Investigation on flexural behavior of a new-type composite column with steel angles

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**ABSTRACT**
A novel precast composite column with L-shape steels and infilled reinforced concrete (LSRC) is proposed and assessed by experimental research and numerical simulation. The LSRC column is constructed with four L-shape steels placing at corners, a reinforced cage placing outside and the casted concrete. The L-shape steels are connected by batten plates and the reinforcement cage is made up of longitudinal bars, hoops, and two-way ties. In this paper, 3 LSRC slender columns and a conventional concrete-filled steel tube column with encased concrete (CFTEC) are tested and compared to investigate their failure modes, flexural strength, and strains of concrete and steel at mid-span. The results show that the LSRC column has more sufficient plastic deformation ability and steel utilization than the CFTEC column, and exhibits the typical flexural failure mode. Furthermore, the 3D finite element model is also established to analyze the contribution of components, interaction mechanism between steel angle and concrete, and the effect of different factors on flexural behavior of LSRC columns. Based on the test and numerical results, the formula for calculating the bending capacity of LSRC column is proposed, and the predicted results agree well with the experimental and numerical values.

**KEYWORDS**
Composite column; flexural behavior; interaction mechanism; material strain; FE analysis; prefabricated structure

1. Introduction

In recent years, the steel reinforced concrete (SRC) columns have been widely used for high-rise buildings and large-span structures because of their advantages in bearing and seismic performance (Ellobby and Young 2011; Soliman, Araf, and Elrakib 2013; Chen, Chen, and Hoang 2016; Zhu, Jia, and Gao et al. 2016; Lai, Richard Liew, and Wang 2019; Yue, Qian, and Beskos 2019; Shichun, Xiongjun, and Zhanqin 1996; Zixiong, Zhiwei, and Yang). For conventional SRC columns Figure 1(a), the steel section was placed at the center of cross section and the reinforcement cage was placed at the edges of cross section, which effectively improved overall shear resistance. In order to make full use of the bending resistance of steel section, extensive researches have been conducted and SRC columns with different forms have been proposed based on experimental studies, numerical simulations, and theoretical predictions (Feng et al. 2015; Zhou and Liu 2010; Zhou, Yan, and Liu 2015; Qi et al. 2011; Liu et al. 2019; Yang et al. 2018; Ding et al. 2017; Zhu et al. 2010; Chang, Wei, and Yun 2012; Chang et al. 2013; Yan, Zhou, and Liu 2019; Wang, Liu, and Zhou 2016). This study develops a new type of precast L-shape steel and infilled reinforced concrete (LSRC) composite column Figure 1(d), where the flexural resistance of L-shape steel placed at corners of the cross section is significantly larger than that of the steel section placed at the center of the cross section. The four L-shape steels are connected by batten plates to form a whole structure, in the meanwhile providing the shear resistance and lateral confinement. The concrete encasement effectively inhibits the corrosion of steel and improves the fire resistance. Furthermore, the concrete encasement also prevents both local and overall buckling of the L-shape steel. The reinforced cage made up of longitudinal bars, hoops, and two-way ties is embedded in the cover, and it is used to prevent the cover concrete spalling and strengthen the cooperative deformation capacity between the steel and concrete.

Poon (1999) studied the precast steel angle composite section column connected by tie plates (battens). The axial compression tests for angle encased concrete column were carried out by Weiping, Peiji, and Yuli (1982) and Shaofei, Shuqing, and Zhiyang (1997). The results suggested that the bearing capacity of angle encased concrete column was larger than that of the conventional RC column, and the ultimate bearing capacity of angle encased concrete column increased with the decreasing hoop spacing. Simplified formulas for predicting the load-carrying capacity were also proposed and the accuracy was confirmed by comparing with experimental results. Montuori and Piluso (2009) used wrapped angle to retrofit and improve the existing RC columns Figure 1(b). Their results showed that the presence of wrapped angle at the corners apparently strengthened the bearing capacity and ductility of the whole column, but local buckling of angle was
susceptible to occur due to the lack of lateral constraint. Eom et al. (2014) developed a new type of prefabricated steel-reinforced concrete column (PSRC) Figure 1(c) and conducted flexural analyses to investigate the mechanical properties of the PSRC column. It turned out that the bending strength and stiffness were larger than that of the conventional composite columns. However, the PSRC specimens were susceptible to bond failure between the angle and concrete during plastic deformation, and weld fracture of the angle at the weld connection to the tie bars was also produced during this period. Hwang, Eom, and Park (2016) conducted a series of axial compression tests for PSRC columns and found out that although the structural performance of new PSRC column was superior to those of conventional SRC columns, the early spalling of cover concrete may affect the load-carrying and deformation capacity of members. Xingrong, Weiqing, and Yiji (2004), Xingrong and Baolin (2009), Xingrong (2004) carried out axial compression, eccentric compression and seismic performance tests on spatial steel frame concrete columns, and explored the mechanical behavior such as the failure patterns, bearing capacity, deformation capacity and ductility and seismic performance of specimens. Meanwhile, the formulas of axial and eccentric compression bearing capacity for spatial steel frame concrete columns were presented.

For existing research (Poon 1999; Weiping, Peiji, and Yuli 1982; Shaofei, Shuqing, and Zhiyang 1997; Montuori and Piluso 2009), the test results showed that wrapped angle connected by steel plates (battens) had significant effects on the mechanical properties of angle encased concrete column. However, the exposed angle without concrete encasement led to a relatively poorer fire and chemical corrosion resistance of the composite columns compared with traditional SRC columns. Moreover, wrapped angle was vulnerable to buckling under axial loading, making material properties underutilized sufficiently. On the other hand, from experimental results (Eom et al. 2014; Hwang, Eom, and Park 2016; Xingrong, Weiqing, and Yiji 2004; Xingrong and Baolin 2009; Xingrong 2004), it could be seen that although composite column with concrete encasement had good fire and corrosion resistance, the early spalling of concrete encasement would decrease the bearing capacity under axial loading, and the bond resistance between angle and concrete would be limited under bending moment. Therefore, in the proposed LSRC column, the reinforced cage, which is made up of longitudinal bars, hoops, and two-way ties, is embedded in the cover concrete to improve these research mentioned above.

Mechanical properties of LSRC columns under axial and eccentric compression have been investigated in a previous work (Ding 2019). In order to plot the M-N curve of the new composite column, three LSRC columns and one concrete-filled steel tube column with encased concrete (CFTEC) were designed and tested to investigate their pure flexural behavior through four point bending tests (Eom et al. 2014), and the axial load was not considered in this paper.

Figure 1. Different composite columns. (a) conventional SRC column; (b) retrofitted RC column using steel angle; (c) PSRC column; (d) LSRC column.
Specifically, based on the test results, several aspects were analyzed, including the failure modes, flexural capacity and strains of concrete and steel at mid-span. Furthermore, the FE software ABAQUS was used to investigate the bending contribution of different components and the interaction mechanism of interfaces between the steel and concrete. Meanwhile, parametric analysis was also conducted to explore the influence of different factors on the flexural behavior of LSRC columns. Based on experiment research and FE analysis, a simplified calculation formula for predicting the bending capacity of LSRC members was proposed, which met the accuracy requirements, to provide reference for practical engineering application.

2. Experimental program
2.1. Test specimens

Table 1 and Figure 2 show the geometric properties of a concrete-filled steel tube specimen (CES) with encased concrete (CFTEC) and three LSRC specimens. The research parameter is the spacing of batten plates. The cross-section of each specimen is 300 × 300 mm, and the overall height is 3570 mm. In the CFTEC specimen S1, the dimension of square steel tube is 240 × 240 × 5 mm (depth × width × thickness, and cross-sectional area = 4700 mm²), and it is placed at the edge of cross section (steel ratio = 5.2%).

