Research Article

Shear Strength Prediction for SFRC Shear Wall with CFST Columns by Softened Strut and Tie Model

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The steel fiber reinforced concrete (SFRC) shear wall with concrete filled steel tube (CFST) columns is an innovative composite structure. In order to calculate the shear strength of SFRC shear wall with CFST columns, the softened strut and tie model (SSTM) of SFRC shear wall with CFST columns was proposed based on the analysis of shear mechanism of SFRC shear wall with CFST columns. The SSTM was composed of diagonal, horizontal, and vertical mechanisms, in which the contributions of concrete, reinforcement, and steel fiber to the shear strength of SFRC web of shear wall were identified. The shear capacities of 24 shear walls were calculated and compared with the available test results, and reasonable agreement was obtained. The results also showed that the steel fibers distributed randomly in concrete could be treated as longitudinal and transverse reinforcement in the shear strength analysis of SFRC web, and the SSTM was reasonable and useful to analyze and predict the shear strength of SFRC shear wall with CFST columns.

1. Introduction

Reinforced concrete (RC) shear wall system is a common one to resist lateral force, which has been widely used in the regions of high seismic activity in the past decade. However, due to low tensile strength and large brittleness of normal-strength concrete, the ductility, energy dissipation, and load bearing capacities of RC shear wall are poor. In particular, the transverse dimension of shear wall can be decreased with the application of high strength concrete, but its brittleness increases inevitably. Correspondingly, the higher requirement for the seismic performance of shear wall especially with high strength concrete (HSC) would be put forward. In order to improve the seismic performance of shear wall, the use of RC shear wall with concrete filled steel tube (CFST) columns and steel fiber reinforced concrete (SFRC) shear wall has attracted the interest of structural engineers in recent years.

First, theoretical analysis and experimental study [1–4] show that the CFST columns serving as boundary members can provide the effective constraint to reinforced high strength concrete (RHSC) wall, restrain the cracking development, and also bear part of transverse load after the RHSC wall deteriorates gradually, indicating that the RHSC shear wall with CFST columns can form double-channel seismic line and have favorable seismic performance. Secondly, the application of SFRC is a good way to enhance the seismic behavior of conventional shear wall structure, especially the RHSC shear wall [5]. Several researchers [6–10] have conducted the experiments on the seismic performance of SFRC shear wall. In these experiments, the SFRC shear walls were subjected to reversed cyclic loading. As expected, because SFRC has good properties, such as stretching, shear resistance, cracking resistance, toughness [11], fire resistance, durability [12, 13], and earthquake resistance, it can remarkably improve the seismic behavior of RC shear wall

and reduce the phenomenon of reinforcement congestion and construction difficulty. The load bearing capacity, ductility, and energy dissipation capacities of SFRC shear walls increase with the increasing of steel fiber volume fraction.

In this study, an innovative composite shear wall referred to as SFRC shear wall with CFST columns is introduced, which consists of SFRC web and CFST boundary elements. In this new composite shear wall, SFRC web can mainly improve the ductility and energy dissipation capacities, CFST boundary elements can enhance the load bearing capacity. This kind of composite shear wall will adequately make use of the advantages of SFRC shear wall and CFST columns. In order to more fully understand the seismic behavior of this structure form, six SFRC shear wall with CFST column specimens were tested under constant axial force and reversed cyclic loading. The failure mode of shear wall specimens was analyzed, and the influencing factors, such as the steel fiber volume fraction and concrete strength on the shear strength of shear wall specimens, were studied. Since the load transferring mechanism of shear walls under the combined action of compression, bending, and shearing is very complicated, the researches on the load bearing mechanism and the calculation methods for shear capacity of shear walls are relatively few. In addition, the only existing calculation methods for shear capacity proposed are mostly semiempirical relations based on the experimental results and are lacking in the reasonable theoretical model. Furthermore, the analysis and calculations are not very accurate.

The softened strut and tie model (SSTM) for determining the shear strengths of RC members failing in diagonal compressions was proposed by Hwang and Lee, which originated from the strut and tie concept, considered the compression softening characteristics of concrete, and was derived to satisfy equilibrium, compatibility, and constitutive laws of cracked reinforced concrete [14–19]. The SSTM has been used in shear strength prediction of various kinds of concrete members and the accuracy has been checked by comparing calculated shear strengths with experimental data reported in the previous literature [20–24].

In this paper, the SSTM of SFRC shear wall with CFST columns was proposed based on the analysis for its shear mechanism, in which the steel fibers distributed randomly in concrete were equivalent to the longitudinal and transverse reinforcements. Furthermore, a new calculation method based on the SSTM for the shear strength of SFRC shear wall with CFST columns, RC shear wall with CFST columns, and SFRC shear wall was established. Finally, the shear capacities of 24 low-rise shear walls were calculated and the calculated values showed a reasonable agreement with the available experimental results [2, 4, 9, 10].

2. General Test Situation

2.1. Specimen Details. The dimensions and configurations of the shear wall specimens are shown in Figure 1. Each specimen mainly consists of five parts: one SFRC web in the centre, two CFST columns which are located in the right and left sides of middle SFRC web, and two RC beams which are

located in the top and bottom sides of middle SFRC web. In order to ensure the reliable connections between the CFST columns and the shear wall web, the U-shaped connectors [25] with the same space as the horizontal reinforcing bars are also welded to the wall of steel tubes. The photos of completed steel frame and U-shaped connector are shown in Figures 2(a) and 2(b), respectively.

