EXPERIMENTAL STUDY ON THE SEISMIC BEHAVIOR OF A SHEAR WALL WITH CONCRETE-FILLED STEEL TUBULAR FRAMES AND SLITS

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ABSTRACT

This paper proposes a new type of composite shear wall with concrete-filled steel tubular frames and artificial vertical slits. The seismic behaviour of this new shear wall is studied by using a cyclic loading test. The failure mode, load-carrying capacity, ductility, stiffness degradation, hysteresis behaviour and energy dissipating capacity shown in the course of the test are studied. And this paper proposes the calculation formula of normal section bearing capacity. The main research results are as follows. Firstly, the failure mode of this new shear wall is the flexural failure. Secondly, compared with the ordinary reinforced concrete shear wall with slits, the new shear walls with concrete-filled steel tubular frames and slits show a better performance in load-carrying capacity, ductility, stiffness degradation and energy dissipating capacity.

KEYWORDS

Shear wall, Seismic behaviour, Cyclic lateral loading, Concrete-filled steel tubular frame, Artificial vertical seams

INTRODUCTION

Shear walls and core tubes composed of shear walls are core anti-earthquake vertical system of high-rise buildings, so it is very important to improve their seismic behaviour for preventing the structures from seismic collapse. In recent years, the studies about this field have got high attention and made a series of achievements. There are two new types of shear walls which are closely related to this paper and have been studied for their seismic behaviour.

Jiang et al. [1] studied a new type of slit shear wall. It is one with vertical slit purposely cast within the wall, and the rubber belt penetrated by a part of web shear reinforcement as seismic energy-dissipation device is filled in the slit. It was found that the seismic performance of the slit wall improved significantly. Cho et al. [2] used hollow steel tubes as the boundary elements of reinforced concrete shear walls. The seismic behaviour of this new type of shear wall is equivalent to that of a traditional reinforced concrete shear wall. An experimental investigation including four test models was thus conducted on circular CFST columns and RC shear wall mixed structures subjected to a constant axial load and cyclic lateral load [3]. It was found that all the tested specimens failed in a shear-dominant mode. The ductility and energy dissipation capacity of the specimens decreased with an increase of axial load level or decrease of height–width ratio. Han [4] presented two building models with 30 stories consisting of composite frames and RC shear walls on shaking table tests. It was found that the composite frames cooperated well with the core RC shear wall structure under earthquakes, and the two building models exhibited excellent seismic performance. Nie [5] proposed a new detailed concrete-filled double-steel-plate (CFDSP) composite wall using high-strength concrete. This CFDSP composite wall is composed of
concrete-filled steel tubular columns at the two boundaries. All the specimens exhibited good energy dissipation ability and deformation capacity with full hysteretic curves and large ultimate drift ratios.

The weakness of the shear wall with slits is low bearing capacity and stiffness, while the concrete-filled steel tubular column has a higher bearing capacity and deformability. Thereby a new type of composite shear wall is proposed through combining the advantages of concrete-filled steel tubular frames and artificial vertical slits. In order to study its seismic performance, three shear wall specimens were constructed and then tested for failure under cyclic lateral loading.

**TEST PROGRAM**

**Description of shear wall specimens**

Three shear wall specimens are respectively denoted as SWS1, SWS2, and SWS3. Specimen SWS1 is ordinary reinforced concrete shear wall with slits. Specimens SWS2 and SWS3 are the shear walls with concrete-filled steel tubular frames and slits. The thickness of specimens is 50mm. Essential parameters of the specimens are shown in Table 1.

<table>
<thead>
<tr>
<th>Specimen number</th>
<th>Thickness of steel tube /mm</th>
<th>Shear span ratio</th>
<th>Cross-sections of steel tubular frames</th>
<th>Dimension of steel tubular frames /mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SWS1</td>
<td>——</td>
<td>2.5</td>
<td>rectangle</td>
<td>50×50</td>
</tr>
<tr>
<td>SWS2</td>
<td>1.2</td>
<td>2.5</td>
<td>rectangle</td>
<td>50×50</td>
</tr>
<tr>
<td>SWS3</td>
<td>1.2</td>
<td>2.5</td>
<td>circle</td>
<td>Φ50</td>
</tr>
</tbody>
</table>

