Experimental and analytical investigation of semi-rigid CFST frames with external SCWPs

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ARTICLE INFO

Keywords:
Concrete-filled steel tubular (CFST) Seismic behavior
Blind bolts
Finite element (FE) analysis

ABSTRACT

An experimental and numerical research on the seismic performance of semi-rigid concrete-filled steel tubular (CFST) frames with external sandwich composite wall panels (SCWPs) was reported. Four specimens of semi-rigid CFST frames with external SCWPs and one specimen of pure semi-rigid CFST frame subjected to low-cyclic loading were conducted. Failure modes, horizontal load versus displacement relation curves were analyzed. The test specimens exhibited good hysteretic behavior, energy dissipation and ductility. Finite element (FE) analysis modeling was developed and the results obtained from the FE model matched well with the experimental results. Extensive parametric studies have been carried out to investigate the effect of steel strength, column slenderness ratio and steel wire diameter of wall, etc. on the strength and stiffness of the typed composite frames. The opening ratio and location of the SCWPs were also discussed. The experimental study and numerical analysis will provide the scientific basis for design theory and application of the SCWPs in fabricated steel structure building.

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1. Introduction

Widespread and un anticipated brittle fractures occurred in welded steel beam-column connections in the 1994 Northridge and the 1995 Kobe earthquakes. To avoid extensive welding and the required high tolerance, the static and seismic behavior of blind bolted joints to CFST columns has been studied by experiments and FE analysis, such as Mirza and Uy [1], Lee et al. [2], Wang et al. [3–6], Ataei et al. [7], and Wang et al. [8]. However, little studies focused on the semi-rigid concrete-filled steel tubular (CFST) frames with external sandwich composite wall panels (SCWPs).

The development and construction of lightweight pre-fabricated sandwich panels in building construction are a growing trend in China due to its high strength, reduced weight, good thermal insulation, money-saving and better fire resistance, etc. The SCWPs studied in this paper consisted of two outside layers separated by an insulation layer. The outside layers were constructed of precast concrete and the core layer was polystyrene foam. Diagonal steel wire with alternating direction was welded to steel wire mesh which embedded into each concrete layer to form space truss connectors. The SCWPs could be prefabricated in factory and assembled in spot and speed construction schedule up to maximum extent. However, previous earthquake damage surveys indicated that the wall destruction and collapse caused large casualties and property losses and scare studies have been done to investigate failure mechanism of composite frame with SCWPs. Thus, the cooperative work and failure modes of semi-rigid CFST frames with external SCWPs under the earthquake action are a research topic with a high priority.

Up to now, a great deal of study on the seismic behavior and interaction between the panels or blocks and H-shaped steel frame have been conducted. Flanagan et al. [9] tested nine steel frames with hollow clay tile under in-plane loading; the experimental results showed that all infills failed by corner crushing. Markulak [10] reported the hysteretic behavior of steel frames infilled with three different masonry infill types: perforated clay blocks, lightweight AAC blocks and newly proposed combination of these materials. Tasnimi et al. [11] conducted six brick-infilled steel frames with openings. Moghadam [12] carried out eleven experimental tests on masonry and concrete infilled steel frames. Fang et al. [13] completed a shaking table test of a full-scale steel frame with ALC external panels. Tong et al. [14] and Sun et al. [15] investigated seismic behavior of the semi-rigid steel frame with RC infill walls. Matteis and Landolfo [16] investigated the behavior of sandwich panels inserted into a pin-jointed frame systems. Hou et al. [17] described a cyclic loading test results of H-shaped steel frames with sandwich composite panels.

Apart from experimental research, many accurate theoretical models and FE analysis on the behavior of steel frames with various infill walls have been proposed. Saneinejad and Hobbs [18] proposed a method to predict the strength and stiffness of concrete or masonry infilled steel frames. Dawe et al. [19] set up a series of complex...
calculation model for steel frames with masonry infilling walls. Doudoumis [20] simulated single-bay, single-story infilled frames through a precise FE micromodel to investigate the elastic behavior of infilled frames. Moghaddam [12] proposed an approximate method based on FE analysis to evaluate cracking strength and shear resistance of steel frames with masonry or concrete infills. Chen and Liu [21] developed a FE model to study the effect of opening size and opening location on the seismic behavior of masonry infilled steel frames. Matteis [22] reported a profitable methodology to predict seismic response of lightweight sandwich shear walls infilled jointed steel frames and accounted for the contributing effect to the structural behavior by using ABAQUS.

These above-mentioned experimental studies and numerical analysis mainly focused on the behavior of rigid, semi-rigid or hinged H-shaped steel frame with infilled various type of masonry, brick, RC walls and ALC panels, etc. In addition, few scholars studied the complicated interaction of composite structure with ALC walls. Wang et al. [23] investigated the rigid circular CFST frames with ALC panels under in-plane cyclically increasing lateral loads by five specimens, the failure modes, hysteretic curves, energy dissipation and ductility were analyzed. However, little literature on the experiment and numerical analysis of semi-rigid CFST frames with SCWPs has been reported.

This paper is to investigate the seismic performance of semi-rigid CFST frames with external SCWPs. Four specimens of semi-rigid CFST frames with external SCWPs and one specimen of pure semi-rigid CFST frame were tested under cyclic loading. Moreover, the FE program ABAQUS was applied in the analysis. Comparisons between the FE analytical results and the experimental results indicated that the FE model could well predict the horizontal load versus corresponding displacement relations of the semi-rigid CFST frames with SCWPs. In addition, twelve parameters were also completed to investigate the effect of variation of parameters on the structure performance, such as steel strength, column slenderness ratio, steel wire diameter on walls and column axial load level. The opening ratio and location of the SCWPs were also discussed.

