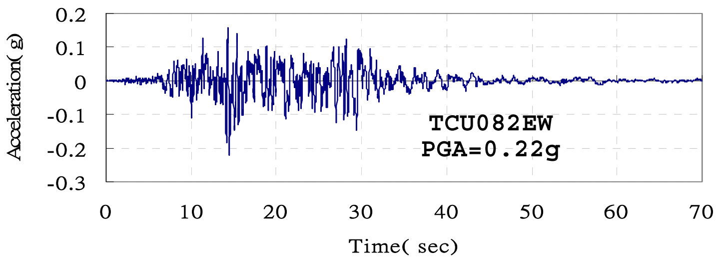


Figure 14. Original Ground Acceleration Time History.

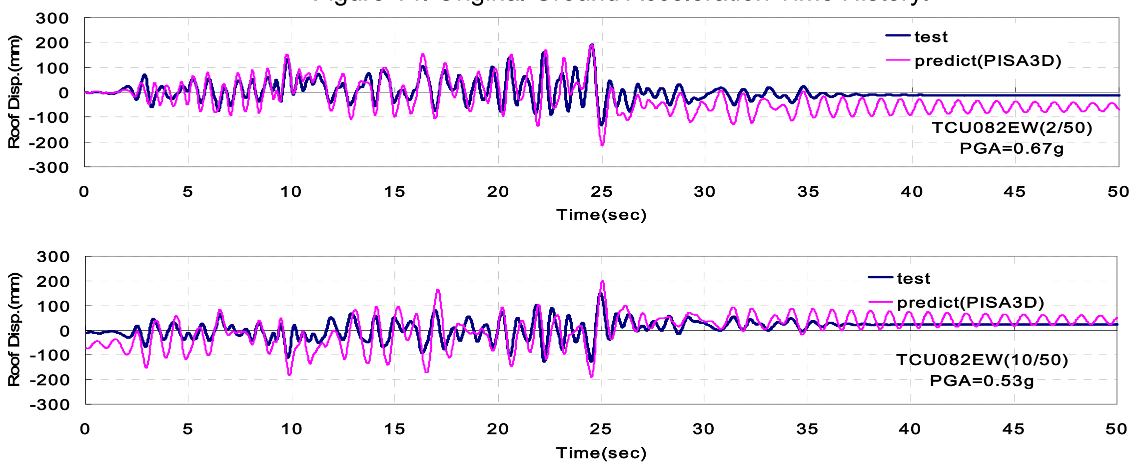


Figure 15. Roof displacement time histories (2/50&10/50).

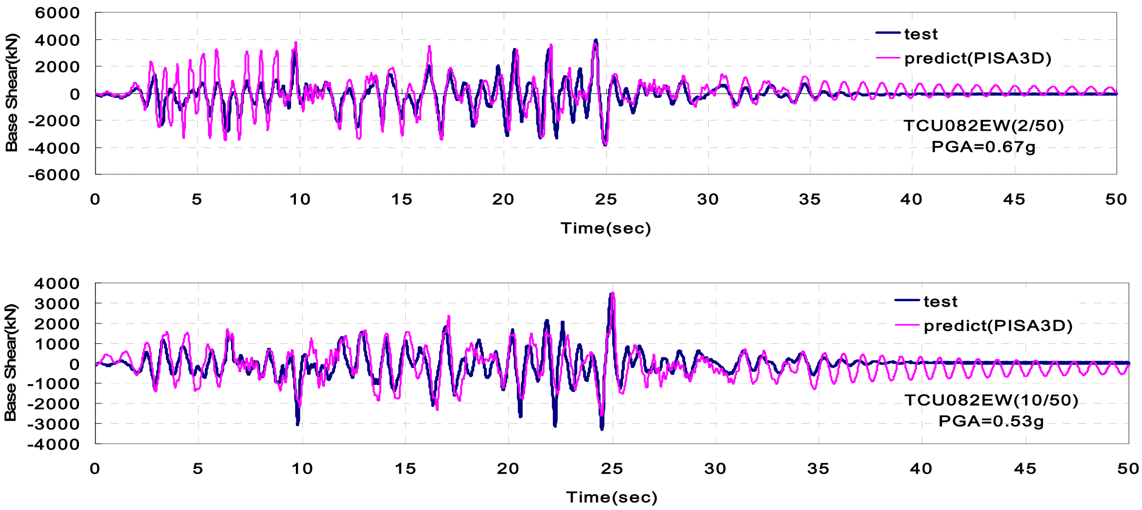


Figure 16. Base Shear time histories (2/50&10/50).