Artificial vertical slits of specimens were made by pre-embedding thin veneers in the specimens. Thin veneers were not taken out after pouring concrete. The vertical and horizontal web reinforcement of specimens was not cut off at artificial vertical slits. There were U shaped shear keys between concrete-filled steel tubular frames and reinforced concrete wall. The dimensions and reinforcement details of specimens are shown in Figure 1.
Fig.1 - The dimensions and reinforcement details of specimens

(a) SWS1

(b) SWS2 and SWS3
Material properties

The same batch of concrete was used to fill three test specimens. The mixing amount of water reducing agent is 12kg per cubic meter concrete. Test blocks were made at the same time of filling specimens. The dimensions of the test blocks were 150mm×150mm×150mm. Test blocks and specimens were maintained in the same environment.

The average compressive strength of concrete test blocks was 51.4 MPa on the day of the test. The material properties of steel are shown in Table 2. The vertical and horizontal web reinforcement of specimens was replaced by galvanized steel wire mesh of Φ2@40.

<table>
<thead>
<tr>
<th>Type of steel</th>
<th>Yield strength/MPa</th>
<th>Ultimate strength/MPa</th>
<th>Modulus of elasticity/MPa</th>
<th>Elongation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel tube</td>
<td>216</td>
<td>433</td>
<td>1.95×10⁵</td>
<td>23%</td>
</tr>
<tr>
<td>Φ2@40 Steel wire mesh</td>
<td>305</td>
<td>400</td>
<td>1.85×10⁵</td>
<td>21%</td>
</tr>
<tr>
<td>Φ4 cold drawn low carbon</td>
<td>335</td>
<td>580</td>
<td>1.8×10⁵</td>
<td>10%</td>
</tr>
</tbody>
</table>

Test setup and loading sequence

The tests were conducted at the Laboratory of the School of Civil Engineering at Inner Mongolia University of Technology, China. Figure 2 shows the test device including the horizontal actuator, the vertical actuator, the load frame and the reaction wall. In order to apply cyclic lateral loading to the specimens, the loading position was at the mid of span of the top section, which was 1270mm far from the top surface of the foundation beam. West is in the positive loading direction, and east is in the negative direction. The vertical actuator was supported to the load frame through a roller to minimize the friction and allow free movement of specimens in cyclic loading. Horizontal displacement was recorded by displacement meters.

The axial load was applied first and maintained a constant value throughout the test. And the axial compression ratio was at 0.1. When the axial compression ratio is calculated, the contribution of the steel tube is considered [6]. Therefore, the axial compression ratio is calculated in Equation (1)

\[ n = \frac{N}{f_c A_c + f_a A_a} \]  

In Equation (1), \( f_c \) is the axial compressive strength of concrete; \( A_c \) is the area of concrete; \( f_a \) is the yield strength of steel tubes; \( A_a \) is the area of steel tubes.

Subsequently, the cyclic loading was applied to the horizontal actuator under displacement control. The loading process continued until the specimens were broken or the strength of the specimens decreased to 85% of the peak lateral load.
TEST RESULTS

Experimental observations

The failure mode of specimens in the last stage of the cyclic lateral loading test is shown in Figure 3. The development of the cracks as well as the failure mechanism of three specimens is demonstrated as follows.

(1) Specimen SWS1

The first flexural crack in SWS1 appeared in the tension zone which was 45mm away from the top surface of the foundation beam. Then many tiny cracks continuously appeared and gradually extended as the displacement increased. In the middle stage of loading, there are many vertical cracks near the artificial vertical slits. This shows that artificial vertical joints begin to take effect and the wall is divided into three parts. The shear span ratio of this specimen is enlarged. The ductility of this specimen is improved. When the specimen was broken, the reinforced concrete concealed column was crushed at the bottom, and the longitudinal reinforcement in the reinforced concrete concealed column was buckled. As seen from the distribution of cracks, the failure mode of this specimen is the flexural failure. Besides, cracks are distributed at 2/3 height of the specimen. Most of the cracks at the bottom of the wall were horizontal direction.

