Research Article

Numerical Simulation and Shear Capacity Study of a New Double-Steel-Plate Shear Wall

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This paper presents a new type of double-steel-plate shear wall. Through the finite element software, the stress and plastic strain of steel plate and concrete in different loading stages of the wall are analyzed, the mechanical properties and failure rules of the wall are revealed, and the failure mechanism of the wall is obtained. The comparison with the test results verifies the correctness of the finite element analysis. Through nonlinear finite element parameterization analysis, the composition of shear capacity of each composite wall is obtained. Based on the superposition theory, practical calculation formula of its shear capacity is given, and through a comparative analysis with simulation test results, the goodness-of-fit is satisfying.

1. Introduction

The double-plate composite shear wall is a composite shear wall filled with concrete in the middle of the double steel plates. As a vertical stress-bearing member bearing vertical load and horizontal load, it has been applied to all kinds of buildings. At the beginning of the 1990s, Link and Elwi [1] proposed a double-plate composite shear wall, the middle of which was connected using ribbed stiffeners, conducted nonlinear finite element analysis of this shear wall, and analyzed a series of indexes such as failure mode, bearing capacity, and stress distribution. Link et al. [1–9] studied the concrete filled in double profiled steel plates. The two sides of this shear wall consisted of concrete in the profiled steel plates. Then based on axial compression test and shear test, they investigated various indexes like its strength, stiffness, failure mode, strain, and internal concrete contact and proposed related design method. Eom et al. [10] implemented a pseudostatic test of three linear double-plate shear walls and two T-shaped double-plate shear walls. They found that the main failure modes were the tensile failure of weld joints between wall and foundation beam or local buckling failure of steel plates and that bottom anchorage of the wall

had a high bearing on its ductility. Nie et al. [11, 12] conducted an experimental study of double-plate composite shear walls of different forms, and they proposed design suggestions, restriction of axial compression ratio, and corresponding calculation formula through numerical simulation and parameter analysis on test basis. This type of wall is mainly applied to super high-rise buildings. Clubley and Moy et al. studied bi-steel composite shear walls. As for this type of shear wall, internal filled concretes were connected through horizontal shear keys between double steel plates. Based on an experimental study of 12 shear walls with different thicknesses of steel plate, shear key spacing and steel plate spacing, strains of steel plates, and shear keys were analyzed through numerical simulation, and suggestions were proposed for the selection of plate spacing and thickness. Ji et al. [13] put forward a steel tube-double steel plates-concrete composite shear wall. They investigated its indexes like failure mode, bearing capacity, ductility, and energy consumption through a pseudostatic test, where the final failure modes were mainly steel plate buckling, steel plate tension, and concrete crushing. Subsequently, they put forward the calculation formula of normal section bearing capacity. Emori [14] conducted an experimental

compression and shear test of 1/4-scale double-plate shear wall set with longitudinal and transverse ribbed stiffeners in the middle, and the results indicated that this shear wall had very good ductility and bearing capacity. The results of the shear test were compared through finite element analysis, and the calculation formula of bearing capacity of this shear wall was derived. Qian et al. [15] implemented pseudostatic test of 6 concrete filled steel tube shear walls configured with concrete filled circular steel tubes within edge constrained members and pointed out that both bearing capacity and deformation performance of this shear wall were greater than those of reinforced concrete shears walls with the same parameters. Through a comparison with the calculation result obtained through the calculation formula of flexural capacity, it is verified that the test bearing capacity of concrete filled steel tube accorded with formula calculation very well. BLC-C T-shaped composite wall structure is the leading member resisting vertical force and lateral force as a structural system. The wall body is formed by connecting lipped channels with different sectional forms, and it is poured with concrete inside. Relative to ordinary shear walls, it is characterized by flexible arrangement. The end enhancement can help to better bear vertical load and resist seismic forces in two directions at the same time. Therefore, it can serve as vertical and horizontal stress-bearing member of high-rise buildings. Based on an experimental study of five 1:1 full-size BLC-C T-shaped composite wall models under low-cyclic loading, the failure mechanism and stress mechanism of the T-shaped wall were analyzed. According to the data results of five tests, a finite element model was constructed for numerical simulation, and test results were used to verify the correctness of theoretical analysis results, followed by a systematic parameter analysis, and analysis of the effects of axial compression ratio, shear span ratio, steel strength, concrete strength, thickness of steel plate, and sectional dimensions of U-shaped steel on seismic performance of BLC-C T-shaped composite walls. Based on test data and finite element parameter analysis, the main factors influencing the bearing capacity of BLC-C T-shaped composite walls were analyzed. The calculation formula of flexural capacity of BLC-C T-shaped composite wall was derived based on the superposition theory. The composition of shear capacity of the BLC-C T-shaped composite walls was obtained through analysis of the stress-bearing mode of BLC-C T-shaped composite walls with low shear span ratios and nonlinear finite element analysis of various factors influencing shear capacity, and the formula of shear capacity is given by statistical regression.