In LSRC specimen S2, four L-shape steels of L-70 × 70 × 5 (each with a cross-sectional area of 675 mm²) are used at the corners of the section (steel ratio = 3.7%). Rectangular batten plates with spacing of 200 mm are welded to L-shape steels, and each of them has a size of 100 × 90 × 5 (width × height × thickness).

| Table 1. Specimens for flexural tests. |
| No | Steel angle (mm) | Batten plates (mm) | Longitudinal rebars (mm) | Hoops and tie bars (mm) | Steel ratio (%) |
|----|-----------------|--------------------|------------------------|------------------------|----------------|
| S1 | 240 × 240 × 5   | N/A                | N/A                    | N/A                    | 5.2            |
| S2 | 4 L-70 × 70 × 5 | 100 × 90 × 5       | 8-D10                  | D6@150                 | 3.7            |
| S3 | 4 L-70 × 70 × 5 | 100 × 90 × 5       | 8-D10                  | D6@150                 | 3.7            |
| S4 | 4 L-70 × 70 × 5 | 100 × 90 × 5       | 8-D10                  | D6@150                 | 3.7            |

Figure 2. Geometric properties of test specimens. (a) S1; (b) S2 (s = 200 mm); (c) S3 (s = 300 mm); (d) S4 (s = 500 mm); (e) A-A sections of CFTEC and LSRC columns; (f) The welding details of CFTEC and LSRC columns.
Eight longitudinal bars with the diameter of 10 mm are placed at the edges of the cross section. For lateral bars, D6 hoops and cross ties are arranged to connect the longitudinal bars. The spacing of lateral bars is 150 mm. For LSRC specimens S3 and S4, the spacing of batten plates is 300 and 500 mm, respectively, and the other dimensions of them are the same as those of S2.

Figure 3 shows the preparation procedure of LSRC specimens. In Figure 3(a), the steel was fabricated in two steps: Firstly, flat steel plates were modeled into L-shaped plates. Secondly, four L-shaped plates were welded by batten plates. For the CFTEC specimen, the fabrication of square steel tube was illustrated in detail by Ding et al. (2017). Then, L-shaped plates or steel tubes were welded to the two end plates. The butt-welding method was carried out according to GB50017-2003 (2003), and the welded area was smoothed after welding. Before casting the concrete, the L-shaped plates were placed horizontally for LSRC specimens and the longitudinal and lateral bars were tied outside, as shown in Figure 3(b). The concrete was poured into specimens and sufficiently vibrated by the vibrator to make the distribution uniform after the formwork Figure 3(c.d). In CFTEC specimen, the steel tube was slotted on the top surface and concrete was injected and fully vibrated after the formwork. Then, steel plates were used to close the slots. Afterward, the concrete was continuously poured from the top of formwork to form cover concrete. In order to clearly observe the crack development and spalling of cover, after 4 weeks curing period, one side of specimen was painted with a grid of 50 mm × 50 mm in pure bending zone.

### 2.2. Materials

Before the flexural tests, the mechanical properties of steel and concrete were tested through a standard method. In this study, low-carbon steel was used for steel sections and rebar, and the uniaxial tensile tests were carried out on three specimens according to GB/T228-2002 (2002). Table 2 lists the yield strength ($f_y$), elastic modulus ($E$), and Poisson's ratio ($\nu$) of both steel and rebar obtained by tensile tests. In addition, a total of six standard concrete cubes with the dimension of 150 mm, which had same environment and conditions as those of the test specimens, were casted and cured. The compressive strength of concrete cubes was measured in accordance with GB/T50081-2002 (2002) and the average compressive strength was 21.3 MPa.

#### Table 2. Material properties of steel.

| Material | $f_y$ (MPa) | $E$ (MPa) | $\nu$ |
|----------|-------------|-----------|-------|
| Steel$^a$ | 390 | $2.0 \times 10^5$ | 0.296 |
| Steel$^b$ | 361 | $2.0 \times 10^5$ | 0.298 |
| Rebar | 332 | $2.0 \times 10^5$ | 0.293 |

$^a$Square steel tube.  
$^b$Steel section (L-shape steel and batten plate).

2.3. Experimental setup, instrumentation and procedure

The flexural tests on all specimens were conducted at the Structural Test Centre of Zhejiang University, using a 1000-ton universal loading machine. The four-point bending configuration was adopted in a simple support condition, as shown in Figure 4(a). All specimens were strictly aligned in the loading machine. The load ($P$) was transferred to the two loading points through distribution beam, which composed a pure bending zone of length ($l_p$) in the specimen center. The distance between the roller supports is 3250 mm, and the length of shear span is $l_s$.

In order to accurately obtain the deflections of the specimens, five LVDTs ($\delta_1 \sim \delta_5$) were located at the mid-span, 1/2 shear span, and two roller supports Figure 4(a). Figure 5 depicts the layout of both concrete and steel gauges. The five concrete strain gauges were attached

Figure 3. Preparation of LSRC specimens. (a) Welding of L-shape steel and batten plates; (b) reinforcement cage binding; (c) formwork construction; (d) concrete casting.
along mid-span sectional height to measure the strain distribution of each specimen. For CFTEC specimen, two steel strain gauges were placed on the surface of steel tube at the top and bottom of pure bending zone. In each LSRC specimen, two steel strain gauges were attached to the position which was the same as those of the CFTEC column, and one steel strain gauge was placed on the surface of batten plate at the uniform flexural moment region.

During each four-point flexural test, the displacement control mode was adopted with a loading increment of 4 mm from the beginning of test to failure.
after the peak point. After each loading step, the load was maintained for 5 minutes to observe and record the crack of cover concrete.

3. Test results and discussion

3.1. Failure modes

During the test procedure, no composite column specimen produced any unexpected lateral movement of the cross section or any other unstable responses, indicating that all the specimens fully develop their bending resistance of the composite section. Figures 6 and 7 show the damages and failure modes of the tested CFTEC column and LSRC columns, and the failure modes were observed at the ultimate failure point of specimens.

In CFTEC specimen S1, the cover concrete spalling was observed at the bottom of specimen in the constant bending moment region, and the crushing of cover concrete was observed at the top of mid-span, as shown in Figure 6(a). When testing was terminated due to the ultimate failure of specimen, the cover concrete was cut off in the pure bending zone.

![Figure 6. Failure mode of specimens at the end of test.](image)

![Figure 7. Damage pattern of specimens.](image)
Welding failures, tensile fractures, and local buckling did not occur on the surface of steel tubes. After cutting open the square steel tube, it could be found that core concrete cracked on the surface of tension zone and was crushed in the region of pressure Figure 6(a), which was similar to that of cover concrete.

For the specimens LSRC, cover concrete cracking was presented at the bottom surface of specimens, and cover concrete crushing was observed almost at the same position of S1, as can be seen in Figures 6 and 7(b,c,d). After removing cover concrete in the pure bending zone, it could be observed that the buckling of longitudinal bar occurred in the pressure zone. Meanwhile, the fractures of the L-shape steel and longitudinal bar were detected in the tension region of S2 and S3. The results indicated that the spacing of batten plates had little effect on failure modes for LSRC column. In S4, L-shape steel fracture were not found because its clear height (=2000 mm) was less than that of other LSRC specimens (=3250 mm), which led to lower deflection of mid-span for S4.

