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Theoretical Stress–Strain Model for Concrete in Steel-Reinforced Concrete Columns

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Abstract:

This paper presents an investigation on the confinement mechanism of steel-confined concrete in steel reinforced concrete (SRC) columns. Six axially loaded SRC core columns were tested and the corresponding structural performance data were collected and assessed. Considering the confinement mechanism of both steel-confined and hoop-confined concrete, the concrete in the SRC columns was divided into four regions: highly steel-confined concrete (HSCC), partially steel-confined concrete (PSCC), partially confined concrete (PCC) and unconfined concrete (UCC). Based on Mander's model, a theoretical stress-strain model for each concrete region was proposed and implemented in an analytical model to predict the mechanical behaviors of axially loaded SRC columns from the results of both the tests described in this paper and previous tests; satisfactory agreement was observed.

Keywords: Steel-confined concrete; Confinement mechanism; Region partition; Stress-strain model.

Introduction

A steel reinforced concrete (SRC) column is a type of composite column that is reinforced with an encased structural steel section. In an SRC column, the strength of the structural steel can be fully developed because the concrete can prevent local buckling of structural steel shapes. Moreover, due to on the confinement from the structural steel shapes and hoops, the strength and ductility of the concrete can be enhanced. Therefore, an SRC column possesses a higher bearing capacity and ductility compared with the properties of ordinary reinforced concrete columns. This feature has promoted the wider use of SRC columns in various applications in recent years.

In the past decades, considerable experimental and numerical studies, such as the studies by Mirza and Skrabek (1992), Ricles and Paboojian (1994), El-Tawil and Deierlein (1996), El-Tawil and Deierlein (1999), Chen and Lin (2006), Ellobody and Young (2011) and Zhu et al. (2015), have been conducted on the mechanical performance of SRC columns. Some key parameters affecting the behavior and design of SRC columns have been studied, including the material strength, encased steel ratio, volumetric ratio and axial compression ratio. Although the static and cyclic behaviors of SRC columns have been extensively studied, the results from experimental, analytical and numerical analyses on SRC columns are not generally consistent (Chen et al. 2008). The reason for this inconsistency is attributed to the lack of an accurate understanding of the confinement mechanism of steel-confined and hoop-confined concrete and the improper application of stress-strain models to the concrete in SRC columns. Therefore, it is essential to investigate the concrete confinement mechanism in SRC columns with an aim to elucidating their mechanical performance. At present, two methodologies are used to define the stress-strain model for the concrete in SRC columns:

The first methodology (Type I) considers the concrete confinement from the hoops only and ignores the concrete confinement from the structural steel shapes (Ricles et al. 1994; Lee et al. 2001). The concrete in SRC columns is classified in only 2 regions: unconfined concrete (UCC) and confined concrete (hoop-confined concrete), as shown in Fig. 1(a).

The second methodology (Type II) considers the concrete confinement from both structural steel shapes and hoops (El-Tawil and Deierlein 1996; Chen et al. 2006; Chen and Wu 2017). The concrete in SRC columns is divided into 3 regions: unconfined concrete (UCC), partially confined concrete (PCC, hoop-confined concrete) and highly confined concrete (HCC, both hoop-confined and steel-confined concrete), as shown in Fig. 1(b). El-Tawil and Deierlein (1996) conducted a research on the confinement mechanism of H-shaped steel-confined concrete and considered that the concrete confinement of the HCC region was a linear superposition of those provided by the hoops and structural steel shapes. Chen et al. (2006) proposed an analytical model by using a section analysis method to predict the axial bearing capacity for SRC columns and defined some confinement factors that were validated by experimental results. Moreover, Chen and Wu (2017) proposed a stress-strain model for highly confined concrete through the interaction mechanism between the steel section and confined concrete.

Compared with the first methodology, the second methodology possess more reliability because it considers the concrete confinement from the both structural steel shapes and hoops in some extent. However, the previous stress-strain models for highly confined concrete, except that of El-Tawil and Deierlein (1996), are lack of the theoretical basis of the confinement mechanism of steel-confined concrete and just adopt some subjective assumptions to consider the concrete confinement from the structural steel. Similarly, El-Tawil and Deierlein (1996) only studied the confinement mechanism of H-shaped steel-confined concrete but the other kinds of steel profile have not been investigated in detail. With the rapid development of economic and higher standards for architectural requirements, the application of SRC columns encased with T-shaped steel profile and cross-shaped steel profile is gradually increased. Therefore, the confinement mechanism of steel with arbitrary sections confined concrete requires further study. Recently, the Zhao and Wen (2018) initiated a research program focused on the confinement mechanism of steel-confined concrete and developed a stress-strain model for steel-confined concrete by modifying the key parameters of Mander's model; this modified model yielded satisfactory results compared with experimental results (Zhao et al. 2014). However, the material strength of the specimens was normal. Considering more extensive use of high strength materials, the confinement mechanism in high-strength steel-concrete composite columns is worth studying.

Clearly, if the confinement mechanism of both the steel-confined and hoop-confined concrete can be understood and properly considered, the theoretical stress-strain model for concrete in SRC columns can be proposed. It is well known that the confinement mechanism of hoop-confined concrete has been intensively studied by Kent and Park (1971), Sheikh and Uzumeri (1982), Scott el al. (1982), Park et al. (1982), Mander et al. (1988) and Cusson and Paultre (1995). In contrast, limited analytical studies (El-Tawil and Deierlein 1996; Chen and Wu 2017) have been conducted on the confinement mechanism of steel-confined concrete. Therefore, the current stress-strain model for the concrete in SRC columns, especially for HCC, may be not appropriate. The stress-strain model for the concrete in SRC columns also requires further study.

In this paper, an experimental study was carried out on 6 axially loaded high-strength SRC core columns to investigate the confinement mechanism of steel-confined concrete. Then, considering the confinement mechanism of both steel-confined and hoop-confined concrete, the concrete in the SRC columns was divided into four regions according to the confinement level, and a theoretical stress-strain model for each concrete region was proposed by considering the model of Mander et al. (1988). This model possesses the wide application and reliability according to the theoretical basis of confinement mechanism of both steel-confined and hoop-confined concrete. Finally, these stress-strain relationships according to the proposed methodology were implemented in the analytical model proposed by Chen et al. (2006) and Chen and Wu (2017) and the comparisons of the axial load-displacement relationships of the axially loaded SRC short columns between the experimental and predicted results were conducted to validate the theoretical stress-strain model.

Experimental Program

Specimen Design

A total of 6 high-strength SRC core short columns were tested under axial compression. Because this experiment was conducted to investigate the confinement mechanism of steel-confined concrete, the influence of hoop-confined concrete was eliminated and the specimens were designed to only consist of two components: a structural steel section and concrete. For the specimens in Zhao et al. (2014), the grade of concrete ranged from C35 to C50 and the grade of the structural steel section ranged from Q235 to Q345. To reflect the high-strength in this study, the grade of concrete ranged from C60 to C70 and the grade of the structural steel section ranged from Q390 to Q460. Considering the influential factors of the confinement on the concrete, i.e. the area ratio of steel to concrete, confined dimension of the steel flange and the strength combination of the steel and concrete, the configurations of the specimens were designed and shown in Fig. 2, where l_f is the clear width of the steel flange and it is also the confined dimension of the steel flange, l_c is the dimension of the steel flange to its opposite steel web, t_w is the thickness of the steel web, t_f is the thickness of the steel flange, and b_f is the width of the steel flange. The height of the specimens was 1200 mm. The material properties of the structural steel shapes, including the elastic modulus E_s, the yield strength of the steel web and flange ($f_{y,w}$ and $f_{y,f}$), the ultimate tensile strength of steel web and flange ($f_{u,w}$ and $f_{u,f}$), the percentage elongation of structural steel δ_{10} and the cylinder compressive strength of concrete $f_c^{'}$ are listed in Table 1. The aluminum bars were cast within the concrete inside the specimens to measure the strain of internal concrete. The diameter of aluminum was 4 mm. The strength corresponding to residual strain of 0.002 was 242MPa and the ultimate strength was 320MPa.

