Cyclic Test of a Coupled Steel Plate Shear Wall Substructure

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Abstract

This research aimed to investigate the seismic behavior and design of the Coupled Steel Plate Shear Wall (C-SPSW). A prototype six-story C-SPSW building was designed based on the model building code. A 40% scale specimen was constructed as the bottom two-and-half-story substructure of the six-story C-SPSW prototype. The reduced scale sub-structural specimen was cyclically tested using the Multi-Axial Testing System (MATS) at the National Center for Research on Earthquake Engineering (NCREE). In addition to the cyclic lateral forces, the constant vertical loading and cyclic overturning moments were applied on the specimen simultaneously. The test results show that the C-SPSW specimen behaved in a ductile manner and dissipated significant amounts of hysteresis energy during the cyclic loadings. Finally, based on the experimental results, the implications in the capacity design for the bottom boundary column are discussed. A numerical simulation using finite element model was conducted. The analytical results quite match the overall and local experimental responses.

Keywords: Steel Plate Shear Wall, Coupled Steel Plate Shear Wall, Coupling Beam

1. INTRODUCTION

Steel plate shear wall (SPSW) has seen increased usage in North America and Asia in recent years. An SPSW is composed of a structural frame and infill steel plates. The beams and columns surrounding the infill plates are named boundary beams and boundary columns, respectively. SPSW can effectively resist horizontal earthquake forces by allowing the development of diagonal tension field action after the infill plate buckles in shear. The input energy is then dissipated through the cyclic yielding of the infill plates in tension. However, infilling the steel plate into the structural frame would conflict with the architectural...
demand for the doorways. In a tall and slender SPSW, the significant overturning moments will result in high axial forces in the boundary columns. Moreover, the tall SPSWs likely behave in the flexure-dominated deformation mode under the lateral forces. It would cause the steel plates in the top stories of the wall to fail in developing the plastic tension field action. Thus, this research proposed Coupled Steel Plate Shear Wall (C-SPSW) as a solution to the application of SPSW in the high-rise buildings.

As shown in Fig. 1, a C-SPSW consists of two or more SPSWs in series coupled by the coupling beams between the walls at the story levels. The coupling beams restrain the individual cantilever action of each wall by forcing the system to work as composite section. The total stiffness of a C-SPSW exceeds the summation of its individual wall stiffness.

Figure 1: Plastic Mechanism of C-SPSW  
Figure 2: Plastic Mechanism of C-SPSW

2. BEHAVIORS OF C-SPSW

For the Reinforced Concrete (RC) Coupled Shear Wall, which is commonly seen in practice, the desired plastic mechanism consists of flexural yielding in the coupling beams and plastic hinge at the base of the wall. However, when a C-SPSW develops plastic mechanism under lateral forces, as shown in Fig. 1, the infill plates at all stories develop the plastic tension field and the plastic hinges forms on the coupling beams, boundary beams and the column bases. The difference in plastic mechanism between C-SPSW and RC coupled shear wall prevents the extension of the research results for concrete systems to the steel systems. By extending the provisions in the current U.S. building code (AISC, 2005a) for the link beam of the eccentrically braced frame (EBF) to the prediction for the yielding mechanism of the coupling beam, the coupling beam will yield in shear when its length, \( e \), is smaller than \( 1.6 \frac{M_p}{V_p} \), where \( M_p \) and \( V_p \) are the plastic flexural strength and plastic shear strength of the coupling beam, respectively; and the coupling beam will develop flexural plastic hinges at its both ends when \( e > 2.6 \frac{M_p}{V_p} \).

As shown in Fig. 2, when a C-SPSW is subjected to the lateral forces, the ultimate axial forces in the outer columns come from the plastic panel forces and the shears on the boundary beams. For the inner columns, the axial forces due to the coupling beams are opposite to those from the steel plates and boundary beams. Thus, the ultimate axial forces in the inner columns will be less than those in the outer columns.
3. SPECIMEN DESIGN

A prototype 6-story C-SPSW building was designed based on the model U.S. building code (AISC, 2005a). The site is located at the east zone of the Chiayi City in Taiwan. The analytical strip model (Thorburn et al., 1983) of the C-SPSW was constructed to check that the whole structure remains elastic under the code prescribed LRFD load combinations (AISC 2005b). The coupling beams were designed to be shear links. The member size of the boundary columns at the first story was chosen based on the capacity design proposed in this research. The designs of the remaining columns in the other stories were governed by the Strong-column/Weak-beam principle. The objective of the proposed capacity design for the bottom column is to limit the plastic hinge on the compressed bottom column to form within the lowest quarter column height.