2. Experimental Overview and Strain Analysis

2.1. Experimental Overview. Five full-size BLC-C T-shaped composite walls were designed and numbered from BLC-C1-T1 to BLC-C1-T5. 1:1 full-size specimens in practical engineering were studied, the height of the wall body was 2,600 mm, flange lengths were unified as 770 mm, and web length ranged from 1,250 mm to 2,050 mm. The BLC-C T-shaped composite walls were formed by connecting lipped channels with different cross sections

(20 mm × 130 mm, 160 mm × 130 mm, 200 mm × 130 mm, 130 mm × 1300 mm, etc.). The thickness of each steel plate was 3 mm–5 mm, the connection mode was welding, and each was filled with concrete. Typical cross sections are shown in Figure 1. All specimens used steel and concrete of the same materials. The main variable parameters included length and thickness of the lipped channel, web-end length of T-shaped shear wall, and arrangement of cotter or not. The failure modes and stress mechanisms of the walls were determined through an experimental study [16].

2.2. Stress-Strain Analysis of Shear Walls. Strain gauges and strain rosettes were arranged on steel plates of the composite walls to test their strain conditions in the test process. A representative member BLC-C1-T1 was selected for strain analysis. The layout of strain gauges and rosettes of steel plates is shown in Figure 2.

The following conclusions can be obtained through vertical strains at different positions of different T-shaped steel plate shear walls, as shown in Figure 3.

2.2.1. Strain Gauges at 150 mm Height of the Wall. For strain gauges at the web end of the T-shaped wall, in the early elastic loading phase, namely, before the member entered the yield phase, the strain presented linear growth. When the elastic phase was ended, the compressive strain of steel plate was approximately higher than yield strain, entering the yield phase. However, when the member was stretched, the tensile strain was still in the elastic phase.

For strain gauges at flange end of T-shaped wall, the strain at flange end was smaller than web-end strain under the same load. It still presented linear growth in the elastic phase of the member, namely, the steel plate strain at flange end of the member was smaller than that at the web end.

From yield load to peak load, for strain gauges at the web end of each member, the vertical strain of steel plate was increased rapidly. When the steel plate was under compression, the vertical strain rapidly increased, according to the phenomenon that local buckling appeared at the side of steel plate under peak load of the specimen. Under the same load, the vertical direction of the lower steel plate presented elastic change when the steel plate was stretched. For web-end strain gauges, the member strain presented linear growth. The steel plate entered the yield phase when the peak load was reached.

As the horizontal load started declining from peak load and entered the decline phase, for strain gauges at web end of each member, when the steel plate was stretched, its vertical strain abruptly increased, and the compressive strain at the edge of steel tube at wall end was remarkably higher than tensile strain, and this was caused by severe buckling of steel plate at wall bottom; for strain gauges at flange end, compressive strain and stress started rising rapidly in the later phase of hysteresis test, indicating that buckling started appearing on the steel plate at specimen bottom.

FIGURE 1: Configuration details of BLC-C1-T1 and BLC-C1-T2. (a) Front view of BLC-C-T1 and BLC-CT-2. (b) Sectional configuration of BLC-C-T1 and BLC-C-T2.

FIGURE 2: Layout drawing of strain gauges and strain rosettes on steel plates.

2.2.2. Strain Gauges at 450 mm Height of the Wall. In the elastic phase of the member, for strain gauges at the web end, the strain presented linear growth in the linear-elastic loading phase, not entering the yield phase. In comparison with strain gauges at 150 mm height, the strain was small under the same load. For strain gauges at flange end, the strain was smaller than that at web end under the same load, and it still presented linear growth in the elastic phase of the member, indicating that the stress-strain of lower steel plate at flange end of the member was far smaller than that at web end.