Each specimen was labeled by a common form as SRC-*i*-*j*, where *i* is the type of structural steel and *j* represents the grade of the material strengths. For instance, the label "SRC-A-1" refers to the specimen with a type-A encased structural steel section, for which the grade of the concrete is C60 and the grade of the steel is Q390.

Test Setup and Loading System

Fig. 3(a) presents the experimental setup and loading system. The FCS-30000kN electrohydraulic servo coordinate loading system with a capacity of a 30,000 kN load and 400 mm of maximum displacement in the vertical direction was used for this test. The two end plates of the specimens were fixed to the testing machine. The upper endplate was restrained against all degrees of freedom, and the lower endplate, the loading end, could move in only the vertical direction during the loading process.

The displacement-controlled method was adopted as the loading method in this test. The loading rate was changed from 0.6 mm/min to 3 mm/min when a decrease in the measured axial load was observed during the loading process. The application of the load was stopped when the vertical displacement of the specimens exceeded 60 mm, which was regarded to indicate specimen failure.

Measurement Arrangement

For each specimen, the displacement of the specimen and the strains of the structural steel shapes and concrete were continuously measured and recorded during the loading process. The arrangement of the displacement transducers is presented in Fig. 3(b). Transducers D1~D4 were placed on four sides along the height of the specimen to measure the overall vertical displacement, which was also used to verify the concentricity of the applied axial load during the early loading process. Transducers D5~D16 were used to measure the horizontal displacement of the steel flange at the mid-height of the specimen to investigate the out-of-plane bending deformation of the steel flange due to the horizontal expansion of the concrete under compression.

The arrangement of the strain gauges is presented in Fig. 3(c). In section 1-1, gauges S1~S8 were used to obtain the strains of the steel flange; gauges S9~S12 were used to measure the strains of the steel web; and gauges S13~S17, which were attached to 5 vertical aluminum bars, were used to measure the vertical strains of the concrete, given that the inside of the aluminum bars deform with the concrete. In section 2-2, gauges S52~S61, which were attached to 6 horizontal aluminum bars, were used to measure the horizontal strains of the concrete. The 11 strain rosettes, S18~S28, were used to trace the strain distribution of the steel flange; S38~S41 were used to measure the vertical and lateral strains of the concrete surface; and gauges S29~S37 and S42~S51 were designed to not only obtain the vertical strains of the structural steel section but also ensure that the application of the axial load was concentric.

Experimental Results

General Behavior

Two idealized axial load-displacement relationships are presented in Fig. 4(a) as curve A and curve B and the test results of axial load-displacement relationships are shown in Fig. 4(b). Curve A and curve B are the same before point V. Point Y refers to the yield load, N_y, which is calculated by $N_y = f_y \cdot A_s + f_c^{'} \cdot A_c$, where f_y is the yield strength of the structural steel section, A_s is the cross-sectional area of the structural steel, $f_c^{'}$ is the cylindrical compressive strength of the UCC, and A_c is the cross-sectional area of the concrete. Point P is the peak point and indicates that the whole section of the specimen is in a plastic state and the concrete confinement reaches its maximum value. Along the curve B, the load starts to fall at a constant rate after point P because the load increase from the structural steel strengthening is less than the load reduction due to the degradation of the concrete. In contrast, along curve A, the axial load will not decrease until it reaches the balanced load N_v, which indicates that the effect on the capacity of the specimen achieves a balance between the degradation of the concrete and strengthening of the steel; then, the load will increase because the effect of the steel strengthening will be superior to that of the concrete degradation after point V.

The specimen SRC-A-1 is used as an example to illustrate the representative phenomenon during each stage of the curve:

1. Elastic stage (OY): Both the structural steel shapes and concrete exhibited linear behavior in the early stage of the loading process and the load-displacement curve is linear tendency prior to the yield load N_y. No cracks were observed on the concrete surface, as shown in Fig. 5(a).
2. Elasto-plastic stage (YP): The load-displacement curve became nonlinear after the load reached point Y, owing to the nonlinear characteristics of the materials. The concrete confinement provided by the structural steel shapes achieved its maximum value when the load reached point P. Several vertical cracks were observed near the mid-height of the specimen (see Fig. 5(b)) and formed gradually from one side of the specimen to the other sides due to the initial imperfection.
3. Strength degradation stage (after point P): In this stage, because the increase in load due to the steel strengthening was always less than the reduction of the load due to concrete degradation after point P, the load continuously decreased at a relatively slow rate until the specimen failed. The cracks in the concrete developed rapidly, and several large pieces of concrete started to spall, leading to the separation between the steel flange and concrete, as shown in Fig. 5(c).

Confinement Mechanism of Steel-Confined Concrete

The specimens expanded horizontally under the axial compression during the loading process. The lateral displacement of the steel flange and the strain of the steel and concrete were measured to investigate the confinement mechanism of steel-confined concrete.

Fig. 6(a) shows the relationship between N / N_y and Δ_d, where N / N_y is the ratio of the applied load N to the yield load N_y, and Δ_d is the average difference in the horizontal expansion between the average measured value of the two sideward displacement transducers and that of the middle displacement transducer. There is little difference in the horizontal expansion prior to the yield load N_y (see Fig. 4(a)); however, once the yield load N_y is reached, the horizontal expansion difference Δ_d starts to increase, which indicates that the steel flanges deform out-of-plane and provide the effective confinement to the concrete. Then, when the load reaches the peak load N_p, the concrete confinement peaks and begins to decrease because the steel flanges yield and cannot provide enough confinement to the concrete as bending deformation rapidly increases, indicating that the compressive strength of the concrete starts to decrease due to the degeneration of the concrete confinement.

Except for the out-of-plane bending deformation of steel flange, the strains of steel flange can also reflect the confinement mechanism of steel-confined concrete. Since the Poisson’s ratio of materials cannot be obtained directly from the test, the absolute value of the strain ratio of the horizontal to vertical strains is obtained to reflect the change in Poisson’s ratio. Fig. 6(b) presents the relationship between N / N_y and $\left|\varepsilon_{ch} / \varepsilon_{cv}\right|$, where ε_{ch} and ε_{cv} are the average horizontal and vertical strains of the concrete. Fig. 6(c) shows the relationship between N / N_y and $|\varepsilon_{sh} / \varepsilon_{sv}|$, where ε_{sh} and ε_{sv} are the average horizontal and vertical strain of the structural steel section. In general, $|\varepsilon_{sh} / \varepsilon_{sv}|$ is close to 0.3 and $|\varepsilon_{ch} / \varepsilon_{cv}|$ is close to 0.2 in the elastic stage. The absolute values of the strain ratio for either the steel or concrete show a good agreement with the typical Poisson's ratios. Then, $|\varepsilon_{ch} / \varepsilon_{cv}|$ begins to increase rapidly when the axial load reaches 0.75 N_y; however, because the structural steel section enters a plastic state later than that of the concrete, $|\varepsilon_{sh} / \varepsilon_{sv}|$ does not change substantially until the load reaches the yield load N_y. Therefore, the increase in $|\varepsilon_{sh} / \varepsilon_{sv}|$ after 0.75 N_y represents that the concrete confinement increases gradually. After the load reaches N_p, $|\varepsilon_{sh} / \varepsilon_{sv}|$ starts to increase rapidly, indicating that the concrete confinement provided by the structural steel section starts to decrease due to the excessive plastic deformation of the steel flanges.