Compared to S1 using steel tube, large area spalling of cover concrete did not take place in LSRC specimens due to the existence of reinforcement cage. Besides, the fractures of L-shape steel and longitudinal bar implied that the new composite column made more full use of steel than the CFTEC specimen.

### 3.2. Moment (M) versus mid-span deflection (U_m) relationship

Figure 8 shows the relationships of bending moment (M) at the center of the specimens and the mid-span deflection (U_m). The mid-span deflection was measured by using LVDT, and the corresponding flexural moment was calculated as follows:

\[ M = \frac{P L}{2} \]  

where \( P \) is the vertical load from jack and \( L \) is shear span length of the specimens Figure 4(b). Table 3 lists the test results, including peak vertical load \( P_{u} \), peak bending moment capacity \( M_{u} \), yielding mid-span deflection \( \Delta_{yv} \), deflection \( \Delta_{u} \) at failure of specimens, lateral stiffness \( K_{e} \) at mid-span, ductility index \( D_{I} \), and failure process. The ductility index \( D_{I} \) is defined as \( D_{I} = \frac{\Delta_{yv}}{\Delta_{u}} \), and the \( \Delta_{yv} \) is taken as \( \Delta_{yv} = \frac{P_{u}}{K_{e}}, \) in which \( K_{e} \) is secant stiffness corresponding to 0.75\( P_{u} \). Figure 8(e). As shown in Figure 8, composite columns were considered to

![Figure 8. Moment-mid-span deflection relationships of specimens. (a) S1; (b) S2; (c) S3; (d) S4; (e) definition of ductility index.](image)

**Table 3. Test results.**

| Specimens | \( P_{u} \) (kN) | \( M_{u} \) (kN.m) | \( K_{e} \) (kN/m) | \( \Delta_{yv} \) (mm) | \( \Delta_{u} \) (mm) | \( D_{I} \) | Failure process |
|-----------|------------------|------------------|-----------------|-----------------|-----------------|---------|-----------------|
| S1        | 376              | 211              | 14,150          | 26.57           | 50.28           | 58.68   | 1.89            |
| S2        | 326              | 183              | 16,321          | 19.97           | 72.66           | 80.20   | 3.64            |
| S3        | 308              | 173              | 13,065          | 23.57           | 72.99           | 86.70   | 3.10            |
| S4        | 502              | 175              | 41,102          | 12.21           | 39.14           | 51.69   | 3.21            |

\*Initial failure due to cover concrete spalling or crushing.
\*Ultimate failure at the last point of the loading.
experience three phases from starting of loading to failure as follows: (1) elastic stage; (2) plastic stage; and (3) failure stage. Because of the ductility behavior exhibited after flexural yielding, all the specimens were suitable to conduct ductility evaluation.

CFTEC specimen S1 using square steel tube exhibited plastic behavior with the obvious decreases in flexural stiffness after flexural yielding, as shown in Figure 8(a). The peak strength was $M_p = 211\text{kN-m}$ at $U_m = 50.28\text{ mm}$ ($DI = 1.89$). When the curve reached its peak load, the load-carrying capacity was sharply degraded because of cover concrete crushing and spalling in the region of uniform moment Figure 6(a), 7(a), and 8(a)). Finally, the loading termination was conducted at $U_m = 58.68\text{ mm}$ ($DI = 2.21$) due to the large area spalling of cover concrete.

The test results of LSRC specimen S2 with L-shape steel are shown in Figure 8(b). The ductility index $DI = 3.64$ at peak point was 48.1% greater than that of S1, indicating that specimen S2 had stronger plastic deformation capacity. This was caused by the existence of reinforcement cage which effectively delayed the crushing and spalling of cover concrete. However, the peak strength $M_p = 183\text{kN-m}$ was 13.3% less than that of S1 for the following reasons: the yield strength of L-shape steel ($f_y = 361\text{ MPa}$) used for S2 was 8.0% less than that of square steel tube ($f_y = 390\text{ MPa}$) used for S1. Meanwhile, the L-shape steels placed at the corners of the cross section can provide nominal flexural capacity 51.3% lower than steel tube used at the edges of the section. The bending moment capacity instantly decreased at $U_m = 72.66\text{ mm}$ when cover crushing and longitudinal bar fracture with crisp sound occurred in the mid-span Figure 6(b), 7(b) and 8(b). Subsequently, S2 failed at $U_m = 80.20\text{ mm}$ ($DI = 4.02$) due to tensile fracture of the L-shape steels in the bottom region of uniform moment. The tensile fracture of L-shape steels initiated at the location of weld connection to the batten plates, which was the typical flexural failure mode of LSRC columns.

Figure 8(c) shows the moment-deflection relationship of S3 with the batten plates spacing of 300 mm. Because the increased plates spacing reduced the confinement effects on core concrete, the peak strength $M_p = 173\text{kN-m}$ at $U_m = 72.99\text{ mm}$ ($DI = 3.10$) of S3 was slightly less than that of S2. After the peak load, the load-carrying capacity suddenly decreased to a certain value as a result of cover crushing on the top surface of mid-span. Ultimately, the longitudinal bar fracture and followed tensile fracture of L-shape steel at $U_m = 86.7\text{ mm}$ ($DI = 3.68$) symbolized the failure of specimen S3.

In the case of S4 with the greatest plates spacing of 500 mm, the peak strength $M_p = 175\text{kN-m}$ at $U_m = 39.14\text{ mm}$ ($DI = 3.21$) was slightly larger than that of S3, which can be attributed to the shorter clear height of S4. With the cover crushing, the bending moment capacity decreased at a sharp rate. After that, the ultimate failure was caused by tensile fracture of longitudinal bar at the bottom of mid-span at $U_m = 51.69\text{ mm}$ ($DI = 4.23$).

Based on above analysis and test result from Table 3, it can be concluded that LSRC column specimens exhibited more apparent ductility behavior than CFTEC specimen with higher ductility index $DI$, indicating that a satisfactory plastic deformation capacity has been achieved for LSRC columns. For composite specimens, because the mid-span deflection change at failure stage was significantly less than that at plastic stage, the failure behaviors of specimens were governed by cover concrete spalling and crushing. Furthermore, the overall failure process of LSRC specimens were similar, which led to their identical flexural failure modes.

### 3.3. Deflection curves

Figure 9 shows two typical sets of deflection distribution curves along the column at different bending moment levels for CFTEC column S1 and LSRC column S2. In the Figure 9, $H$ denotes the distance away from the left end support, and $U$ is the vertical deflection of specimens. It was detected that test curves were symmetrical about mid-span and resembled a half-sine wave, which was similar to that of composite beams tested in previous studies (Ahmed et al., 2018). Moreover, the deflection varieties for tested columns were significant from 80% of $M_u$ to ultimate bending capacity due to ductility behavior after yielding.

### 3.4. Moment-strain analysis

#### 3.4.1. Concrete strain analysis

Figure 10 plots concrete strain distribution results along the mid-span cross-section for all tested specimens. In CFTEC specimen S1, cover concrete strain at the height of 200 mm away from bottom surface was not obtained due to the failure of strain gauge in the early stage of loading. It can be observed that the overall measured region was under compression axially, which led a great compressive zone of cross section for S1. As shown in Figure 10(a), the concrete strain variety above the midcourt line was obviously greater than that beneath the midcourt line of S1, which was mainly caused by the significant bond slip between steel tube and concrete at the depth of 50 mm away from top surface.