When the horizontal load was between yield load and peak load, for strain gauges at web end, the compressive strain of steel plate reached from elastic phase to yield strain, reached peak load subsequently, and then started increasing rapidly. For strain gauges at flange end and when the peak load was reached, the strain still presented linear growth, but not entering the yield phase, and the strain of steel plate at flange end of the member was much smaller than that at web end.

When horizontal load started declining from peak load, for strain gauges at web end, it could be seen that steel plate started experiencing compressive buckling rapidly. For strain gauges at flange end, as the member started going through failure, steel plate entered the yield phase and its strain reached yield strength. At the end of the test, the steel plate entered the yield phase.

3. Numerical Simulation

3.1. Material Constitutive. The elastoplastic constitutive relation of steel plate can be accurately given, using Mises yield conditions and related flow rules. Steel tube material was Q345 in this test, and the stress-strain relation curve could be generally divided into elastic phase (oa), elasto-plastic phase (ab), plastic phase (bc), reinforcement phase (cd), and secondary plastic flow (de) [17]. In a concrete filled steel tube, due to constraining effect of steel tube on concrete, mechanical properties tend to be changed, and the concrete constrained in steel tube tend to be different from

FIGURE 3: Continued.

FIGURE 3: Vertical strain of BLC-C1-T3. (a) Gauge strain of no. 61. (b) Gauge strain of no. 48. (c) Gauge strain of no. 46. (d) Gauge strain of no. 19. (e) Gauge strain of no. 62. (f) Gauge strain of no. 47. (g) Gauge strain of no. 45. (h) Gauge strain of No. 29.

ordinary concrete in plastic performance. The concrete stress-strain relation proposed in literature [18] was used in this paper, and the core concrete damage factor was obtained according to literature [18].

3.2. Contact Model, Element Type Selection, Boundary Conditions, and Loading Mode. The contact of BLC-C T-shaped composite walls in the study was mainly contacted between steel tube and concrete. The interfacial model between steel tube and concrete consisted of contact in the normal direction of the interface and bonding slippage in a tangential direction. The normal contact of steel plate-concrete contact interface was hard contact. The Coulomb friction model was used to simulate tangential force, where the friction coefficient was 0.4. Element selection: in the finite element simulation, 3D solid elements (C3D8R) of 8-node reduced integration were used in the finite element simulation of concrete and steel plate. Finite slippage was used for slippage on the surface of steel tube and concrete. The boundary conditions and loading mode of BLC-C T-shaped composite walls in this paper were quite clear. Fixed-end constraint was used at the bottom of the foundation beam of each BLC-C T-shaped composite wall, and top of cover plate was free end applied with axial uniformly distributed load, while the horizontal load was applied to the cover plate laterally. Firstly, contact was established and axial compression was applied, and then the reciprocal load was applied according to the test loading system at the end of loading beam.

3.3. Numerical Simulation and Comparison of Test Results

3.3.1. Comparison of Skeleton Curve and Hysteretic Curve. To verify the effectiveness of the finite element model and investigate its working mechanism, Abaqus finite element software was used to simulate the whole low-cyclic loading

process. The load-displacement curves of BLC-C T-shaped composite walls under the action of hysteretic load were compared, followed by finite element analysis of working performances in different phases, as shown in Figures 4 and 5. It could be seen from figures and tables that the result obtained through finite element analysis software accorded with overall shape of the test curve very well. As shown in the skeleton curve and hysteretic curve charts, the skeleton curve obtained through the member test was overlapped with that obtained through finite element analysis very well. As error accumulation and initial defects of members were unavoidable in the test process, the specimens would generate welding distortion and residual stress in the fabrication process. Due to the accumulation of errors in the test process and the inevitable initial defects of components, the finite element result is slightly larger than the test result. Then, the finite element result was slightly higher than test result, but most errors were within 10%, indicating high goodness-of-fit. It could be proved that the finite element model established through Abaqus could reflect hysteretic characteristics of BLC-C T-shaped composite walls very well. Due to the effects of foundation deformation and the initial defects of the specimen, the finite element results have higher lateral stiffness than the test results.

