

Experimental Investigation of Full Scale Two-story Steel Plate Shear Wall

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Summary

In order to address the replaceability of the infill panels following an earthquake and the seismic behavior of intermediate beams, a two-phase experimental program was carried out on a full scale two-story steel plate shear wall with reduced beam section connections and composite floors. In Phase I the specimen was pseudo-dynamically tested subjected to three ground motions of progressively decreasing intensity. In Phase II, the buckled panels were replaced by new panels prior to subjecting the specimen to subsequent pseudo-dynamic and cyclic tests until failure. It is shown that the repaired specimen can survive and dissipate significant amounts of hysteretic energy in a new earthquake without severe damage to the boundary frame or overall strength degradation. It is also found that the specimen had exceptional redundancy and exhibited stable displacement-force behavior for drifts up to 5.2% and 5.0% at the first and second story, respectively.

Introduction

Steel plate shear walls (SPSW) consist of steel panels surrounded by boundary frame members. These panels are allowed to buckle in shear and subsequently form a diagonal tension field. SPSW are progressively being used as the primary lateral force resisting systems in buildings (Sabelli and Bruneau 2006). Past monotonic, cyclic and shaking table tests on SPSW in United States, Canada, Japan, Taiwan and other countries have shown that this structural system can exhibit high initial stiffness, behave in a ductile manner and dissipate significant amounts of hysteretic energy (Literature reviews are available in Berman and Bruneau 2003). Analytical research on SPSW has also validated useful models for the design and analysis of this lateral load resisting system (Thorburn et al.1983; Driver et al. 1997; Berman and Bruneau 2003).

However, no research has directly addressed the replaceability of infill panels following an earthquake, and there remain uncertainties regarding the seismic behavior of intermediate beams in SPSW (intermediate beams are those to which are welded steel plates above and below, by opposition to anchor beams that have steel plates only below or above respectively). The latter problem was analytically addressed by Lopez Garcia and Bruneau (2006) using simple models, but experimental investigations on the behavior of intermediate beams, particularly for beams having reduced beam section (RBS) connections and composite concrete slabs, can provide much needed information on behavior of SPSW system and how to best design the intermediate beams. To address the above issues, a two-phase experimental program was developed to test a two-story SPSW specimen having an intermediate composite beam with RBS connections. The testing

program also investigated how to replace a steel panel after a severe earthquake and how the repaired SPSW would behave in a second earthquake. This paper summarizes the tests conducted and the observed ultimate behavior.

Specimen Description and Test Setup

A full scale two-story one-bay SPSW specimen was fabricated in Taiwan and a two-phase experimental program (Phase I and II tests) was conducted at the laboratory of the National Center for Research in Earthquake Engineering (NCREE) in Taipei, Taiwan. The specimen with equal height and width panels at each story was measured 8000mm high and 4000mm wide between boundary frame member centerlines. The infill panels and boundary frame members were sized based on the recommendations provided by Berman and Bruneau (2003). Beams and columns were of A572 Gr.50 steel members. Infill panels were specified to be SS400 steel which is similar to ASTM A36 steel in this case. The RBS connection design procedure was used to detail the beam-to-column connections at top, intermediate and bottom level respectively. The infill panels were designed to be 3mm and 2mm thick at the first and second story respectively. Prior to Phase II tests, the buckled infill panels were removed using flame-cut and replaced by new panels. Fish plates were used along boundary frame members to connect infill panels. The infill panels of Phase I were welded on one side of the fish plates and those of Phase II on the other side.

The specimen was mounted on the strong floor. In-plane (south-north) servo controlled hydraulic actuators were mounted between the specimen and a reaction wall. Based on the ultimate strength of the specimen assessed using the plastic analysis procedures (Berman and Bruneau 2003), three 1000kN hydraulic actuators were employed to apply earthquake load or cyclic load on the specimen at each story. Ancillary trusses as part of the floor slab system were used to transfer in-plane loads to the specimen at floor levels. In order to avoid out-of-plane (east-west) displacement at floor levels, two hydraulic actuators, mounted between the ancillary truss and a reaction frame, were used at floor levels. A vertical load of 1400kN was applied by a reaction beam at the top of each column to simulate gravity load. Each reaction beam transferred the load exerted from two vertical actuators mounted between the reaction beam and the anchor rods pinned to the strong floor. The specimen schematic and test setup are illustrated in Figures 1a-1c. The designation of H shapes correspond to US designation W shapes reflecting the depth, flange width, as well as web and flange thicknesses.