(2) Specimens SWS2 and SWS3

The failure mode of specimens SWS2 and SWS3 was similar. The first flexural crack appeared in tension zone at above the foundation beam. As the force increased, many horizontal tiny cracks continuously appeared and gradually extended. And cracks gradually developed to the artificial vertical slits. As the force continued to increase, the concrete near the vertical slits and at the bottom of the wall began to flake. At the later stage of the loading, the cracks were almost all over the wall. Simultaneously, the bottom of the concrete-filled steel tubular frames of the specimen is bulging. And the wall was split into three small wall limbs which had a bigger height-width ratio. Seismic energy dissipation areas were formed by extrusion and friction between small wall limbs. There was no relative displacement between concrete-filled steel tubular frames and reinforced concrete walls because of the role of shear keys. Compared with SWS1, the other specimens exhibited more dense and homogeneous distribution of cracks. Cracks can be seen in the entire wall. This shows that artificial vertical slits and the concrete-filled steel tubular frames lead to the development trend of cracks and limit the development of long cracks. The energy dissipation capacity of the concrete wall of the other specimens was more fully utilized.
It can be seen from the actual situation of the experiment, the destruction of the three specimens is as follows: The destruction of specimen SWS1 mainly concentrated in the corner of the wall. The height of the plastic zone was about 767mm; the destruction of specimen SWS2 and SWS3 is mainly concentrated in artificial slits. The height of the plastic zone was about 1150 mm. Therefore, concrete-filled steel tubular frames can avoid the local damage of the shear wall. So that the energy dissipation zone of the shear wall is enlarged, and the energy dissipation capacity of the specimen is increased.

![Failure modes of specimens](image)

**Fig. 3 - Failure modes of specimens**

**Hysteretic behaviour and energy dissipating capacity**

Hysteresis curves of three specimens are shown in Figure 4. The hysteresis curves of the specimens are elongated and spindle shape before specimens are cracked, and they are basically in an elastic working state. During the loading cycle of 0.0073 to 0.0088 drifts, specimens yielded. Stiffness of specimens decreased rapidly. The residual deformation increased gradually after unloading. The area of the hysteresis curve is continuously increasing and energy consumption gradually larger. As seen in Figure 4, hysteresis curves of specimens SWS2 and SWS3 are plumper than that of specimen SWS1. Hysteresis curve of specimen SWS1 exhibited palpable pinching phenomenon. This shows that the steel tube can obviously increase the bearing capacity and energy consumption.

![Hysteretic curves of each specimen](image)

**Fig. 4 - Hysteretic curves of each specimen**
The amount of energy absorbed by a specimen during any cycle is defined as the area enclosed within the hysteretic loop for that cycle. So the area of the hysteretic curve formed by the outside envelope is taken to be the main index \( E_S \) of energy dissipating capacity for three specimens. The index \( E_S \) of specimens and their relative values are shown in Table 3.

As seen in Table 3, energy dissipating capacity of specimen SWS3 increases by 195.23% compared with specimen SWS1 and increases by 8.21% compared with specimen SWS2. It can be concluded that the concrete-filled steel tubular frames can increase the energy dissipation capacity of shear wall specimens significantly, and the concrete-filled steel tubular frames with circular cross-sections are more effective than concrete-filled steel tubular frames with rectangular cross-sections on achieving this goal.

<table>
<thead>
<tr>
<th>Number of Specimen</th>
<th>Energy Dissipating ( E_S )/kN·mm</th>
<th>Relative Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>SWS1</td>
<td>1001.8</td>
<td>1.00</td>
</tr>
<tr>
<td>SWS2</td>
<td>2733.3</td>
<td>2.73</td>
</tr>
<tr>
<td>SWS3</td>
<td>2957.6</td>
<td>2.95</td>
</tr>
</tbody>
</table>

**Skeleton curves**

Skeleton curves of three specimens are shown in Figure 5. As seen in Figure 5, the initial stiffness of specimens SWS2 and SWS3 is basically same, and both are larger than SWS1. The specimens SWS2 and SWS3 are more excellent than specimen SWS1 in respect of the load-carrying capacity, ultimate displacement and the area enclosed by the skeleton curve and the coordinate axis. Besides, the specimen SWS3 is better than specimen SWS2 in respect of the indexes above. The skeleton curves of specimens continue to confirm the conclusions presented in this paper previously.