3.3.2. Comparison of Failure Modes. The reliability of finite elements was verified by comparing calculation results obtained by nonlinear finite element software and failure modes obtained through the test. BLC-C-T1 was selected for such comparative analysis.

Figure 6 indicates that the lower steel plate at web end reached ultimate stress in the finite element analysis of BLC-C-T1, and apparent buckling appeared at the steel plate bundles at the lower side of web end. Steel plate in most regions of the flange end of the wall did not reach ultimate stress with small wall deformation. The stress of

FIGURE 4: Comparison of hysteretic curves: (a) specimen BLC-C-T-1, (b) specimen BLC-C-T-2, (c) specimen BLC-C-T-3.

steel plate 1,000 mm above the wall did not enter the yield phase without any deformation above the wall. This was basically identical to the phenomenon that evident buckling appeared at the web end, and no deformation appeared at flange end or on the wall under the final failure of BLC-C-T1. Therefore, hysteretic characteristics and failure modes of the BLC-C T-shaped composite wall simulated through Abaqus finite element software were identical to failure phenomena, failure modes, and stress conditions in the low-cyclic loading test of the wall, and the finite element simulation in this paper could accurately simulate the stress of the BLC-C T-shaped composite wall.

(1) *Compared with the Test Failure.* When the specimen was loaded to failure, the side wall at the bottom of the wall buckled significantly at 150 mm. The first, second, third, and fourth steel tube bundles at the heights of 150 mm, 250 mm, and 350 mm are very obvious buckling, as shown in Figures 7(a) and 7(b). In the finite element model, at the end of hysteretic test, the positive horizontal

load is 1050 kN and the horizontal displacement is 42 mm. Figure 7(c) and 7(d) are steel plate Mises strain diagram and equivalent plastic strain diagram, respectively. As can be seen from the figure, all the steel plates at the bottom of the web end of the whole wall have yielded, and the steel strength at the bottom of most of the web area has reached its ultimate strength and entered the failure stage, which was in accordance with the experimental phenomenon that rows of buckling occurred at the bottom of the shear wall and steel plates entered yield deformation at the end of the test loading. It can be seen from the equivalent plastic strain diagram of steel plate that the plastic deformation at the bottom of steel plate increases significantly at about 200 mm height, indicating that the deformation and failure phenomenon at the bottom of steel web end at 200 mm–300 mm height is the most serious. The stress at the flange end of steel plate is less than that at the web end, and only part of the area enters into yield, which corresponds to the experimental phenomenon that the failure mainly occurs at the web end.

FIGURE 5: Comparison of skeleton curves: (a) specimen BLC-C-T-1, (b) specimen BLC-C-T-2, (c) specimen BLC-C-T-3.

4. Simplified Calculation Formula of Shear Capacity

A large number of parameters were analyzed based on a BLC-C T-shaped composite wall with a low shear span ratio. According to nonlinear parameter analysis results and experimental study results, data were provided to formula derivation of shear capacity.

4.1. Parameter Analysis. Sectional dimensions of the wall were set as 2700 mm × 2884 mm × 130 mm, and shear span ratio was set as 0.94. Each BLC-C T-shaped composite wall was formed by connecting 18 lipped channels with the size of 160 mm × 130 mm. Thickness and strength of steel plate were 4 mm and Q345, and the concrete strength was C40. Based on the finite element model verified through the test, the influences of parameters like concrete strength, yield strength of steel plate, sectional steel ratio, and shear span ratio on stress properties of the shear wall were analyzed. In the parameter analysis, only one parameter was changed, while other parameters were kept unchanged.

4.1.1. Influence of Axial Compression Ratio. When other parameters remained unchanged, the axial compression ratio was changed to study the influence of the axial compression ratio on the seismic property of the BLC-C T-shaped composite wall. Figure 8 and Table 1 display the load-displacement curve under axial compression ratio of 0.1–0.7. It could be seen from the figure and table that the axial compression ratio had a certain influence on bearing capacity and ductility of the BLC-C T-shaped composite wall. When the axial compression ratio was small (0.1–0.4), axial compression ratio had a minimal effect on bearing capacity of the member. Under axial compression ratio of 0.3, horizontal load reached the maximum value. Under axial compression ratio of 0.3–0.5, horizontal load and load descent stage were slightly reduced in comparison with that under 0.3; when the axial compression ratio was within 0.6–0.7, peak load was obviously reduced, and more obvious descent stage appeared on the skeleton curve. It could be known from parameter analysis that under the axial compression ratio of 0.3 the horizontal bearing capacity of the BLC-C T-shaped composite wall

FIGURE 6: Comparison of failure mode of BLC-C-T1: (a) schematic of final damage, (b) final failure stress charge of finite elements.

was the maximum. As the axial compression ratio rose to 0.7, the horizontal load declined to 87% of peak load.

