ANALYTICAL STUDY ON STEEL PLATE SHEAR WALLS USING DUAL STRIP MODEL AND 3D FE MODEL

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ABSTRACT

Steel plate shear walls (SPSW) have been used as the primary lateral force resisting system in buildings. Extensive analytical studies on steel plate shear walls in the United States, Canada, Taiwan and other countries have already validated the simple analytical model known as strip model to predict the monotonic behavior of this kind of structural system. However, more research is still needed to better understand the local behavior of SPSW as it related to global behavior, most notably the behavior of intermediate beam in multi-story SPSW. This paper presents some analyses on the Phase II tests of MCEER/NCREE cooperative experimental program on the full scale two-story SPSW specimen. A dual strip model using tension-only strips was developed using the commercially available finite element soft package ABAQUS/standard to replicate the Phase II pseudo-dynamic test. Beam element (B31) and truss element (T3D2) were used to represent the boundary frames and dual strips respectively. It was found that the simulation results showed good agreement with experimental results. This paper also presents the results of monotonic pushover analysis conducted using 3D finite element model in ABAQUS/Standard. Shell element (S4R) was employed for all structural sub-assemblage in this model. Linear eigenvalue buckling analysis was performed prior to the pushover analysis to introduce initial imperfection. The global structural responses from the analysis were compared with those from the MCEER/NCREE Phase II cyclic test. It was found the lateral capacity of the steel plate shear wall can be accurately estimated by 3D finite element model.

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INTRODUCTION

A steel plate shear wall (SPSW) consists of infill steel panels surrounded by boundary beams and columns. These panels are allowed to buckle in shear and subsequently form a diagonal tension field. SPSWs are progressively being used as the primary lateral force resisting system in buildings (Sabelli and Bruneau 2006). Past monotonic, cyclic and shaking table tests on SPSW in the United States, Canada, Taiwan and other countries have shown that this type of structural system can exhibit high initial stiffness, behave in a ductile manner and dissipate significant amounts of hysteretic energy, which make it a suitable option for the design of new buildings as well as for the retrofit of existing constructions (extensive literature reviews are available in Sabelli and Bruneau 2006 and Berman and Bruneau 2003a, to name a few). Analytical research on SPSW has also validated useful models for the design and analysis of this lateral load resisting system (Thorburn et al.1983; Elgaaly et al.1993; Driver et al. 1997; Berman and Bruneau 2003b). Recent design procedures for SPSW are provided by the CSA Limit States Design of Steel Structures (CSA 2003) and the AISC Seismic Provision for Structural Steel Buildings (AISC 2005). Innovative SPSW designs have also been proposed and experimentally validated to expand the range of applicability of SPSW (Berman and Bruneau 2003a; Vian and Bruneau 2005).

However, some impediments still exist that may limit the widespread acceptance of this structural system. For example, there remain uncertainties regarding the seismic behavior of intermediate beams in SPSW (intermediate beams are all the beams in a continuous SPSW except the top and bottom beams. This differentiation is needed because they are loaded differently by the yielding plates.). This problem was analytically addressed by Lopez Garcia and Bruneau (2006) using simple models, but further investigations on the behavior of intermediate beams, particularly for those beams having reduced beam section (RBS) connections, can provide much needed information on behavior of this structural system and how to best design the intermediate beams.

To address the above issues with regards to SPSW performance, a two-phase experimental program was developed under the cooperative efforts of MCEER and NCREE to test a two-story SPSW specimen having an intermediate composite beam with RBS connections. This paper summarizes the analytical work on the Phase II tests conducted and the adequacy of simple models as well as 3D finite element (FE) model to replicate the global behavior of the SPSW considered.

SPECIMEN DESCRIPTION

A full scale two-story one-bay SPSW specimen was designed and 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 in Taipei, Taiwan.

The specimen, with equal height and width panels at each story, was 8000 mm high (4000 mm at each story) and 4000 mm wide, measured between boundary frame member centerlines. The infill panels and boundary frame members were sized based on the recommendations provided by Berman and Bruneau (2003b). The RBS connection design procedure of the Federal Emergency Management Agency (FEMA) Document, FEMA 350 (FEMA, 2000) was used to detail the beam-to-column connections at top, intermediate and bottom level respectively. This detail was designed to ensure all inelastic beam action would occur at these locations.