2. Experimental program

2.1. Test specimens

In order to explore the effect of the wall concrete type, the wall connection type, the brace setting and the wall setting on the seismic behavior and failure modes of the typed composite structure, four specimens of semi-rigid CFST frames with external SCWPs and one specimen of semi-rigid CFST frame were tested and analyzed in this paper. Details of the specimen are illustrated in Fig. 1. The columns for all specimens are concrete-filled square steel tubes with a cross section of $150 \times 150 \times 6$ mm and its length is 1785 mm. The self-compacting concrete was filled in the square steel columns. The beams were designed as hot-rolled H-shaped steel section with a cross-section of $150 \times 75 \times 5 \times 7$ mm and the length of the beams is 1350 mm. The steel beams and columns were fabricated through extended end plate connections with blind bolts which were two rows of M16 Grade 10.9 high strength bolts. The ratio of the yielding strength to the ultimate strength of the bolt is 0.9. All bolts for the beam-to-column connections were finally tightened to a torque of 228 Nm according to specification GB50017 [24].

The extended end plate was fastened to square steel tubes by blind bolts with hooked extensions into the concrete core for the purpose of reducing the deformations of the tube wall and the end plate, as seen in Fig. 2. The hook-extended type to the bolt is high strength reinforcing rebar with 16 mm in diameter, 70 mm in horizontal length and 35 mm in hooked length and the yield strength of the extensions is of grade 335 N/mm². The high strength bolt welded with a hooked reinforcing rebar to form a complete unit. Gardner and Goldsworthy [25, 26] concluded that the hooked anchorage welded to the blind bolts into CFST column could obviously improve the strength and the initial stiffness of the joints.

The sandwich composite wall panel is a three-layer element comprising of two outside layers and one core insulation layer. The outside layers were constructed of precast concrete and the core layer was...
Blind bolts
Extended end plate
Square CFST column
$150 \times 150 \times 6$
Steel beam
HN$150 \times 75 \times 5 \times 7$

M$16$ Grade$10.9$

(a) Extended end plate connection
(b) Extended end plate

Fig. 2. Detail of blind bolted end plate joint (unit: mm).

(a) Wall set with embedded part
(b) Wall set with embedded part and steel braces

(c) Wall section A-A
(d) Wall section B-B

Fig. 3. Detail of sandwich composite wall panels (unit: mm).
polystyrene foam. A square welded steel wire mesh of 3 mm diameter bars with 50 × 50 mm openings was established as the longitudinal and transverse reinforcement and embedded in the two outside layers. The connectors were made of 3 mm diameter steel bar and welded to steel wire mesh with an inclined angle of 45° to a horizontal line. Besides, connectors were welded along with alternating direction and over a distance of 100 mm. These connectors running the full height of the panels were used to tie the two outside concrete layers to form space truss connectors and to provide shear transfer between the layers. Fig. 3 illustrates the details of SCWPs. The embedded parts were pre-buried at the four corners of SCWPs. The connecting plates were welded on the column flange or beam flange. Then, the SCWPs were in assembly to semi-rigid CFST frames through bolts drawn into the embedded part and connecting plate to form complete structures, as shown in Fig. 1.

A summary of the test specimens is represented in Table 1. The wall connection type of WSF1, WSF2 and WSF4 was that the SCWPs was connected with both beams and columns, however, specimen WSF3 was that the SCWPs was connected with columns. Meanwhile, a pair of steel braces with a width of 150 mm and a thickness of 15 mm were set on specimen WSF4. Specimen WSF5 was a pure blind-bolted CFST frame without SCWPs. In addition, ceramsite concrete was casted in the specimen WSF2, ordinary concrete was used in specimens WSF1, WSF3 and WSF4.

2.2. Experimental setup and loading protocol

The arrangement of the experimental setup is illustrated in Fig. 4 and pictures of test site for some specimens were shown in Fig. 5. One hydraulic actuator of 1000 kN in capacity was employed to apply the load to the column end and was controlled to simulate seismic loading. The actuator mounted between the specimen and RC reaction wall. Eight anchor bolts and four hydraulic jacks were set on the steel ground beam to transfer the shear forces from the specimen to the ground floor and prevented slip displacement. Due to the limitation of loading conditions and consideration of safety, no vertical load was applied on the CFST column during the test.

For investigating the behavior of the semi-rigid CFST frames with external SCWPs and make experiments going smoothly, the preloading stage and the formal loading stage were applied during the loading process. In the preloading stage, the hydraulic actuator pushed +5 mm to the structures and after that dropped to zero, and then pulled −5 mm on the frames and after a while reduced to zero again. Repeating this process twice ensures that the apparatus was working well in subsequent experimental procedures. In the formal loading stage, the displacement control mode was applied in accordance with ATC-24 [27] guidelines until test specimens damaging or having larger deformations. The displacement of specimen corresponding to the yield load (0.7P_{max} and P_{max} is assumed as theoretical value of maximum load of specimens) is defined as the theoretical yield displacement (Δ_γ). The theoretical yield displacement Δ_γ and ultimate loading capacity P_{max} were determined based on finite element analysis. And the Δ_γ was determined as 12 mm during the loading process. Two cycles were employed to specimen at each displacement level of 0.25Δ_γ, 0.5Δ_γ and 0.7Δ_γ; three cycles were employed to specimen at each displacement level of 1Δ_γ, 1.5Δ_γ and 2Δ_γ; two cycles were imposed at each displacement level of 3Δ_γ, 5Δ_γ, 6Δ_γ, 7Δ_γ.

2.3. Material properties

The measured steel coupons applied in the specimens were listed in Table 2. Tensile coupons cut from steel tubes and sheets (used in beams, columns, end plates, connecting plates, embedded parts and steel wire) were conducted to obtain their yield stress (f_y), ultimate stress (f_u), modulus of elasticity (E_s) and elongation at fracture (δ_u). The testing yield stress and ultimate strength of the Grade 10.9 M16 blind bolts were measured as 928 N/mm^2 and 1015 N/mm^2, respectively.