**Load-carrying capacity**

The measured value of cracking load \( F_c \), yield load \( F_y \), ultimate load \( F_u \), and the calculated value of the yield ratio \( \mu_{yu} = F_y/F_u \) are shown in Table 4. The yield displacement is determined based on the reduced stiffness equivalent elasto-plastic yield approach proposed by Park [7]. The following conclusions can be drawn from Table 4.

1. The cracking load, yield load and ultimate load of specimen SWS2 increase by 14.43%, 23.60%, 28.40%, while yield ratio decrease by 3.73%, respectively, compared with
specimen SWS1. The cracking load, yield load and ultimate load of specimen SWS3 increase by 19.25%, 33.54%, 38.21%, while yield ratio decrease by 3.37%, respectively, compared with specimen SWS1. The bearing capacity of SWS2 and SWS3 is greater than SWS1. Its reasons are as follows: When the shear wall is subjected to cyclic loading, the tensile strain and compressive strain at the edge are the largest in the same horizontal cross section. The edge of the cross-section is prone to damage. The longitudinal reinforcement ratio of restrained edge member of specimen SWS1 is 2.01%. Steel ratio of restrained edge member of specimens SWS2 and SWS3 is 10.34%. Restrained edge member with high steel ratio can reduce the strain of the restrained edge member under the same load. The steel tube can restrain the concrete in the tube and the compressive strength of concrete in the tube is improved. Therefore, specimens of concrete-filled steel tubular frames have higher bearing capacity than the specimen of reinforced concrete concealed column.

It can be concluded that concrete-filled steel tubular frames are of great benefit to the improvement of load-carrying capacity. This advantage can enhance the emergency capacity of specimens. In addition, concrete-filled steel tubular frames still have a certain vertical bearing capacity at the end of the test, which was more conducive to the target of avoiding the collapse in response to a strong earthquake.

(2) The cracking load, yield load and ultimate load of specimen SWS3 increase by 4.21%, 8.04%, 7.64%, while the value of the yield ratio is very close, respectively, compared with specimen SWS2.

It shows that the binding effect of concrete-filled steel tubular frames with circular cross-sections is more effective than that of concrete-filled steel tubular frames with rectangular cross-sections.

### Tab. 4 - Load-carrying capacity of specimens

<table>
<thead>
<tr>
<th>Number of Specimens</th>
<th>$F_c$ /kN</th>
<th>$F_y$ /kN</th>
<th>$F_u$ /kN</th>
<th>Relative value to $F_u$</th>
<th>$\mu_{yu}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>SWS1</td>
<td>18.7</td>
<td>48.3</td>
<td>58.1</td>
<td>1.000</td>
<td>0.831</td>
</tr>
<tr>
<td>SWS2</td>
<td>21.4</td>
<td>59.7</td>
<td>74.6</td>
<td>1.284</td>
<td>0.800</td>
</tr>
<tr>
<td>SWS3</td>
<td>22.3</td>
<td>64.5</td>
<td>80.3</td>
<td>1.382</td>
<td>0.803</td>
</tr>
</tbody>
</table>

**Ductility**

The value of displacement, ductility coefficient and maximum displacement angle of three specimens are shown in Table 5. Where $U_c$ is cracking displacement, $U_y$ is yield displacement, $U_u$ is the ultimate displacement corresponding to the point where the load capacity decreases to 85% of the maximum lateral load, $\mu=U_d/U_y$ is ductility coefficient and $\theta_p$ is the maximum displacement angle. The following conclusions can be drawn from Table 5.

(1) The cracking displacement, yield displacement, ultimate displacement, ductility coefficient and maximum displacement angle of specimen SWS2 increase by 0.73%, 15.63%, 40.79%, 21.78%, and 45.10% respectively, compared with specimen SWS1. The cracking displacement, yield displacement, ultimate displacement, ductility coefficient and maximum displacement angle of specimen SWS3 increase by 2.92%, 19.70%, 57.12%, 31.28%, and 57.45% respectively, compared with specimen SWS1.

It can be concluded that concrete-filled steel tubular frames can make specimens in the elastic-plastic stage fully relying on its own deformation to consume seismic energy. The energy dissipation capacity of specimens is obviously increased.
The cracking displacement, yield displacement, ultimate displacement, ductility coefficient and maximum displacement angle of specimen SWS3 increase by 2.17%, 3.52%, 11.60%, 7.80%, and 8.51% respectively, compared with specimen SWS2.

