Development Research of the Double “L” Shaped Steel Plate-Concrete Composite Shear Wall

Wenhui He1∗, Yikun Wan1, Yuyu Li2 and Lin Chen3

1 State Key Laboratory of Subtropical Building Science, South China University of Technology, Guangzhou, 510640, China
2 College of Water Conservancy and Civil Engineering, South China Agricultural University, Guangzhou, 510640, China
3 Guangdong Jianke Architectural Design Institute of Guangdong Province Co., Ltd, Guangzhou, 510500, China.
Email: ctwhhe@scut.edu.cn

Abstract: A double steel plates composite shear wall member composed of “L” elements (D-L-SCW) was proposed in this paper and its seismic behavior was preliminarily studied. With the shear-span ratio as test parameter, two specimens with actual ratio of 0.5 were constructed and tested in horizontal low-cycle reverse loading test. The failure modes, hysteretic behavior, skeleton curve, ductility and energy dissipation of the specimens were studied. Results showed that the suggested composite shear wall has significant seismic performance and can make more full use of the properties of both materials. The shear-span ratio mainly affected the peak load of the specimens, whilst it didn't affect the overall deformation trend.

Keywords. Double “L” shaped steel plate, development research, composite shear wall, horizontal low cycle reverse test.

1. Introduction

The double steel plates composite shear wall developed rapidly in recent years. This kind of shear wall mainly consists of steel plates and in-filled concrete. It can fully combine the characteristics of the two materials. In-filled concrete can effectively inhibit buckling of the steel plates, while steel plates can restrain lateral deformation of the concrete and enhance the compressive strength of the concrete [1].

Various experts and scholars have carried out a lot of studies on the double steel plate composite shear wall. Nie Jianguo et al. carried out a series of horizontal horizontal low cycle reverse tests on composite shear walls with prefix plates [2, 3]. The results showed that the setting of prefix plates restrained the in-filled concrete and made the shear walls have better ductility and bearing capacity. Xu Man et al. study the seismic performance of the composite shear walls connected with steel plates with experiment methods, the results showed that seismic behaviors of the wall is fine, and the steel plates connected on sides could effectively eliminate the additional shear action on the edge members [4]. Zhang Wenyuan et al. carried out experimental studies on the a type of composite shear wall, composed of a pair of steel plates, vertical strengthening steel plates, which connected the side plates and in-filled concrete, the results showed that the wall have benign seismic behaviors [5]. The research results of Link and Emori showed that the composite shear wall with integral inner bulkhead could maintain the stable late bearing capacity until large deformation was achieved after the peak load, which proved that the shear wall had better energy dissipation capacity and ductility [6, 7].
Clubley proposed a novel type of composite shear wall with Bi-Steel, which were formed by steel plates connected with shear connecting rods between plates and in-filled concrete. The results showed that the wall had qualified seismic behaviors [8, 9].

However, most of the structures of double steel plates composite shear wall at present were complicated and difficult to assemble. Based on these, D-L-SCW with simple structure, flexible arrangement and assembly was proposed in this paper, as shown in figure 1.

![Figure 1. Geometry and construction details of specimens (All dimensions in mm).](image)

2. Experiment Overview

2.1. Specimen Design

Two specimens with the actual ratio of 0.5 were designed, including the specimen PLW1 with the wall width of 723 mm and the specimen PLW2 with the wall width of 843 mm. The height and the thickness of both specimens were 1850 mm and 65 mm, respectively. The steel plate thickness was 3 mm. The Chinese standard Q235B grade steel with a nominal yield strength $f_y$ of 235 MPa was used, and the nominal compressive strength $f_c$ of the in-filled concrete was 40 MPa. The axial compression ratio of shear wall was calculated according to equation (1), and the shear span ratio adopted the ratio of height to thickness. The specific structural details are shown in figure 1.

$$n = \frac{1.25N}{f_y A_y / 1.4 + f_c A_c / 1.1}$$

2.2. Material Properties

The steel in this test is Q235B, and the mechanical test of material properties was carried out according to Metallic Materials Part 1: Test Method at Room Temperature (GB / t228.1-2010) [10]. The results of material property test are shown in table 1.

The design strength grade of concrete is 40 MPa. Three $150\times150\times150$ mm cube were made, which were maintained in the same environment with the corresponding specimens. The mechanical test of material properties was carried out for the blocks, and the compressive strength of the three specimens were 48.2 MPa, 47.4 MPa and 45.6 MPa, respectively. The average value was 47.1 MPa.