Using the 6-story C-SPSW prototype as a basis, a 40% scale specimen was constructed as the bottom 10.5 m high substructure of the original C-SPSW. The region of the substructure contains the lowest two and half stories of the original structure (shown in Fig. 3). The plates were 3.5 mm thick low yield strength (LYS) steel (measured yield stress $F_y = 220$ MPa). All the boundary elements and coupling beams were made of A572 Grade 50 steel. The design results of the test specimen are shown in Fig. 4. The Reduced Beam Section (RBS) detailing was employed at the ends of the boundary beams. The infill plates were welded at the edges to the boundary elements using 7 mm thick fishplate connection details.

![Figure 3: Schematic of the test setup](image)

![Figure 4: Schematic of the test specimen](image)

4. EXPERIMENTAL PROGRAM

In order to simulate the effects of the upper substructure acting on the bottom two-and-half story substructure, as illustrated in Fig. 3, the specimen was subjected to the cyclic lateral forces, $F_{lh}$, the cyclic overturning moments, $M_{OT}$, and a constant 1400 kN vertical force, $P_V$, which represents the gravity load effect. The specimen was tested using the Multi-Axial Testing System (MATS) at the National Center for Research on Earthquake Engineering (NCREE). The specimen was set upside down in the MATS. The base beam of the specimen was mounted on the cross beam of MATS. The top boundary of each SPSW was connected with a transfer beam. The column top ends were pin-connected with the transfer beam ends; and the top steel plate was welded to the transfer beam using the fishplate connection. The mid-span of the transfer beam was pin-supported on the platen. A lateral support system was constructed on the
platen and the reaction frame (A-frame) of MATS. The lateral supports were provided at each beam-to-column joint. Thus, the unbraced length of the columns was equal to the story height.

The actuator system applied forces on the platen. Two horizontal actuators were employed to apply the lateral displacement on the specimen. Two cycles of 0.1%, 0.2%, 0.3%, 0.5%, 0.75%, 1.0%, 1.5%, 2.0%, 3.0%, 4.0% and 5.0% radian roof drifts were imposed sequentially on the specimen. The vertical actuators can be classified into 3 categories: (1) two rows of vertical pancake type actuators pushing the bottom of the platen. The difference in the applied forces between the two rows of actuators induced the overturning moment effects on the platen; (2) two built-in hold down actuators which were mounted on the A-frame of MATS and pushing on the top of the platen; and (3) two additional actuators which were anchored between the platen and the cross beam. The two additional actuators provided a constant 1960 kN vertical compressive force on the platen. It should be noticed that the pancake type actuators must be always in contact with the platen in compression. The hold down actuators and additional actuators provided vertical compressive forces on the platen to insure the six pancake type actuators were always in compression.

The resultant forces of these vertical actuators applied a constant vertical force \( P_V = 1400 \text{ kN} \). The relationship between the resultant moment \( M_{OT} \) of these vertical actuators and the lateral force \( F_H \) applied by horizontal actuators is: \( M_{OT} = F_H \times (2.51 \text{ m}) \). The relationship is calculated based on the assumption that the inverted triangle vertical distribution of the lateral forces acting on the original 6-story building is constant during the cyclic loading. The vertical distribution of the lateral forces was determined from the recommendations of the current Taiwan’s seismic code.

5. TEST RESULTS AND LEARNED DESIGN IMPLICATIONS

5.1. Force versus displacement relationship

Figure 5 illustrates the force versus displacement relationship of the specimen. The C-SPSW specimen exhibited an excellently ductile behavior and dissipated significant amount of hysteresis energy during the cyclic loading test. The lateral and vertical load-carrying capacities did not notably deteriorate when the overall drift of the specimen reached 5% rad.

After the test, as shown in Fig 6, the flaking of the whitewashes on the specimen showed that the specimen had developed the plastic mechanism. The plastic flexure hinges formed at the ends of the boundary beams (Fig. 6(c)) and the column bases (Figs. 6(d), 6(e) and 6(f)). The coupling beams developed plastic shear hinges (Fig. 6(c)).