The side length and height of the square CFST columns were 120 mm and 750 mm, respectively. The steel plates for hollow sections of square CFST columns and U-shaped connectors were made by cold-formed steel tubes with the thickness of 3 mm and 2 mm, respectively. The strength grade of reinforcement was HRB400, and the grade of steel plates was Q235B based on the Chinese code. The material properties of steel bar and plate are listed in Table 1. The steel fiber is hooked at both ends with length of 35 mm, diameter of 0.55 mm, and aspect ratio of 64.

The test parameters included steel fiber volume fraction ρ_{sf} (0, 0.5%, 1.0%, and 1.5%) and concrete strength grade (CF40, CF60, and CF80). According to the test parameters, six shear wall specimens were cast, tested, and numbered as SS-1.0-00-CF60, SS-1.0-05-CF60, SS-1.0-10-CF60, SS-1.0-15-CF60, SS-1.0-10-CF40, and SS-1.0-10-CF80. In the serial number of specimens, the first part SS represents that steel tube and steel fiber; the second part 1.0 refers to the shear span ratio of 1.0, the third part represents the steel fiber volume fraction of the web in shear wall, and the fourth part represents the concrete, in which C represents plain concrete, whilst CF represents SFRC, and the following number denotes target concrete strength of shear wall specimen. The summary of information of each shear wall specimen is listed in Table 2. The mechanical properties of the hardened concrete used in the shear wall of all specimens are also listed in Table 2. All the hardened concretes reached its target compressive strength.

2.2. Test Scheme. In the test, the load exerted on the specimens included horizontal and vertical one, and the test set-up is shown in Figure 3. The horizontal load was cyclic one and applied by an electrohydraulic servo control actuator on the top beam of specimen; the axial compressive force was kept constant and applied by hydraulic Jack. The target axial compression ratio, n , was taken as 0.2 for all specimens. The rolling support between hydraulic Jack and vertical reaction steel frame assured the smooth horizontal slipping of specimens. Rotation control mode was adopted and the typical rotation controlled loading history is shown in Figure 4. A nominal yield rotation angle (θ_{ym}) was designated to 0.0025 rad according to the research in literature [1], which was corresponding to a nominal yield lateral displacement (Δ_{ym}) at the loading point, $\Delta_{ym} = \theta_{ym} \times 750 \text{ mm} = 1.875 \text{ mm}$. Before θ_{ym} , the rotation levels of $0.5\theta_{ym}$ and $0.75\theta_{ym}$ were imposed. Hereafter, the rotation increment was 0.0025 rad for each rotation level and three cycles were imposed at each rotation level. The tests were conducted under the rotation controlled cyclic loading until the axial compressive force could not be maintained or the lateral force degraded below 85% of peak lateral load.

FIGURE 1: Specimen configuration (unit: mm). (a) Elevation. (b) A-A section. (c) One-dimensional and three-dimensional graph of U-shaped connector.

2.3. Failure Process. The whole process from the initiation to failure of specimens was observed. During the initial loading period, the inclined cracks gradually appeared and constantly extended under cyclic lateral loading. Then several pairs of visible intercrossing diagonal cracks developed approximately along the diagonal directions of the web in shear wall, the horizontal reinforcing bars in the web of the shear wall yielded successively with the loading, and the diagonal strut mechanism in the web of the shear wall was formed. Hereafter, new cracks seldom occurred and the

width of the diagonal cracks at the middle part of the web in shear wall increased rapidly with further loading. The web of the shear wall began to fail in diagonal compression, the concrete in the web was crushed and spalled, and the web lost bearing capacity gradually. Upon further cycles, slight local buckling was observed at the bottom of CFST columns and plastic hinges were formed. Finally, the lateral load rapidly dropped to about 85% of the peak lateral load and the specimen failed. All specimens mainly exhibited a typical diagonal cracking pattern during the test process and

FIGURE 2: Steel frame and U-shaped connector. (a) Steel frame. (b) U-shaped connector.

TABLE 1: Material properties of steel plate and bar.

Type	Yield strength f_y (N/mm ²)	Ultimate strength f_u (N/mm ²)	Elastic modulus E_s (N/mm ²)
3 mm plate	307.67	392.00	1.98×10^5
2 mm plate	236.67	323.20	1.88×10^5
C 6 bar	369.17	521.60	1.85×10^5

TABLE 2: Summary of specimen information.

Identification number	ρ_{sf} (%)	Columns f_c (MPa)	Columns f_t (MPa)	Web f_c (MPa)	Web f_t (MPa)	$V_{jh, test}$ (kN)
SS-1.0-00-C60	0	55.9	2.76	56.3	2.81	902
SS-1.0-05-CF60	0.5	55.5	2.73	55.2	3.67	948
SS-1.0-10-CF60	1.0	55.5	2.74	55.1	6.15	1005
SS-1.0-15-CF60	1.5	56.0	2.77	56.5	7.88	1048
SS-1.0-10-CF40	1.0	38.1	2.15	38.3	3.78	810
SS-1.0-10-CF80	1.0	64.6	3.96	65.6	6.84	1107

FIGURE 3: Test set-up. (a) Schematic view. (b) Test scene.

showed a shear-dominant failure mode. The failure pattern and crack distribution at the peak lateral load and ultimate state (the lateral force degraded to 85% of the peak lateral load) of all tested specimens are shown in Figure 5.

