

Article



# **Experimental and Numerical Analysis of Large-Scale Circular Concrete-Filled Steel Tubular Columns with Various Constructural Measures under High Axial Load Ratios**

## Lidong Zhao<sup>®</sup>, Wanlin Cao<sup>\*</sup>, Huazhen Guo, Yang Zhao, Yu Song and Zhaoyuan Yang

College of Architecture and Civil Engineering, Beijing University of Technology, Beijing 100124, China; zhao2732@sina.com (L.Z.); boringguo@163.com (H.G.); zy18811776916@163.com (Y.Z.); sy18811506626@163.com (Y.S.); yangsir1995@foxmail.com (Z.Y.)

\* Correspondence: wlcao@bjut.edu.cn; Tel.: +86-10-6739-2819

Received: 8 August 2018; Accepted: 21 September 2018; Published: 12 October 2018



**Abstract:** To investigate the effect of constructional measures (including horizontal and vertical stiffeners, rebar cages, embedded steel tubes, and cavity welded steel plates) under high axial load ratios on the seismic performance of concrete-filled steel tubular (CFST) columns, quasi-static tests for six large-scale CFST columns with various constructional measures are performed. All specimens are subjected to identical axial forces. The failure mode, hysteresis characteristics, bearing capacity, stiffness degradation, ductility, and energy dissipation of specimens are analyzed. The study shows that the horizontal stiffener delays the occurrence and severity of column base buckling, the vertical stiffener improves the bending resistance capacity and initial stiffness of the member, the rebar cage improves the ductility, and energy dissipation. When an internal circular steel tube and cavity welded steel plate are applied in tandem, the section steel ratio increases by 4.42% and the bearing capacity improves by 42.72%. A finite element model is created to verify test results, and simulation results match the test results well.

**Keywords:** Concrete-filled steel tubular (CFST); constructional measures; seismic performance; quasi-static test; high axial load ratio; finite element analysis

## 1. Introduction

Concrete-filled steel tubular (CFST) structures have advantages, including its high bearing capacity, excellent ductility, and superior fireproof properties, compared to steel structures, and it has been widely deployed in high-rise and super high-rise buildings. Currently, China is the country with the fastest pace of super high-rise building development in the world. With the development of society and the economy, the height of super high-rise buildings continuously increases, for example, the Shanghai Center (height 632 m), Wuhan Greenland Center (height 606 m), and Shenzhen Ping'an International Financial Center (height 648 m). Super high-rise buildings exceeding 600 m are normally based on a mega frame-core tube-mega support architectural structures [1]. CFST columns and steel reinforced concrete (SRC) columns are commonly used lateral force-resistant members in super high-rise buildings. Figure 1 shows the deployment of mega columns in an actual project. CFST columns place the core concrete under tri-axial stress, which significantly improves the compressive strength of concrete. Additionally, the core concrete is supported by steel tubes, which enhance its stability. The compressive bearing capacity of CFST columns significantly exceeds the sum of the bearing capacities of steel tubes and concrete. Therefore, the cross-sectional area is reduced

significantly. External steel tubes function not only as longitudinal reinforcements and stirrups but also as molds to facilitate construction. CFST sections can have circular, square, and irregular shapes. Among them, circular sections have the optimal core concrete constraint effect. An analysis by Webb and Peyton [2] suggested that, as building layers increase, the cost of construction with concrete-filled steel tubes essentially becomes the same as that with reinforced concrete.



**Figure 1.** Mega columns in practical engineering. (**a**) Concrete-filled steel tubular (CFST) columns; and (**b**) steel reinforced concrete (SRC) columns.