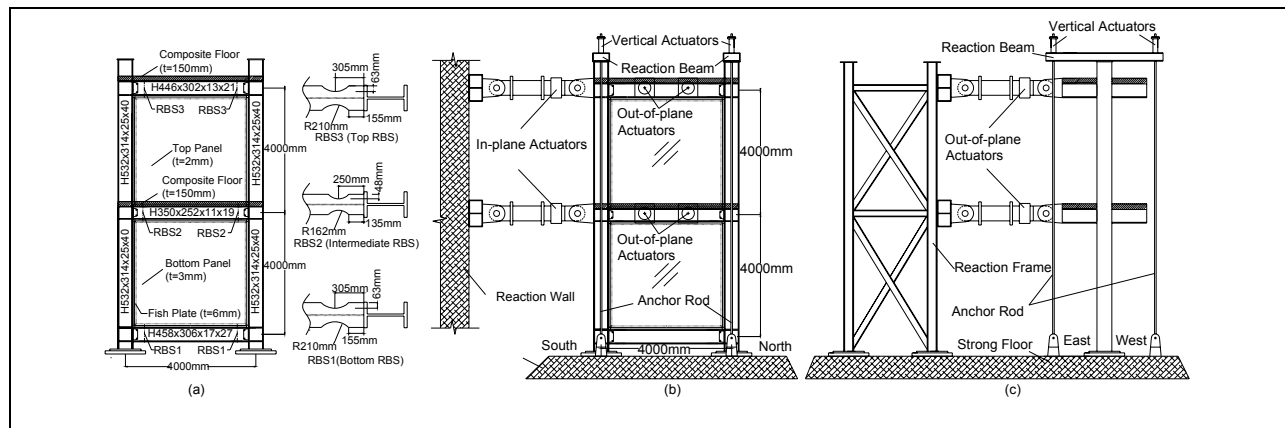


Figure 1: Specimen Schematic (a); In-plane Setup (b); Out-of-Plane Setup (c)

Phase I Tests

In Phase I, the specimen was tested under three pseudo-dynamic loads using the Chi-Chi earthquake record (TCU082EW) scaled up to levels of excitations representative of seismic hazards having 2%, 10% and 50% probabilities of exceedances in 50 years, subjecting the wall to earthquakes of progressively decreasing intensity. Despite the numerous ancillary calculations that checked the adequacy of the specimen, the intermediate concrete slab and the south column base suffered premature failures at the time step of 9.5sec. and 24sec. of the first earthquake record respectively. The tests resumed after the specimen was strengthened at these locations. The SPSW specimen behaved similarly to the Phase II pseudo-dynamic test described in greater length below. No fracture was found in the boundary frame and it was deemed to be in satisfactory condition allowing for the replacement of infill panels for the subsequent phase of testing. Detailed information about specimen design and results from the Phase I tests are presented elsewhere (Lin et al 2006).

Phase II Pseudo-dynamic Test

In order to investigate how the repaired SPSW specimen would behave in a second earthquake in the first stage of Phase II, the specimen was tested under pseudo-dynamic load corresponding to the Chi-Chi earthquake record (TCU082EW) scaled up to the seismic hazard of 2% probability of occurrence in 50 years which was equivalent to the first earthquake record considered in the Phase I tests after the buckled panels were replaced by new panels.