During the test process, the connections between the CFST columns and web were reliable and no significant separation at the interface occurred, which indicated that the

performance of the U-shaped connector designed was effective. Moreover, no evident local buckling on the CFST columns of the SFRC shear wall specimens was observed at the ultimate state. The SFRC shear wall specimens had smaller destruction compared with that without steel fibers, which indicated that the SFRC shear wall specimens had the better reparability.

FIGURE 4: Loading history.

FIGURE 5: Failure pattern and crack distribution of shear wall specimens. (a) Failure pattern at peak state. (b) Failure pattern at ultimate state. (c) Schematic view of crack distribution.

2.4. Influence Factors. The shear capacities of all specimens are listed in Table 2. It can be seen from Table 2 that the shear performance of specimens is improved obviously with the increase of steel fiber volume fraction from 0% to 1.5%. Compared with the specimen SS-1.0-00-C60, the shear capacity of specimen SS-1.0-05-CF60, SS-1.0-10-CF60, and SS-1.0-15-CF60 is increased by 5.10%, 11.42%, and 16.19%,

respectively. In addition, steel fiber could effectively control the crack width and improve the distribution shape of cracks in the web of shear wall. The cracks became much thinner, denser, of more quantity, and wider distribution with the increase of steel fiber volume fraction, whilst the crushing and spalling of concrete were also reduced. For instance, the maximum crack width of RC shear wall specimen numbered

SS-1.0-00-C60 was 2.0 mm at the peak load, whilst the maximum crack width of SFRC shear wall specimen numbered SS-1.0-05-CF60, SS-1.0-10-CF60, and SS-1.0-15-CF60 was 1.6 mm, 1.2 mm, and 1.0 mm, respectively. The maximum crack width decreased significantly with the increase of steel fiber volume fraction; the similar results were reported in the literature [9, 10].

Furthermore, it can be seen from Table 2 that the shear performance of specimens is improved obviously with the enhancing of concrete strength from CF40 to CF80. Compared with the specimen SS-1.0-10-CF40, the shear capacity of SS-1.0-10-CF60 and SS-1.0-10-CF80 is increased by 24.07% and 36.67%, respectively. And as shown in Figure 5, the SFRC shear wall specimens with different concrete strength had the similar failure appearance at the peak and ultimate state under the condition of the same steel fiber volume fraction. It can be seen that the lateral load of SFRC shear wall specimen with higher strength concrete deteriorated more obviously after the peak load, the crushing, and spalling of concrete were more serious at the ultimate state, especially for the specimen SS-1.0-00-CF80.

3. Shear Resisting Mechanism of SFRC Shear Wall with CFST Columns

It was observed in the test that the failing process of SFRC shear wall with CFST columns can be divided into two stages: at early loading stage, the all bending moments acted on the SFRC shear wall with CFST columns were carried by the CFST columns, nearly all the horizontal shear forces were carried by the web, and the axial forces were shared by the CFST columns and web; at medium and late loading stage, the concrete in the web continuously cracked, resulting in a decrease of the stiffness, and the shear forces gradually transferred from the web to CFST columns. Therefore, the calculation method for shear strength of SFRC shear wall with CFST columns can be defined as

$$V_{jh,calc} = V_{jh} + V_{col}, \quad (1)$$

where $V_{jh,calc}$ is the calculated value for shear strength of SFRC shear wall with CFST columns; V_{jh} is the shear strength of web; and V_{col} is the shear strength of CFST columns.

The shear strength of CFST columns can be defined as [4]

$$V_{col} = \alpha_v \cdot A_{sc} \cdot \tau_{scy}, \quad (2)$$

where α_v is the calculation coefficient for shear capacity, $\alpha_v = 0.97 + 0.2 \ln(\xi)$, ξ is the confinement factor, $\xi = \alpha_s f_y / f_c$, f_y is the yield strength of steel tube, f_c is the axial compressive strength of concrete, α_s is the steel ratio of CFST column, $\alpha_s = A'_s / A'_c$, A'_s and A'_c are the cross-sectional areas of steel tube and concrete, respectively, A_{sc} is the total cross-sectional area of CFST column, and τ_{scy} is the shear yield limit, $\tau_{scy} = (0.422 + 0.313 \alpha_s^{2.33}) \xi^{0.134} (1.14 + 1.02 \xi) f_c$.

The load transferring mechanism of SFRC web under the combined action of compression, bending, and shearing is very complicated, which is similar to RC web in the literature [2, 3]. In the SSTM, the shear strength of the web is assumed

to be that of concrete compressive struts, and the concrete in the web can continue to withstand shear force after the tension ties yield. Therefore, the main characteristic of web failure is that the concrete in the web reaches its compressive strength and is crushed and spalled. For diagonal compression failure, the shear strength of the web is defined as the concrete compressive stress on the nodal zone as the concrete reaches its capacity. The concrete bearing capacity is the summation of compressions from the concrete compressive struts in the diagonal, horizontal, and vertical mechanisms [14–19]. Hence, three strut and tie load paths are proposed to simulate the force transfer within the web, including the diagonal, horizontal, and vertical mechanisms, as shown in Figure 6.