As described above, the confinement mechanism of steel-confined concrete in high-strength SRC core columns, which is similar with that in normal-strength SRC core columns (Zhao et al. 2014), can be described as follows: both the structural steel shapes and concrete are in an elastic stage before the axial load achieves 0.75 N_y; since the difference in the lateral expansion deformation between the steel flange and concrete is limited, the confinement effect of the structural steel shapes on the concrete is limited before the 0.75 N_y; after 0.75 N_y, $|\varepsilon_{ch} / \varepsilon_{cv}|$ starts to increase due to the plastic behavior of the concrete, but $|\varepsilon_{sh} / \varepsilon_{sv}|$ remains approximately unchanged. Therefore, the concrete confinement begins to increase gradually and reaches its maximum value when the load reaches the peak load N_p. Then, the steel flanges yield and deform rapidly, resulting in the reduction of the concrete confinement and the degeneration of compressive strength of the concrete.

However, the concrete confinement cannot be provided uniformly in all directions due to the open-type section of the structural steel shapes. Therefore, considering the distribution of the confinement effect in different directions, the steel-confined concrete can be divided into three regions according to the confinement mechanism, as indicated by the red lines in section 2-2 in Fig. 3(c), i.e. highly steel-confined concrete (HSCC, region I), partially steel-confined concrete (PSCC, region II) and UCC (region III). The region I (blue part) can be considered to sustain the lateral confining stress in any directions; the region II (pink part) can be considered to sustain the lateral steel confining stress in only one direction; and the region III (yellow part) can be considered to sustain no lateral steel confining stress in any directions.

Based on the regional partition of the steel-confined concrete mentioned above, Fig. 7 illustrates the relationships between the axial load and horizontal strain of the different steel-confined concrete regions. The horizontal strain is the average measured value of the strain gauges attached on the horizontal aluminum bars in the same region. The concrete horizontal strain increases from the HSCC to PSCC to UCC under the same axial load; therefore, different regions of the steel-confined concrete experience different levels of confinement effect. As such, the experimental investigation in this section provides a theoretical basis for the regional partition of the steel-confined concrete proposed in this paper.

Influential factors on the confinement of concrete

The test results are listed in Table 2. The ratio of the peak load to yield load varied from 1.04 to 1.15. The strength and deformation capacity of concrete were enhanced due to the confinement provided by the structural steel section, but the enhancement of strength could not be directly measured from the experiment. The axial compressive load of specimens was shared by the steel section and concrete. When the peak load was reached, the load shared by the steel section N_s was calculated by the measured yield strength times the area of the steel section and thus the load shared by the concrete N_c was obtained by subtracting N_s from the peak load N_p. The enhanced strength of concrete f_{ci} was got by dividing N_c by the area of concrete. The strength incremental factor k_{ci} was defined as the ratio of $f_{ci} / f_c^{'}$ to reflect the strength enhancement of concrete. Accordingly, the displacement ratio of Δ_p / Δ_y, where Δ_y and Δ_p are the displacements corresponding to the yield load and peak load, revealed the improvement of peak strain.

Fig. 8(a) presents the test results of specimens SRC-A-1, SRC-B-1, SRC-A-2 and SRC-B-2, which illustrates the effect of the area ratio A_s / A_c on the strength incremental factor and displacement ratio Δ_p / Δ_y. Obviously, k_{ci} increases with the increasing area ratio A_s / A_c, and the strength enhancement of concrete is more significant for the strength combination of the grade of steel Q390 and grade of concrete C60. However, the area ratio possesses limited effect on the Δ_p / Δ_y; for the strength combination of grade of steel Q390 and grade of concrete C60, the Δ_p / Δ_y decreases by 1% when the area ratio A_s / A_c varies from 0.16 to 0.33, and 1% increase was observed for strength combination of grade of steel Q460 and grade of concrete C70. Therefore, the area ratio A_s / A_c has more positive effect on k_{ci} compared with Δ_p / Δ_y. As shown in Fig. 8(b), the test results of specimens SRC-A-1 and SRC-C-1 are discussed to investigate the effect of confined dimension of steel flange l_f on the k_{ci} and Δ_p / Δ_y. Both k_{ci} and Δ_p / Δ_y are enhanced when the l_f increases from 50mm to 75mm. The k_{ci} increases by 23% and Δ_p / Δ_y increases by 8%. It is thus evident that larger confined dimension of the steel flange can offer more confinement for concrete. The effect of the strength combination on the k_{ci} and Δ_p / Δ_y was investigated by comparing with the test results of specimens SRC-B-1, SRC-B-2 and SRC-B-3 as presented in Fig. 8(c). From the comparisons between the specimens SRC-B-1 and SRC-B-3, it is clear that k_{ci} and Δ_p / Δ_y decrease with individual increase of steel strength. However, the individual increase of concrete strength is only beneficial for Δ_p / Δ_y from the comparisons between the specimens SRC-B-2 and SRC-B-3.

Proposed Theoretical Stress-Strain Model for Concrete in SRC Columns

The confinement mechanism of steel-confined concrete from normal-strength to high-strength SRC columns has been investigated. The confinement mechanism of both steel-confined and hoop-confined concrete can be considered and adopted to propose a theoretical stress-strain relationship for the concrete in SRC columns. The proposed procedure for the theoretical stress-strain model is shown in Fig. 9. The four steps, including the region partition, effective lateral confining stress, strength incremental factor and stress-strain relationship, will be presented in detail as follows.

Referring to Fig. 1(b), the HCC is confined by both structural steel shapes and hoops and it can be classified into two components, HSCC and PSCC. Therefore, the concrete in SRC columns is considered to be comprised of 4 distinct regions, as shown in Fig. 10: HSCC, PSCC, PCC and

UCC. Each of these regions can be represented by the stress-strain model according to different confinement levels by considering the model of Mander et al. (1988).

Mander's model contains two key parameters: the effective lateral confining stress $f_l^{'}$ and strength incremental factor k. The former parameter is the pressure of the concrete in the horizontal direction from the hoops, and the later parameter refers to the enhancement of the concrete compressive strength due to confinement. Hence, if the effective confining stress can be obtained by considering the theoretical basis, the confinement factors will then be determined and the stress-strain model for concrete confined by arbitrary objects, such as hoops, steel tubes and structural steel shapes, can be proposed accordingly.

For Mander's model, the compressive strength of concrete is given by:

$$\sigma = \frac{f_{cc}^{'} \cdot x \cdot r}{r - 1 + x^r} \tag{1}$$

with

$$f_{cc}^{'} = k f_c^{'} \tag{2}$$

$$x = \frac{\varepsilon}{\varepsilon_{cc}} \tag{3}$$

$$r = \frac{E_c}{E_c - E_{\sec}} \tag{4}$$

$$E_c = 5000 f_c^{'0.5} \tag{5}$$

$$E_{\sec} = \frac{f_{cc}^{'}}{\varepsilon_{cc}} \tag{6}$$

where $f_{cc}^{'}$ is the compressive strength of the confined concrete, ε_{cc} is the compressive strain of the confined concrete corresponding to $f_{cc}^{'}$, E_c is the elasticity modulus of UCC and is estimated from Equation (5), and $E_{\sec}$ is the secant modulus of the confined concrete.

Mander et al. (1988) determined that the strength incremental factor of confined concrete k was influenced by two effective lateral confining stresses, f_{l1} and f_{l2} ($f_{l1} \leq f_{l2} \leq 0.3 f_c^{'}$), as shown in Fig. 11. Then, El-Tawil and Deierlein (1996) described those curves by a set of equations, considering the following two cases: (1) confined concrete under f_{l1} and $f_{l2} = 0.3 f_c^{'}$ and (2) confined concrete under $f_{l1} = f_{l2} \leq 0.3 f_c^{'}$. These cases represent upper and lower limiting conditions for the confining stress f_{l1}, respectively.