The seismic performance of CFST columns is commonly the focus of researchers worldwide. In the realm of theoretical research, Young and Sung [3] proposed strength formulae and simplified strength interaction curves, which provides reasonably conservative estimates for circular CFST columns under combined axial compression and bending. Bruneau and Marson [4] presented new equations to calculate the bearing capacity of concrete-filled steel tubes when subjected to axial forces and flexure. The new proposed design equations provided improved correlation between predicted strength and experimental data. Jiho Moon [5] established a validated model to predict the strength and inelastic performance of concrete-filled tubes affected by combined loading. The model was verified using previous tests. Based on the model, the author proposed a modified P-M interaction curve which could provide more accurate predictions than current design methods. In the realm of experimental studies, Xueping Li and Xilin Lu et al. [6] performed seismic performance tests for 16 rectangular CFST columns with side lengths up to 300 mm; the parameters under investigation included the section length-width ratio, the steel ratio, and the loading direction. With the increased of axial load ratio and section length-width ratio, the ductility of the specimen decreased obviously. Ahmed Elremaily and Atorod Azizinamini [7] performed quasi-static tests for six circular CFST columns with diameters of 324 mm; the axial load ratio ranged from 0.2–0.4, and the effect of the diameter-thickness ratio on the seismic performance of CFST columns was investigated. The test results showed that when the diameter-thickness ratio was larger, the member still had good ductility. Amir Fam [8] performed seismic performance tests for five circular CFST beam-column specimens with diameters of 152 mm; the axial load ratio was 0.25, and the study focused on the effects of steel tube and concrete bonding strength on the seismic performance of CFST columns. The test results suggested that whether there was bond between steel tube and concrete had no significant effect on bending strength. P. Gajalakshmi and H. Jane Helena [9] performed quasi-static tests for 16 circular CFST column specimens with diameters of 114 mm to investigate the accumulated damage failure mechanism of a structure; the axial load ratio in the test was 0.3. The experimental results showed that the steel fiber reinforced CFST columns exhibited about 1.5 to 2 times more energy dissipation capacity than the CFST columns. Konstantinos A. Skalomenos [10] performed quasi-static tests for three circular CFST column specimens with diameters of 150 mm; the axial load ratio was 0.25. The external steel tubes of some specimens

were based on super high-strength steel H-SA700. The test results suggested that, compared to conventional steel, structures with super high-strength steel had superior bearing capacity and delayed external steel tube local buckling time. To investigate the effect of mixing rubber particles in concrete on the seismic performance of a circular CFST column, A.P.C. Duarte [11] performed quasi-static tests for 20 circular CFST columns with diameters of 141 mm or 219 mm; the axial load ratio was set to 0.1 or 0.2. The test results showed that adding 5% rubber particles into the concrete had the least effect on the strength and could significantly improve the ductility of the members. Feiyu Liao [12] performed seismic performance tests for 10 stainless CFST columns with diameters of 120 mm. The test results suggested that, compared to low carbon steel specimens, stainless CFST columns had superior ductility, deformation capability and residual strength. Six square hybrid double-skin tubular columns with side length of 150 mm were tested under combined axial compression and cyclic lateral loading by Yunita and Togay [13], the axial load ratio ranged from 0.14–0.42. The results showed that the specimens with thicker inner steel tube had a plumper hysteretic curve. Additionally, the plastic hinge lengths were effected by the axial load level and the size of inner steel tube. Omar and Mohamed [14,15] performed seismic performance tests for hollow-core FRP-concrete-steel (HC-FCS) columns; the axial load ratio was 0.05. The tests results suggested that the total energy dissipation of the well-confined HC-FCS column was 87% higher than that of the conventional RC. Increasing the steel tube thickness by 100% increased the flexural strength by 62%. Teng and Yu et al. [16,17] proposed hybrid FRP-concrete-steel double-skin tubular columns with a diameter of 152.5 mm, which consist of an inner steel tube and an outer FRP tube. Axial compression and bending experiments were conducted and the test results showed that these members were found to have a very ductile behavior. Yang [18,19] studied the compression behavior of square and T-shaped CFST columns with reinforcement stiffeners. The tests results showed that the stiffeners can delay the tube bulking and improve the confinement for concrete and ductility of specimens.