For LSRC specimens S2, S3, and S4, concrete strain gauges gradually became invalid along the cross section from bottom to top because of the cover cracking at the bottom of mid-span. Therefore, for observing concrete strain distribution at different moment, consistent ratios of mid-span moment ($M$) to the ultimate $M_u$ was not
adopted for different LSRC specimens. As shown in Figure 10(b,c,e), the neutral axes of specimens S2 and S3 were higher than their midcourt line, which indicated that the height of compressive region was lower than that of tensile region. Meanwhile, the compressive zone of S2 and S3 basically satisfied the Assumption of Plane Cross-section according to approximately linear strain variety (Zhao 2013; CECS 38:2004 2004), which reflected the favorable synergistic deformation between steel and concrete in this region. In the elastic phase, the tensile strains appeared relatively apparent nonlinear for all LSRC specimens Figure 10(b,c,d,f), which was caused by the cover concrete cracking at the bottom of mid-span. Moreover, the nonlinear behavior of S2 and S4 was more obvious because of notable cracking. With an increase of loading, the neutral axes of S2 and S4 presented a tendency of ascension before reaching the peak point, as shown in Figure 10(e). This can also be attributed to the bottom concrete cracking of mid-span.

3.4.2. Steel strain analysis
The strains of steel tube (S1) is shown in Figure 11. L-shape steels and batten plates (S2–S4) varied with mid-span moment M at the bottom and top of the pure moment zone. As the strain gauges damaged during the vibration, it cannot be measured that tensile strains of L-shape steel at the bottom of S2 and S4 and strains of batten plates in the region of uniform moment of S3 and S4. As shown in Figure 11(a), the compressive strains of steel tube and L-shape steels at the top of S1–S4 significantly varied after flexural yielding of specimens. The maximum compressive strain in the L-shape steels of LSRC columns was greater than that in the steel tube of CFTEC column before the ultimate moment of specimens. In the case of S4, the compressive strain of L-shape steel reached the yield strain of steel (1805 με) after flexural yielding. Therefore, the buckling of L-shape steel appeared with large spacing of batten plates Figure 6.
Furthermore, when reaching the yield bending moment, specimens S1–S3 presented the adverse tendency of strain variety due to stress redistribution. As shown in Figure 11(b), the rates of tensile strain of L-shape steel in S3 was apparently greater than that of steel tube in S1. The maximum tensile strain of L-shape steel reached 3987 με before peak point, which was significantly greater than that of steel tube (2372 με). The result indicated that specimen S3 had more efficient use of material properties than S1, and the L-shape steels used at the corners of section made a greater contribution to flexural stiffness and strength (Eom et al. 2014). In specimens S1 and S3, the growth rate of tensile strain of steel tube and L-shape steel significantly improved after overall flexural yielding, which can be attributed to the core concrete cracking and L-shape steel yielding at the bottom of pure moment zone, respectively. In addition, batten plate in S2 was under longitudinal tension Figure 11(b), which indicated that it restricted the deformation of L-shape steel to some extent.

4. Finite element (FE) modeling

Finite element modeling (FEM) of LSRC specimens was performed using the ABAQUS software (ABAQUS 2013). The object of FEM was to verify the models, investigate the bond slip between concrete and L-shape steel and perform the parametric analysis. Figure 12 shows the schematic view of the finite element model considering geometric and material nonlinearity. The model consists of core concrete, L-shape steels, batten plates, cover confined concrete, cover unconfined concrete, heel block, and reinforcement cage. The reinforcement cage comprises longitudinal bars, transverse bars, and two-way tie bars.

4.1. Material models

4.1.1. Steel section and reinforcement cage

Based on the elastic-plastic model provided by ABAQUS (using the ELASTIC and PLASTIC options), the nonlinear stress-strain relationship curves was adopted to describe the constitutive behavior of steel sections and reinforcement bars. The curves were given in the Code for design of concrete structures in China (GB50010-2010 2010), respectively. In this nonlinear curve, elastic modules \(E_s\), Poisson’s ratio \(\nu_s\), yield stress \(f_y\), and the postyield strain-hardening stiffness \(E_{sh}\) were used according to the material test results.

4.1.2. Material model of concrete

On the basis of the study by Huang, Zhou, and Liu (2017) and Ellobody and Young (2011), the concrete of LSRC column can be divided into three main zones including unconfined concrete, partially confined concrete and highly confined concrete, as shown in Figure 13. Firstly, the unconfined concrete was taken as cover concrete outside the centerlines of hoops. Secondly, the partially confined concrete was taken from outer side of L-shape
steel to centerlines of hoops. Thirdly, the highly confined concrete was determined as the zone surrounded by L-shape steel.

Figure 14 shows the typical stress-strain relationship for both unconfined and confined concrete. The compressive strength of core concrete and cover confined concrete was significantly improved because of the great confinement effect of batten plates and transverse bars. For compressive constitutive behaviors of different concrete, the Young’s modulus in the elastic stage \(E_c\) is taken as 4700 \(\sqrt{f_{cd}}\) given by ACI Committee (ACI318-11 2011), where \(f_{cd}\) is concrete strength under uniaxial compression. The Poisson’s ratio \(v_c\) is determined as 0.2. \(\sigma\) and \(\varepsilon\) are the stress and strain of concrete, respectively. For highly confined, partially confined and unconfined concrete, the specific stress-strain relationships can be defined as follows, respectively.

1. Unconfined concrete

A stress-strain model of unconfined concrete recommended by GB50010-2010(2010) was applied for constitutive behavior of cover un-confined concrete. The following equation revealed the details of stress-strain model.

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

\(\varepsilon = \begin{cases} 
\frac{1 - \rho_c}{n - 1} \varepsilon, & \varepsilon \leq 1 \\
\frac{n}{\rho_c(x - 1)^2 + x}, & \varepsilon > 1 
\end{cases}
\]

Figure 14. Compression stress-strain model of concrete.

where \(n = \frac{E_c k_f}{E_c - k_f}, \ \rho_c = \frac{f_c}{E_c}, \ \varepsilon = \varepsilon / \varepsilon_c, \ \varepsilon_c = (700 + 172 \sqrt{f_c}) \times 10^{-6}, \ \alpha_c = 0.15710.785 - 0.905\)

In which \(f_c\) is the uniaxial compressive strength of concrete, \(f_c = f_{cd}\), which can be derived according to the results of material tests.

(2) Partially confined concrete

For cover confined concrete in LSRC, the increase of concrete strength mainly depended on the confinement of transverse bars in the reinforcement cage. Therefore, a stress-strain model proposed by Mander, Priestley, and Park (1804–1826) was adopted and defined as follows:

\[
\sigma = \frac{f_{cc}\varepsilon}{r - 1 + x^2}
\]

In which \(x = \varepsilon / \varepsilon_{cc}, \ r = \frac{\varepsilon_c}{\varepsilon_c - \varepsilon_{sec}}, \ \varepsilon_{sec} = f_{cc} / E_{cc}\)

\(f_{cc}\) is the maximum compressive strength of confined concrete, which can be calculated using Eq. (5); \(\varepsilon_{cc}\) is the compressive strain corresponding to \(f_{cc}\), which can be calculated using Eq. (6).

\[
f_{cc} = f_{cd} \left(-2.254 + 2.254 \sqrt{1 + \frac{7.94f_c}{f_{cd}} - 2f_c/f_{cd}} \right)
\]

\[
\varepsilon_{cc} = \varepsilon_c \left[1 + 5 \left(\frac{f_{cc}}{f_{cd}} - 1 \right)\right]
\]

Where \(f'\) is the effective confining stress of transverse bars, which can be calculated as follows:

\[
f' = \rho f_{yh} k_e
\]

Where \(\rho\) is the volumetric hoop ratio, which can be calculated as \(A_{bh}/A_b\), in which \(A_{bh}\) is the overall sectional area of hoops in a unit body, \(s\) is the spacing of hoops, and \(b_c\) is the side length of hoops; \(f_{yh}\) is the yield strength of hoops; \(k_e\) is the effective confinement coefficient that can be calculated using Eq. (8).

\[
k_e = \frac{\left(1 - \frac{\sum_i w_i^2}{6s^2}\right) \left(1 - \frac{s}{2\varepsilon_c}\right)^2}{1 - \rho_{cc}}
\]

In which \(\rho_{cc}\) is the ratio of the area of longitudinal bars to the area of concrete surrounded by centerlines of hoops; \(n\) is the number of longitudinal bars; \(w_i\) is the clear distance between adjacent longitudinal bars; \(s'\) is the clear spacing of hoops.