Columns of all specimens were filled with self-compacting concrete and ordinary concrete or ceramsite concrete were utilized on the SCWPs. For obtaining the material properties of concrete used in the test, each group of tests had three concrete cubes with size of 150 × 150 × 150 mm for compressive strength and 100 × 100 × 300 mm for elastic modulus. The compressive strength of core concrete in steel tube was 43.31 N/mm^2 at 28 days and the modulus of elasticity was 33,015 N/mm^2. The compressive strength of ordinary concrete used in the SCWPs was found to be 32.53 N/mm^2 at 28 days and the modulus of elasticity was 30,611 N/mm^2. The compressive strength of ceramsite concrete was 51.50 N/mm^2 at 28 days and the modulus of elasticity was 34,797 N/mm^2, as shown in Table 3.

3. Finite element modeling

3.1. General descriptions

In order to deeply study the mechanical behavior and failure modes of semi-rigid CFST frames with external SCWPs, the key factors for modeling the typed structure include the material constitutive law, connection between elements, element type and mesh, boundary conditions and solution method.

3.2. Material modeling of steel

The constitutive relation of steel was developed by elastic-plastic model in software ABAQUS. A simplified bilinear stress-strain curve was introduced to the high strength blind bolt with hooked extension and the steel wire and the hardening stage modulus was determined as 0.01E_s where E_s is the measured elastic modulus of steel. A multi-linear stress-strain relationship was adopted to steel beam, steel tube, end plate, connecting plate and embedded part. The first part of the multi-linear stress-strain relationship represents the elastic part with a measured elastic modulus (E_s) and the rest of model for steel assumes associated plastic flow. Abdel-Rahman and Sivakumaran [28] proposed a multi-linear stress-strain relation model to simulate accurately mechanical behavior of cold-formed steel tubes, where a cold-formed cross section was divided into two zones: a corner zone and a flat zone. Sully and Hancock [29] concluded that a residual stress of 0.4f_y could be applied in the corner zone and a residual stress of (0.24–0.0006B)f_y be used for the flat zone (where f_y is the yield strength in unit MPa and B is the thickness of the steel tube in unit mm, respectively).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Column section B × D × t (mm)</th>
<th>Beam section h_0 × t_0 × b_1 (mm)</th>
<th>Wall concrete type</th>
<th>Connection type</th>
<th>Brace setting</th>
</tr>
</thead>
<tbody>
<tr>
<td>WSF1</td>
<td>□ 150 × 150 × 6</td>
<td>H150 × 75 × 5 × 7</td>
<td>Ordinary concrete</td>
<td>Connect with beams and columns</td>
<td>–</td>
</tr>
<tr>
<td>WSF2</td>
<td>□ 150 × 150 × 6</td>
<td>H150 × 75 × 5 × 7</td>
<td>Ceramsite concrete</td>
<td>Connect with beams and columns</td>
<td>–</td>
</tr>
<tr>
<td>WSF3</td>
<td>□ 150 × 150 × 6</td>
<td>H150 × 75 × 5 × 7</td>
<td>Ordinary concrete</td>
<td>Connect with columns</td>
<td>–</td>
</tr>
<tr>
<td>WSF4</td>
<td>□ 150 × 150 × 6</td>
<td>H150 × 75 × 5 × 7</td>
<td>Ordinary concrete</td>
<td>Connect with beams and columns</td>
<td>Steel braces</td>
</tr>
<tr>
<td>WSF5</td>
<td>□ 150 × 150 × 6</td>
<td>H150 × 75 × 5 × 7</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>
3.3. Material modeling of concrete

The concrete damaged plastic model could be used to simulate the concrete mechanical properties under cyclic or monotonic loads through the ABAQUS library. The model enables to input a multi-linear uniaxial compressive stress-strain curve. Due to the confinement of the steel tube wall, the transverse deformation of concrete was limited and the initial defect of concrete was slowed down. The constitutive law of the concrete filled in the steel tube and the concrete used in the SCWPs were different. For the concrete filled steel tube, a practical and accurate model is proposed by Han et al. [30]. The compressive stress-strain curve of core concrete is expressed as follows:

\[
y = \begin{cases} 
\frac{2x - x^2}{\chi} & (x \leq 1) \\
\frac{\beta(x-1)^\eta + x}{\chi} & (x>1)
\end{cases}
\]

where \( \chi = \varepsilon_0/\varepsilon_0, \) \( \beta = \sigma_0/\varepsilon_0 = 8000^{0.2} \times 10^{-6}, \) \( \varepsilon_c = \begin{cases} 0.005 + 1.25f_c' \times 10^{-6} & \text{for CFST}, \\
0.005 + 1.5\gamma & \text{for square CFST}, \end{cases} \) \( \beta = (f_{fc,c})^{1/2}/(A_p f_{fc,c}), \) \( A_p, \) and \( A_c, \) are respectively the cross-sectional areas of the steel tube and core concrete; \( f_p \) and \( f_{fc,c} \) are respectively the yield strength of the steel tube and the characteristic strength of the concrete. The elastic modulus and Poisson’s ratio were equal to 4730 \( \sqrt{f_{fc,c}} \) and 0.2, respectively, which was determined by ACI 318 [31]. Where \( f_{fc,c} \) denotes the cylinder compressive strength of concrete. The relationship between cube compression strength of concrete and cylinder compression strength of concrete is given by CEB-FIP [32]. For the core concrete in tensile condition, the tension stress is supposed to increase linearly with respect to strain until the concrete cracking, and then the tension stress reduces proportionally to zero at a strain which is ten times of the strain at concrete cracking. In addition, the damaged plastic model proposed by specification GB50010-2010 [33] was applied to simulate the compressive and tensile behavior of ordinary concrete or ceramsite concrete casted on the SCWPs. It could better reflect a certain degree of concrete damage by introducing damage factor in the process of compressing or tensioning. The uniaxial compressive stress-strain curve of concrete used in SCWPs was defined as follows:

\[
\sigma = (1 - d_c)E_c \cdot \varepsilon
\]

\[
d_c = \begin{cases} 
1 - \frac{\rho_c n}{m - 1 + x^2} & (x \leq 1) \\
1 - \frac{\rho_c}{\alpha_c(x-1)^2 + x} & (x>1)
\end{cases}
\]