Previous experimental studies of seismic performance show that researchers worldwide have conducted comprehensive studies on the diameter-thickness ratio, bond slip behavior, and material properties of steel tubes and concrete. However, the following problems remain: (1) The dimensions of most models are small. The diameters of circular steel tubes are in the range of 114–324 mm. The dimension effect cannot be ignored. Large-scale specimen experimental studies are scarce. (2) Studies on constructional measures, such as horizontal stiffeners, vertical stiffeners, core structure rebar, embedded circular steel tubes, and cavity welded steel plates, are scarce. (3) Experimental studies on the seismic performance of CFST columns under high axial load ratios are scarce. To address these problems, in this paper, based on mega framework circular CFST columns in super high-rise buildings, six large-scale circular CFST column specimens with various cavity constructional measures are designed and prepared. Quasi-static tests under high axial load ratios are performed to investigate the failure mode, bearing capacity, hysteresis characteristics, stiffness degradation, ductility, and energy dissipation of each specimen. In this paper, the steel tube internal structure is investigated systematically. The effects of various constructional measures on the seismic performance of CFST columns are compared, which provides reference for an actual project.

#### 2. Experimental Programs

#### 2.1. Test Specimen

Based on a super high-rise tower building in China, six large-scale circular CFST column specimens are prepared. The scale of the specimen is 1/8. The specimens are named CFST-N (no constructional measures), CFST-HS (horizontal stiffeners), CFST-VS (vertical stiffeners), CFST-SR (stiffeners and rebar), CFST-DT (double-skin tubes), and CFST-MC (multiple cavities). The steel tube parameters are as follows: The exterior diameter is D = 508 mm; the wall thickness is t = 9 mm; D/t = 56.4; and the column clear height is 1270 mm. The base is built of a cube composed of six welded, square steel plates filled with concrete. The base is 600 mm high. An opening (diameter 511 mm) is

drilled at the center of the upper steel plate. The steel tube penetrates through the opening and is welded to the floor. One vertical stiffener plate is deployed at each side of the base loading direction. The stiffener plate is welded to the steel pipe, at the floor and roof. The test loading method is as follows: The top of column is fixed, and the lower end is pushed at the base by a horizontal actuator to perform the quasi-static tests. Therefore, to facilitate fixation of the column top, four square steel plates (thickness of 20 mm) are welded around the column top. The gap between the upper and lower surfaces undergoes lofting and is welded with a 20 mm-thick steel plate with a corresponding shape. Consider specimen CFST-MC for example: The geometrical dimensions and structure are shown in Figure 2. Figure 2a is a top view of the specimen; and Figure 2b is a three-dimensional diagram of the specimen cavity structure. Deployment of a cavity welded steel plate between the internal circular steel tube and external circular steel tube solves the hydration heat problem associated with large-volume concrete. A rectangular opening at the cavity steel plate connects the concrete in each cavity and improves the integrity of the structure. Figure 2e shows a three-dimensional diagram of the entire specimen.



**Figure 2.** Dimensions and details of CFST-MC. (**a**) top view of specimen; (**b**) stereogram of cavity structure; (**c**) section 1-1; (**d**) section 2-2; and (**e**) stereogram of whole model.

Figure 3 shows the section structure of each specimen. Specimen CFST-N is a steel tube concrete section without any structure. Horizontal stiffeners are welded to the interior wall of the external steel tube in specimen CFST-HS. The rib plate width is b = 40 mm and the thickness is t = 4 mm. The first horizontal stiffener is deployed at 100 mm above the base of the top surface. Then, more are deployed upward every 200 mm, with six total. Horizontal stiffeners and vertical stiffeners are welded on the interior wall of the external steel tube of specimen CFST-VS. The vertical stiffener width is b = 37 mm,