The strength incremental factor k_h corresponding to case (1) can be described by the following equation (El-Tawil and Deierlein 1996):

$$k_h = 1.3 + 5.55 \frac{f_{l1}}{f_c^{'}} - 7.5 \left(\frac{f_{l1}}{f_c^{'}} \right)^2 \tag{7}$$

The strength incremental factor k_l corresponding to case (2) can be described by the following equation (Mander et al. 1988):

$$k_l = -1.254 + 2.254 \sqrt{1 + \frac{7.94 f_{l1}}{f_c^{'}}} - 2 \frac{f_{l1}}{f_c^{'}} \tag{8}$$

The required strength incremental factor k corresponding to the two effective lateral confining stresses f_{l1} and f_{l2} is obtained by interpolating between the upper and lower limits (El-Tawil and Deierlein 1996):

$$k = k_l + \left(k_h - k_l \right) \sqrt{\frac{\left(\frac{f_{l2}}{f_c^{'}} - \frac{f_{l1}}{f_c^{'}} \right)}{\left(0.3 - \frac{f_{l1}}{f_c^{'}} \right)}} \tag{9}$$

The compressive strain of the confined concrete corresponding to $f_{cc}^{'}$ is given by

$$\varepsilon_{cc} = \varepsilon_{co} \cdot \left[1 + 5 \left(k - 1 \right) \right] \tag{10}$$

where ε_{co} is the compressive strain corresponding to the cylinder compressive strength of UCC and is 0.002 for concrete with a cylinder compressive strength under 28 MPa and 0.003 for concrete with a cylinder compressive strength over 82 MPa (El-Tawil and Deierlein 1996).

Model for HSCC and PSCC

- *Effective Lateral Confining Stress on HSCC and PSCC*

In SRC columns, the HSCC and PSCC regions are confined by both the structural steel section and hoops, and the lateral confining stress is schematically shown in Fig. 12. Hence, the effective lateral confining stresses on the HSCC and PSCC, $f'_{l,hs}$ and $f'_{l,ps}$, can be given by (El-Tawil and Deierlein 1996):

for HSCC $$f'_{l,hs} = f'_{l,h} + f'_{l,w} \tag{11}$$

for PSCC $$f'_{l,ps} = f'_{l,h} + f'_{l,f} \tag{12}$$

where $f'_{l,h}$, $f'_{l,w}$ and $f'_{l,f}$ are the effective lateral confining stresses from the hoops, steel web and steel flange, respectively.

Considering the interaction between the structural steel section and concrete in the SRC columns, the effective lateral confining stress from the steel web and steel flange can be determined as follows:

$$f'_{l,w} = K'_{e,hs} f_{l,f} \tag{13}$$

$$f'_{l,f} = K'_{e,ps} f_{l,f} \tag{14}$$

Where $f_{l,f}$ is the lateral confining stress from the steel flange, $K'_{e,hs}$ and $K'_{e,ps}$ are the confinement effectiveness coefficients for the HSCC and PSCC, respectively.

With the expansion of concrete under axial compression, the steel flange can be considered as a cantilever beam and the lateral confining stress is calculated based on the resistance provided by the steel flange (El-Tawil and Deierlein 1996; Zhao and Wen 2018). The bending moment applied at the base of the flange can be equivalent to the resistance developed by the steel flange (see Fig. 13) and the first yielding of the steel flange is considered as the limiting condition:

$$\frac{f_{l,f} \cdot l_f^2}{2} = \frac{t_f^2 \cdot f_{y,f}}{6} \tag{15}$$

The lateral confining stress is

$$f_{l,f} = \frac{t_f^2 \cdot f_{y,f}}{3 \cdot l_f^2} \tag{16}$$

where $f_{y,f}$ is the yield strength of the steel flange.

From the experiment performed in this study, a quarter of the steel-confined concrete is highlighted in the following discussion due to the biaxial symmetry of the cross section. According to the equilibrium condition and finite element analysis (Zhao and Wen 2018), the actual lateral confining stress distributes mainly around the intersection between the flanges and adjacent webs, as shown in Fig. 14(a). More specifically, the lateral confining stress is mostly distributed at the sides of the steel web EG and EF rather than those of the steel web AG and FD. Therefore, on the basis of the regional partition as shown in Fig. 3(c), the lateral confining stress provided by the steel web, AG and FD, is ignored, and the distribution of the lateral confining stress is simplified as several rectangular areas in accordance with El-Tawil's assumption as shown in Fig. 14(b). The lateral confining stress $f_{l,f}$ can also be obtained by Eq. (16).

- *Confinement Effectiveness Coefficient of HSCC and PSCC*

Regarding the confinement effectiveness coefficient of the concrete confined by H-shaped steel, Mirza and Skrabek (1992) assumed that the effective confined concrete area was the total concrete area minus the area of the two parabolas with heights equal to one quarter their length (see Fig. 15(a)). Using a similar procedure to that proposed by Mander et al. (1988) to define the confinement effectiveness, the confinement effectiveness coefficient $K_e^{'}$ is

$$K_e^{'} = \frac{\text{effective confined concrete area}}{\text{total confined concrete area}} \tag{17}$$

It is necessary to consider the effect on the distribution of the lateral confining stress due to the out-of-plane bending deformation of the steel flange. Fig. 15(b) shows that the arching action is assumed to occur in the form of a second-degree parabola with an initial tangent slope of 45° at each side (Huang et al. 2008). Considering both the arching action and out-of-plane bending deformation of the steel flange, the effective confined region of the H-shaped steel-confined concrete is defined as shown in Fig. 15(c) (Zhao and Wen 2018).

As show in Fig. 16 (a), the blue part in the upper left is the HSCC. The sides GE and EF are connected with the steel web, but the sides GO and OF are connected with the PSCC. The HSCC sustains the lateral confining stress all around. More specifically, the sides GE and EF sustain the confinement provided by the steel web and the sides GO and OF sustain the confinement essentially provided by the steel flange. Therefore, in accordance with the region partition of concrete, the black region is equivalent to the steel component to present the function of the lateral confining stress. Considering that the steel web is located at the symmetric axis of the cross section, the steel web cannot undergo bending deformation. However, the steel flanges will provide confinement for concrete with out-of-plane bending deformation and the sides GO and OF are disconnected. Hence, the effective confined region of the HSCC is defined as shown in Fig. 16(a) (Zhao and Wen 2018), based on which, the confinement effectiveness coefficient of the HSCC $K_{e,hs}^{'}$ can be obtained by Eq. (17).

Regarding the PSCC, the pink part in the upper left is the PSCC. The side AG is connected with steel web and the side AB is connected with the steel flange. The PSCC sustains the lateral confining stress in only one direction. Based on the same region partition, the simplified distribution of the lateral confining stress was put forward as shown in Fig. 16(b), where the black region is also equivalent to the steel component to present the function of lateral confining stress. The side AB will undergo bending deformation, but the side GO will not deform substantially due to the restriction of the surrounding concrete. The effective confined region of PSCC is considered as half of the effective confined region of the H-shaped confined concrete (see Fig. 15(c)) and is defined as shown in Fig. 16(b) (Zhao and Wen 2018). Therefore, the confinement effectiveness coefficient of PSCC $K_{e,ps}^{'}$ can also be obtained by Eq. (17).

The strength incremental factors of the HSCC and PSCC, k_{hs} and k_{ps}, can be obtained by the Eqs. (7)~(9).

Model for PCC and UCC

The stress-strain relationship for PCC is proposed based on Mander's model. However, the experimental studies conducted by Sheikh and Uzumeri (1982) indicated that the hoops did not yield when the concrete confinement provided by the hoops peaked, reflecting that some modification is needed to consider the actual stress in the hoops to calculate the confining stress from hoops to concrete. Cusson and Paultre (1995) proposed an iterative procedure to determine the stress in hoops, but this procedure is extremely complicated when the hoops do not yield. Chen and Wu (2017) put forward a simplified method to obtain the actual stress in the hoops f_{rh}. A relationship between f_{rh} and ξ was proposed, where ξ was defined as the confinement effectiveness factor for PCC and is determined by $\xi = K'_{e,h}\rho_{sh}$. $K'_{e,h}$ is the confinement effectiveness coefficient for PCC, and ρ_{sh} is the volume ratio of the hoops. The details of this method to determine the actual stress in the hoops f_{rh} are provided in Chen and Wu (2017). The effective lateral confining stress from the hoops $f'_{l,h}$ is expressed by

$$f'_{l,h} = K'_{e,h} f_{rh} \tag{18}$$

The confinement effectiveness coefficient for PCC $K'_{e,h}$ can be determined according to Mander's model (Mander et al. 1988). Furthermore, the strength incremental factor of PCC k_p can be obtained from the Eqs. (7)~(9), but the strength incremental factor k_c is 1.0 because the UCC is not confined by either structural steel shapes or hoops.