where \( \rho_c = f_{fc,c}/(E_c \cdot \varepsilon_{c,cr}), n = (E_c \cdot \varepsilon_{c,cr} - f_{fc,c})/E_c \cdot \varepsilon_{c,cr} - f_{fc,c}, \) \( x = \varepsilon/\varepsilon_{c,cr}, \) \( f_{fc,c} \) can be defined as \( f_{fc,c} \) and equaled to 20.1 MPa and 32.4 MPa, respectively, for ordinary concrete and ceramsite concrete according to GB50010-2010 [33]. The \( \alpha_c \) and \( \varepsilon_{c,cr} \) equaled to 0.74 and \( 1470 \times 10^{-6} \) for ordinary concrete.
concrete; the $c_c$ and $e_c$ equaled to 1.50 and $1680 \times 10^{-6}$ for ceramsite concrete, according to GB50010-2010 [33]. The uniaxial tensile stress-strain curve of concrete used in SCWPs was defined as follows:

$$\sigma = (1-d_i)E_c \varepsilon$$

(4)

where $d_i = \begin{cases} 1 - \rho_1 \left[ 1.2 - 0.2\varepsilon \right]^2 \times 1 \\ 1 - \frac{\rho_2}{\alpha_c (x-1)} + x \end{cases}$

(5)

where $\rho_1 = f_{cu} / (E_c e_{cu})$, $x = \varepsilon / e_{cu}$. The $f_{cu}$ can be defined as $f_{cu}$ and equaled to 2.01 MPa and 2.64 MPa, respectively, for ordinary concrete and ceramsite concrete according to GB50010-2010 [33]. The $c_c$ and $e_{cu}$ equaled to 1.25 and $95 \times 10^{-6}$ for ordinary concrete; the $c_c$ and $e_{cu}$ equaled to 2.19 and $110 \times 10^{-6}$ for ceramsite concrete, according to GB50010-2010 [33].

3.4. Numerical model description

Based on the experimental specimens, finite element (FE) analysis of the semi-rigid CFST frames with external SCWPs was developed and analyzed. A fine mesh of 3D8R is used for all steel parts and concrete part described above and the three-dimensional two-node linear element (T3D2) is used to simulate steel wire truss applied in SCWPs. A reasonable mesh density was ensured by meshing test, typical meshes of composite frame. The friction coefficient between steel and concrete is taken as 0.45 according to GB50017-2003 [24]. Many scholars studied the interaction between SCWP and steel beam web; the contact between steel tube and core concrete, the contact between bolt shank and bolt holes on the steel tube and extended end plate, the contact between bolt nut and steel tube, the contact between embedded part and connecting plate. The continuity and convergence were taken into account on the surface contact. The friction coefficient between steel components is taken as 0.45 according to GB50017-2003 [24]. The friction coefficient between the two components was respectively taken as 0.2, 0.25, 0.3, 0.47 and 0.6 to coincide better with each test result [34–38]. The friction coefficient between steel tube and core concrete was calculated as 0.25, 0.47, 0.6 and 0.8 for the finite element model of specimen WSF1 to investigate the effect of friction coefficient between steel and core concrete on the overall behavior of the typed composite frame. The FE calculation results demonstrated that FE model with different friction coefficient marginally affected the horizontal load versus displacement of the typed composite frame. The friction coefficient between steel and concrete; the $c_c$ and $e_c$ equaled to 1.50 and $1680 \times 10^{-6}$ for ceramsite concrete, according to GB50010-2010 [33].

Table 2

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Thickness (mm)</th>
<th>Yield stress (N/mm²)</th>
<th>Ultimate stress (N/mm²)</th>
<th>Young's modulus (N/mm²)</th>
<th>Elongation at fracture (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel tube</td>
<td>6</td>
<td>361.3</td>
<td>492.3</td>
<td>1.97 x 10⁶</td>
<td>17.5</td>
</tr>
<tr>
<td>Steel beam flange</td>
<td>7</td>
<td>371.5</td>
<td>508.3</td>
<td>2.01 x 10⁶</td>
<td>17.4</td>
</tr>
<tr>
<td>Steel beam web</td>
<td>5</td>
<td>308.5</td>
<td>498.2</td>
<td>2.14 x 10⁶</td>
<td>16.9</td>
</tr>
<tr>
<td>End plate</td>
<td>16</td>
<td>364.4</td>
<td>474.4</td>
<td>2.08 x 10⁶</td>
<td>22.8</td>
</tr>
<tr>
<td>Steel brace</td>
<td>8</td>
<td>377.2</td>
<td>482.9</td>
<td>1.85 x 10⁶</td>
<td>18.6</td>
</tr>
<tr>
<td>Connecting plate</td>
<td>12</td>
<td>341.2</td>
<td>452.6</td>
<td>2.15 x 10⁶</td>
<td>19.4</td>
</tr>
<tr>
<td>Embedded part</td>
<td>12</td>
<td>354.2</td>
<td>487.6</td>
<td>2.08 x 10⁵</td>
<td>20.1</td>
</tr>
<tr>
<td>Steel wire</td>
<td>3</td>
<td>461.6</td>
<td>592.5</td>
<td>1.88 x 10⁵</td>
<td>18.2</td>
</tr>
</tbody>
</table>

Table 3

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Specimen dimension (mm)</th>
<th>Age (day)</th>
<th>$f_{cu}$ (N/mm²)</th>
<th>$E_c$ (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>150 x 150 x 150</td>
<td>28</td>
<td>45.26</td>
<td></td>
</tr>
<tr>
<td>C2</td>
<td>150 x 150 x 150</td>
<td>28</td>
<td>41.96</td>
<td></td>
</tr>
<tr>
<td>C3</td>
<td>150 x 150 x 150</td>
<td>28</td>
<td>42.71</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td>43.31</td>
<td>33,015</td>
</tr>
<tr>
<td>C1</td>
<td>150 x 150 x 150</td>
<td>28</td>
<td>32.78</td>
<td></td>
</tr>
<tr>
<td>C2</td>
<td>150 x 150 x 150</td>
<td>28</td>
<td>33.31</td>
<td></td>
</tr>
<tr>
<td>C3</td>
<td>150 x 150 x 150</td>
<td>28</td>
<td>31.51</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td>32.53</td>
<td>30,611</td>
</tr>
<tr>
<td>C1</td>
<td>150 x 150 x 150</td>
<td>28</td>
<td>50.22</td>
<td></td>
</tr>
<tr>
<td>C2</td>
<td>150 x 150 x 150</td>
<td>28</td>
<td>52.98</td>
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<tr>
<td>C3</td>
<td>150 x 150 x 150</td>
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<td>51.31</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td>51.50</td>
<td>34,797</td>
</tr>
</tbody>
</table>

Note: C1, C2 and C3 represent material test of self-consolidating core concrete, ordinary concrete and ceramsite concrete, respectively.