In order to investigate the seismic behavior of SPSW in severe earthquake and aftershocks in the Phase I tests, 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 a 2%, 10% and 50% probabilities of exceedances in 50 years, subjecting the wall to earthquakes of progressively decreasing intensity. The infill panels dissipated energy and buckled as anticipated. No fracture was found in the boundary frame, and it was deemed to be in satisfactory condition allowing for the subsequent phase of testing. Detailed information about specimen design and results of the Phase I tests are presented elsewhere (Lin et al 2006 and Lin et al 2007).

Prior to Phase II tests, the buckled infill panels were removed using flame-cut and replaced by new panels at the first and second story respectively. Fish plates were used along the boundary frame members to connect infill panels. The infill panels of Phase I were weld on one side of the fish plates and the new panels installed as part of Phase II were welded on the other side (after Phase I panels were cut-out). The specimen was tested under pseudo-dynamic load and subsequent cyclic load to failure in Phase II. Detailed information about the results of Phase II tests is presented elsewhere (Qu et al 2007).

The test setup is illustrated in Figure 1.The specimen was mounted on the strong floor. In-plane (north-south) servo controlled hydraulic actuators were mounted between the specimen and a reaction wall. Three 1000kN hydraulic actuators were employed to apply earthquake load or cyclic load on the specimen at each story. Ancillary trusses were used to transfer in-plane loads to the specimen at the floor levels. In order to avoid out-of-plane (east-west) displacements of the SPSW at floor levels, two hydraulic actuators were mounted at each floor level between the edge of the floor (ancillary truss) and a reaction frame. A vertical load of 1400 kN was applied at the top of each column to simulate gravity load that would be present in the prototype.

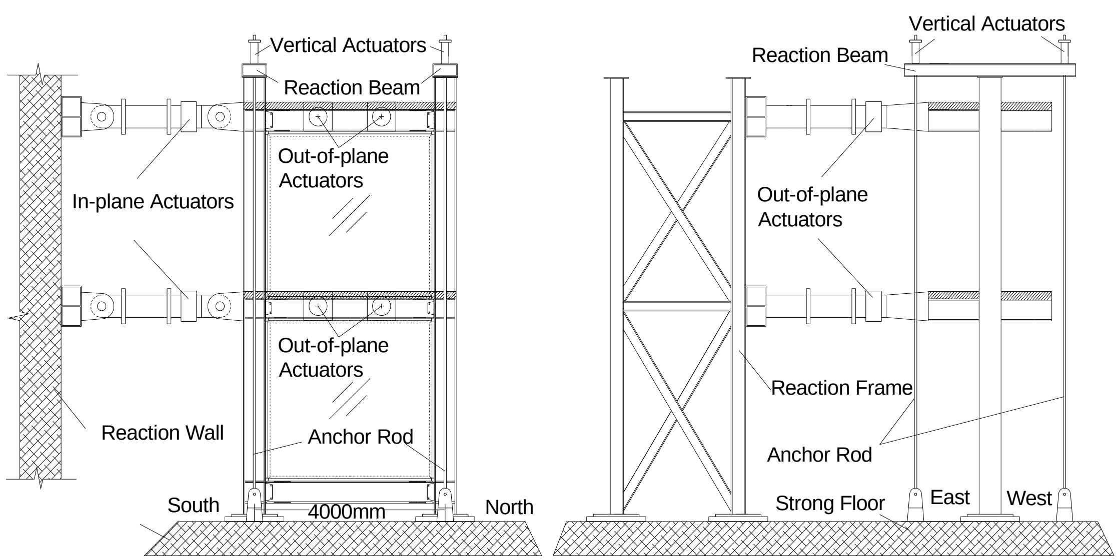


Figure 1. Test setup

SIMULATION OF 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 the pseudo-dynamic loads corresponding to the Chi-Chi earthquake record (TCU082EW) scaled to a seismic hazard having a 2% probability of occurrence in 50 years (i.e. equivalent to the first earthquake record considered in the Phase I tests). This scaled earthquake record had peak ground acceleration (PGA) of 0.63g and the peak pseudo acceleration (PSa) response of 1.85g at the fundamental period of 0.52 sec. The original ground motion record and the drift histories are shown in Figure 2.