4.1.2. Influence of Yield Strength of Steel Plate. In order to investigate the influence of yield strength of steel plate on the horizontal bearing capacity of the BLC-C T-shaped composite wall, yield strength and tensile strength of steel plate were changed while other conditions were kept unchanged to study the influence of steel strength on seismic performance of the wall. Steel strength was Q235, Q35, Q390, and Q420, respectively. According to Figure 9 and Table 2, as the steel strength grade was elevated, the peak horizontal load of the BLC-C T-shaped composite wall was increased evidently, and wall stiffness was enhanced in elastic phase. As steel strength grade rose from Q345 to Q390 and Q420, peak horizontal load was elevated by 11% and 18%, in comparison with that under Q345 condition, respectively. It shows that, with the elevation of steel strength grade, horizontal load presented linear growth; the descent stage of skeleton curve became more gentle with the elevation of steel strength grade, meaning that as the steel strength grade increased, the ability of steel to constrain the concrete was enhanced.

4.1.3. Influence of Thickness of Steel Plate. Skeleton curves under the thickness of steel plate of 4 mm–8 mm were selected to investigate the influence of the thickness of steel plate on seismic performance of the BLC-C T-shaped composite wall. As shown in Figure 10 and Table 3, when the thickness of steel plate was increased, the steel ratio of the member would be elevated, so were elastic stiffness and peak

load of the shear wall to an obvious degree; as the thickness of steel plate increased from 4 mm to 5 mm, 6 mm, 7 mm, and 8 mm, the peak load of the shear wall was increased by 20%, 41%, 61%, and 81%, respectively. Horizontal bearing capacity of the BLC-C T-shaped composite wall and its stiffness in elastic phase presented linear growth with the thickness of steel plate, the ability of steel plate to constrain the concrete would be enhanced, the descent stage of skeleton curve would gradually become gentle, the ductility of the composite wall would be improved to a certain degree, and the increase of thickness of steel plate had the most obvious influence on the performance of the shear wall.

4.1.4. Influence of Concrete Strength. To study the influence of concrete strength on bearing capacity of the BLC-C T-shaped composite wall, we changed the concrete strength while other conditions were kept unchanged. Table 4 and Figure 11 display the influences of concrete strength grade on the horizontal bearing capacity of the BLC-C T-shaped composite wall and its skeleton curve. It could be known that, with the increase of concrete strength grade and its elasticity modulus, the peak horizontal load of the BLC-C T-shaped composite wall would be elevated. In contrast, wall stiffness in elastic phase was slightly increased, indicating that, with the increase of concrete strength grade, elasticity modulus, and compressive strength, the horizontal load presented linear growth. However, the descent stage of skeleton curve declined rapidly, and the degradation of bearing capacity was accelerated, indicating that as the concrete strength grade increased, the constraining ability of steel plate for the concrete would be continuously reduced.

FIGURE 7: Comparison diagram with experimental failure phenomenon. (c) Steel plate Mises strain diagram. (d) Equivalent plastic strain diagram.

4.1.5. Influence of Thickness of Shear Span Ratio. To investigate the influence of shear span ratio on the BLC-C T-shaped composite wall, we changed the number of bundled lipped channels while other factors were kept unchanged. The skeleton curves with the number of steel tube bundles ranging from 16 to 20 were selected to study the influence of the shear span ratio on seismic performance of the BLC-C T-shaped shear wall. Figure 12 and Table 5 show that as the shear span ratio decreased, the elastic stiffness and a peak load of the composite wall were obviously elevated. With the decrease of shear span ratio, the horizontal bearing capacity and elastic stiffness of composite wall increased linearly. As the shear span ratio decreased, the failure mode of the composite wall was turned from bending failure into shear failure, and the shear capacity of steel plate and concrete would be improved. From the descent stage of skeleton curve, the change of shear span ratio had no influence on descent stage of skeleton curve.