The concrete in the web is subjected to the compression force; thus the diagonal compressive strut is formed, as shown in Figure 6(a). Due to the higher tensile strength of SFRC, the diagonal tension forces in the web are mainly carried by the SFRC before the cracking of concrete, the action of reinforcing bars is much smaller at this time. The initial inclined cracks in the web occur when the diagonal tension force is greater than the tensile strength of concrete. After the initial cracking of concrete, the diagonal tension forces in the web are mainly carried by the horizontal and vertical ties. The horizontal ties are composed of horizontal reinforcing bars and steel fibers as shown in Figure 6(b), whilst the vertical ties are composed of vertical reinforcing bars and steel fibers as shown in Figure 6(c). In summary, the reinforcing bars and steel fibers are subjected to tension and the concrete acts as compressive struts after the development of the first cracking pattern in the web; thus a strut and tie action is formed.

The diagonal mechanism (Figure 6(a)) is a single SFRC diagonal compression strut whose angle of inclination θ is defined as

$$\theta = \tan^{-1} \left(\frac{H}{l} \right), \quad (3)$$

where H is the distance from the lateral force loading point to the shear wall base; l is internal lever arm of the vertical shear couple, $l = 0.8 l_w$; and l_w is the length of the entire web in the direction of the applied lateral force.

The effective area of the SFRC diagonal compression strut A_{str} is defined as

$$A_{str} = \alpha_s \times b_s, \quad (4)$$

where a_s is the depth of diagonal strut and b_s is the width of diagonal strut that can be taken as the width of the web t_w .

The depth of the diagonal strut a_s depends on its end condition provided by the compression zone at the base of the web. It can be intuitively assumed that

$$\alpha_s = \alpha_w, \quad (5)$$

where a_w is the depth of the compression zone at the base of the web, which can be approximately determined by [15]

$$\alpha_w = \left(0.25 + 0.85 \frac{N'}{A_w f'_c} \right) l_w, \quad (6)$$

FIGURE 6: Shear resisting mechanisms of web. (a) Diagonal. (b) Horizontal. (c) Vertical.

where N' is the axial compressive force for the web; A_w is the cross-sectional area of the web, $A_w = t_w \times l_w$; and f'_c is the compressive strength of a standard concrete cylinder (MPa).

The axial compressive force for the web N' can be calculated as

$$N' = \frac{0.85E_c t_w l_w}{0.85E_c t_w l_w + 2E'_s A'_s + 2E'_c A'_c} N, \quad (7)$$

where N is the axial compressive force for the shear wall specimen; E_c is the elastic modulus of web concrete; 0.85 is the reduction coefficient of axial compressive stiffness for web concrete [4]; E'_s is the elastic modulus of steel tube; and E'_c is the elastic modulus of concrete inside the steel tube.

The horizontal mechanism (Figure 6(b)) includes one horizontal tie and two flat struts. Since steel fiber improves the tensile strength of the concrete, which makes the performance of SFRC shear wall different from that of RC shear wall. The horizontal tie is made up of the horizontal reinforcing bars and steel fibers and can be described as the following:

$$F_h = F_{s,h} + F_{sf,h}, \quad (8)$$

where F_h is the tension force in the horizontal tie; $F_{s,h}$ is the tension force of horizontal reinforcing bars, $F_{s,h} = A_{s,h} \times f_{s,h}$; $A_{s,h}$ is the sectional area of horizontal reinforcing bars; $f_{s,h}$ is the tensile strength of horizontal reinforcement; $F_{sf,h}$ is the tension force of horizontal steel fibers, $F_{sf,h} = A_{sf,h} \times f_{sf}$; $A_{sf,h}$ is the sectional area of horizontal steel fibers; and f_{sf} is the tensile strength of steel fiber.

The experimental results indicated that the horizontal reinforcing bars in the web did not fully yield when the web damaged, so the effective coefficient for shear resistance of horizontal reinforcement, η_1 , is introduced. Thus $F_{s,h}$ can be defined as $F_{s,h} = \eta_1 \times A_{s,h} \times f_{s,h}$, where η_1 is taken as 0.75 based on the analysis for the test results in the literature [15].

For simplicity, the steel fibers randomly distributed in the three dimensions of the web can be equivalent to horizontal and vertical microreinforcing bars, as shown in Figure 7. Thus, the sectional area of horizontal steel fibers, $A_{sf,h}$, can be defined as

$$A_{sf,h} = n_{sf} A_{sf} \quad (9)$$

FIGURE 7: Steel fiber ties.

where A_{sf} is the cross-sectional area of a single steel fiber and n_{sf} is the equivalent steel fiber number, which can be calculated as

$$n_{sf} = \eta_2 \rho_{sf} \frac{t_w H}{A_{sf} \sin \theta}, \quad (10)$$

where ρ_{sf} is the steel fiber volume fraction and η_2 is the equivalent coefficient, which can be approximately designated as 0.41 according to the study in the literature [26].