Verification of Proposed Stress-Strain Model for Concrete

To verify the theoretical stress-strain model for the concrete in SRC columns proposed in this paper, it was implemented into the analytical model proposed by Chen et al. (2006) and Chen and Wu (2017) to predict the axial load-displacement relationships and ultimate load from previous experiments on axially loaded SRC short columns. In previous studies, adopting this analytical model, the cylinder compressive strength of concrete strength ranged from 23.5MPa to 94.0MPa and the yield strength of structural steel ranged from 235MPa to 517MPa. The short columns referred to the columns whose ratios of height to width were less than 4.5.

Analytical Modeling for SRC Columns

According to the section analysis method, the analytical model can accurately reflect the load-displacement response and axial capacity of SRC columns by adding the contributions of different components, including the steel flange, steel web, longitudinal bars, HSCC, PSCC, PCC and UCC. Chen and Wu (2017) proposed the following assumptions: (1) the axial compressive strains of each component are uniform; (2) the compressive stress of each component can be obtained according to the stress-strain model for each material and (3) the second-order effect is ignored.

The analytical axial load P is given by

$$P = A_{cc,hs} \cdot \sigma_{cc,hs} + A_{cc,ps} \cdot \sigma_{cc,ps} + A_{cc,p} \cdot \sigma_{cc,p} + A_{cc,c} \cdot \sigma_{cc,c} + A_{s,f} \cdot \sigma_{s,f} + A_{s,w} \cdot \sigma_{s,w} + A_{s,l} \cdot \sigma_{s,l} \tag{19}$$

where $A_{cc,hs}$ and $\sigma_{cc,hs}$ are the cross-sectional area and compressive stress of the HSCC, $A_{cc,ps}$ and $\sigma_{cc,ps}$ are the cross-sectional area and compressive stress of the PSCC, $A_{cc,p}$ and $\sigma_{cc,p}$ are the cross-sectional area and compressive stress of the PCC, $A_{cc,c}$ and $\sigma_{cc,c}$ are the cross-sectional area and compressive stress of the UCC, $A_{s,f}$ and $\sigma_{s,f}$ are the cross-sectional area and compressive stress of the steel flange, $A_{s,w}$ and $\sigma_{s,w}$ are the cross-sectional area and compressive stress of the steel web, and $A_{s,l}$ and $\sigma_{s,l}$ are the cross-sectional area and compressive stress of the longitudinal bars.

- *Stress-Strain Model for Concrete*

According to the discussion provided above, the concrete in SRC columns is divided into 4 regions and the theoretical stress-strain model is proposed by the following process:

1. Divide the concrete according to the confinement mechanism of steel-confined and hoop-confined concrete, i.e., into HSCC, PSCC, PCC and UCC regions, as shown in Fig. 10.

2. Calculate the effective lateral confining stress of each concrete region by the steps provided in the analytical modeling process.
3. Calculate the strength incremental factor of each concrete region by using Eqs. (7)~(9).
4. Propose a stress-strain model for each concrete region based on Mander's model.

The stress-strain models for HSCC, PSCC, PCC and UCC proposed by the above steps are presented in Fig. 17(a), where $\varepsilon_{cc,hs}$, $\varepsilon_{cc,ps}$, $\varepsilon_{cc,p}$ and $\varepsilon_{cc,c}$ are the strains corresponding to the compressive strength of HSCC, PSCC, PCC and UCC. Given the difference in concrete confinement among the four concrete regions, the ultimate strains of PSCC and HSCC, $\varepsilon_{cu,hs}$ and $\varepsilon_{cu,ps}$, are assumed to correspond to $0.7k_{hs} f_c^{'}$ and $0.7k_{ps} f_c^{'}$; the ultimate strains of PCC and UCC, $\varepsilon_{cu,p}$ and $\varepsilon_{cu,c}$, are assumed to correspond to $0.5k_p f_c^{'}$ and $0.2k_c f_c^{'}$.

- *Stress-Strain Model for Structural Steel Sections*

With the expansion of the inner concrete under compression, the steel flange provides confinement for the concrete that undergoes out-of-plane bending deformation. In this case, the steel web is in a tensile state in the horizontal direction, weakening the compressive strength of the steel web. Chen and Wu (2017) considered the biaxial stress state of steel web and adopted the modified Giuffrè-Menegotto-Pinto model to propose the stress-strain model for the steel web. The horizontal tensile stress of steel web $f_{yt,w}$, corresponding to the limiting condition adopted in Eq. (15), can be calculated according to force equilibrium and is given by

$$f_{yt,w} = \frac{2 \cdot l_f \cdot f_{l,f}^{'}}{t_w} \tag{20}$$

The axial compressive stress of the steel web $f_{yv,w}$ can be obtained according to the biaxial stress ellipse theory (Chen and Wu 2017), which is given by

$$f_{yv,w}{}^{2} + f_{yt,w}{}^{2} - f_{yv,w} \cdot f_{yt,w} = f_y^2 \tag{21}$$

As for steel flange, the Giuffrè-Menegotto-Pinto model (Giuffrè and Pinto, 1970) is also applicable. Drawn on the definition of Chen and Wu (2017), the parameters in the Giuffrè-Menegotto-Pinto model are defined as follows. The constant representing the radius of the transitional area between the elastic range and hardening branch $R = 10$, the ratio β between the hardening stiffness E_{sh} and the elastic modulus E_s of steel is given by

$$\beta = \frac{E_{sh}}{E_s} = \frac{\left(f_u - f_y\right)/\left(\varepsilon_u - \varepsilon_y\right)}{E_s} \tag{22}$$

where f_u is the ultimate strength of the structural steel, and ε_y and ε_u are the strains corresponding to the yield and ultimate strength of the structural steel. The elastic modulus of the steel E_s is 206 GPa. The proposed stress-strain models for steel section are illustrated in Fig. 17(b). In these models, ε_y and $\varepsilon_{yv,w}$ are the strains corresponding to f_y and $f_{yv,w}$, and $E_{sh,f}$ and $E_{sh,w}$ are the hardening stiffness of the steel flange and steel web.

- *Stress-Strain Model for Longitudinal Bars*

The stress-strain model for longitudinal bars proposed by Chen and Lin (2006) is adopted in this analytical model, as shown in Fig. 17(c). It is assumed that the longitudinal bar will undergo yielding during a yield plateau under compression. The compressive stress of longitudinal bars start to decrease when the axial compressive strain is ε_{co}. Considering the bucking of the bars after the concrete cover undergoes crushing, the compressive stress of the bar will drop to 20% of the yield strength and then maintain a constant value (Chen and Lin, 2006).

Comparison of Analytical Results with Experimental Results

Using the above analytical model, the predicted results of the axial compressive behavior of SRC short columns are obtained and compared with the experimental results of axially loaded SRC short columns to validate the stress-strain model proposed in this study.

The specimens are classified into two groups. One group describes the SRC core columns that are comprised of only structural steel section and concrete, but the other group describes the ordinary SRC columns. A total of 101 specimens consisting of 34 SRC core columns (Chen et al. 2010; Zhao et al. 2014 and the experiments studied in this paper) and 67 SRC columns (Chen and Yeh 1996; Tsai et al. 1996; Weng et al. 2006; Chen et al. 2010; Shih et al. 2013; Liang et al. 2014; Chen et al. 2015; Liu et al. 2015; Zhu et al. 2015; Xiao et al. 2017) are investigated in this section. Apart from the specimens studied in this paper, the geometry and material properties of several representative specimens are summarized in Table 3 and the whole data are listed in Table S1, including the cross-sectional dimensions, measured yield strength and ultimate tensile strength of the steel flange and steel web, the yield strength and configuration of the longitudinal bars and hoops, and the cylinder compressive strength of the UCC. The cylinder compressive strength of the UCC ranges from 20.6 MPa to 94.0 MPa and the yield strength of the structural steel ranges from 235 MPa to 517 MPa, which is suitable for the range of parameters this analytical model has been validated. More details can be found in the corresponding references.