4.2. Calculation Formula of Shear Capacity. According to test results and parameter analysis, the main influence factors of the BLC-C T-shaped composite wall included strength of steel plate and concrete material, cross-sectional area, shear span ratio, and axial compression ratio, etc. Based on parameter analysis results and the formula mentioned

previously, the superposed calculation formula of shear capacity can be obtained as follows:

$$V \leq V_c + V_s + V_n, \quad (1)$$

where V is shear capacity of the member, V_c is shear capacity of the concrete, V_s is shear capacity of the steel plate, and V_n is increment of shear capacity by axial compression.

4.2.1. Axial Compression Term. The shear capacity V_n contributed by axial compression is calculated using $V_n = kN$ in ACI Specifications, Technical Specifications for Concrete Structures of High-Rise Building and Technical Specifications for Composite Structures. In Technical Specifications for Concrete Structures of High-Rise Building, $V_n = (1/(\lambda - 0.5))0.13N$ when the anti-seismic effect is not considered, and $V_n = (1/\gamma_{RE})((1/(\lambda - 0.5))0.10N)$ when it is considered. By reference to the specifications and finite element analysis, the shear capacity contributed by axial compression is taken as

$$V_n = \frac{1}{\lambda + a_1} a_3 N, \quad (2)$$

where λ is the shear span ratio of the wall and N is axial compression borne by the wall.

FIGURE 8: Skeleton curves of different axial load ratios.

TABLE 1: Calculation results for different axial compression ratios.

Model no.	Axial load ratio	Axial load (kN)	Positive peak load (kN)	Negative peak load (kN)
BLC-1-1	0.1	1417.515	4412.3	-4413.63
BLC-1-2	0.3	4252.546	4475.33	-4450.05
BLC-1-3	0.5	7087.576	4342.4	-4322.61
BLC-0	0.6	8505.092	4127	-4118
BLC-1-4	0.7	9922.607	3909.47	-3912.71

FIGURE 9: Skeleton curves of different yield strengths of steel plate.

TABLE 2: Calculation results for different yield strengths of steel plate.

Model no.	Steel strength (MPa)	Tensile strength (MPa)	Positive peak load (kN)	Negative peak load (kN)
BLC-2-1	235	335	3144	-3146
BLC-0	345	490	4127	-4118
BLC-2-2	390	557	4594	-4580
BLC-2-3	420	600	4877	-4853

FIGURE 10: Skeleton curves of different steel thicknesses.

4.2.2. *Steel Plate Term.* The formula of shear capacity V_s contributed by steel plate refers to American AISC-310, Technical Specifications for Steel Plate Shear Walls and finite element analysis results; the shear capacity provided by steel plate is

$$V_s = \frac{a_4}{\lambda + a_1} A_s f_s, \quad (3)$$

where a is a specific parameter, A_s is sectional area of steel plate, and f_s is design yield strength of steel plate.

4.2.3. *Concrete Term.* The shear capacity provided by the concrete is calculated by reference to ACI Specifications, Technical Specifications for Concrete Structures of High-Rise Building and Technical Specifications for Composite Structures. The formula of concrete shear capacity is obtained by combining the finite element analysis result as follows:

$$V_c = \frac{a_2}{\lambda + a_1} f_t b_w h_{w0}, \quad (4)$$

where f_t is axial tensile strength of concrete, b_w is concrete thickness in wall, and h_{w0} is effective height of concrete section in wall. Hence, the formula of shear capacity of the BLC-C T-shaped composite wall is

$$V_u = \frac{1}{\lambda + a_1} (a_2 f_t b_w h_{w0} + a_3 N + a_4 f_y A_y). \quad (5)$$

Therefore, a_1 , a_2 , a_3 , and a_4 are four unknown numbers. Formulas (5) taken as the equation, the finite element model calculated based on multiple BLC-C T-shaped composite walls is taken as a dependent variable to perform nonlinear regression of data, and the result is obtained.

$$\begin{aligned} a_1 &= 0.271, \\ a_2 &= 1.999, \\ a_3 &= -0.065, \\ a_4 &= 0.585. \end{aligned} \quad (6)$$

The calculation formula of oblique sectional bearing capacity of the BLC-C T-shaped composite wall is as follows:

$$V_u = \frac{1}{\lambda + 0.271} (1.999 f_t b_w h_{w0} - 0.065 N + 0.585 f_y A_y). \quad (7)$$

The comparison between wall shear capacity calculated through formula and the finite element result is shown in Table 6. Table 6 indicates that the calculated value has a high goodness-of-fit with the finite element result.