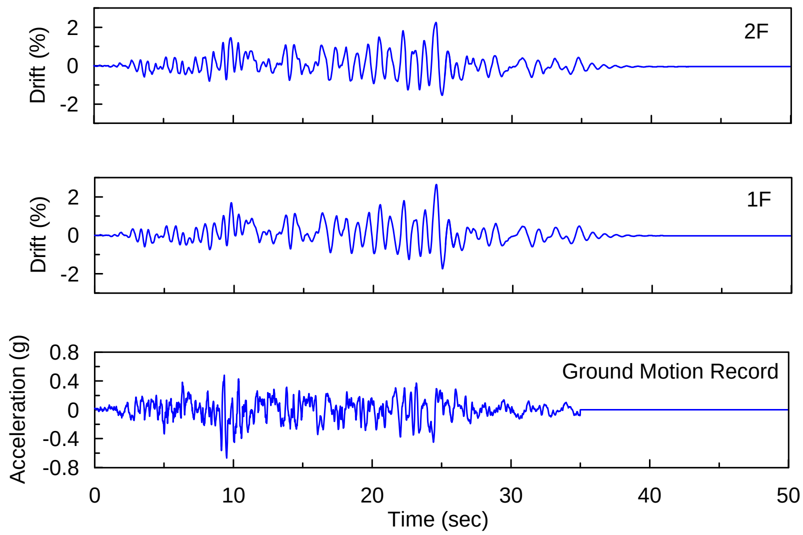


Figure 2. Ground motion record and drift histories.

To check the adequacy of the strip model to predict the nonlinear behavior of SPSW under the Phase II pseudo-dynamic load, a dual strip model using tension-only strips was developed using the commercially available finite element software package ABAQUS/Standard. Thirty strips (15 strips in each direction) were used at each story as shown in Figure. 3(a)

Strip Model

The strip model also known as multi-strip model was firstly proposed by Thorburn et al (1983). In this model, the infill panels can be represented as a series of pin-ended tension members inclined at angle α relative to vertical and having a cross-sectional area equal to the strip spacing times the panel thickness. The inclination angle α , as described above, can be estimated by the procedures provided in the AISC Seismic Provision (2005)

Boundary Conditions

The boundary frame was fixed at column bases to replicate the test conditions. Boundary conditions preventing out-of-plane displacements were imposed at floor levels. Gravity loads were firstly applied at the top of the columns. Then the drift histories obtained from the test, as shown in Figure 2 were used as displacement input at floor levels.

Assumption and Simplification

The arc cutouts of the RBS connections were simplified as rectangular cutoffs for the purpose of this analysis. The length and width of the approximate reduced beam flange using rectangular cutoffs were equal to the length and minimal width of the original reduced beam flange respectively, recognizing that this is a somewhat more severe reduction than the actual RBS used.

To consider the contribution of the concrete slabs to the global behavior of the SPSW specimen, the thickness of the top flange of the intermediate and top beams were increased to provide the same positive plastic section moment capacity as the real composite beam section. Composite action was neglected in negative flexure.