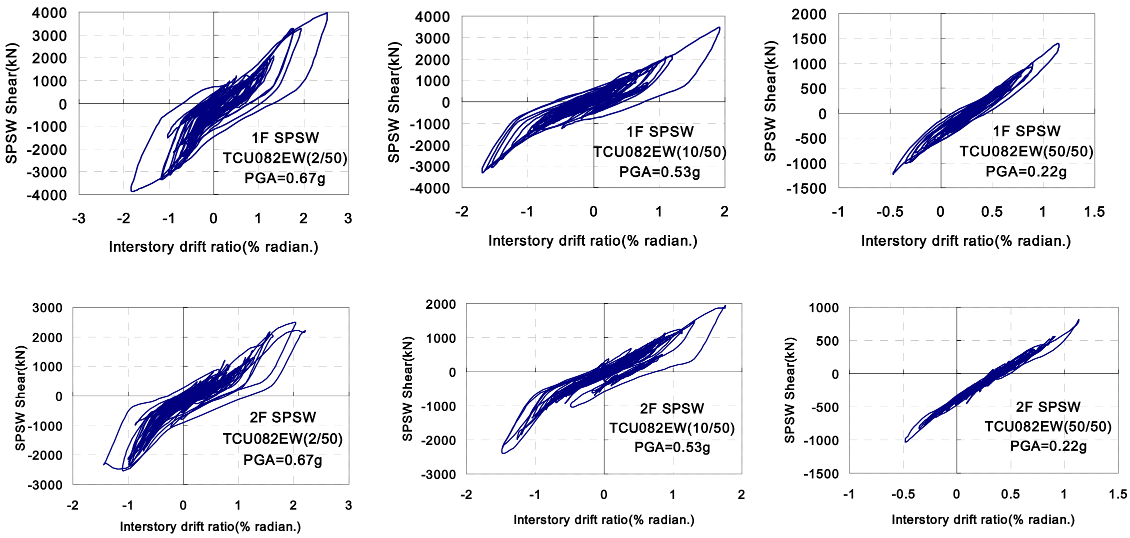


Figure 17. Inter-story drift ratio vs. story shear relationships of three tests.

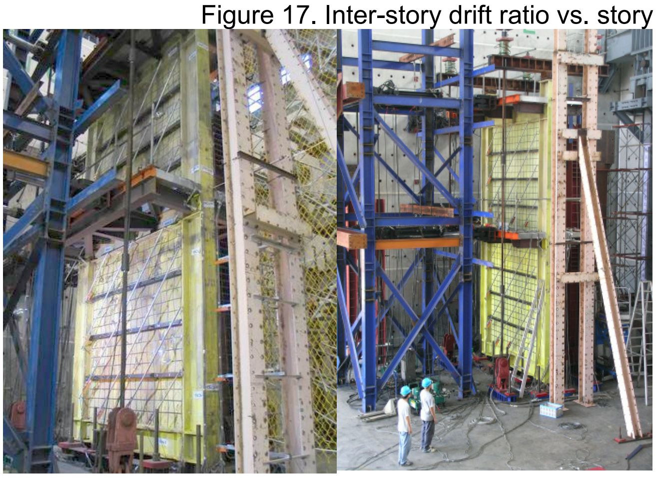


Photo 1. Two-story SPSW specimen.

Photo 2. Steel plate buckling.

Photo 3. Steel plate crack.

Photo 4. Top Beam web yielding.

Photo 5. Top beam web yielding.

Photo 6. Middle beam web yielding.

Photo 7. Column web yielding near the column base.

Photo 8. Column flange yielding.