TABLE 3: Calculation results for different steel thicknesses.

Model no.	Steel thickness (mm)	Positive peak load (kN)	Negative peak load (kN)
BLC-0	4	4127	-4118
BLC-3-1	5	4990	-5003
BLC-3-2	6	5818	-5822
BLC-3-3	7	6651	-6647
BLC-3-4	8	7453	-7444

TABLE 4: Calculation results for different concrete strengths.

Model no.	Concrete strength	Axial compression standard value (MPa)	Positive peak load (kN)	Negative peak load (kN)
BLC-4-1	C30	20.10	3992	-3992
BLC-0	C40	26.80	4127	-4118
BLC-4-2	C50	32.40	4187.01	-4170
BLC-4-3	C60	38.50	4268	-4275

FIGURE 11: Skeleton curves of different concrete strengths.

FIGURE 12: Skeleton curves of different shear span ratios.

TABLE 5: Calculation results for different shear span ratios.

Model no.	Dimension (mm)	Shear span ratio	Quantity of steel tube bundles	Positive peak load (kN)	Negative peak load (kN)
BLC-5-1	2564 * 130	1.05	16 * 160	3390	-3406
BLC-5-2	2724 * 130	0.99	17 * 160	3765	-3768
BLC-0	2884 * 130	0.94	18 * 160	4127	-4118
BLC-5-3	3044 * 130	0.89	19 * 160	4551	-4529

TABLE 6: Comparison of calculated value and finite element result of shear capacity.

Model no.	Finite-element result V_s (kN)	Calculated result V_c (kN)	V_s/V_c
YZQ-B00	4127	4158	1.007
BLC-1-1	4412	4539	1.029
BLC-1-2	4475	4387	0.980
BLC-1-3	4342	4234	0.975
BLC-1-4	3909	4081	1.044
BLC-2-1	3144	3122	0.993
BLC-2-2	4594	4594	1.000
BLC-2-3	4877	4921	1.009
BLC-3-1	4990	4989	1.000
BLC-3-2	5818	5821	1.000
BLC-3-3	6651	6653	1.000
BLC-3-4	7453	7485	1.004
BLC-4-1	3993	4049	1.014
BLC-4-2	4187	4217	1.007
BLC-4-3	4268	4254	0.997
BLC-5-1	3390	3385	0.998
BLC-5-2	3765	3763	1.000
BLC-5-3	4551	4566	1.003

5. Conclusions

According to strain analysis and finite element analysis of the members, the following conclusions are drawn:

- (1) The strain analysis of the BLC-C T-shaped composite walls verified that the wall failure mainly occurs at the bottom of wall web end. Besides, the wall strain at the flange end is smaller than that at the web end, and the steel plate in the middle of the wall or above the wall does not experience yield failure.
- (2) The effectiveness of the test result is verified by a comparison with finite element analysis. The test results of the hysteretic curve and skeleton curve of BLC-C T-shaped composite walls accord well with finite element results. Following the analysis of stresses and plastic strains of steel plate and concrete in different loading phases of the walls, mechanical properties and failure laws of the walls are revealed and failure mechanism is obtained.
- (3) The effects of concrete strength, the strength of steel plate, shear span ratio of the wall, thickness of steel plate, sectional dimensions of the lipped channel, and axial compression ratio on mechanical properties of each composite wall are analyzed through the parameter analysis.
- (4) Based on analysis of stress-bearing mode of BLC-C T-shaped composite walls with low shear span ratios and finite element analysis of various factors

influencing shear capacity, the composition of shear capacity of each composite wall is obtained. Parameter values are calculated based on the superposition theory through statistical regression, practical calculation formula of its shear capacity is given, and through a comparative analysis with simulation test results, the goodness-of-fit is satisfying.

Data Availability

The experimental and finite element data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare no conflicts of interest.