The sectional area of horizontal steel fibers, $A_{sf,h}$, can be determined as

$$A_{sf,h} = \frac{0.41 \rho_{sf} t_w H}{\sin \theta}. \quad (11)$$

The vertical mechanism (Figure 6(c)) is composed of one vertical tie and two steep struts. Similar to the horizontal tie, the vertical tie includes the vertical reinforcing bars and steel fibers within the web and can be described as the following:

$$F_v = F_{s,v} + F_{sf,v}, \quad (12)$$

where F_v is the tension force in the vertical tie; $F_{s,v}$ is the tension force of vertical reinforcing bars, $F_{s,v} = A_{s,v} \times f_{s,v}$; $A_{s,v}$ is the sectional area of vertical reinforcing bars; $f_{s,v}$ is the tensile strength of vertical reinforcement; $F_{sf,v}$ is the tension force of

vertical steel fibers, $F_{sf,v} = A_{sf,v} \times f_{sf}$, $A_{sf,v}$ is the sectional area of vertical steel fibers; and $A_{sf,v} = 0.41 \rho_{sf} t_w l_w / \cos \theta$ based on the above analysis.

The research in literature [15] shows that the vertical reinforcing bars are not fully effective in constituting the vertical tie, and the effective coefficient for shear resistance of vertical reinforcing bars, η_3 , is introduced. Thus, $F_{s,v}$ can be defined as $F_{s,v} = \eta_3 \times A_{s,v} \times f_{s,v}$, where η_3 is taken as 0.8 [15].

4. Calculation Method for Shear Strength of SFRC Shear Wall with CFST Columns

The SSTM is used to determine the shear strength of web failing in diagonal compression.

4.1. Equilibrium Conditions. Based on the proposed strut and tie model for a web of shear wall, the horizontal and vertical shear forces can be calculated as follows:

$$V_{jh} = D \cos \theta + F_h + F_v \cot \theta, \quad (13)$$

$$V_{jv} = D \sin \theta + F_v + F_h \tan \theta, \quad (14)$$

where V_{jh} is the horizontal shear force; V_{jv} is the vertical shear force; and D is the compression force in SFRC diagonal strut. The details of force diagram are shown in Figure 8.

The ratios of the horizontal shear V_{jh} assigned among the three resisting mechanisms can be defined as [15]

$$D \cos \theta : F_h : F_v \cot \theta = R_d : R_h : R_v, \quad (15)$$

where R_d , R_h , and R_v are the web shear ratios resisted by the diagonal, horizontal, and vertical mechanisms, respectively, which can be defined as the following [15]:

$$R_d = \frac{(1 - \gamma_h)(1 - \gamma_v)}{1 - \gamma_h \gamma_v}, \quad (16)$$

$$R_h = \frac{\gamma_h(1 - \gamma_v)}{1 - \gamma_h \gamma_v}, \quad (17)$$

$$R_v = \frac{\gamma_v(1 - \gamma_h)}{1 - \gamma_h \gamma_v}, \quad (18)$$

where γ_h is the fraction of horizontal shear transferred by the horizontal tie in the absence of the vertical tie and γ_v is the fraction of vertical shear carried by the vertical tie in the absence of the horizontal tie. The values of γ_h and γ_v can be obtained as [27]

$$\gamma_h = \frac{2 \tan \theta - 1}{3}, \quad \text{for } 0 \leq \gamma_h \leq 1, \quad (19)$$

$$\gamma_h = \frac{2 \tan \theta - 1}{3}, \quad \text{for } 0 \leq \gamma_h \leq 1. \quad (20)$$

By solving (13)–(15), the values of D , F_h , and F_v can be obtained as

$$D = \frac{1}{\cos \theta} \times \left(\frac{R_d}{R_d + R_h + R_v} \right) \times V_{jh}, \quad (21)$$

$$F_h = \left(\frac{R_h}{R_d + R_h + R_v} \right) \times V_{jh}, \quad (22)$$

$$F_v = \frac{1}{\cot \theta} \times \left(\frac{R_v}{R_d + R_h + R_v} \right) \times V_{jh}. \quad (23)$$

To check whether the web of shear wall reached failure strength, the bearing pressure on the nodal zone (Figure 6), where the compressive forces from the diagonal, flat, and steep struts intersect (Figure 8), must be checked. The maximum compressive stress $\sigma_{d,max}$ acting on the nodal zone can be calculated as

$$\sigma_{d,max} = \frac{1}{A_{str}} \left[D + \frac{F_h}{\cos \theta} \left(1 - \frac{\sin^2 \theta}{2} \right) + \frac{F_v}{\sin \theta} \left(1 - \frac{\cos^2 \theta}{2} \right) \right]. \quad (24)$$