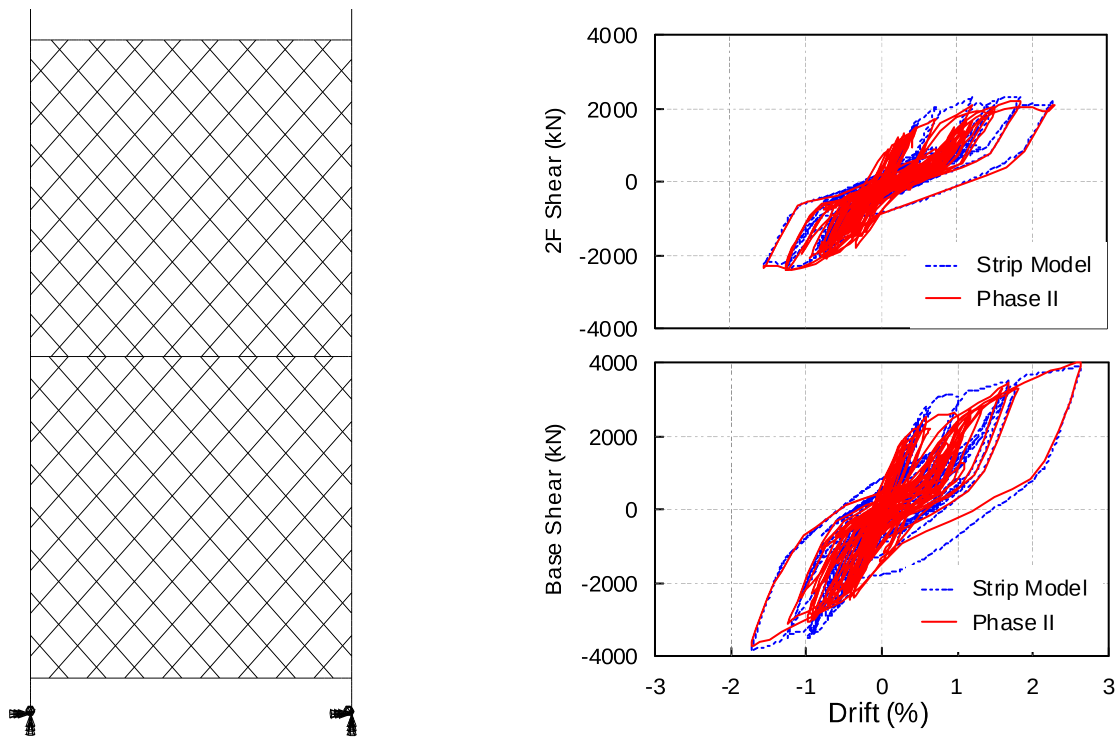
Element

Beam element (ABAQUS element B31) and truss element (ABAQUS element T3D2) were used to represent the boundary frame and dual strips respectively. B31 is a 2-node 3D linear beam element which allows biaxial bending, axial strain and transverse shear deformations. T3D2 is a 2-node 3D linear truss element.

Material

Nominal stress-strain curves for the infill panels as well as the boundary frame members were obtained from standard coupon tests. Steel was modeled as an isotropic material with a simple rate-independent constitutive behavior. Von-Misses yield surface was adopted as the yield criterion for the boundary frame members with identical strengths in tension and compression. Tension-only strength was given to the diagonal strips.

The hysteretic curves obtained from the above dual strip model are plotted on top of those experimentally obtained from the Phase II pseudo-dynamic test in Figure 3(b). It is found that global behavior of the SPSW specimen can be satisfactorily predicted by this dual strip model.



(a) (b)

Figure 3. Analytical model and the simulation results of the Phase II pseudo-dynamic test
(a) Dual strip model; (b) Hystereses from strip model and test

3D FE ANALYSIS OF PHASE II CYCLIC TEST

The second stage of the Phase II tests involved cyclic test of the SPSW specimen to investigate the ultimate capacity of the SPSW.A displacement-controlled scheme was selected for the cyclic test. Because the first mode response dominated the global response of the SPSW in the prior pseudo-dynamic test (although some higher mode effects were observed) and to allow testing both panels even if failure progressively develops at one of the two stories, a displacement constraint was exerted to keep the in-plane actuators displacing in a ratio corresponding to a first mode of response through-out the entire test. 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.

To further assess the global behavior of SPSW specimen, a 3D FE model as shown in Figure 4(a) was developed in ABAQUS/Standard to obtain the responses of the specimen subjected to the Phase II cyclic test.

Vian and Bruneau (2005) demonstrated that although the entire cyclic response of SPSW can be replicated using such finite element models, the monotonic response obtained from a pushover analysis using such a model can adequately capture the global behavior of a SPSW at the peak drifts of a cyclic test - hence only monotonic analysis was conducted here.

Elements

Shell element (ABAQUS element S4R) was employed for all structural sub-assemblages. S4R is a 4-node, quadrilateral shell element with reduced integration and a large-strain formulation. A total of 30,553 elements were used for this model.