and the thickness is t = 4 mm, which is deployed uniformly at six locations around the perimeter. Specimen CFST-SR is a scaled-down section of a prototype in an actual project. Horizontal and vertical stiffeners are deployed, and three layers of rebar cages are deployed in the steel tube. The inner ring longitudinal reinforcement is 6Deight, the mid-ring longitudinal reinforcement is 14Deight, and the outer-ring longitudinal reinforcement is 20Deight. The distances between the longitudinal reinforcement central lines and the section origin are 65 mm, 123 mm, and 180 mm, respectively. The reinforcement ratio is 1.07%, the stirrup is Bsix@120, and the volume stirrup ratio is 0.52%. Specimen CFST-DT is a double-skin circular CFST column specimen designed using the same amount of steel for its internal structure as specimen CFST-SR (including horizontal and vertical stiffeners, longitudinal rebar and stirrup). The exterior diameter of the internal circular steel tube is D = 245 mm, and the wall thickness is t = 6 mm. In specimen CFST-MC, a cavity welded steel plate is deployed between the internal steel tube and the external steel tube. Additionally, a rectangular opening is drilled at the steel plate, and the cavity dimension is 30 mm  $\times$  100 mm.



Figure 3. Cross sections of specimens. (a) CFST-N; (b) CFST-HS; (c) CFST-VS; (d) CFST-SR; (e) CFST-DT; and (f) CFST-MC.

#### 2.2. Material Properties

After specimen preparation, concrete is transported in a tank from the high strength concrete mixing station to the construction site for pouring. The concrete strength grade is C45 self-compacting, and the concrete mixture ratio is listed in Table 1.

The base and steel tube interior are based on concrete of the same grade. Measurements show that the average compressive strength of three cubic concrete specimens under the same curing conditions (side length of 150 mm) is  $f_{cu,m} = 61.85$  MPa. The concrete strength variation coefficient  $\zeta$  is determined through testing, and the value is 0.039. The standard value of the concrete cube compressive strength is  $f_{cu,k} = f_{cu,m} (1-1.645 \zeta) = 57.88$  MPa. The average concrete axial compressive strength is  $f_{c,m} = (0.66 + 0.002 f_{cu,k}) f_{cu,m} = 47.98$  MPa [20], and the elastic modulus is  $E_c = 33.5$  GPa.

Table 1. Mix proportion of concrete	Table 1.	Mix	proportion	of concrete
-------------------------------------	----------	-----	------------	-------------

Concrete Grade	Water (kg/m <sup>3</sup> )	Cement (kg/m <sup>3</sup> )	Fine Aggregates (kg/m <sup>3</sup> )	Coarse Aggregates (kg/m <sup>3</sup> )
C45	162	321	738	1019

The steel grade is Q345, and the rebar grade is HRB400. The steel undergoes a series of tensile performance tests. The yield strength  $f_y$ , ultimate tensile strength  $f_u$  and elongation ratio  $\delta$  of the steel tube, stiffener, and rebar measured in the tests are listed in Table 2.

Property Parameters	Steel Tubes		Stiffeners	Rebar	
	T = 9 mm	T = 6 mm	T = 4 mm	D = 8 mm	D = 6 mm
$f_{y}$ (MPa)	328	343	358	716	608
$f_u$ (MPa)	462	445	476	966	870
δ (%)	23.4	25.3	14.6	15.9	16.7

Table 2. Mechanical properties of steel.

## 2.3. Test Device and Loading Procedure

The quasi-static test is performed with a 4000 t large-scale test device from the Beijing University of Technology. This device has a 4000 t vertical loading capability and a 400 t horizontal cyclic loading capability. The specimen is fixed on a test bed that can move horizontally. The horizontal loading device pushes the base to achieve horizontal cyclic loading. The test loading direction is South-North. The top of the test device is a spherical hinge; the vertical loading system applies a load to the top end of the specimen and restricts the top end of the specimen from horizontal displacement. A diagram of the test loading device is shown in Figure 4, and a field image is shown in Figure 5a. In the test, draw-wire displacement meters one and two are deployed on the ground and test bed to measure the base horizontal displacement and relative slip between the base and test bed. The test results show that the base is steadily fixed and that the relative slip between the specimen and test bed can be ignored.