(3) Highly confined concrete

The confinement of core concrete was mainly provided by batten plates in LSRC columns. A stress-strain model given by Kai Wang (2015) was used to represent the uniaxial stress-strain relation of highly confined core concrete. The same form of \(\sigma - \varepsilon\) relation expression was adopted as Eq. (4).
4.2. Element types and mesh

The FE model viewed L-shape steels, batten plates, cover and core concrete and heel block as 3D solid element C3D8R with 8-node reduced integration, and viewed reinforcement cage including longitudinal bars and transverse bars as 3D truss element T3D2 which can be used to model nonprestressed reinforcement. Based on the assumption that there was a great synergistic deformation capacity between rebar and concrete, the rebar elements were embedded to cover concrete using embedded constraint technique, where the node relationships were established between the main element and the embedded element. Papanikolou and Kappos (2009) validated that this technique was suitable to confinement modeling. In order to save time and ensure the calculation accuracy, models with different grid sizes were conducted and compared in this paper, and then an appropriated mesh was acquired as shown in Figure 12.

4.3. Boundary conditions and load application

The heel blocks were assumed to be elastic rigid body which had adequate stiffness so that the deformation can be neglected during the whole loading. Each top surface of upper heel block was coupled to a reference point in order that all the degrees of freedom (DOFs) can be represented using point DOFs. The three DOFs, which included a translational DOF (U,) and two rotational DOFs (URz and URy), were restrained for reference points of upper heel blocks. At the lower heel block of left side, all the DOFs in the centerlines of bottom surfaces were restrained except the rotational DOF (URz). The right support released the translation DOF (U,0) on the basis of restraint conditions of left one. The model was loaded as a hinged column in a manner that applied a displacement control.

4.4. Modeling of interaction

The Mohr-Coulomb friction model was used to simulate the interaction behavior in the tangential direction between concrete and steel section consisting of L-shape steel and batten plates, and the friction coefficient of bond model were taken as 0.25 (Ellobody and Young 2011; Huang, Zhou, and Liu 2017; Sharif, Al-Mekhlafi, and Al-Osta 2019; Cai, Pan, and Wu 2015; Liu et al. 2018). In the normal direction, the interaction modeling between steel section and concrete was specified as hard contact. The contact pressure model can prevent contact surfaces from penetrating and allow them to separate each other after contacting. The contact between core and cover concrete was assigned to use a tie interaction, which prohibited contact surfaces to move relatively. In addition, the tie interaction was also applied for the contact between heel blocks and cover concrete.

4.5. Validation of FE modeling

In order to evaluate the accuracy of FE models, this paper performed the comparison of predicted and measured P – Um curves for S2–S4 specimens, as shown in Figure 15. The FE modeling results presented a good agreement with the experimental results before the ultimate load. Thereafter, the FE modeling curves declined slightly, which was different from the experimental results. The reason for this difference is that the constitutive model of steel used in this paper does not take tension fracture into consideration. Figure 16 shows the comparison of typical failure mode between predicted result and experimental result for S2 specimen. As can be seen, the cover concrete was crushed at the top of pure moment zone and cracked in the bottom region of uniform moment. The tensile failure of longitudinal bar occurred at the bottom of mid-span, and L-shape steel fracture initiated at the location of weld connection to batten plates. It is clear that the typical fracture feature of FE modeling maintained the good consistency with the test result. In summary, it can be concluded that the established FE model can accurately
simulate the mechanical behavior in the elastic and plastic stages, predict the ultimate carrying capacity and capture the failure modes of LSRC columns.

4.6. Analysis of moment-curvature curves

Figure 17(a) shows the typical mid-span moment of the whole LSRC model and its components versus curvature ($\varphi$) relations (taking the LSRC specimen S2 as an example). Horizontal axis $\varphi$ is the curvature at the center region, and it can be calculated as follows:

$$\varphi = \left[ \frac{(\delta_m - \delta_{p1})}{l_{VDT}} + \frac{(\delta_m - \delta_{p2})}{l_{VDT}} \right]$$

(13)

Where $\delta_{p1}$ and $\delta_{p2}$ are the deflections at the loading point, and $\delta_m$ is the deflection at the mid-span, as shown in Figure 4. $l_{VDT}$ is the distance from loading point to mid-span ($=l_p/2$).

It can be seen from the figure that the elastic stiffness of L-shape steel component was significantly greater than that of both core and cover concrete components because of the larger elastic modulus of steel. When the flexural yielding of S2 was occurred at the yield curvature
(\(\varphi\)) given by Eom et al. (2014), L-shape steel component yielded with the decreased stiffness, and bending capacity of cover concrete declined due to the cover cracking in the bottom region of pure moment zone. In addition, although the bending capacity of core concrete was obviously less than that of L-shape steel, it presented a gradually rising tendency during the loading process up to the ultimate moment. When \(\varphi\) reached 0.0604, the moment contribution of core concrete exceeded that of cover concrete. The reason is that core concrete of compression region maintained the bending resistance with confinement effects of batten plates and stirrups when it was crushed. In general, it can be concluded that L-shape steel occupied the largest proportion of bending moment contribution among the components proposed in Figure 17(a).

Figure 17(b) shows the \(M_{\text{LS}}/M_{\text{LSRC}}\) versus mid-span curvature (\(\varphi\)) curve of LSRC specimen S2, where \(M_{\text{LS}}\) and \(M_{\text{LSRC}}\) are the bending moment of L-shape steel component and the whole LSRC column, respectively. From the figure, the growth rate of \(M_{\text{LS}}/M_{\text{LSRC}}\) was significantly quick in the early stage of loading, which was caused by the larger elastic modulus of steel and the cracking of cover concrete in the bottom region of mid-span. The value of \(M_{\text{LS}}/M_{\text{LSRC}}\) was about 0.46 at point A. After point A, the curve presented a relative gentle slope. The slower increase of bending moment contribution can be attributed to the gradually yielding steel, and it can also be contributed to the reason that strength of core concrete was improved under triaxial compression in the compression zone when it comes into plasticity. The value of \(M_{\text{LS}}/M_{\text{LSRC}}\) was 0.55 at point B, and it presented a slow upward trend after flexural yielding. It is apparent that at this stage, the L-shape steel inside the LSRC column provided more than half of the bending moment, becoming the majority component assuming the flexural capacity.