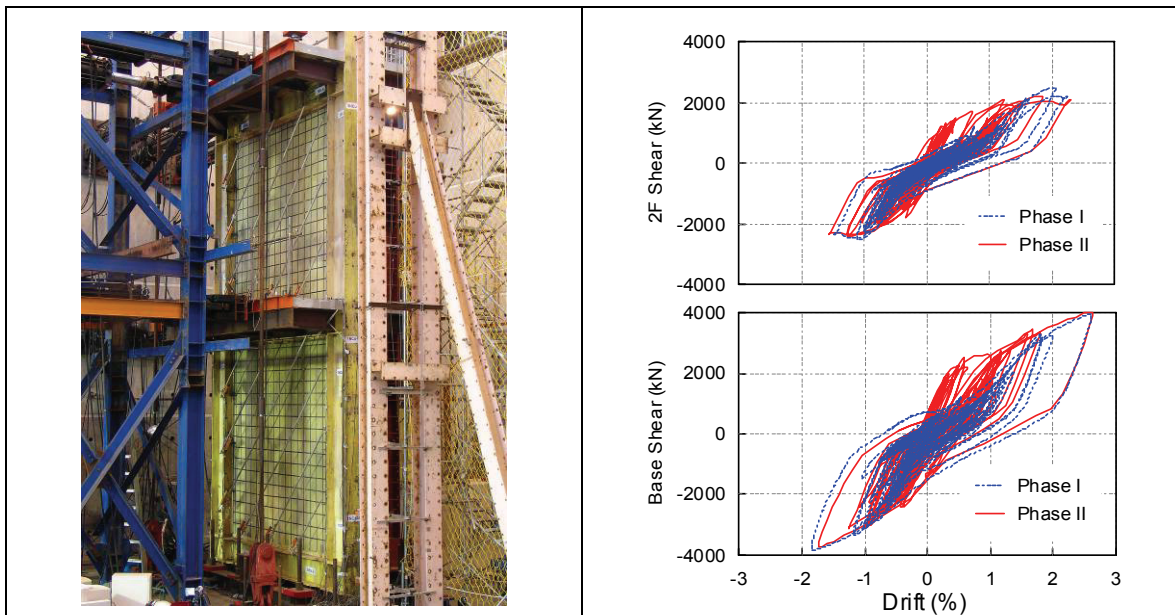


Figure 2a. Specimen prior to Phase II tests

Figure 2b. Hystereses of Phase I and Phase II Pseudo-dynamic test

The specimen prior to Phase II tests and hysteretic curves from Phase II pseudo-dynamic test along with the counterpart results from Phase I are shown in Figure 2a-2b. Drifts designated as “+” or “-” refer to loading in the north and south directions respectively. Observation of the hysteretic curves obtained from the Phase II shows that the first story dissipated more hysteretic energy than the

second story. Both the first and second story exhibited stable displacement-force behavior, with some pinching of the hysteretic loops as the magnitude of drifts increased, particularly after the development of a small fracture along the bottom of the shear tab at the north end of the intermediate beam at drifts of 2.6% and 2.3% at the first and second story respectively. After the pseudo-dynamic test, the boundary frame was in good condition except for the aforementioned damage in the shear tab of the intermediate beam. There were notable plastic deformations at the column bases and RBS connections at all levels. All welds within the SPSW specimen were intact after the test. Comparing the hysteretic curves from the Phase I and Phase II tests shown together in Figure 2b, the two specimens are found to behave similarly under the same strong ground motion except that the initial stiffness of the repaired specimen is higher than that of the original one. This is because the results shown for the specimen in Phase I are those obtained after the specimen was repaired due to the unexpected failures mentioned in Phase I tests. Therefore the infill panels had already experienced some inelastic deformation before these unexpected failures occurred.

Phase II Cyclic Test

The next stage of Phase II tests involved cyclic test on the SPSW specimen in order to investigate the ultimate behavior of intermediate beam and the cyclic behavior and ultimate capacity of SPSW system after severe earthquakes. As mentioned in the observations of Phase II pseudo-dynamic test, the boundary frame members were in good condition after the pseudo-dynamic test except for a small fracture that was found along the bottom of the shear tab at the north end of the intermediate beam. To correct this limited damage and to get a better assessment of the possible ultimate capacity of SPSW, the damaged shear tab was replaced by a new one prior to conducting the cyclic test. A displacement-controlled scheme was selected for the cyclic test.

The hysteretic curves resulting from the Phase II cyclic test, along with the results of Phase II pseudo-dynamic tests described in detail in the prior section, are shown in Figure 3. It is observed the initial stiffness of the SPSW specimen in the cyclic test was smaller than that in pseudo-dynamic test. Because the previous pseudo-dynamic test stretched the infill panels up to specimen drifts of 2.6% and 2.3% at the first and second story respectively, the hysteretic loops exhibited pinching up to those drifts. Hysteretic loops were then full until drifts of 2.8% and 2.6% at the first and second story respectively in Cycle 7, when complete fracture occurred along the shear tab at the north end of the intermediate beam. A similar fracture developed along the shear tab at the south end of the intermediate beam when the specimen was pulled towards to the reaction wall in this cycle.