The load ratios of the predicted axial bearing capacity to yield load $N_{p,a} / N_y$ are presented in Table 2. The load ratio $N_{p,a} / N_y$ ranges from 1.06~1.12 and the average error calculation between $N_{p,a} / N_y$ and $N_{p,t} / N_y$ is 2.67%, which indicates that the analytical model can accurately reflect the enhancement of the bearing capacity of specimens in this paper due to the enhancement of the concrete strength. The parameters obtained according to the analytical model are tabulated in Table 4. For some specimens, the stresses in the hoops are less than the yield strength, reflecting that it is effective to adopt the procedure of Chen and Wu (2017) to avoid overestimating the lateral confining stress from the hoops. The strength incremental factors for the HSCC range from 1.23 to 2.07, the strength incremental factors for the PSCC range from 1.09 to 1.48, and the strength incremental factors for the PCC range from 1.05 to 1.36. Clearly, for most specimens, the confinement of the HSCC is highest because HSCC is confined by both the steel section and hoops in two directions. Therefore, the concrete confinement provided by the structural steel section cannot be ignored by the comparisons among the strength incremental factors for the HSCC, PSCC and PCC.

Fig. 18(a) shows the comparisons of the experimental and analytical axial load-displacement curves for the tests studied in this paper and some previously performed representative tests. All results substantially begin at the origin but the starting points of each result are at certain intervals to clearly present the comparisons. The predicted results obtained by using the analytical model are in good agreement with the experimental results in terms of the initial stiffness, axial bearing capacity and loading path. Additionally, the axial bearing capacities are also compared, as shown in Fig. 18(b). Table 4 shows the results of the axial bearing capacity from the experiment and analytical model, $N_{p,t}$ and $N_{p,a}$, for the specimens corresponding to Table 3. Overall, the absolute error of 82% of specimens is less than 7%, and the absolute average error of all specimens is 4.21%. The average value of $N_{p,t} / N_{p,a}$ is 1.014 and the coefficient of variation is 0.0514, which indicates that the predicted capacities agree well with the test results. Fig. 18(c) also presents the comparisons of the axial load-displacement curves among the test results and results from three analytical models (Chen et al. 2006, Chen et al. 2017 and this paper). Both three analytical models can accurately predict the mechanical behaviors of SRC columns. Compared with the Chen et al. (2017) and this paper, the use of degrading model for structural steel applied in Chen et al. (2006) may overestimate the strength degeneration for the post-peak behavior, which can be found in the comparison of specimen HSRC-SP2. The average ratio of the predicted capacities according to analytical model (Chen et al. 2006) to test results is 1.013 and the coefficient of variation is 0.0368; the average ratio of the predicted capacities according to analytical model (Chen et al. 2017) to test results is 0.993 and the coefficient of variation is 0.0532. Even though the error levels of the predicted capacity among these three analytical models are similar, the stress-strain model for concrete proposed in this paper, unlike the stress-strain relationships in the other two models, possesses the theoretical basis of the confinement mechanism of both steel-confined and hoop-confined concrete. According to it, the analytical model proposed in this paper is suitable for the arbitrary SRC columns regardless of the configuration of the reinforcement, material strength and configuration of the encased steel section.

In general, the analytical model established in this section effectively predicts the axial compressive behavior of SRC columns, and the theoretical stress-strain model proposed in this study for the concrete in SRC columns is accurate and satisfactory.

Conclusions

An experimental study on 6 SRC core columns was carried out to investigate the confinement mechanism of steel-confined concrete in high-strength SRC columns; based on this work, a theoretical stress-strain model was proposed, considering the confinement mechanism of both structural steel shapes and hoop-confined concrete. Furthermore, this model was implemented in an analytical model to predict the axial compressive behavior of axially loaded SRC columns. Finally, the comparisons of the axial behavior between the experimental results and predicted results prove that the theoretical stress-strain model proposed in this study for the concrete in the SRC columns is accurate and effective. The conclusions in this paper can be drawn as follows:

1. Open-type structural steel shapes can provide confinement for concrete by the out-of-plane bending deformation of steel flanges to enhance the compressive strength and ductility of the concrete.
2. Based on the confinement mechanism of steel-confined and hoop-confined concrete, the concrete in the SRC columns is divided into four regions, namely, HSCC, PSCC, PCC and UCC.
3. The theoretical stress-strain model for the concrete in SRC columns is proposed according to the confinement mechanism in SRC columns, and comparisons of the axial behavior of SRC

columns between experimental and analytical results prove that this model is accurate and effective.

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Notation

The following symbols are used in this paper:

A_c = cross-sectional area of concrete

$A_{cc,c}$ = cross-sectional area of unconfined concrete

$A_{cc,hs}$ = cross-sectional area of highly steel-confined concrete

$A_{cc,p}$ = cross-sectional area of partially confined concrete

$A_{cc,ps}$ = cross-sectional area of partially steel-confined concrete

A_s = cross-sectional area of steel

$A_{s,f}$ = cross-sectional area of steel flange

$A_{s,l}$ = cross-sectional area of longitudinal bars

$A_{s,w}$ = cross-sectional area of steel web

b_f = width of steel flange

E_c = elastic modulus of unconfined concrete

E_s = elastic modulus of structural steel

E_{sh} = hardening stiffness of structural steel

$E_{sh,f}$ = hardening stiffness of steel flange

$E_{sh,w}$ = hardening stiffness of steel web

$E_{\sec}$ = secant modulus of confined concrete

f_{ci} = enhanced strength of concrete

$f_c^{'}$ = cylindrical compressive strength of unconfined concrete

$f_{cc}^{'}$ = compressive strength of confined concrete

f_l = lateral confining pressure

$f_l^{'}$ = effective lateral confining pressure

$f_{l,f}$ = lateral confining pressure from steel flange

$f_{l,f}^{'}$ = effective lateral confining pressure from steel flange

$f_{l,h}^{'}$ = effective lateral confining pressure from hoops

$f_{l,hs}^{'}$ = effective lateral confining pressure on highly steel-confined concrete

$f_{l,ps}^{'}$ = effective lateral confining pressure on partially steel-confined concrete

$f_{l,w}^{'}$ = effective lateral confining pressure from the steel web

f_{rh} = actual stress of hoops

f_u = ultimate strength of the structural steel

$f_{u,f}$ = ultimate strength of steel flange

$f_{u,w}$ = ultimate strength of steel web

f_y = yield strength of structural steel

$f_{y,f}$ = yield strength of steel flange

$f_{y,w}$ = yield strength of steel web

$f_{yt,w}$ = horizontal tensile stress of steel web

$f_{yv,w}$ = axial compressive stress of the steel web

$K_e^{'}$ = confinement effectiveness coefficient

$K_{e,c}^{'}$ = confinement effectiveness coefficient of unconfined concrete

$K_{e,h}^{'}$ = confinement effectiveness coefficient of hoop-confined concrete

$K_{e,hs}^{'}$ = confinement effectiveness coefficient of highly steel-confined concrete