4.2. Constitutive Laws. The softened stress-strain relation of cracked SFRC concrete can be represented as the following [28]:

$$\sigma_d = \zeta f'_c \left[2 \left(\frac{\varepsilon_d}{\zeta \varepsilon_0} \right) - \left(\frac{\varepsilon_d}{\zeta \varepsilon_0} \right)^2 \right], \quad \text{for } \frac{\varepsilon_d}{\zeta \varepsilon_0} \leq 1, \quad (25)$$

where σ_d is the average principal stress of concrete in the d-direction; ζ is the softening coefficient; ε_d is the average principal strain in the d-direction; and ε_0 is the concrete cylinder strain corresponding to the cylinder strength f'_c , which can be defined approximately as [29]

$$\varepsilon_0 = -0.002 - 0.001 \left(\frac{f'_c - 20}{80} \right), \quad \text{for } 20 \text{ MPa} \leq f'_c \leq 100 \text{ MPa}. \quad (26)$$

The softening coefficient ζ can be calculated as [28]

$$\zeta = \frac{5.8}{\sqrt{f'_c}} \frac{1}{\sqrt{1 + 400 \varepsilon_r}} \leq \frac{0.9}{\sqrt{1 + 400 \varepsilon_r}}, \quad (27)$$

where ε_r is the average principal strain in the r-direction.

Whether the shear strength of the web of shear wall reaches the compressive stress, the strain of the concrete diagonal strut can be conformed by the following formula:

$$\sigma_d = \zeta f'_c, \quad (28)$$

$$\varepsilon_d = \zeta \varepsilon_0. \quad (29)$$

The stress-strain relation of reinforcing bar can be described as

$$\begin{cases} f_s = E_s \varepsilon_s, & \varepsilon_s < \varepsilon_y, \\ f_s = f_y, & \varepsilon_s \geq \varepsilon_y, \end{cases} \quad (30)$$

FIGURE 8: Strut and tie model for the web of shear wall.

where E_s is the elastic modulus of reinforcing bars; f_y is the yield strength of reinforcing bars; and f_s and ϵ_s are the stress and strain in mild steel, respectively. f_s becomes $f_{s,h}$ or $f_{s,v}$, E_s becomes $E_{s,h}$ or $E_{s,v}$, ϵ_s becomes $\epsilon_{s,h}$ or $\epsilon_{s,v}$, and f_y becomes $f_{y,h}$ or $f_{y,v}$ when equation (30) is applied to horizontal or vertical reinforcing bars, respectively.

The stress-strain relation of steel fiber can be expressed as

$$f_{sf} = E_{sf} \epsilon_{sf}, \quad (31)$$

where E_{sf} is the elastic modulus of steel fiber and f_{sf} and ϵ_{sf} are the stress and strain of steel fiber, respectively.

The experiment results show that steel fibers are usually pulled out from concrete rather than broken due to their good tensile strength. Thus, the shear resistant action of the steel fiber depends on the bond strength between steel fiber and concrete. The stress of steel fiber, f_{sf} , should satisfy the relationship as the following:

$$A_{sf} f_{sf} \leq \lambda_{sf} A_{spf} \tau_{sf,max}, \quad (32)$$

where $\tau_{sf,max}$ is the maximum bond strength between steel fiber and concrete, $\tau_{sf,max} = 2.5f_{ct}$ [30]; f_{ct} is the matrix tensile strength of SFRC; λ_{sf} is the influence coefficient of steel fiber type; the values of λ_{sf} for long straight, wave-shaped, and hooked steel fibers are 0.5, 0.75, and 1.0, respectively; A_{spf} is the surface area of steel fiber, $A_{spf} = \pi d_{sf} l_{sfo}$; d_{sf} is the nominal diameter of steel fiber; l_{sfo} is the effective anchorage

length of steel fiber, $l_{sfo} = 0.25l_{sf}$, and l_{sf} is the length of steel fiber.

By adopting equation (33), the stress of steel fiber f_{sf} can be defined as

$$f_{sf} \leq \lambda_{sf} \left(\frac{l_{sf}}{d_{sf}} \right) \tau_{sf,max}. \quad (33)$$

The relationship between forces and strains of the tension ties becomes

$$F_h = F_{s,h} + F_{sf,h} = 0.75 A_{s,h} E_{s,h} \epsilon_{s,h} + \frac{0.41 \rho_{sf} t_w H E_{sf} \epsilon_{sf,h}}{\sin \theta} \leq F_{yh}, \quad (34)$$

$$F_v = F_{s,v} + F_{sf,v} = 0.8 A_{s,v} E_{s,v} \epsilon_{s,v} + \frac{0.41 \rho_{sf} t_w l_w E_{sf} \epsilon_{sf,v}}{\cos \theta} \leq F_{yv}, \quad (35)$$

$$\begin{aligned} \epsilon_{s,h} &= \epsilon_{sf,h} = \epsilon_h, \\ \epsilon_{s,v} &= \epsilon_{sf,v} = \epsilon_v, \end{aligned} \quad (36)$$

where $\epsilon_{s,h}$ and $\epsilon_{sf,h}$ are the strains of the reinforcing bars and the steel fibers in the h-direction, respectively; $\epsilon_{s,v}$ and $\epsilon_{sf,v}$ are the strains of the reinforcing bars and the steel fibers in the v-direction, respectively; ϵ_h and ϵ_v are the average normal strains in the h- and v-directions, respectively; and F_{yh} and

FIGURE 9: Flow chart showing efficient algorithm.

F_{yv} are the yielding forces of the horizontal and vertical ties, respectively.