It shows that the concrete-filled steel tubular frames with circular cross-sections are more effective than concrete-filled steel tubular frames with rectangular cross-sections on improving the ductility of specimens.

<table>
<thead>
<tr>
<th>Number of Specimens</th>
<th>$U_c$/mm</th>
<th>$U_y$/mm</th>
<th>$U_d$/mm</th>
<th>$\mu$</th>
<th>$\theta_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>SWS1</td>
<td>1.37</td>
<td>9.34</td>
<td>15.74</td>
<td>1.685</td>
<td>1/74</td>
</tr>
<tr>
<td>SWS2</td>
<td>1.38</td>
<td>10.80</td>
<td>22.16</td>
<td>2.052</td>
<td>1/51</td>
</tr>
<tr>
<td>SWS3</td>
<td>1.41</td>
<td>11.18</td>
<td>24.73</td>
<td>2.212</td>
<td>1/47</td>
</tr>
</tbody>
</table>

**Stiffness degradation**

It is noted that each specimen experienced significant degeneration in their stiffness as drift levels increased. It is of interest to determine the magnitude of stiffness degradation experienced by each specimen. Stiffness of specimens at typical stages is shown in Table 6. Where $K_o$ is initial elastic stiffness, $K_c$ is initial cracking stiffness and $K_y$ is yield stiffness. The curves of stiffness degradation of all specimens with the increase of drifts are shown in Figure 6. The following conclusions can be drawn from Table 6 and Figure 6.

<table>
<thead>
<tr>
<th>Number of Specimens</th>
<th>$K_o$/kN/mm</th>
<th>$K_c$/kN/mm</th>
<th>$K_y$/kN/mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>SWS1</td>
<td>29.72</td>
<td>13.65</td>
<td>5.17</td>
</tr>
<tr>
<td>SWS2</td>
<td>31.07</td>
<td>15.51</td>
<td>5.53</td>
</tr>
<tr>
<td>SWS3</td>
<td>32.58</td>
<td>15.82</td>
<td>5.77</td>
</tr>
</tbody>
</table>

**Fig. 6 - Curves of stiffness degradation**
(1) The initial elastic stiffness, initial cracking stiffness and yield stiffness of specimen SWS2 increase by 4.54%, 13.63%, and 6.96% respectively, compared with specimen SWS1. The initial elastic stiffness, initial cracking stiffness and yield stiffness of specimen SWS3 increase by 9.62%, 15.90%, and 11.61% respectively, compared with specimen SWS1. Besides, the curves of stiffness degradation of specimen SWS2 and SWS3 are gentler than that of specimen SWS1. It can be concluded that concrete-filled steel tubular frames increase the stiffness of specimens and the development of diagonal cracks in the wall is restrained. Consequently, the rate of stiffness degradation is slowed, and the specimens got stable stiffness in the later stage.

(2) The initial elastic stiffness, initial cracking stiffness and yield stiffness of specimen SWS3 increase by 4.86%, 2.00%, and 4.34% respectively, compared with specimen SWS2. Besides, the curve of stiffness degradation of specimen SWS3 is gentler than that of specimen SWS2. It shows that concrete-filled steel tubular frames with circular cross-sections are more effective than concrete-filled steel tubular frames with rectangular cross-sections on providing stable stiffness for the specimens.

**Calculation formula of normal section bearing capacity of a shear wall with concrete-filled steel tubular frames and slits**

The experimental results show that specimen failure mode is of the flexure type. According to the form and stress characteristics of specimens, the following basic assumptions are made to calculate the normal section bearing capacity of specimens:

- (1) Conforming to the plane section assumption.
- (2) The stiffness of top beam is infinite.
- (3) The friction between the wall and the thin board is not considered.
- (4) The tensile force and dowel action of horizontal web reinforcement in the vertical slit is not considered.
- (5) The tensile action of concrete in tension zone is not considered.
- (6) The longitudinal web reinforcement in compression zone tends to be buckled, so the compressive contribution is not taken into account.

As shown in Figure 7, two vertical slits are used to divide the wall into three small wall limbs.