$K_{e,ps}^{'}$ = confinement effectiveness coefficient of partially steel-confined concrete

k = strength incremental factor

k_c = strength incremental factor of unconfined concrete

k_{ci} = strength incremental factor of confined concrete from test results

k_{hs} = strength incremental factor of highly steel-confined concrete

k_p = strength incremental factor of partially confined concrete

k_{ps} = strength incremental factor of partially steel-confined concrete

l_c = dimension of steel flange to its opposite steel web

l_f = clear width of steel flange

N_c = load shared by concrete

N_p = peak load

$N_{p,a}$ = predicted peak load

$N_{p,t}$ = peak load of test results

N_s = load shared by steel

N_v = balanced load

N_y = yield load

P = analytical axial load

t_f = thickness of steel flange

t_w = thickness of steel web

Δ = axial displacement

Δ_b = displacement corresponding to balanced load

Δ_d = average difference in the horizontal expansion

Δ_p = displacement corresponding to peak load

Δ_y = displacement corresponding to yield load

$\sigma_{cc,c}$ = compressive stress of unconfined concrete

$\sigma_{cc,hs}$ = compressive stress of highly steel-confined concrete

$\sigma_{cc,p}$ = compressive stress of partially confined concrete

$\sigma_{cc,ps}$ = compressive stress of partially steel-confined concrete

$\sigma_{s,f}$ = compressive stress of steel flange

$\sigma_{s,l}$ = compressive stress of longitudinal bars

$\sigma_{s,w}$ = compressive stress of steel web

ε_{cc} = peak strain of the confined concrete

$\varepsilon_{cc,c}$ = peak strain of unconfined concrete

$\varepsilon_{cc,hs}$ = peak strain of highly steel-confined concrete

$\varepsilon_{cc,p}$ = peak strain of partially confined concrete

$\varepsilon_{cc,ps}$ = peak strain of partially steel-confined concrete

ε_{ch} = average horizontal strain of concrete

$\varepsilon_{cu,c}$ = ultimate strain of unconfined concrete

$\varepsilon_{cu,hs}$ = ultimate strain of partially steel-confined concrete

$\varepsilon_{cu,p}$ = ultimate strain of partially confined concrete

$\varepsilon_{cu,ps}$ = ultimate strain of partially steel-confined concrete

ε_{cv} = average vertical strain of concrete

ε_{sh} = average horizontal strain of structural steel

ε_{sv} = average vertical strain of structural steel

ε_{u} = ultimate strain of structural steel

ε_{y} = yield strain of structural steel

$\varepsilon_{yv,w}$ = strain corresponding to axial compressive stress of the steel web

ρ_{sh} = volume ratio of the hoops

ξ = confinement effectiveness for partially confined concrete

Supplemental Data

Tables S1 and S2 are available online in the ASCE Library (https://ascelibrary.com).

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Figure Captions

Fig. 1. Regions for the concrete in SRC columns: (a) Type I, UCC and confined concrete; (b) Type II, UCC, PCC and HCC

Fig. 2. (a) Configuration and (b) dimensions of specimens (in mm).

Fig. 3. Test arrangement: (a) test setup; (b) arrangement of displacement transducers; (c) arrangement of strain gages.

Fig. 4. Axial load-displacement relationships: (a) typical axial load-displacement relationship; (b) test results.

Fig. 5. Failure process of SRC-A-1: (a) elastic stage; (b) elasto-plastic stage; (c) strength degradation stage.

Fig. 6. Data analysis of confinement mechanism: ratio of the applied load to the yield load for (a) horizontal displacement difference of steel flange curves; (b) strain ratio curves for concrete; (c) strain ratio curves for steel flange.

Fig. 7. Axial load ratio–horizontal strain curves for each zone of steel confined concrete.

Fig. 8. Analyses of influential factors: (a) effect of the area ratio of steel to concrete; (b) effect of the confined dimension of the steel flange; (c) effect of the strength combination of steel and concrete.

Fig. 9. Proposed procedure for the theoretical stress-strain model for concrete in SRC columns.

Fig. 10. Regional partition of the concrete in SRC columns with different steel profiles.

Fig. 11. Relationship between effective lateral confining stresses and strength incremental factors. (Reprinted from Mander et al. 1988, © ASCE.)

Fig. 12. Lateral confining stress in an SRC column.

Fig. 13. Mechanism of steel-confined concrete.

Fig. 14. Distribution of lateral confining pressure: (a) actual distribution; (b) simplified distribution.

Fig. 15. Region for effective confined concrete: (a)H-shaped steel confined concrete; (b) steel tube confined concrete; (c) cross-shaped steel confined concrete.

Fig. 16. Simplified stress state and effective confined region: (a) HSCC; (b) PSCC.

Fig. 17. Theoretical stress-strain relationships in the analytical model: (a) concrete; (b) structural steel section; (c) longitudinal bars.

Fig. 18. Comparisons between experiments and analyses: (a) axial load-displacement curves; (b) axial compressive capacities; (c) curves for specimens SRC4 and HSRC-SP2

Table 1. Parameters and material properties of specimens

Specimen designation	Steel grade	Concrete grade	Area ratio A_s / A_c	l_f (mm)	Steel profile						Concrete
					E_s (MPa)	$f_{y,f}$ (MPa)	$f_{u,f}$ (MPa)	$f_{y,w}$ (MPa)	$f_{u,w}$ (MPa)	ε_u	$f_c^{'}$ (MPa)
SRC-A-1	Q390	C60	0.16	75	204×10^3	437	700	437	700	0.169	47.4
SRC-A-2	Q460	C70	0.16	75	201×10^3	460	724	460	724	0.172	64.7
SRC-B-1	Q390	C60	0.33	75	206×10^3	457	706	457	706	0.179	47.4
SRC-B-2	Q460	C70	0.33	75	204×10^3	508	730	508	730	0.168	64.7
SRC-B-3	Q460	C60	0.33	75	207×10^3	515	738	515	738	0.185	45.7
SRC-C-1	Q460	C60	0.15	50	206×10^3	437	700	437	700	0.174	47.4

Table 2. Test results

Specimen designation	Yield load N_y (kN)	Yield displacement Δ_y (mm)	Peak load N_p (kN)	Peak displacement Δ_y (mm)	Balanced load N_v (kN)	Load ratio $N_{p,t} / N_y$ (mm)	Load ratio $N_{p,a} / N_y$ (mm)	Displacement ratio Δ_p / Δ_y (mm)	Load of steel N_s (kN)	Load of concrete N_c (kN)	Strength incremental factor k_{ci}
SRC-A-1	8498	2.81	9741	4.20	-	1.15	1.12	1.49	5463	4279	1.25
SRC-A-2	9802	3.24	10428	4.33	-	1.06	1.10	1.34	5750	4678	1.02
SRC-B-1	14917	3.63	16997	5.34	16369	1.14	1.10	1.47	11882	5115	1.49
SRC-B-2	17260	3.36	18214	4.57	15238	1.06	1.07	1.36	13208	5006	1.10
SRC-B-3	16318	4.88	17024	6.16	16068	1.04	1.06	1.26	13390	3634	1.10
SRC-C-1	7286	3.25	7698	4.47	6559	1.06	1.07	1.38	4589	3109	1.02