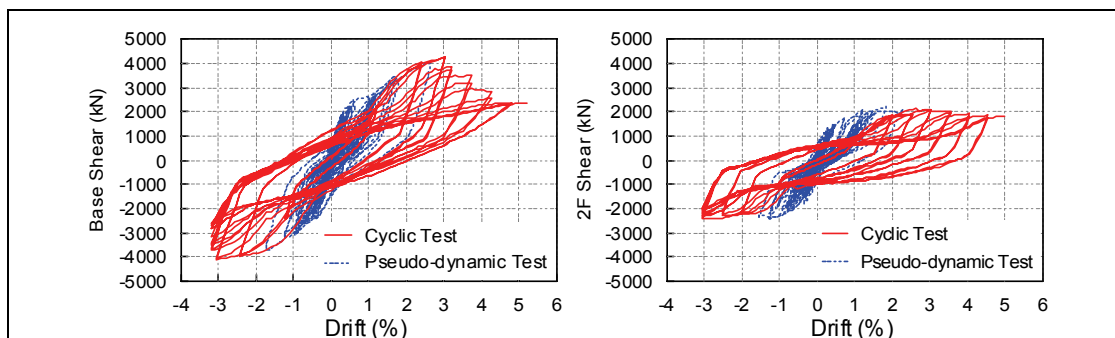


Figure 3. Hystereses of the Phase II tests

Rupture of the shear tabs triggered fracture of the bottom flange at the north end of the intermediate beam. At drifts of 3.3% and 3.1% at the first and second story respectively in Cycle 9, the bottom flange at the north end of the intermediate beam fractured as shown in Figure 4. However, no fractures developed in the reduced beam flange regions of the intermediate beam. The welds connecting the infill panels to the fish plates around the north end of the intermediate beam also fractured over a substantial length to a more severe extent after the specimen experienced drifts of 5.2% and 5.0% at the first and second story respectively as shown in Figure 5. These events significantly changed the load path within the system. However, the SPSW specimen was still able to exhibit stable displacement-force behavior as evidenced by the hysteretic curves shown in Figure 3, which demonstrates the redundancy of this kind of structural system. The cyclic test ended at drifts of 5.2% and 5.0% at the first and second story respectively, when a sudden failure occurred in the load transfer mechanism, i.e. when a fatal longitudinal crack developed along the top concrete slab of the specimen as shown in Figure 6.

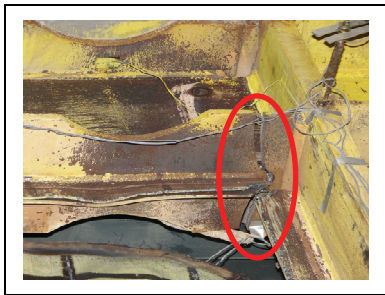


Figure 4. Ruptures at the north end of intermediate beam



Figure 6. Crack at the top slab

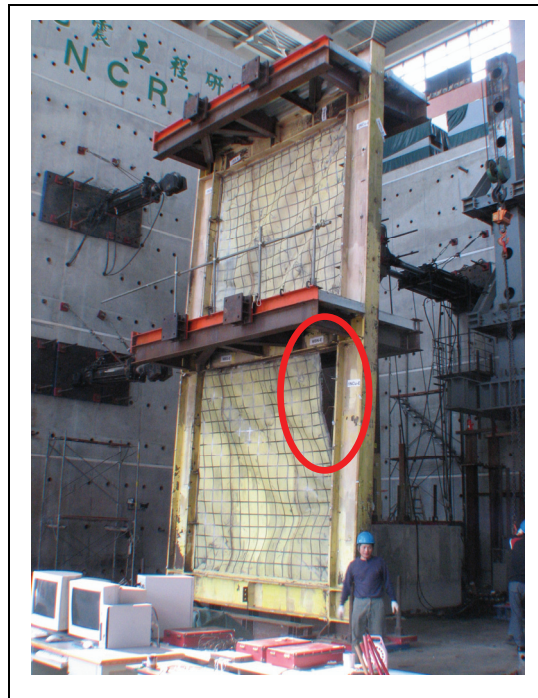


Figure 5. Fracture of the welds connecting the infill panels to fish plates

Concluding Remarks

A full scale two-story SPSW specimen was designed and investigated in a two-phase experimental program. The pseudo-dynamic tests show that a SPSW repaired by replacing the infill panels buckled in a prior earthquake by new ones can be a viable option to provide adequate resistance to the lateral loads imparted on this structure during new seismic excitations (note that possible undesirable aesthetic issues related to the residual drifts prior to replacing the panels are beyond the scope of this work). The repaired SPSW behaved quite similarly to the original one. Testing showed that the repaired SPSW can survive and dissipate a similar amount of energy in the subsequent earthquake without severe damage to the boundary frame and without overall strength degradation.

Results from the cyclic test allowed to investigate the ultimate displacement capacity of the SPSW specimen. Though the hysteretic curves were pinched at low drift levels due to the inelastic deformations that the infill panels experienced during the pseudo-dynamic test, and even though the strength of the SPSW dropped as the ends of the intermediate beam fractured, the SPSW structure exhibited stable displacement-force behavior and provided a significant energy dissipation capacity, exhibiting substantial redundancy.

The columns and anchor beams, as well as top and bottom RBS connections performed as intended. However, the intermediate beams failed unexpectedly. The ends of the intermediate beams having RBS connections ultimately developed fractures in the shear tabs followed by fractures at the end of the bottom beam flange. No fractures developed in the reduced beam flange region. Further investigation is required to clarify the local behavior of intermediate beams in SPSW, to allow developing a better understanding of how such intermediate beams should be designed.

Acknowledgements

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