![Figure 5: Experimental and analytical force versus displacement relationship](image-url)
5.2. Design implications on the bottom column

Figure 6(d) shows the flaking of the whitewashes on the bottom (first story) columns of the northern SPSW after the test. As shown in Fig. 6(e), the plastic zone on the outer bottom column spread over a range from 40 to 100 mm measured from the column base. On the other hand, the plastic zone on the inner bottom column concentrated at the column base (Fig. 6(f)). The plastic zone on the outer column was wider than that on the inner column. In addition, the plastic zone on the outer column was located at a higher position than that on the inner column. The difference in the distribution of the plastic zone between the outer and inner columns could be attributed to that the axial force in the inner column was smaller than that in the outer column because of the coupling beam effect illustrated in Fig. 2. Hence, the reduced flexural strength of the outer column was smaller than that of the inner column.
Figure 7: Residual relative deflections of the bottom columns in the northern SPSW in the various drift levels

Figure 8: Shear deformation of the second floor coupling beam versus the first story drift relationship

Figure 7 shows the relative deflections of the first story columns in northern SPSW when the lateral forces approached zero during the first cycle of the various drift levels. As shown in the top right of Fig. 7, the relative deflection is defined as the difference between the absolute deflection and the reference line, which is the straight line from the top end to the bottom end of the column. The relative deflection can be utilized to estimate the inward flexural deformation in the column induced by the horizontal tension field forces of the infill plate. The column deflections measured when the lateral forces approaches zero in the various drift levels could represent the residual deflection after the specimen has experienced such level of deformation.

As shown in Fig. 7, it can be found that residual “pull-in” deformations on the outer column were much larger than those on the inner column. This should be attributed to that the axial forces in the outer column were much higher, thus, the flexural strength of the outer column is smaller due to the axial-flexural interaction effect. However, form the Fig. 7, it can be found that the maximum residual deflection of the first story outer column was about 10 mm (= h/200) after the specimen had suffered a 2.1% rad. first story drift). The 1/200 of the story height could serve as the deflection index limiting the development of large secondary forces in the columns (Ellingwood, 2003). Moreover, the past test (Tsai et al., 2006) has shown that peak story drift a well-designed SPSW specimen during the collapse prevention level (2/50 hazard level) earthquake was about 2.0 to 2.5% rad. The test result suggests that, even if the plastic zones on the bottom columns had spread over the bottom half height of the column, the residual “pull-in” column deflection would not cause a significant secondary effect.

5.3. Design implications on the coupling beam

Figure 8 shows the shear deformation of the second floor (2F) coupling beam versus the first story drift relationship. The slope of the line composed of the data the various peak drifts had a sudden change as the story drift reached about 0.75% rad. It indicates that the plastic shear hinge developed at this drift level. From the data at the peak drifts in the various drift levels after the shear yielding, it can be found that the shear deformation of the 2F coupling beam is closed to the story drift. It suggests that the rotational demand of the coupling beam at the lowest level of a C-SPSW can be estimated as the design story drift. The design of the stiffeners in the coupling beam could be based on the estimated rotational demand.
6. NUMERICAL SIMULATION

In this research, a finite element (FE) model for the C-SPSW specimen was constructed using the commercially available finite element software package ABAQUS/Standard. As shown in Fig. 9(a), the majority of this FE model, including the infill plates and the boundary elements, was constructed using the 4-node, quadrilateral, stress/displacement shell elements with the reduced integration and a large-strain formulation (ABAQUS S4R Element). Beam elements were employed to represent the transfer beams. Special constraints were set to represent the pin-connections between the transfer beams and the boundary columns. The platen in the MATS was modeled as a rigid beam pin-connected with the beam elements representing the transfer beam. The vertical load, lateral force and the overturning moment were applied on the rigid beam. Force-controlled pushover analysis was conducted on the FE model. In order to analyze that a thin plate buckles in shear under a very low lateral load, initial imperfections were specified on the infill panels in the FE models. The initial imperfections were determined from the mode shapes obtained from a buckle analysis prior to the pushover analysis.

Figure 5 shows that the analytical pushover results agree well with the experimental hysteresis loop. In addition, as shown in the right of Fig. 5, the FE model can decently predict the individual story drift (1F drift or 2F drift) versus overall drift relationship. As shown in Fig. 9(b) the plastic zone on the FE model near a 5% rad. overall drift was similar to the pattern of the flaking of whitewashes on the specimen after the test (shown in Fig. 6). Figure 8 shows that the shear deformation on the 2F coupling beam obtained from the FE analysis quite match the test results.

7. CONCLUSIONS

The test results show that the C-SPSW specimen behaved well in the cyclic test. The limited residual pull-in deflections of the boundary columns on which the plastic zone spread over the bottom half height
would not cause the significant secondary effects. This suggests that the proposed column capacity design, which aims at limiting the plastic hinge on the column to form below the bottom quarter column height, could be a choice of design in the practice. The test results suggest that the rotational demand of the coupling beam at the lowest level of a C-SPSW can be estimated as the design story drift. Finite element analysis can predict the overall and local responses of the specimen very well. Further studies on the seismic design and behaviours of the C-SPSW can be conducted using the finite element analysis.

REFERENCES