4.7. Interactions between L-shape steel and concrete

Figure 18 shows the composite actions between L-shape steel and concrete in LSRC column, where \(\tau_1\) and \(\tau_2\) are the tangential stresses between core and cover concrete and L-shape steel along the axial direction of column, respectively. \(p_1\) and \(p_2\) are the normal stresses between core and cover concrete and L-shape steel, respectively. Figure 19(a) gives the change of total area in contact (CAREA) between concrete and L-shape steel with the increased mid-span curvature. It can be seen that CAREA of inner and outer interfaces presented a quickly declined trend during the stage I, which contributed the fact that there were less action area of normal confined stresses on the contact surfaces. The reason was that the Poisson’s ratio of L-shape steel was greater than core concrete in the elastic phase for inner interface and cover concrete was crushed and cracked in the pure moment zone for outer interface. At the stage II, the slope of curve became gently generally, indicating that the normal contact region of interfaces tended to be stable with increased curvature. In addition, the CAREA of cover concrete was generally greater than that of core concrete during the loading process until the peak point, which can be attributed to the confinement effect of reinforced cage.

In ABAQUS, COPEN was defined as the distance between slave surface and master surface, and it can also be used to describe the contact status of interfaces. Therefore, the COPEN distribution corresponding to yield and ultimate moment on the inner and outer interfaces are shown in Figure 19(b). From the figure, the region that COPEN exceeded zero was mainly concentrated in the uniform moment zone, which demonstrated that there was a poor cooperative deformation capacity in this region, and this result can also be validated in section 3.4.1. In summary, with the increased mid-span curvature, the normal disengagement area between steel and concrete progressively increases in the center region of column. Compared to inner interface, cover concrete is able to work together better with L-shape steel with the confinement of hoops.

Figure 20(a) gives the axial total contact frictional stresses (CFS) of inner and outer interfaces versus mid-span curvature curves. At stage I, it can be found that

Figure 18. Composite actions between concrete and steel. (a) CAREA of interfaces versus curvature (\(\varphi\)) relation; (b) COPEN distribution of interfaces at yield and ultimate points.
CFS of the two interfaces existed. This is because that both core and cover concrete cracked in the tensioned zone and axial strain ratio of concrete was larger than that of steel in the early stage. At stage II, CFS of outer interface overall showed an ascending trend. The reason is that the strain ratio of steel was larger than that of concrete after L-shape steel yielded. Meanwhile, the existence of reinforced cage confined the expansion of cover concrete. The resulting tangential friction contributed to the increase of the total frictional stresses. For inner interface, when core concrete came to plasticity, the Poisson's ratio of L-shape steel was smaller than that of concrete. The expansion rate of core concrete was faster than that of L-shape steel in the region of compression, which resulted in the significantly improvement of compressive pressure ($p_1$) between core concrete and L-shape steel. Thus CFS of inner interface had a greater growth rate than that of outer interface after $\phi = 0.037$ according to the Coulomb friction formula given by ABAQUS (2013). When $\phi$ reached 0.076, the CFS of inner interface declined gradually with crushing of concrete in the compressive zone.

ABAQUS adopted CSLIP as the relative tangential slip to reflect the bond slip interaction of interfaces. Figure 20(b) shows the tangential slip (CSLIP) distribution of inner and outer interfaces at the yield and ultimate moments, respectively. When LSRC column occurred flexural yielding, the bond slip of interfaces was mainly centralized in the region of shear span, while it was seldom appeared in the uniform moment region due to the disengagement between L-shape steel and concrete Figure 19(b). After the peak point, the bond slip of interfaces presented a trend toward moving across the middle, which indicated that the capacity to coordinate deformation at the center region was weakened. In general, it can be concluded that the collaborative working ability between core concrete and L-shape steel was poorer after flexural yielding, and the bond slip was inclined to pure moment zone at peak value.

The batten plates not only provided the confinement to the core concrete, but also constrained the axial deformation of L-shape steel. The confinement effect of batten plates was explored by analyzing the plate stress ($\sigma_p$) versus mid-span curvature ($\phi$) relation, as shown in Figure 21. The confinement supplied by batten plates almost existed in the whole loading procedure. When $\phi = 0.0102$ at point A, because the L-shape steel yielded in the bottom of mid-span, its
axial strain rate increased. This contributed to the fact that batten plates undertook the greater tension effect from top and bottom L-shape steels. Thus the growth rate of plates stress increased, and batten plates provided stronger confinement to L-shape steels.

Figure 22 shows the four typical types of tangential interaction between steel and concrete in the LSRC column, including frictionless contact, contact with static friction, contact with slip friction and disengagement. The more advanced the contact type is, the better the overall performances among components are. In order to further investigate interaction of interfaces in the mid-span, the tangential stress ($\tau$) changes at top and bottom of middle section with the increase of curvature are shown in Figure 23. In the figure, the critical shear stress ($\tau_{\text{max}}$) can be calculated using $\tau_{\text{max}} = p \times \mu$, in which $\mu$ is the coefficient of friction. For outer interface, it can be seen that the tangential toughing of top surface belonged to the first type before flexural yielding, which demonstrated that a satisfactory collaborative performance has been achieved compared to the disengaged state of bottom surface. For inner interface of middle section, the disengagement between steel and core concrete were observed in the compressive region due to the larger Poisson’s ratio of steel than that of concrete in the elastic phase. After flexural yielding, the compressive interface was basically in a frictionless contact state, which was better than that of tension zone with static friction contact. From the perspective of the whole loading procedure, the coordinative work capacity of compression zone is more superior than that of tension zone.

5. Parametric analysis

The factors affecting the bending resistance of the composite column are concrete strength, steel strength, cross-sectional area of longitudinal bar, batten plate spacing and the thickness and length of L-shape steel. To optimize mechanical properties and economic benefits, this section analyzed 19 LSRC columns to explore the effects of above parameters on their flexural behaviors. Figure 24 gives the effects of different parameters on initial flexural stiffness ($K_u$), yield deflection ($\Delta_y$), ultimate moment ($M_u$), ductility index ($DI$), and the $M_{LS}/M_{LSRC}$ versus $\phi$ relations are shown in Figure 25. The design parameters and results are listed in Table 4.

5.1. Concrete strength

As concrete strength ($f_c$) increased, the initial flexural stiffness ($K_u$) increased Figure 24(a), and yield deflection ($\Delta_y$) gradually decreased Figure 24(b). This trend indicated that specimens yielded earlier and their ductility gradually improved at plasticity phase with the increased concrete strength Figure 24(d). The increase of concrete strength slightly improved the ultimate bending capacity ($M_u$) of LSRC columns Figure 24(c).
However, as concrete strength ($f_c$) grew, the $M_{\text{LSS}}/M_{\text{LSRC}}$ decreased Figure 25, and the reduction of contribution provided by L-shape steel was more significant after flexural yielding of specimens.

5.2. Strength of L-shape steel

As L-shape steel strength ($f_y$) increased from 361 to 460 MPa, the initial flexural stiffness ($K_e$) increased slightly
as shown in Figure 24(a). The improvement of ultimate bending capacity ($M_b$) was not significant because the $M_b$ of specimens were dominated by ultimate strength of steel. It can be seen from Figure 25(b) that the steel strength had little influence on $M_{LS}/M_{SRC}$ before the yield point, after which the contribution of L-shape steel increased with the increase of steel strength.

5.3. Sectional area of longitudinal bars

As the sectional area of longitudinal bars ($A_l$) increased, the flexural stiffness ($K_s$), yield deflection ($\Delta_y$), and ultimate moment ($M_u$) increased Figure 24(a,b,c). However, the ductility index showed an upward trend before the section area of longitudinal bars reached 153.9 mm² Figure 24(d), which can be attributed to the fact that the increase of $A_l$ delay the crushing of the concrete in the compression zone. As the sectional area of longitudinal bars ($A_l$) increased, $M_{LS}/M_{SRC}$ decreased as shown in Figure 25(c).