Fig. 6. FE analysis model.
tube and core concrete was taken as 0.6 according to Han et al. [39] and coincided better with the test results. The friction coefficient between steel and core concrete on other models of tested specimens and parametric study models were all 0.6. The surface of bolt extension and core concrete was tied to approximately simulate the provision of hooked anchorage on the blind bolts into core concrete. The contact between embedded part and wall concrete was tied to couple fully. The contact between embedded part and wall concrete was tied to couple fully. The surface of bolt extension and core concrete was tied to approximately simulate the provision of hooked anchorage on the blind bolts into core concrete. The contact between embedded part and wall concrete was tied to couple fully. For the welding between steel elements and the contact between the outside concrete layers and foam-core layer, all degree of freedom for the same nodes at the contact surface were tied to couple fully. The steel wire truss was embedded in the SCWPs based on the embedding principle.

The bottom surface of the CFST columns was fixed against all degrees-of-freedom to simulate the spot condition of experimental specimens. The horizontal load is transmitted to the typed frame by a rigid block whose elastic modulus is $1 \times 10^5$ N/mm$^2$ and Poisson’s ratio is 0.0001. Two steps were set to complete the FE analysis, pre-tightening force of 155 kN was imposed on the high strength blind bolt at the beam-to-column joints at the first step, then, a horizontal displacement was loaded on the column end at the second step.

4. Experimental and numerical results

4.1. Failure modes

The observed and numerical failure modes of the specimen were compared, as illustrated in Fig. 8. The failure modes that occurred in the test process are matched well with the numerical results. Cracks occurred on the wall surface around the embedded parts and then penetrated to the opposite side of wall to form slitted cracks with the increasing of horizontal load. The crushing of concrete around the embedded parts was also observed due to larger horizontal displacement and connection rotation. Local buckling occurred on the beam flanges and webs at the beam end. For the specimen WSF2 namely that the semi-rigid CFST frame with external precast ceramsite concrete SCWPs and the SCWPs is connected with beams and columns, the number and distribution of cracks on the precast ceramsite concrete SCWPs are less than that of the precast ordinary concrete SCWPs of specimen WSF1. For the specimen WSF3 namely that the semi-rigid CFST frame with external precast ordinary concrete SCWPs and the SCWPs is connected with columns, the number and distribution of cracks on the...
Fig. 8. Observed and predicted failure modes.
SCWPs are more than that of the SCWPs connected with beams and columns (WSF1). For the WSF4 that SCWPs is set with diagonal braces, the cracks propagated to the middle part of the SCWPs and stress concentration around the connections can be eased.

From Fig. 8, it can be found that the damage Mises stress of WSF2 was higher than that of WSF1, WSF3, WSF4 because of the usage of ceramsite concrete in SCWPs for WSF2. While comparing Fig. 8(a) with Fig. 8(d), the damage Mises stress WSF4 was lower than that of WSF1 and WSF3, though the steel braces were set on specimen WSF4 and the ultimate bearing capacity was higher than that of WSF1 and WSF3, deformation of the SCWPs was restricted and the SCWPs showed as more brittle and these phenomena can also be found in the test. For specimen WSF5, buckling deformation at the beam top flange and bottom flange were shown in Fig. 8(e). The FE analysis agreed reasonably with the test results in term of the location of the local buckling and the magnitude of the buckling deformation. All specimens behaved in an acceptable manner and the test of specimens WSF1, WSF2, WSF3, WSF4 were stopped when the signs of severe crush and spalling of concrete occurred around the embedded parts and local buckling occurred on the beam end, while the specimen WSF5 was stopped because that beam flange fractured partly and too large overall deformation occurring on the frame to resist horizontal load. No unexpected failures occurred in the test and FE process. It could be suggested that a generally good consistency is reached between the tested and predicted results.

4.2. Loading-displacement relationship

The horizontal loads and corresponding displacements were used to establish the horizontal load-displacement curves for the typed structures, as shown in Fig. 9. Fig. 9 shows the effects of the wall concrete type, the wall connection type, the steel brace setting and the wall setting on the load-displacement curve for the typed structure. The ultimate strength and elastic stiffness of the semi-rigid CFST frames with external SCWPs significantly improved when compared with pure semi-rigid CFST frame. When the SCWPs connected with beam and

![Fig. 9. Load (P)-displacement (Δ) relationships of test specimens.](image)
column, compared with specimen WSF1 that the SCWPs used ordinary concrete, the ultimate strength of specimen WSF2 was improved obviously owing to its SCWPs using higher strength ceramsite concrete. When the SCWPs adopted the same ordinary concrete, specimen WSF1 which adopted SCWP-beam and column connection type possessed better integrity than that of specimen WSF3 which used SCWP-column connection type. Thus, the seismic behavior of specimen WSF1 was superior to specimen WSF3. The hysteresis loop of specimen WSF4 was fuller and without significant pinch, it showed that the ultimate strength of specimen WSF4 was improved when the wall set with diagonal steel braces. The seismic behavior of specimen WSF4 was better than that of specimen WSF1 for the reason that the steel braces shared certain earthquake action for the whole structure. Specimen WSF5 without SCWPs had good deformation capacity, but the ultimate strength and elastic stiffness were the smallest in all test specimens. The test results confirmed that the external SCWPs could dramatically improve the strength and stiffness of the CFST frames.