4.3. Compatibility Condition. The compatibility conditions adopted herein are given as the following [31]:

$$\varepsilon_r = \varepsilon_h + (\varepsilon_h - \varepsilon_d) \cot^2 \theta, \quad (37)$$

$$\varepsilon_r = \varepsilon_v + (\varepsilon_v - \varepsilon_d) \tan^2 \theta. \quad (38)$$

4.4. Solution Procedures. The solution procedures are proposed [15, 19], as shown in Figures 9 and 10. The algorithm in Figure 9 starts with a selection of the horizontal shear V_{jh} and consists of three major steps. Firstly, $\sigma_{d,max}$ is calculated by employing the equilibrium equations. By assuming the strength of the concrete strut is reached, an initial value of the softening coefficient ζ is obtained through $\zeta = \sigma_{d,max} / f'_c$. Secondly, the strains of the struts and ties are calculated by applying the corresponding constitutive laws. Finally, the

FIGURE 10: Algorithm for postyielding cases.

TABLE 3: Calculation type.

Type	Type E	Type YH	Type YV	Type YHV	Type YVH
Condition	$F_h < F_{yh}$ and $F_v < F_{yv}$	$F_h = F_{yh}$ and $F_v < F_{yv}$	$F_h < F_{yh}$ and $F_v = F_{yv}$	$F_h = F_{yh}$ and then $F_v = F_{yv}$	$F_v = F_{yv}$ and then $F_h = F_{yh}$

TABLE 4: Experimental verification.

Specimen	Web f_c (MPa)	Columns f_c (MPa)	H (mm)	$t_w \times l_w$ (mm)	$a \times t$ (mm)	n	Horizontal reinforcement	f_{vh} (MPa)	Vertical reinforcement	f_{yv} (MPa)	Steel fiber I_{sf}/d_{sf} ρ_{sf} (%)	$V_{jh, test}$ (kN)	$V_{jh, calc}$ (kN)	$V_{jh, test}/V_{jh, calc}$
SS-1.0-00-C60	56.3	55.9	750	120 × 510	120 × 3.0	0.2	C6@100	369.17	12 C 6	369.17	—	902	848	1.06
SS-1.0-05-CF60	55.2	55.5	750	120 × 510	120 × 3.0	0.2	C6@100	369.17	12 C 6	369.17	64	948	896	1.06
SS-1.0-10-CF60	55.1	55.5	750	120 × 510	120 × 3.0	0.2	C6@100	369.17	12 C 6	369.17	64	1005	931	1.08
SS-1.0-15-CF60	56.5	56.0	750	120 × 510	120 × 3.0	0.2	C6@100	369.17	12 C 6	369.17	64	1048	947	1.11
SS-1.0-10-CF40	38.3	38.1	750	120 × 510	120 × 3.0	0.2	C6@100	369.17	12 C 6	369.17	64	810	795	1.02
SS-1.0-10-CF80	65.6	64.6	750	120 × 510	120 × 3.0	0.2	C6@100	369.17	12 C 6	369.17	64	1107	1008	1.10
SW1.0-2 [2]	42.14	42.14	740	140 × 460	159 × 3.7	0.35	A6@120	478.73	A4@80	683.17	—	872.09	974	0.90
CFST-S-S1 [4]	39.71	39.71	920	85 × 860	120 × 3.0	0.31	A6@120	397	A6@120	397	—	770.6	786	0.98
CFST-S-S2 [4]	39.71	39.71	920	85 × 860	120 × 3.0	0.62	A6@120	397	A6@120	397	—	891.4	826	1.08
CFST-S-L1 [4]	39.71	39.71	920	85 × 1320	120 × 3.0	0.31	A6@120	397	A6@120	397	—	991	1044	0.95
CFST-S-L2 [4]	39.71	39.71	920	85 × 1320	120 × 3.0	0.62	A6@120	397	A6@120	397	—	1265	1091	1.16
CFST-C-S1 [4]	39.71	39.71	920	85 × 860	140 × 2.0	0.29	A6@120	397	A6@120	397	—	788.5	792	1.00
CFST-C-S2 [4]	39.71	39.71	920	85 × 860	140 × 2.0	0.58	A6@120	397	A6@120	397	—	895.1	834	1.07
CFST-C-L1 [4]	39.71	39.71	920	85 × 1320	140 × 2.0	0.29	A6@120	397	A6@120	397	—	987.5	1053	0.94
CFST-C-L2 [4]	39.71	39.71	920	85 × 1320	140 × 2.0	0.58	A6@120	397	A6@120	397	—	1225	1100	1.11
SW-05-40 [9]	21.2	—	900	—	—	0.1	A8@150	340	6 B 14	373.5	57	730	608	1.20
SW-10-40 [9]	26.8	—	900	—	—	0.1	A8@150	340	6 B 14	373.5	57	745	730	1.02
SW-15-40 [9]	25.1	—	900	—	—	0.1	A8@150	340	6 B 14	373.5	57	770	748	1.03
SW-20-40 [9]	26.9	—	900	—	—	0.1	A8@150	340	6 B 14	373.5	57	808	792	1.02
SW-10-30 [9]	17.8	—	900	—	—	0.1	A8@150	340	6 B 14	373.5	57	730	586	1.25
FSW1 [10]	36	—	600	—	—	0.09	6A6.5	310	6A6.5	310	64	335	431	0.78
FSW2 [10]	33.5	—	600	—	—	0.09	6A6.5	310	6A6.5	310	64	330	436	0.76

TABLE 4: Continued.