5.4. Thickness of L-shape leg

As shown in Figure 24(a), as thickness of L-shape leg increased, the growth of flexural stiffness ($K_s$) was significant owing to the increased sectional moment of inertia. In addition, the ultimate flexural moment ($M_u$) presented significant growth with the increase of L-shape steel thickness Figure 24(c), which can be attributed to the fact that the thicker L-shape steel provided a larger contribution of flexural resistance, as shown in Figure 25(d). As thickness of steel section increased, the yield deflection ($\Delta_y$) of specimens increased Figure 24(b).

5.5. Length of L-shape leg

As the length of L-shape leg increased, the initial stiffness ($K_s$) and bending capacity ($M_b$) increased Figure 24(a,c) due to the larger steel ratio. The length of L-shape leg had little influence on $M_{LS}/M_{SRC}$ Figure 25(e). As the length of L-shape leg increased, the yield deflection of mid-span ($\Delta_y$) increased as shown in Figure 24(b).

5.6. Spacing of batten plates

Owing to the fact that LSRC column with spacing =500 mm had a shorter distance between supports in the test part, the effects of batten plates spacing on the flexural performance could not be expressed accurately. Therefore, the FE models with the same span and different plate spacing were established and analyzed. As the spacing of batten plates increased, the initial stiffness ($K_s$) and bearing capacity ($M_u$) decreased Figure 24(a,c) because of the smaller confinement to core concrete. Moreover, the yield deflection of mid-span exhibited a gradual upward trend with the increase of batten plate spacing Figure 24(b), which led to a decrease in the ductility of members Figure 24(d). The influence of batten plates spacing on $M_{LS}/M_{SRC}$ was not significant, as shown in Figure 25(f).

In summary, it is clear that the increase of L-shape leg thickness and concrete strength can effectively improve the flexural stiffness of LSRC column. Furthermore, the bending capacity of specimens can be improved by increasing the sectional area of longitudinal bars and thickness of L-shape leg.

6. Flexural strength prediction

6.1. Basic assumption

In this paper, the strength of core concrete increased due to the confinement effect provided by batten plates. However, on the basis of the experimental results, the improvement of core concrete strength was limited due to the limited compression region, which resulted in the lower moment contribution of core concrete. For LSRC column, the frictional shear stress of interfaces between L-shape steel and concrete

| No | $f_y$/MPa | $f_y$/MPa | $A_l$/mm² | $t$/mm | $b$/mm | $s_l$/mm | $K_s$/kN/m | $\Delta_y$/mm | $M_b$/kN | $D_l$ |
|----|--------|--------|---------|--------|--------|--------|---------|----------|--------|-------|
| S-base | 21.3   | 361    | 78.5    | 5      | 70     | 200    | 14,707  | 22.1     | 182.7  | 3.87  |
| S-C1 | 1.3    | 361    | 78.5    | 5      | 70     | 200    | 16,314  | 20.3     | 186.7  | 3.88  |
| S-C2 | 40     | 361    | 78.5    | 5      | 70     | 200    | 17,825  | 19.1     | 191.0  | 3.89  |
| S-C3 | 50     | 361    | 78.5    | 5      | 70     | 200    | 18,623  | 18.7     | 196.3  | 4.01  |
| S-S1 | 21.3   | 410    | 78.5    | 5      | 70     | 200    | 15,696  | 21.0     | 185.2  | 4.04  |
| S-S2 | 21.3   | 460    | 78.5    | 5      | 70     | 200    | 16,017  | 20.9     | 186.8  | 3.63  |
| S-S3 | 21.3   | 510    | 78.5    | 5      | 70     | 200    | 15,240  | 23.1     | 198.2  | 4.50  |
| S-A1 | 21.3   | 361    | 113.1   | 5      | 70     | 200    | 16,040  | 23.8     | 214.7  | 4.63  |
| S-A2 | 21.3   | 361    | 201.1   | 5      | 70     | 200    | 16,739  | 24.9     | 234.0  | 4.14  |
| S-T1 | 21.3   | 361    | 78.5    | 7      | 70     | 200    | 17,454  | 23.8     | 233.5  | 4.13  |
| S-T2 | 21.3   | 361    | 78.5    | 9      | 70     | 200    | 20,105  | 24.9     | 281.1  | 4.04  |
| S-T3 | 21.3   | 361    | 78.5    | 11     | 70     | 200    | 20,050  | 26.3     | 326.0  | 3.90  |
| S-L1 | 21.3   | 361    | 78.5    | 5      | 75     | 200    | 15,250  | 22.6     | 193.6  | 4.45  |
| S-L2 | 21.3   | 361    | 78.5    | 5      | 80     | 200    | 15,376  | 23.5     | 202.9  | 4.43  |
| S-L3 | 21.3   | 361    | 78.5    | 5      | 85     | 200    | 15,584  | 24.2     | 211.7  | 4.40  |
| S-S1 | 21.3   | 361    | 78.5    | 5      | 70     | 300    | 14,258  | 22.5     | 180.7  | 3.79  |
| S-S2 | 21.3   | 361    | 78.5    | 5      | 70     | 435    | 13,774  | 23.0     | 177.9  | 3.70  |
| S-S3 | 21.3   | 361    | 78.5    | 5      | 70     | 500    | 13,513  | 23.1     | 175.7  | 3.70  |
was significantly less than strength of steel and concrete. Therefore, the interaction of interfaces had little influence on the contribution of bending capacity. In this section, in order to simplify the calculation model of LSRC specimens, some simple and reasonable assumptions were listed as follows:

1. The confinement effect of concrete at the compression zone was neglected, and the strength of concrete at the tension zone was not considered.
2. The influence of frictional interaction between steel and concrete was not taken into account.
3. The part of L-shape steels and longitudinal bars at the compression zone were considered as compressed to yield, and the rest of L-shape steels and longitudinal bars at the tension region were considered as tensed to yield.

6.2. Calculation method

According to the above assumptions, a calculation formula for predicting the ultimate bending capacity of LSRC column was proposed. In the formula, the rectangle stress block method proposed by Li (2010) and Zheng, Li, and Lu (2011) was adopted to calculate the contribution of concrete. Figure 26 gives the calculation diagram of normal section at peak point. According to the balance relationship of force and moment, the height of compression zone \((x)\) and ultimate bending capacity \((M_u)\) can be calculated as follows:

\[
af_Bx + f_yA_1 + f_yA_s - f_yA'_s - f_p(Bx + A'_1) = 0
\]

\[
M_u = af_Bx^2/2 + f_yA_1|x - a_s| + f_yA_s|x - a_s| + f_pB|x - a_s|
\]

\[
+ f_pA'_1[B - a'_s - x] + f_pA'_s[B - a'_s - x]
\]

where \(x\) is the height of compression region; \(a\) is the factor of rectangle stress block method (=1.0); \(A_1\) is the area of longitudinal bars at the compression region; \(A'_1\) and \(A'_s\) are the areas of longitudinal bars at the bottom of tension zone and center line, respectively; \(A_s\) and \(A'_s\)

are the areas of L-shape steel at the compression and tension zone, respectively; \(f_y\) and \(f_p\) are the compressive and tensile yield strength of L-shape steel; \(f_o\) and \(f_o'\) are the compressive and tensile yield strength of longitudinal bars; \(B\) is the width of cross-section; \(a_s\) and \(a'_s\) are the distance from centroid of longitudinal bars and L-shape steel at the compression region to top surface of section, respectively; \(a_s\) and \(a'_s\) are the distance from centroid of longitudinal bars and L-shape steel at the tension zone to bottom of section, respectively; and \(a'_s\) is the distance from center line to bottom of cross-section (=\(B/2\)).