Figure 4. Test setup. Note: 1. Specimen; 2. M48 bolts; 3. Test pedestal; 4. Test car; 5. Rail; 6. Load transducer; 7. Spherical hinge; 8. Beam; 9. LVDT-1; 10. LVDT-2; 11. Reaction frame; 12. Balance weight; and 13. Wire rope.

When the test starts, axial pressure is applied to the specimen until it reaches a predefined value, which then remains stable during horizontal cyclic loading. Then, the horizontal actuator pushes the base, fixed on a scooter, to apply a horizontal cyclic force to the specimen. The six specimens are under an identical axial pressure of 9180 kN. The axial load ratio n of each specimen is calculated via Formula (1). In this formula, the steel tube constraint effect on the core concrete is considered [21]; N is the specimen vertical axial force;  $A_{stiff}$  and  $A_{rebar}$  are the total areas of the vertical stiffener and longitudinal rebar, respectively; and  $f_{stiff}$  and  $f_{rebar}$  are the yield strengths of the vertical stiffener and longitudinal rebar, respectively. The constraint effect coefficient is  $\theta = A_s f_y / A_c f_{c,m}$ , where  $A_s$  and  $A_c$  are the areas of the steel tube and core concrete, respectively, and  $f_y$  is the steel tube yield strength.

$$n = N/[0.9f_{c.m}A_c(1+1.8\theta) + f_{stiff}A_{stiff} + f_{rebar}A_{rebar}]$$
(1)

The structure steel ratio is defined as  $\rho = V_s/V_c$ , where  $V_s$  is the overall steel volume of the CFST column and  $V_c$  is the volume of concrete. The steel ratio and axial load ratio of each specimen are listed in Table 3. The test is based on a displacement loading procedure [22]. When the lateral drifts are 1/1000, 1/500, 1/300, and 1/250, cyclic loading is applied once. When the lateral drifts are 1/200, 1/100, 1/75, 1/50, 1/33, 1/25, 1/20, and 1/16.7, cyclic loading is applied twice. The loading procedure is shown in Figure 5b.



Figure 5. CFST column test. (a) Test site; and (b) typical cyclic loading protocol.

## 3. Test Results and Analysis

#### 3.1. Test Observations

Quasi-static tests are performed for six circular CFST column specimens. The observed failure mode of each specimen is as follows:

(1) All six specimens demonstrate a compressive bending failure mode. The failure process of specimen CFST-N is elaborated here. When the lateral drift is smaller than 1/250, the circular steel tube exterior wall has no change. When the lateral drift is 1/200, at 50 mm above the column base in the North, a slight bulge of the steel tube can be felt by hand. When a reverse load is applied, the bulge is flattened, and there is no residual deformation. When the lateral drift is 1/100, bulge deformation is observed at both the South and North ends, and it gradually expands upward to 100 mm above the column base. When the lateral drift is 1/75, bulge deformation at the South and North ends grows continuously; at 100 mm above the column base in the North, paint starts to peel off, and there is residual deformation expands toward the East and West and gradually penetrates, the paint layer peels-off in the North and becomes more severe, and concrete in the steel tube produces a sound. When the lateral drift is 1/33, the paint layers peel off at the East and West ends. When the lateral drift is 1/25, all paint layers within 170 mm of the column base peel off, bulge deformation around the column base completely penetrates, a semi-waveform bulge is formed, and local buckling of the steel tube is severe.

(2) Specimens CFST-HS, CFST-VS and CFST-SR demonstrate similar failure modes. When the lateral drift reaches 1/25, unlike specimen CFST-N, the 3 specimens do not form a connected bulge

deformation around the column base. Bulge deformation occurs at the South and North ends, but the East and West ends show almost no deformation. Bulge deformation at the South and North ends ranges from 70–90 mm above the column base. Peeling of paint layers from the columns of the three specimens is not as severe as for specimen CFST-N, and it occurs primarily at the South and North ends.