Table 3. Main parameters of specimens

Specimen designation	Cross section (mm)	Structural steel section		Longitudinal bars	Hoops		Material strength (Mpa)							Reference
		Type	Size		Configuration	Type	$f_{y,f}$	$f_{u,f}$	$f_{y,w}$	$f_{u,w}$	$f_{y,l}$	$f_{y,h}$	$f_c^{'}$	
SRC1	280×280	H	H150×150×7×10	12d16	D6@140	R	296	350	296	350	350	453	29.5	Chen and
SRC4	280×280	＋	2 H175×90×5×8	12d16	D6@140	R	329	446	329	446	350	453	29.8	Yeh (1996)
src1	280×280	＋	2 H175×90×5×8	4d16	D6@140	R	318	444	378	466	453	606	23.9	Tsai et al.
src2	280×280	＋	2 H175×90×5×8	4d16	D6@100	R	318	444	378	466	453	606	23.5	(1996)
H6 -S17	200×200	H	H120×60×7×10	4d13	D6@170	R	316	448	266	351	388	538	20.6	Weng et al.
C4-S9	200×200	＋	2H120×40×4×7	4d13	D6@90	R	266	351	296	372	388	538	20.6	(2006)
A-04	—	＋	2H450×112×10×14	—	—	—	416	547	418	528	—	—	27.0	Chen et al.
1A-HP4-28	560×560	＋	2H450×168×10×14	4d25	D13@110	R	416	547	418	528	436	302	27.8	(2010)
Src5	600×600	＋	2H350×175×6×9	12d25	D13@150	M	386	529	421	571	469	463	32.3	Shih et al.
Src6	600×600	＋	2H350×175×6×9	12d25	D13@125	M	386	529	421	571	469	463	32.3	(2013)
SRC-1-2(1)	—	＋	2H360×170×20×20	—	—	—	280	515	280	515	—	—	35.0	Zhao et al.
SRC-1-2(2)	—	＋	2H360×170×20×20	—	—	—	280	515	280	515	—	—	27.0	(2014)
DH-SP-75	600×600	＋	2H350×175×6×9	16d25	D13@75	SP	454	574	437	546	451	472	34.6	Liang et al.
DH-SP-95	600×600	＋	2H350×175×6×9	16d25	D13@95	SP	454	574	437	546	451	472	34.6	(2014)
C-I-M40	200×200	H	H100×68×4.5×7.6	12d10	D6.5@40	M	254	368	254	368	427	335	93.0	Zhu et al.
C-+-M60	200×200	＋	2H100×68×4.5×7.6	12d10	D6.5@60	M	254	368	254	368	427	335	94.0	(2014)
HSRC-SP1	D411	＋	2H206×120×12×12	20d10	D10@50	SP	516	635	516	635	426	426	36.5	Chen et al.
HSRC-SP2	D411	＋	2H206×120×12×12	20d10	D10@50	SP	516	635	516	635	426	426	36.5	(2015)
CSRC1	400×400	H	H150×150×7×10	12d16	D8@100	M	301	440	301	440	356	340	26.5	Liu et al.
CSRC3	400×400	H	H175×175×7.5×11	12d16	D8@200	M	297	442	301	440	356	340	34.8	(2015)
E00-1	450×450	H	H120×106×12×20	32d8	D3.3@80	M	408	603	523	603	438	597	48.3	Xiao et al.
E00-2	450×450	H	H120×106×12×20	32d8	D3.3@80	M	398	603	411	603	438	597	44.7	(2017)

Note: H = H-shaped steel section; ＋= Cross-shaped steel section; R =Rectangular hoops; M = Multiple hoops; SP = Spiral hoops; d = diameter of longitudinal bars; D = diameter of hoops.

Table 4. Key parameters of theoretical stress-strain model for confined concrete

Specimen designation	$f_{l,f}$ (MPa)	$f_{r,h}$ (MPa)	$f_{l,h}$ (MPa)	$K'_{e,hs}$	$K'_{e,ps}$	$K'_{e,h}$	HSCC (MPa) f_{l1}	HSCC (MPa) f_{l2}	k_{hs}	PSCC (MPa) f_{l1}	PSCC (MPa) f_{l2}	k_{ps}	PCC (MPa) f_{l1}	PCC (MPa) f_{l2}	k_p	Error[a] (%)	Reference
SRC-A-1	2.59	—	—	0.804	0.562	—	2.08	2.08	1.28	0	1.46	1.10	—	—	—	0.34	This paper
SRC-A-2	2.73	—	—	0.804	0.562	—	2.19	2.19	1.23	0	1.53	1.09	—	—	—	4.20	
SRC-B-1	10.83	—	—	0.804	0.562	—	8.71	8.71	1.91	0	6.09	1.20	—	—	—	-2.33	
SRC-B-2	12.04	—	—	0.804	0.562	—	9.68	9.68	1.83	0	6.77	1.18	—	—	—	0.50	
SRC-B-3	12.21	—	—	0.804	0.562	—	9.82	9.82	2.07	0	6.86	1.22	—	—	—	-0.24	
SRC-C-1	5.83	—	—	0.804	0.282	—	4.69	4.69	1.61	0	1.79	1.11	—	—	—	2.25	
SRC1	1.93	435	0.75	—	0.508	0.379	—	—	—	0.28	1.27	1.16	0.28	0.28	1.07	2.04	Chen and
SRC4	3.89	434	0.75	0.804	0.599	0.361	3.39	3.39	1.63	0.27	2.60	1.21	0.27	0.27	1.06	5.06	Yeh (1996)
src1	3.76	448	0.77	0.804	0.599	0.229	3.28	3.28	1.74	0.18	2.43	1.22	0.18	0.18	1.05	1.84	Tsai et al.
src2	3.76	464	1.12	0.804	0.599	0.229	3.50	3.50	1.82	0.26	2.51	1.24	0.26	0.26	1.07	4.97	(1996)
H6 -S17	15.00	446	0.90	—	0.176	0.329	—	—	—	0.30	2.94	1.29	0.30	0.30	1.10	0.65	Weng et al.
C4-S9	13.41	481	1.83	0.804	0.331	0.329	6.18	6.18	2.29	0.60	5.04	1.43	0.60	0.60	1.19	2.40	(2006)
A-04	10.45	—	—	0.804	0.156	—	8.10	8.10	2.29	0.00	1.63	1.13	—	—	—	1.32	Chen et al.
1A-HP4-28	4.35	302	1.53	0.804	0.400	0.512	4.29	4.29	1.80	0.78	2.53	1.31	0.78	0.78	1.18	0.66	(2010)
Src5	1.46	455	3.71	0.804	0.577	0.360	1.75	1.75	1.33	0.58	1.42	1.20	0.58	0.58	1.12	6.51	Shih et al.
Src6	1.46	463	2.47	0.804	0.577	0.381	1.92	1.92	1.36	0.75	1.59	1.24	0.75	0.75	1.15	8.39	(2013)
SRC-1-2(1)	6.64	—	—	0.804	0.562	—	5.34	5.34	1.79	0	3.73	1.18	—	—	—	-3.40	Zhao et al.
SRC-1-2(2)	6.64	—	—	0.804	0.562	—	5.34	5.34	1.96	0	3.73	1.20	—	—	—	-4.49	(2014)
DH-SP-75	1.72	472	3.15	0.804	0.577	0.513	3.00	3.00	1.50	1.62	2.61	1.38	1.62	1.62	1.29	1.75	Liang et al.
DH-SP-95	1.72	472	2.49	0.804	0.577	0.513	2.66	2.66	1.45	1.28	2.27	1.32	1.28	1.28	1.24	4.25	(2014)
C-I-M40	4.85	335	3.71	—	0.184	0.510	—	—	—	1.89	2.78	1.19	1.89	1.89	1.13	1.17	Zhu et al.
C-+-M60	4.85	335	2.47	0.804	0.483	0.437	4.98	4.98	1.33	1.08	3.42	1.16	1.08	1.08	1.08	-3.17	(2014)
HSRC-SP1	8.49	426	3.71	0.804	0.618	0.238	7.71	7.71	2.01	0.88	6.13	1.35	0.88	0.88	1.16	6.25	Chen et al.
HSRC-SP2	8.49	426	3.71	0.804	0.618	0.238	7.71	7.71	2.01	0.88	6.13	1.35	0.88	0.88	1.16	5.28	(2015)
CSRC1	1.96	340	1.51	—	0.335	0.507	—	—	—	0.76	1.42	1.27	0.76	0.76	1.19	1.81	Liu et al.

CSRC3	1.71	340	0.75	—	0.335	0.349	—	—	—	0.26	0.84	1.12	0.26	0.26	1.05	-1.80	(2015)
E00-1	24.63	421	0.32	—	0.372	0.762	—	—	—	0.24	9.41	1.27	0.24	0.24	1.03	3.31	Xiao et al. (2017)
E00-2	24.63	421	0.32	—	0.372	0.762	—	—	—	0.25	9.41	1.28	0.25	0.25	1.04	15.33	

[a] Error = $N_{p,a} / N_{p,t} - 1$