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References

- [1] R. A. Link and A. E. Elwi, "Composite concrete-steel plate walls: analysis and behavior," *Journal of Structural Engineering*, vol. 121, no. 2, pp. 60–71, 1995.

- [2] H. Wright, "The axial load behaviour of composite walling," *Journal of Constructional Steel Research*, vol. 45, no. 3, pp. 353–375, 1998.
- [3] H. D. Wright and S. C. Gallocher, "The behaviour of composite walling under construction and service loading," *Journal of Constructional Steel Research*, vol. 35, no. 3, pp. 257–273, 1995.
- [4] K. M. Anwar Hossain and H. D. Wright, "Experimental and theoretical behaviour of composite walling under in-plane shear," *Journal of Constructional Steel Research*, vol. 60, no. 1, pp. 59–83, 2004.
- [5] H. D. Wright and K. M. Hossain, "In-plane shear behaviour of profiled steel sheeting," *Thin-Walled Structures*, vol. 29, no. 1, pp. 79–100, 1997.
- [6] K. M. Anwar Hossain, "Design aspects of double skin profiled composite framed shear walls in construction and service stages," *ACI Structural Journal*, vol. 101, no. 1, pp. 94–102, 2004.
- [7] K. M. A. Hossain and H. D. Wright, "Performance of double skin-profiled composite shear walls - experiments and design equations," *Canadian Journal of Civil Engineering*, vol. 31, no. 2, pp. 204–217, 2004.
- [8] K. M. A. Hossain and H. D. Wright, "Behaviour of composite walls under monotonic and cyclic shear loading," *Structural Engineering & Mechanics*, vol. 17, no. 1, pp. 69–85, 2004.
- [9] K. M. Anwar Hossain and H. D. Wright, "Flexural and shear behaviour of profiled double skin composite elements," *Steel and Composite Structures*, vol. 4, no. 2, pp. 113–132, 2004.
- [10] T.-S. Eom, H.-G. Park, C.-H. Lee, J.-H. Kim, and I.-H. Chang, "Behavior of double skin composite wall subjected to in-plane cyclic loading," *Journal of Structural Engineering*, vol. 135, no. 10, pp. 1239–1249, 2009.
- [11] J.-G. Nie, H.-S. Hu, J.-S. Fan, M.-X. Tao, S.-Y. Li, and F.-J. Liu, "Experimental study on seismic behavior of high-strength concrete filled double-steel-plate composite walls," *Journal of Constructional Steel Research*, vol. 88, no. 9, pp. 206–219, 2013.
- [12] J. G. Nie, F. Bu, and J. S. Fan, "Experimental research on seismic behavior of low shear-span ratio composite shear wall with double steel plates and infill concrete," *Journal of Building Structures*, vol. 32, no. 11, pp. 74–81, 2011.
- [13] X. Ji, F. Jiang, J. Qian et al., "Experimental study on seismic behavior of steel tube-double steel plate-concrete composite shear walls," *Journal of Building Structures*, vol. 34, no. 6, pp. 26–38, 2013.
- [14] K. Emori, "Compressive and shear strength of concrete filled steel box wall," *Steel Structures*, vol. 68, no. 2, pp. 29–40, 2002.
- [15] J. Qian, Z. Jiang, and X. Ji, "Behavior of steel tube-reinforced concrete composite walls subjected to high axial force and cyclic loading," *Engineering Structures*, vol. 36, no. 4, pp. 173–184, 2012.
- [16] X. Zhang, Y. Qin, Z. Chen, and L. Jie, "Experimental behavior of innovative T-shaped composite shear walls under in-plane cyclic loading Experimental behavior of innovative T-shaped composite shear walls under in-plane cyclic loading," *Journal of Constructional Steel Research*, vol. 120, pp. 143–159, 2016.
- [17] L.-H. Han, G.-H. Yao, and Z. Tao, "Performance of concrete-filled thin-walled steel tubes under pure torsion," *Thin-Walled Structures*, vol. 45, no. 1, pp. 24–36, 2007.
- [18] J. Li and J. Y. Wu, "Elastoplastic damage constitutive model for concrete based on damage energy release rates, part I: basic formulations," *China Civil Engineering Journal*, vol. 38, no. 9, pp. 14–20, 2005, in Chinese.