(3) Specimens CFST-DT and CFST-MC show bulge deformation at higher heights than the other four specimens. When the lateral drift reaches 1/25, the bulge deformation of specimen CFST-DT ranges from 120–140 mm above the column base; the East and West ends only show slight bulge deformation. Peeling of paint occurs within 140 mm above the column base; the East and West ends show more severe peeling than the South and north. Bulge deformation of specimen CFST-MC occurs in the range of 125–175 mm above the column base; the East tand West ends show almost no deformation. Peeling of paint layers is concentrated at the South and North ends; the East and West ends are nearly unaffected. Failure images of typical specimens CFST-N, CFST-VS and CFST-MC are shown in Figure 6.



Figure 6. Cont.



**Figure 6.** Failure modes of specimens. (**a**) failure of CFST-N; (**b**) detailed measurement of CFST-N; (**c**) failure of CFST-HS; (**d**) detailed measurement of CFST-HS; (**e**) failure of CFST-VS; (**f**) detailed measurement of CFST-VS; (**g**) failure of CFST-SR; (**h**) detailed measurement of CFST-SR; (**i**) failure of CFST-DT; (**j**) detailed measurement of CFST-DT; (**k**) failure of CFST-MC; and (**l**) detailed measurement of CFST-MC.

3.2. *Hysteresis Characteristics* 

Hysteresis curves from the test are shown in Figure 7, where F represents the horizontal load and  $\Delta$  represents the horizontal displacement. Figure 7 shows the following:

(1) The hysteresis curve for each specimen is full and spindle-shaped. There is no notable pinching, and the hysteresis performance is stable.

(2) Before the lateral drift increases to 1/250, the hysteresis curve of each specimen essentially changes linearly, and there is no residual deformation after the load is removed. This means that the specimen is in an elastic state. When the horizontal lateral drift increases gradually and the horizontal load is reduced to zero, the displacement becomes non-zero, which means that the specimen develops residual deformation at this moment.

(3) The loading and offloading curves in the first cycle and the second cycle of the same grade lateral drift for each specimen essentially coincide, which means that the accumulated damage of each specimen is small.

(4) Specimens CFST-DT and CFST-MC have fuller hysteresis curves and higher bearing capacities.



Figure 7. Cont.



**Figure 7.** Measured and FEA model load versus displacement relations. (**a**) CFST-N; (**b**) CFST-HS; (**c**) CFST-VS; (**d**) CFST-SR; (**e**) CFST-DT; and (**f**) CFST-MC.

Skeleton curves of specimens are shown in Figure 8. Before the horizontal force of each specimen reaches 0.6 times the peak load, the skeleton curve is approximately linear. After the horizontal force exceeds 0.6 times the peak load, the specimen is in a completely elastoplastic state, and the skeleton curve becomes nonlinear. After the horizontal force reaches the peak load, the skeleton curve of each specimen enters a declining section; however, the declining section of each specimen has a different degradation speed. Specimen CFST-N has the fastest degradation speed in the declining section. When the lateral drift reaches 1/25, the bearing capacity has dropped to 62.8% of the peak load. Specimen CFST-MC has the slowest degradation speed in the declining section. When an identical lateral drift is reached, the bearing capacity decreases to 88.4% of the peak load. The test results show that the axial load ratio is higher, the second-order effect has a greater impact on the bending resistance and stability of the structure. At the later stages of loading, specimen CFST-N shows severe local buckling of the steel tube base, and the bearing capacity declines rapidly.



Figure 8. Skeleton curves of specimens.

## 3.3. Bearing Capability

Based on the energy equivalence method [23], the nominal yield load of each specimen  $F_y$  is calculated from the skeleton curve. The calculation method is shown in Figure 9. Connecting point O to point C under the condition of area (1) equal to area (2) and ensuring the ordinate value of point C is  $F_p$ . Point B is on the skeleton curve and has the same displacement value as point C. Hence the ordinate value of point B is the nominal yield load of each specimen.  $F_p$  is the peak load; the ultimate load is defined as the load when the horizontal force decreases to 85% of the peak load, represented by the symbol  $F_u$ . Actual measurement results of the specimens are shown in Table 4 and Figure 10.