[illegible]

FIGURE 11: Correlation of experimental and predicted wall shear strengths.

FIGURE 12: Effects on shear strength predictions. (a) Effect of steel fiber volume fraction on shear strength predictions. (b) Effect of concrete strength on shear strength predictions. (c) Effect of axial compression ratio on shear strength predictions.

average principal strain in the r -direction ε_r is calculated with the aid of the compatibility equation. Then, a new value of ζ is calculated by using the softening theoretical formula. If the assumed ζ value is close to the new ζ , the V_{jh} selected is the shear strength of the web, otherwise back to the iterations.

Furthermore, the procedure in Figure 9 is categorized into five types according to varied yielding conditions of the ties [15, 19], as shown in Table 3. Figure 10 presents the force redistribution of the web shears after the yielding of the horizontal tie.

5. Experimental Verification

Although the experimental design parameters of SFRC shear wall specimens, RC shear wall with CFST columns specimens, and SFRC shear wall with CFST columns specimens were different, the failure processes of shear wall specimens in this paper and the literature [2, 4, 9, 10] are similar. Based on the experimental results, the failure process generally could be divided into four stages: initial cracking stage, main diagonal cracks forming stage, limit stage, and failure stage. The web of shear wall was separated into a series of diagonal compression rhombic blocks by the intercrossing inclined cracks under gradually increasing lateral load. As the lateral load increased continuously, the concrete rhombic blocks reached ultimate compressive strength and the concrete in the web was crushed and spalled, resulting in obvious shear-dominant failure mode for the shear wall specimens in this paper and the literature [2, 4, 9, 10]. The existence of steel tubes and steel fibers cannot change the ultimate failure mode of shear wall specimens. Therefore, the model presented in this paper can be used to calculate the shear strength of the abovementioned three types of shear walls failing in diagonal compressions in theory.

A total of 24 test specimens [2, 4, 9, 10] and their results (Table 4) are used to verify the proposed method, including 6 SFRC shear wall with CFST columns specimens in this paper, 9 RC shear wall with CFST columns specimens, and 9 SFRC shear wall specimens in the available literature [2, 4, 9, 10]. The test specimens selected cover a wide range of structure forms, geometry, loading, steel tube, steel fiber, and reinforcement detailing, as summarized in Table 4. The test specimens considered herein have three major common features: (1) all walls showed a web shear-dominant failure mode; (2) they were one-story isolated walls; (3) all contained both horizontal and vertical reinforcement uniformly distributed basically throughout the web.

In Figure 11, the experimental shear strengths from 24 walls are compared to their shear strengths predicted by the method presented in this paper. The relationship between the prismatic strength f_c and the cylinder strength f'_c is obtained from literature [32]. The last column of Table 4 lists the values of $V_{jh, \text{test}}/V_{jh, \text{calc}}$ ratios to indicate the precision of the proposed model. Figure 11 shows satisfactory results for the comparison of the measured and predicted strengths. The average strength ratio ($V_{jh, \text{test}}/V_{jh, \text{calc}}$) is 1.01 with a coefficient of variation (COV) of 0.14 (Table 4). It is

concluded that the proposed model can accurately predict the shear strengths of the shear walls mentioned above.

Figure 12 shows the effects of steel fiber volume fraction, concrete strength, and axial compression ratio on the shear strength predictions of the proposed method. The steel fiber volume fraction, concrete strength, and axial compression ratio are not observed to have significant influences on the strength ratio ($V_{jh, \text{test}}/V_{jh, \text{calc}}$), indicating that the main factors which affect the shear wall bearing capacity are reflected in the proposed model.

6. Conclusions

This paper introduces the tests for an innovative composite shear wall named SFRC shear wall with CFST columns and presents a proposal for determining the shear strength of the shear wall. The proposed SSTM is derived from the concept of struts and ties and satisfies equilibrium, compatibility, and constitutive laws of cracked SFRC. Based on the test results in this paper and available literature and their comparison with the proposed model, the following conclusions can be drawn:

- (1) All the SFRC shear wall with CFST columns specimens showed a shear-dominant failure mode in the current tests, and all specimens mainly exhibited a typical diagonal cracking pattern during the testing process. Steel fiber could effectively control the crack width and improve the distribution shape of cracks in the web of shear wall.
- (2) The shear mechanism of SFRC shear wall with CFST columns can be described by the SSTM, which consists of diagonal, horizontal, and vertical mechanisms.
- (3) The steel fibers distributed randomly in concrete can be treated as longitudinal and transverse reinforcement in the shear strength analysis of SFRC shear wall with CFST columns. And the contribution of steel fiber to the wall shear strength is identified.
- (4) Examination of existing experimental data indicated that the proposed model is capable of predicting the shear strengths of SFRC shear wall with CFST columns, RC shear wall with CFST columns, and SFRC shear walls for diagonal compression failures.

Data Availability

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

Conflicts of Interest

The authors declare that there are no conflicts of interest regarding the publication of this paper.

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