![Table 1. Mechanical properties of steel.](image)
2.3. Loading Device and Loading System
In the test, the horizontal displacement was applied by the MTS electro-hydraulic servo actuator loading device with the stroke of 1000 KN and ±250 mm, and the vertical force was applied by 2000 kN hydraulic jack. MTS actuator was connected to a rigid reaction wall with high strength bolts. A rigid reinforced base with a height of 350 mm was constructed at the bottom of the specimen, which is settled by bolts and ground beams on both sides to ensure that there will be no slide. The reinforced concrete loading beam was connected with the hydraulic servo actuator through the end plate and high-strength bolts. The loading beam acted as a rigid distribution beam to evenly distribute the load to the structure. The loading device can be seen in figure 2.

![Test photo](image1)
![Test schematic](image2)

**Figure 2.** Test setup.

During the experiment, the axial load corresponding to the axial compression ratio was applied firstly, and then the horizontal bidirectional reciprocating loading was applied using displacement control. The stability of axial force was maintained by using electric oil pump and pressure reducing valve connected to hydraulic jack through the whole experiment. The loading displacement increment of each stage was 4.5 mm, and three loading cycles are implemented for each stage displacement. The loading was stopped when the specimen had been damaged to be unsuitable for further loading or when the load had declined below 0.85 of the peak load.

3. Test Results and Analysis

3.1. Experimental Phenomena
At the early stage of the test, there was no obvious deformation of PLW1 (figure 3). After the yield load, local buckling happened at the bottom of both sides of the wall. Thereafter, with the loading continuing, the development of local buckling took the distance between two ribs as half wavelength. As the concrete was crushed, the buckling part of steel plate developed more and more plump. Finally, the steel plate ruptured, exposing the crushed concrete, and then the experiment was terminated corresponding to a drift ratio of 2.7% for safety consideration.

![Half wave buckling occurred at bottom of the wall and the steel plate was torn](image3)
![Plump buckling](image4)

**Figure 3.** Failure patterns of PLW1.
After the displacement reached $2\Delta y$ (yield displacement), the local buckling of both sides of the PLW2 (figure 4) began. With the increase of the displacement, when the compression zone was transformed into the tension zone, the steel plate at the buckling zone was straightened, and the new compression side also gradually produced local buckling. After the peak load, the steel plate at the bottom began to be torn, the in-filled concrete was crushed. When the load was down to 0.85 of the peak load, corresponding to the same drift ratio as PLW1, the test was ceased.

(a) Local buckling occurred in concrete was crushed. (b) The steel plate was torn and the many parts of the wall.

Figure 4. Failure patterns of PLW2.

3.2. Hysteresis Curve
The hysteretic curves are shown in figure 5. At the inception period of experiment, the specimens were in the elastic phase, both materials were under common force, and the hysteretic curves were in a full shuttle shape. With the increase of displacement, the hysteretic curve gradually developed into an inverse “S” shape, which was due to the development of the local buckling.

(a) PLW1  (b) PLW2

Figure 5. Hysteresis curves of specimens.

The development trend of hysteretic curves of the two specimens is similar, but the specimen PLW1 with larger shear span ratio has a more plump shape, which indicates that when the axial compression ratio is 0.3, the larger shear span ratio shows a positive impact on the energy dissipation capacity, and the materials can be brought into more full use.

3.3. Skeleton Curve
Figure 6 shows the details of skeleton curves. The shapes of the curves are inverted “S”, which indicates that the specimens have gone through three phases of elastic, elastic-plastic and failure degradation. The failure of the walls is ductile failure with high safety reserve. The overall development trend of the skeleton curves of the two specimens is relatively close. However, the peak load of specimen PLW2 is 38.78% higher than that of PLW1, and the displacement corresponding to the peak load is small. This is due to and the wider wall and the smaller shear-span ratio, which proves
that the shear-span ratio have no effect on the deformation development obviously, but mainly affects the peak load.

![Figure 6](image_url)

**Figure 6.** Skeleton curves of specimens.

### 3.4. Deformation and Ductility Analysis

Ductility reflects the inelastic deformation capacity of the structure and is expressed by ductility coefficient $\mu$. Because the yield time of different parts are different, the nominal yield stress [11] is used as the yield stress in this paper. With the load falling to 0.85 of the peak load, the shear wall can be considered damaged.