Boundary Conditions

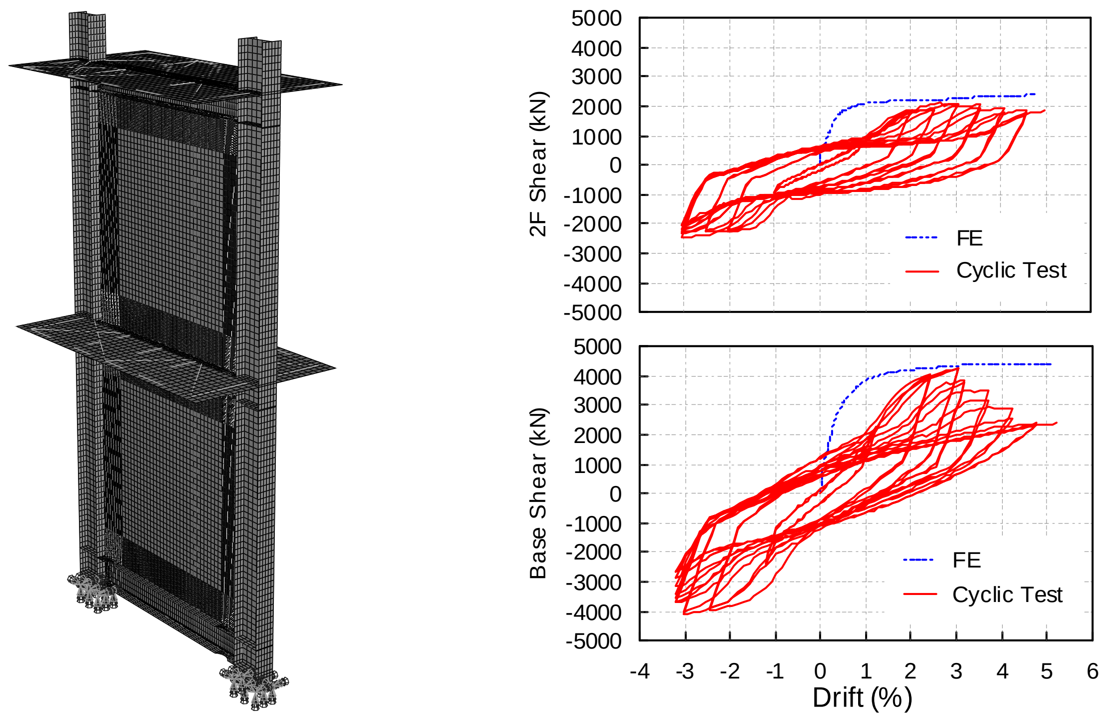
The boundary frame was fixed at column bases. Boundary conditions preventing out-of-plane displacements were used along the intermediate and top concrete slabs respectively. Gravity loads were applied at the top of the columns prior to the in-plane loading. Lateral in-plane displacements were applied at the floor levels in proportion to the same ratio used in the test.

Material Properties

Nominal stress-strain curves for all steel structural sub-assemblages were obtained from coupon test. Steel was modeled as an isotropic material with a simple rate-independent constitutive behavior. Von-Misses yield surface was selected as the yield criterion. In this case, the actual concrete slab was modeled using unconfined concrete model and the compressive strength measured from cylinder tests.

Solution Strategy

Linear eigenvalue buckling analysis was performed prior to the monotonic pushover analysis to introduce initial imperfections in the panel, and ensure reliable modeling of their buckling. The global structural response from this finite element analysis was compared with the experimental results from the cyclic test, as shown in Figure 4(b).



(a) (b)

Figure 4. Analytical model and the results of the Phase II cyclic test
(a) 3D FE model; (b) Hystereses from test and pushover results of 3D FE model

It is observed that the story shears from the FE analysis are greater than those obtained from the cyclic test prior to 2.6% and 2.3% drifts at the first and second story respectively. This is principally because the specimen was loaded into the inelastic range in the prior Phase II pseudo-dynamic test, resulting in the partial absence of tension field in the infill panel at low drift levels. However, the story shears obtained from FE analysis fit well with those obtained from the cyclic test at drifts exceeding the maximum drifts of 2.6% and 2.3% at the first and second story respectively reached in the Phase II pseudo-dynamic test. After drifts of 3% and 2.5% at the first and second story respectively, the story shears from cyclic tests are smaller than those from FE analysis due to the failures in intermediate beam.(Qu et al 2007)

CONCLUSION

To better understand the behavior of SPSW, a series of analyses either using simple strip model or FE model were developed to analyze the Phase II tests of the MCEER/NCREE experimental program on full scale two-story SPSW. The adequacy of the dual strip model using tension-only strips was found accurate to predict the nonlinear behavior of SPSW under earthquake load, as demonstrated by the experimental results of the Phase II pseudo-dynamic test. The ultimate lateral in-plane load capacity of SPSW was shown to be equally well predicted by a monotonic pushover analysis using a 3D FE model with shell elements, when comparing to the experimental results of the Phase II cyclic test.

ACKNOWLEDGEMENT

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