Fig. 9 also gives the comparisons between numerical and experimental load-displacement curves. The failure load (\(P_{f,t}\)) and the failure displacement (\(\Delta_{f,t}\)) of test are respectively the horizontal load and corresponding displacement when the load falls to 85% of maximum load; the failure displacement of verified numerical simulation equaled to the failure displacement (\(\Delta_{f,t}\)) of test, the failure displacement corresponding to the load is defined as the failure load of the verified numerical simulation. On the whole, the numerical analysis results were matched reasonably with the test results. The damage of corner panels and the local buckling of beams can be simulated. It is also noted that the elastic stiffness, ultimate strength and correspond displacement of the finite element models were relatively larger than that of the tested results for the reason that the manufacturing and processing defects of SCWPs and semi-rigid CFST frames were not considered on the FE model. The three layers of SCWP on FE model were tied together and not considered the real interaction between them. The FE theoretical analysis model developed in this paper could be used in subsequent numerical analysis of semi-rigid CFST frames with external SCWPs.

5. Parametric analysis

The FE modeling tools were used to present and discuss the results of parametric analysis on the effect of structural mechanical behavior and working mechanism in this section. The main factors that affect the performance of the typed structure and influence regularity were determined.

Based on the above-mentioned model, a parametric investigation of the main variables in a structure design of semi-rigid CFST frames with SCWPs was conducted to ascertain the discipline on the elastic stiffness, ultimate strength and corresponding displacement. Twelve parameters which may influence the seismic behavior of the typed structure were divided into three aspects: material parameters, geometric parameters and load parameters. Specific parameters were set as follows:

- Material parameters: steel strength \(f_y\) and concrete strength \(f_{cu}\) of SCWPs; and
- Geometric parameters: concrete layer thickness of walls \(t\), steel wire diameter \(d\), slenderness ratio of CFST column \(\lambda\), beam to CFST column linear stiffness ratio \(k_i\), steel ratio \(\alpha\), beam to CFST column yield strength ratio \(k_{0,i}\), end plate thickness \(t_{ep}\) and blind bolt diameter \(d_{b}\); and
- Load parameters: axial compression ratio of the CFST column \(n\) and blind bolt pretension force \(P\).

The characteristic calculating example which is commonly used in practical engineering is described as follows:

- Steel beam: H350 × 175 × 7 × 11 mm; span length: \(L = 6000\) mm; steel strength: \(f_y = 345\) N/mm².
- CFST column: steel tube \(\Theta300 \times 300 \times 10\) mm; column height: \(H = 3000\) mm; steel strength: \(f_y = 345\) N/mm²; concrete strength: \(f_{cu} = 60\) N/mm²; axial compression ratio: \(n = 0.6\).

SCWPs: thickness of the core polystyrene foam layer: \(t = 30\) mm, thickness of the outside concrete layer: \(t = 40\) mm.

Connection details: extended end plate: \(300 \times 600 \times 16\) mm; high strength blind bolt: 10.9 Grade M20; bolt extension: 75 mm length and 20 mm diameter.

In this paper, the slenderness ratio \(\lambda\) is defined as \(4H/B\), where \(B\) is the width of steel tube. The beam to CFST column linear stiffness ratio \(k_i\) is defined as \(i/b_i\), where \(i\) and \(b_i\) are the linear stiffness of steel beam and CFST column. The beam to CFST column yield strength ratio \(k_{0,i}\) is defined as \(M_0/M_i\), where \(M_0\) and \(M_i\) are the sectional yielding moment capacities of steel beam and CFST column by specification DB34/T 1262-2010 [40], respectively. The steel ratio \(\alpha\) is defined as \(A_i/A_0\), where \(A_0\) and \(A_i\) are the net cross-sectional areas of core concrete and steel tube, respectively. The axial compression ratio \(n\) is defined as \(N/N_0\), where \(N\) and \(N_0\) are the compressive load exerted on the CFST column and the design value of axial compression bearing capacity of the CFST column by DB34/T 1262-2010 [40], respectively. \(\lambda, a, k_i\) and \(k_{0,i}\) for this typical model are respectively 35, 0.15, 0.5, 0.6. For the FE models with various parameters, the failure load and corresponding displacement is the maximum point on each horizontal load versus displacement curve.

5.1. Effect of steel strength \(f_y\)

In order to investigate the effect of steel strength on the behavior of the typed structure, four kinds of strength grade of steel commonly employed in the engineering which are 235, 345, 420, 550 MPa in this study. The recorded horizontal load versus horizontal displacement \(P-\Delta\) relationship was compared with change of the steel yield strength, as seen in Fig. 10(a). It could be observed that the elastic stiffness and ultimate strength were increased dramatically with the increase of steel yield strength. When the steel yield strength of the structure was set to 345, 420, 550 MPa, the elastic stiffness of the frame was raised by 19.5%, 24.7%, 28.6%, respectively, compared with the frame using the yield strength 235 MPa and the corresponding values of the ultimate strength was increased by 23.5%, 57.1%, 77.3%, respectively. When steel strength \(f_y \leq 420\) MPa, the ultimate strength of the structure increases greatly when increasing the steel strength, whereas the incremental amplitude of the ultimate strength is slow down when \(f_y > 420\) MPa.

5.2. Effect of concrete strength of SCWPs \(f_{cu}\)

Fig. 10(b) shows the effect of concrete strength \(f_{cu}\) of SCWPs on the \(P-\Delta\) curves for the structure. The cube compressive strengths of concrete are fit to 30, 60, 80, 100 MPa in FE analysis to explore this changes. The results of the comparisons showed that both the stiffness and strength of the structure are dramatically affected by concrete strength \(f_{cu}\) casted on SCWPs. The elastic stiffness and ultimate strength were increased dramatically with the increase of concrete strength \(f_{cu}\). When the concrete strength \(f_{cu}\) of the structure was set to 60, 80, 100 MPa, the elastic stiffness of the frame was raised by 4.5%, 25.2%, 52.3%, respectively, compared with the frame using the concrete strength \(f_{cu} = 30\) MPa and the corresponding values of the ultimate bearing capacity was also increased by 15.5%, 18.6%, 26.8%, respectively.