Based on the calculation formula proposed above, the predicted values were verified with FE results presented in Table 4, and the comparison results were shown in Figure 27. As can be seen from the figure, the values of \(M_u/\) and \(M_u/\) are in range of \(0.84-1.19\). The results show that the formula proposed in this paper has a satisfactory accuracy and can be used to estimate the bending capacity of LSRC column.

7. Conclusion

Flexural behavior of the precast composite column with LSRC and conventional CFTEC have been investigated in this study. Four specimens subjected to flexure were tested, and the FE modeling was performed at the same time. Based on the experimental and numerical simulation results, the following conclusions can be drawn:

- Figure 26. The calculation diagram of normal section at ultimate bending capacity.
- Figure 27. Comparison of flexural strength between predicted and test (FE) results.
(1) The reinforcement cage in LSRC column can effectively prevent the large area peeling of cover concrete. Compared to CFTEC column, LSRC member has a more full use of steel.

(2) After flexural yielding, the plastic deformation capacity of LSRC column is satisfactory, and it is better than that of CFTEC column. For LSRC and CFTEC columns, the failure behaviors of members are governed by the crushing and spalling of cover concrete, and LSRC columns present the identical flexural failure modes.

(3) The neutral axes of LSRC members are basically higher than midcourt line of cross section, and the favorable synergistic deformation capacity is exhibited in the compression region of mid-span, while LSRC column work together poorly at the tension zone. All the tested members have greater variety ratio of compressive strain after flexural yielding. Furthermore, the maximum compressive strain in the L-shape steel of LSRC column is greater than that of steel tube of CFTEC column before the peak moment.

(4) In LSRC column, L-shape steel occupies the largest proportion of bending moment capacity among different components, and its contribution of bending capacity increases with the increased mid-span curvature. After the overall yielding of LSRC column, considered as the majority component of flexural resistance, L-shape steel bears more than half of bending moment.

(5) The normal disengagement area between steel and concrete increases as the mid-span curvature increases in the center region of column. In comparison with inner interface, cover concrete works together better with L-shape steel with the confinement of stirrups. The collaborative working ability of inner interface is poorer after flexural yielding with the bond slip inclined to the bottom of pure moment zone. As L-shape steel yield, batten plate presents the faster growth rate of stress, providing the stronger confinement to steel section.

(6) The increase of L-shape leg thickness and concrete strength can apparently improve the flexural stiffness of LSRC column. In addition, the strength of specimens can be improved by increasing the sectional area of longitudinal bars and thickness of L-shape leg.

(7) Based on the results of experiment and FE modeling, the calculation formula of bending capacity of LSRC column is proposed. The calculation result has good precision and can be used to estimate the bending capacity of LSRC column.

**Nomenclature**

- \( f_y \): Yield strength of steel (L-shape steel and steel tube)
- \( E_s \): Elastic modulus of steel
- \( \nu_s \): Poisson’s ratio of steel
- \( l_s \): Shear span length of specimen
- \( l_p \): Length of pure bending zone in the specimen
- \( P \): Vertical load provided by actuator
- \( L \): Distance between roller supports
- \( \delta_t \): Deflection of LVDTs
- \( M \): Bending moment of specimen (=\( M_{LSRC} \))
- \( U_m \): Deflection of mid-span (=\( \delta_m \))
- \( P_0 \): Peak vertical load
- \( M_y \): Peak bending moment capacity
- \( \Delta_y \): Yield deflection of mid-span
- \( \Delta_u \): Deflection at failure of specimen
- \( K_e \): Initial flexural stiffness
- \( D_I \): Ductility index of specimen
- \( E_c \): Elastic modulus of concrete
- \( f_c \): Concrete strength under uniaxial compression (=\( f_{cd} \))
- \( \sigma \): Stress of concrete
- \( \varepsilon \): Strain of concrete
- \( f'_{cc} \): Peak compressive strength of cover confined concrete
- \( \varepsilon_{cc} \): Compressive strain corresponding to \( f'_{cc} \)
- \( f' \): Effective confining stress of transverse bars
- \( \rho \): Volumetric hoop ratio
- \( A_{sh} \): Overall sectional area of hoops in a unit body
- \( s \): Spacing of hoops
- \( A_l \): Sectional area of longitudinal bar
- \( t \): Thickness of L-shape leg
- \( b \): Length of L-shape leg
- \( x \): Height of compression region
- \( a \): Factor of rectangle stress block method
- \( A_l \): Longitudinal bar area of compression region
- \( A_s \): Longitudinal bar area of tension zone at bottom surface
- \( A_{ls} \): Longitudinal bar area at midcourt line of section
- \( A_i \): Angle area of compression region
- \( A_{ii} \): Angle area of tension region
- \( f_{yl} \): Yield strength of longitudinal bar at compression zone
- \( f_{yt} \): Yield strength of longitudinal bar at tension zone
- \( B \): Width of cross-section
- \( d_{ls} \): Distance from midcourt line to bottom surface
- \( M_{eq} \): Bending capacity of LSRC column from equation
- \( M_{u,FE} \): Bending capacity of LSRC column from FE modeling
- \( M_{u,exp} \): Bending capacity of LSRC column from experiment
- \( b_c \): Side length of hoops
- \( f_{yh} \): Yield strength of hoops
- \( k_e \): Effective confinement coefficient
- \( \rho_{cc} \): Area ratio of longitudinal bars to concrete surrounded by hoops
- \( n \): Number of longitudinal bar
- \( w_i \): Distance between adjacent longitudinal bars
- \( s' \): Clear spacing of hoops
- \( f_{cc} \): Peak compressive strength of core concrete
- \( \varepsilon_{cc} \): Compressive strain corresponding to \( f_{cc} \)
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\( \sigma_c \) Lateral effective confining pressure for core concrete
\( k_{c2} \) Effective confining coefficient of batten plates
\( \sigma_{s2} \) Yield strength of batten plates
\( A_{sh} \) Sectional area of batten plates in a unit body
\( S_h \) Spacing of batten plates
\( b_{c2} \) Distance between two opposite side of plates
\( w_{ct} \) Clear distance between adjacent L-shape steel
\( d \) Width of batten plates
\( \rho_{sc} \) Area ratio of L-shape steel to core concrete
\( l_{w1} \) Weld length of steel tube
\( l_{w2} \) Weld length between angle and batten plates
\( \phi \) Curvature of mid-span
\( \delta_{pl} \) Deflection of loading point
\( l_{LVDT} \) Distance from loading point to mid-span
\( M_{LS} \) Bending capacity provided by L-shape steel
\( p_{1} \) Normal stress of inner interface
\( p_{2} \) Normal stress of outer interface
\( \tau_{1} \) Tangential stress of inner interface
\( \tau_{2} \) Tangential stress of outer interface
\( \text{CAREA} \) Total area in contact of interface
\( \text{OPEN} \) Distance between slave and master surface
\( \text{CFS} \) Axial total contact frictional stress
\( \text{CSLIP} \) Relative tangential slip of interface
\( \sigma_{b} \) Stress of batten plate
\( \tau_{max} \) Critical shear stress
\( \mu \) Coefficient of friction
\( f_{y} \) Yield strength of steel section at tension region
\( a_{c} \) Distance between compressive longitudinal bar and top surface
\( a_{t} \) Distance between compressive angle and top surface
\( \alpha_{i} \) Distance between bottom longitudinal bar and bottom surface
\( H \) Distance away from the left end support
\( U \) Vertical deflection of specimens
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