*Fig. 7 - Classification of small wall limbs*
According to the above basic assumptions, the inflection point of the three small wall limbs will appear at the midpoint. Each half of each small wall limb was taken as the research object. The forced state is shown in Figure 8. It is found that the forced state on each small wall is different and need to be calculated separately.

\[ qh = f_c t x + f_s' A_s' + F_v - f_a A_a - N_{SW} \]  
\[ V_1 \frac{t_w}{2} + \frac{1}{2} f_c t x^2 = \frac{q h^2}{2} + f_a A_a \left( h - \frac{h_1}{2} \right) + N_{SW} \frac{h-h_1+1.5x}{2} \]  
\[ qh = f_c t x + f_s A_s - f_s A_s - N_{SW} \]  
\[ V_2 \frac{t_w}{2} + \frac{1}{2} f_c t x^2 = \frac{q h^2}{2} + f_s A_s \left( h - \frac{h_1}{2} \right) + N_{SW} \frac{h-h_1+1.5x}{2} + F_v h \]  
\[ qh = N_c + f_a A_a' - F_v - f_s A_s - N_{SW} \]  

In the above equations, \( F_v = 0.2 f_c h_s \) \[8\]; \( N_{SW} = f_y \rho_w t (h - h_1 - 1.5x) \)

When the steel tube is square steel tube, \( N_c = f_c t x \)

When the steel tube is round steel tube, \( N_c = N_{c1} + N_{c2} \); \( N_{c1} = f_c t(x - h_1) \)

\[ N_{c2} = (1 + 1.8 \theta_t) f_c A \]

\( \theta_t \) is confinement index.

When the steel tube is square steel tube,

\[ V_3 \frac{t_w}{2} + \frac{1}{2} f_c t x^2 = \frac{q h^2}{2} + f_s A_s \left( h - \frac{h_1}{2} \right) + N_{SW} \frac{h-h_1+1.5x}{2} + F_v h \]  

When the steel tube is round steel tube,
All of the above equations, $h$ is the width of a small wall limb; $f_c$ is the test value of axial compressive strength, $f_c = 0.76 f_{cu}$; $t$ is the thickness of the wall; $f_y$ is the tensile strength of longitudinally web reinforcement; $f'_s$ is the compressive strength of 4mm diameter cold drawn low carbon wire; $A'_s$ is the area of 4mm diameter cold drawn low carbon wire; $h_s$ is the sum of the concrete height at the vertical slit; $f_a$ is the tensile strength of the steel tube; $A_a$ is the area of the tension steel tube; $f_s$ is the tensile strength of 4mm diameter cold drawn low carbon wire; $A_s$ is the area of 4mm diameter cold drawn low carbon wire; $\rho_w$ is the longitudinal reinforcement ratio; $h_1$ and $h_2$ are the width of tension column and compressed concealed column in wallboard column respectively; $f'_a$ is the compressive strength of the steel tube; $A'_a$ is the area of the tensioned steel tube.

Combined with the analysis of the three small wall limbs can be seen, the calculation formula of normal section bearing capacity of a shear wall with concrete-filled steel tubular frames and slits should be:

$$V = V_1 + V_2 + V_3$$

**CONCLUSIONS**

(1) The experimental results show that specimen failure mode is of the flexure type. Compared with SWS1, specimens with concrete-filled steel tubular frames exhibited more dense and homogeneous distribution of cracks. Cracks can be seen in the entire wall. The energy dissipation capacity of the concrete wall of specimens with concrete-filled steel tubular frames was more fully utilized.

(2) Compared with ordinary reinforced concrete shear wall specimen with seams, the shear wall specimens with concrete-filled steel tubular frames and seams show a better performance in load-carrying capacity, ductility, stiffness degradation and energy dissipating capacity. The reason is that seismic behaviour of shear wall specimens has been improved significantly by concrete-filled steel tubular frames and artificial vertical seams.

(3) The concrete-filled steel tubular frames with circular cross-sections are more effective than concrete-filled steel tubular frames with rectangular cross-sections on improving seismic behaviour of shear wall specimens, although steel consumption of the former frames is less.

(4) This paper proposed calculation formula of normal section bearing capacity of a shear wall with concrete-filled steel tubular frames and slits. By comparing the calculated values with the test value, it can be found that the calculated values are basically consistent with the test value. Relative error is less than 10%.

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