5.3. Effect of the concrete layer thickness of SCWPs \(t\)

The effect of the thickness of concrete panel where \(t = 30, 40, 50\) mm was carried out. Fig. 10(c) displays a comparison of the \(P-\Delta\) curves of the structure, indicating that the ultimate strength increases slightly and the elastic stiffness remains basically constant with increasing the thickness of concrete panel. This result demonstrates that both the ultimate strength and elastic stiffness of the structure are not notably affected by the outside concrete layer thickness.
5.4. Effect of steel wire diameter of SCWPs (d)

The change of steel wire diameter of SCWPs could affect the ultimate strength of the structure. Three different diameters are used to plot the horizontal load \( P \) versus horizontal displacement \( \Delta \) relationship, as displayed in Fig. 10(d). It demonstrates that the ultimate strength increases greatly with an increase in the steel wire diameter of SCWPs. While the steel wire diameter had almost no influence on the stiffness of the structure. When the diameters are 6, 14 mm, the ultimate strength is raised by 12.2%, 34.9%, respectively, compared to the SCWPs using 3 mm diameter steel wire.

5.5. Effect of slenderness ratio of CFST column (\( \lambda \))

The study of the impact of slenderness ratio of CFST column where \( \lambda = 20, 40, 60 \) was implemented. By changing the height of the column to achieve the goal of the variation of slenderness ratio in the finite element models. Fig. 10(e) provides a comparison of the horizontal load versus horizontal displacement \( (P-\Delta) \) relationship with the change of slenderness ratio. It is found that the slenderness ratio had significant influence on ultimate strength and elastic stiffness, both ultimate strength and elastic stiffness of the structure dramatically decrease as the \( \lambda \) increase.

5.6. Effect of beam to CFST column linear stiffness ratio (\( k_i \))

Three different beams to CFST column linear stiffness ratios were adopted in the process of simulation which included 0.25, 0.5, 1 and then the horizontal load-displacement curves are depicted, as shown in Fig. 10(f). It is shown that the ultimate carrying capacity is affected greatly by the parameter \( k_i \), but only a minor influence exhibited on the elastic stiffness of the structure. When the beam to CFST column linear stiffness ratio

Fig. 10. Effect of various parameters on P-\( \Delta \) relations for the structures.
linear stiffness ratio of the structure was arranged for 0.5, 1, the ultimate strength of the frame was enhanced by 10.3%, 14.5%, respectively, compared with the $k_i$ equaling to 0.25. The reason for the result of the simulations was that the constraint from beam to column would be enhanced with the increasing of $k_i$.

5.7. Effect of steel ratio ($a$)

The steel ratio of the CFST column cross-section is a significant design parameter in practical engineering. By changing the steel tube thickness to achieve different steel ratio and the FE modeling with steel ratio of 0.05, 0.1, 0.15. Fig. 10(g) illustrates the $P$-$\Delta$ curves gained from the FE results. The ultimate strength increases with increasing steel ratio, while the elastic stiffness of the structure hardly changed at all. Comparing with the steel ratio of 0.05, the ultimate strength is raised by 16.9%, 27.6%, respectively, when the steel ratios of the FE modeling are 0.1 and 0.15.

5.8. Effect of beam to CFST column yield strength ratio ($k_m$)

The horizontal load versus displacement under monotonic loading and corresponding beam to CFST column yield strength ratio with 0.4, 0.6, 0.8 were plotted in Fig. 10(h). It is suggested that the ultimate strength of the frame increase with the incremental ratio $k_m$ while the elastic stiffness of the structure is rarely changed. When the ratio $k_m \leq 0.6$, the ultimate strength observably increases as the ratio $k_m$ is increasing. However, the pace of increase of the ultimate strength is slowed down when $k_m > 0.6$. 

\[ P \text{ vs. } \Delta \text{ for } k_m = 0.4, 0.6, 0.8 \]
5.9. Effect of extended end plate thickness (tep)

Changing the thickness of extended end plate would also impact the ultimate strength of the structure. Fig. 10(i) shows the P-Δ curves of the structure with different tep. It is suggested that the ultimate strength of the structure increases as the extended end plate thickness increases, while the elastic stiffness is not sensitive to different thickness of the extended end plate. Comparing to the ultimate strength of the frame which is tep = 28 mm, the ultimate strength of frames in which the extended end plate thickness was set to 10, 16 mm decreased by 34.2, 6.1%, respectively.

5.10. Effect of blind bolt diameter (db)

Fig. 10(j) exhibits the load-displacement relationship with different blind bolt diameters. It can be seen that the ultimate strength increases with the increase of the blind bolt diameter, but small changes occurred in the elastic stiffness of the frame. When the blind bolt diameter of the frame was designed as 20, 28 mm, the ultimate load was improved by 8.9%, 12.9%, respectively, compared with the frame using 16 mm diameter blind bolts.

5.11. Effect of axial compression ratio of the CFST column (n)

The axial compression ratio of the CFST column has an important influence on the behavior of the frame. n ranges from 0.2 to 0.8 developed in the FE models to illustrate the horizontal load versus displacement curves, as shown in Fig. 10(k). It seems that both the ultimate strength and elastic stiffness of the frames are decreased as the axial compression ratio n increases. The column axial load level of the structure was set to 0.4, 0.6, 0.8, the ultimate strength of the frame was reduced by 6.8%, 17.6%, 27.9%, respectively, when comparing with n = 0.2 and the elastic stiffness was dropped by 5.6%, 12.8%, 21.3%, respectively.

5.12. Effect of blind bolt pretension force (P)

The research of the effect of pre-tightening force of the blind bolt where P = 0.25P0, 0.75P0, 1.0P0, 1.5P0 was carried out. Hereinto, the normal pretension forces P0 = 155 kN for 10.9 Grade M20 high strength blind bolts according to GB50017-2003 [24]. Fig. 10(l) provides a comparison of the horizontal load versus horizontal displacement variation in the blind bolt pretension force. It can be shown that the ultimate strength of the frame increases slightly when increasing pretension force of blind bolts especially for P > 1.0P0.