Table 2 shows that the limit displacement is between 31.17 cm and 33.06 cm, that is, the drift ratios is between 1/48 and 1/45, which is much larger than the 1/120 the ultimate drift ratios limit of shear wall composite structure in the Code for Seismic Design of Buildings [12], and the ductility coefficient of shear wall is more than 3. It shows that the shear wall structure has satisfactory deformation ability.

| Specimens | Load direction | $P_y$ ($kN$) | $\Delta_y$ (mm) | $P_{max}$ ($kN$) | $\Delta_{max}$ (mm) | $P_u$ ($kN$) | $\Delta_u$ (mm) | $\mu = \Delta_u / \Delta_y$ |
|-----------|----------------|--------------|-----------------|------------------|-------------------|--------------|-----------------|-----------------|
| PLW1      | +              | 331.57       | 7.95            | 421.51           | 18.38             | 358.28       | 31.17           | 3.92            |
|          | -              | 337.14       | 10.31           | 402.14           | 23.02             | 341.82       | 33.06           | 3.21            |
| average   |                | 334.36       | 9.13            | 411.83           | 20.93             | 350.05       | 32.12           | 3.57            |
| PLW2      | +              | 412.91       | 6.81            | 570.22           | 18.18             | 484.69       | 31.49           | 4.62            |
|          | -              | 424.49       | 7.07            | 573.05           | 20.57             | 487.09       | 31.96           | 4.52            |
| average   |                | 418.70       | 6.94            | 571.54           | 19.38             | 485.89       | 31.73           | 4.57            |

### 3.5. Energy Consumption

The energy consumed by the specimen can be expressed by the area contained in the hysteretic curve, and the relative dissipation of energy can be expressed by the equivalent damping coefficient. The calculations are as follows (figure 7):

![Figure 7](image_url)

**Figure 7.** Energy dissipation.
\[ E = S_{ADC} + S_{MCB} \] (2)
\[ \zeta = \frac{S_{\Delta ADC} + S_{\Delta ACB}}{2\pi(S_{\Delta OFD} + S_{\Delta MCB})} \] (3)

The results are shown in figure 8. We can see from figure 8(a) that PLW1 consumes more energy than PLW2, which is due to the wider wall. In figure 8(b), the equivalent damping coefficient of PLW1 and PLW2 is basically equal in the early stage of the test, which is due to the elastic stage of the specimen, but in the middle and late stage of the test, PLW1 has larger equivalent damping coefficient than that of the PLW2.

(a) Energy-displacement curve  
(b) Equivalent damping coefficient-displacement curve

Figure 8. Energy dissipation.

4. Conclusion
In this paper, D-L-SCW was proposed and two shear wall specimens with different shear span ratios were tested in horizontal low-cycle reverse loading tests. Conclusions were drawn as follows:

(1) The hysteresis curve of D-L-SCW is plump, drift ratios are within the range of 1/48 to 1/45, ductility coefficients are over 0.3 and equivalent damping coefficients are larger than 0.2, which prove that this kind of shear wall structure has satisfactory seismic behaviors.

(2) The shear span ratio has impact on the peak load, but not the overall deformation trend of the shear wall.

References
[1] Nie J G, et al. 2011 A new progress in the study of double steel plate and concrete composite shear walls Building Structure 41(12) 52-60.
[2] Nie J G, Bu F M and Fan J S 2011 Experimental study on seismic performance of double steel plate concrete composite shear wall with low shear span-ratio Journal of Building Structures 32(11) 74-81.
[3] Bu F M, Nie J G and Fan J S 2013 Experimental Study on seismic performance of double steel plate-concrete composite shear walls with medium and high shear-span ratio at high axial pressure ratio Journal of Building Structures 34(4) 91-98.
[4] Xu M, et al. 2010 Walls connected with two side and combined shear walls Journal of Harbin Institute of Technology 42(8) 216-1220.
[5] Zhang X Y, Wang K, et al. 2019 Experimental study on the seismic behaviour of composite shear walls with stiffened steel plates and infilled concrete Thin-Walled Structures 144 160-279.
[6] Link R A and Elwiae 1995 Composite concrete-steel plate walls: analysis and behaviour Journal of Structural Engineering 121 (2) 60-71.
[7] Emori K 2002 Compressive and Shear strength of concrete infilled steel box wall *Journal of Constructional Steel Structures* **68**(2) 29-40.

[8] Clubley S K, Moy S J and Robert Y 2003 Shear strength of steel-concrete-steel composite panels: part I: testing and numerical modelling *Journal of Constructional Steel Research* **59**(6) 781-794.

[9] Clubley S K, Moy S S J and Robert Y 2003 Shear strength of steel-concrete-steel Composite Panels: part II: detailed numerical modelling of performance. *Journal of Constructional Steel Research* **59**(6):795-808.

[10] Metallic Materials: Tensile Testing: Part 1: Method of Test at Room Temperature 2010 *GB/T228.1—2010* Beijing: China Planning Press.

[11] Han L H 2007 *Concrete-Filled Steel Tubular Structure: Theory and Practice* Beijing: Science Press.

[12] Code for Seismic Design of Buildings 2010 *GB 50011—2010* Beijing: China Architecture and Building Press.