On the whole, the parameters described above were investigated in detail to make it clear that the seismic performance of the semi-rigid CFST frames with external SCWPs. The numerical analysis results demonstrated that the ultimate strength of the structure is obviously affected by steel strength, concrete strength of SCWPs, steel wire diameter of SCWPs, slenderness ratio, steel ratio, beam to CFST column yield strength ratio, extended end plate thickness, blind bolt diameter and column axial compression ratio. The elastic stiffness of the structure is obviously affected by steel strength, concrete strength of SCWPs, column slenderness ratio and column axial compression ratio.

6. Discussion of SCWPs with openings

The walls with openings is very common in the practical engineering. The openings are located in different position of walls to meet the requirements of flexible architectural function. Several experiments and numerical analysis have been conducted and presented to investigate the in-plane behavior and reductions with appearance of openings in infilled walls [11,21]. However, infills in those literatures were masonry with openings. For deeply investigating the behavior of semi-rigid CFST frames infill SCWPs with openings, the opening size and opening location of SCWPs were considered.

6.1. Effect of door opening ratio (αd)

The door opening was set on the center of SCWPs and the opening ratio has a prominent influence on the ultimate strength and elastic stiffness of the typed structure. In this paper, the opening ratio (αd) is defined as Ad/Ap, where Ad and Ap are the area of door opening and whole wall, respectively. Six different opening ratio (αd) were used in the FE model which were 10%, 25%, 40%, 60%, 80% and 100%. Fig. 11 shows that the strength and stiffness of SCWPs decreased markedly owing to the set of an opening. The drop velocity slows down with the increase of opening size, especially when 40% ≤ α ≤ 80%. Compared with the solid frame, when the opening ratio is 10%, 25%, 40%, 60%, 80% and 100%, the ultimate strength of the structure respectively reduced by 4.6%, 19.7%, 36.2%, 37.1%, 40.3% and 65.1%; the elastic stiffness of the structure respectively decreased by 2.3%, 4.3%, 5.5%, 8.7%, 11.4% and 43.2%.

6.2. Effect of door opening location

At the same door opening ratio αd = 40%, three models with different door opening locations including central, left-side and right-side and one model together with door and window opening were considered, as shown in Fig. 12. Fig. 13 provides a comparison of the horizontal load versus displacement curve for the door opening location. It can be found that as the opening transfers from the right-side to center to left-side, the ultimate strength and stiffness increase. That is to say, when the opening location is toward the loaded side, the stiffness and strength are the largest. Compared with the left-side door opening, the ultimate strength of walls with right-side, central, together with door and window opening decreased by 14.7%, 7.8%, 4.5%; the elastic stiffness of the structure respectively decreased by 42.7%, 29.1%, 6.4%.

6.3. Effect of window opening location

At the same window opening ratio of 40%, three models with different window opening locations including central, left-side and right-side and one model together with door and window opening were considered, as shown in Fig. 14. The horizontal load versus displacement curve for the window opening location is shown in Fig. 15. It shows that as the opening transfers from the right-side to center to left-side, the ultimate strength and stiffness decrease. Compared with the right-leaning window opening, the ultimate strength of walls with left-leaning, central, together with door and window opening decreased by 14.2%, 10.5%, 5.3%; the elastic stiffness of the structure respectively decreased by 46.2%, 31.8%, 16.9%.

![Fig. 11. Effect of door opening ratio on P-Δ relation.](image-url)
The numerical analysis of walls with openings shows that both stiffness and strength of the typed structure decrease with an increase in the opening area. Meanwhile, the reduction rate is also linked with the opening location. The ultimate strength and stiffness decrease when the door opening moves away from the loaded side. However, the ultimate strength and stiffness decrease when the window opening closes to the loaded side.

7. Conclusions

Based on the experimental and numerical results, the findings obtained may be summarized within the limitation of the study:

1. The main failure modes of the semi-rigid CFST frames with external SCWPs include the cracks emerged and propagated around the embedded parts, concrete spalling around the embedded parts and the local buckling of beam flange and web. Compared with traditional wall panels, the SCWPs exhibit better integration.

2. The ultimate strength and initial stiffness of the semi-rigid CFST frames with external SCWPs significantly improved when compared with pure semi-rigid CFST frame. As for the SCWPs using ordinary concrete, the ultimate strength of the specimen that the SCWPs connected with columns and beams is larger than that of specimen that the SCWPs connected with columns. When the SCWPs connected with beams and columns, the ultimate strength of the specimen that the SCWPs using high strength ceramsite concrete was larger than that of the specimen that the SCWPs using ordinary concrete. In addition, the set of diagonal steel braces could improve obviously structural ultimate strength but become more brittle.

3. The FE analysis modeling on semi-rigid CFST frames with external SCWPs was established in this study. The FE modeling was validated in terms of the failure modes and horizontal load-displacement relationship curves of the typed structure. It is shown that the FE modeling can be used to predict the behavior of the typed structure with an acceptance precision.

4. The extensive numerical parameter analysis results showed that the ultimate strength of the structure is obviously affected by steel strength, concrete strength of SCWPs, steel wire diameter of SCWPs, column slenderness ratio, steel ratio, beam to column yield strength ratio, end plate thickness, blind bolt diameter and column axial load level. The elastic stiffness of the structure is obviously affected by steel strength, concrete strength of SCWPs, column slenderness ratio and column axial load level.
A reduction in both stiffness and strength of SCWPs with the increase of opening ratio and the reduction rate is also linked with the opening location at the same opening ratio. The ultimate strength and stiffness decrease when the door opening offset moves away from the loaded side while the ultimate strength and stiffness decrease when the window opening closes to the loaded side.

Acknowledgments

This work described in paper is supported by the National Natural Science Foundation of China (project 51478158 and project 51178156). New Century Excellent Talents in University (project NCET-12-0838), is greatly appreciated.

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