Figure 9. Diagram of energy equivalent method.

Table 4. Measured load and displacement at characteristic points.

Spacimons	No	minal Yi	eld		Peak			Ultimate			
No.	Fy (kN)	$\Delta_y$ (mm)	θ <sub>y</sub> (rad)	<i>F<sub>p</sub></i> (kN)	$\Delta_p$ (mm)	θ <sub>p</sub> (rad)	<i>F<sub>u</sub></i> (kN)	$\Delta_u$ (mm)	θ <sub>u</sub> (rad)	$F_y/F_p$	μ
CFST-N	816.3	13.50	1/113	987.9	26.35	1/58	839.8	49.10	1/31	0.826	3.64
CFST-HS	858.1	15.79	1/96	1043.9	31.46	1/48	887.4	61.42	1/25	0.822	3.89
CFST-VS	970.8	13.74	1/111	1152.9	23.60	1/65	980.0	57.09	1/26	0.842	4.16
CFST-SR	983.8	14.49	1/105	1153.5	33.49	1/46	1002.0	63.96	1/24	0.853	4.42
CFST-DT	1097.0	14.01	1/109	1304.9	25.39	1/60	1109.1	60.68	1/25	0.841	4.33
CFST-MC	1202.0	14.20	1/107	1409.9	30.27	1/50	1224.8	64.23	1/23	0.853	4.52



Figure 10. Bar graph of bearing capacity.

Table 4 and Figure 10 show the following: (1) When the steel ratio increases, the yield load, and peak load of each specimen increase accordingly. Compared to CFST-N, specimen CFST-MC shows the following improvements: The section steel ratio increases by 4.42%, and the yield load and peak load improves by 47.25% and 42.72%, respectively. (2) Compared to CFST-N, specimen CFST-HS has the following improvements: The section steel ratio increases by 0.62%, and the yield load and peak load improve by 5.12% and 5.67%, respectively. While the bearing capacity improvement is marginal, the horizontal stiffener has a notable local stabilizing effect on the steel tube column, which delays buckling deformation of the steel tube. The skeleton curve also shows that when the horizontal stiffener is added, the degradation speed in the declining section slows significantly. (3) Compared to specimen CFST-HS, specimen CFST-VS has the following improvements: The section steel ratio increases by 0.53%, and the yield load and peak load improves by 13.13% and 10.44%, respectively. This means that the vertical stiffener specimen significantly improves the bending resistance capacity. (4) The core structure rebar has a small impact on the specimen bearing capacity. (5) The embedded circular steel tube has a significant constraining effect on the core area concrete, which improves the bearing capacity by 32%. (6) Specimen CFST-DT has the same steel ratio as specimen CFST-SR; however, the yield load and peak load improve by 11.51% and 13.13%, respectively. (7) Based on specimen CFST-DT, specimen CFST-MC has a cavity welded steel plate between the internal steel tube and external steel tube to further improve the bearing capacity of the structure. The yield load and peak load improve by 9.57% and 8.05%, respectively. When the lateral drift reaches 1/25, the bearing capacity did not reach the failure load, and the specimen still has a desirable bearing capability.

#### 3.4. Stiffness Degradation

The average of the secant stiffness in the positive and negative directions is used to measure specimen stiffness at each grade of lateral drift. The average secant stiffness  $K_i$  for each specimen at the *i*-th grade of lateral drift is calculated via formula (2):

$$K_{i} = \sum_{j=1}^{m} \left( \left| +P_{i,j} \right| + \left| -P_{i,j} \right| \right) / \sum_{j=1}^{m} \left( \left| +\Delta_{i,j} \right| + \left| -\Delta_{i,j} \right| \right)$$
(2)

where *m* is the number of times that a certain grade of lateral drift is loaded, and  $\Delta_{i,j}$  and  $P_{i,j}$  are the corresponding maximum horizontal displacement and horizontal forces when the i-th grade of lateral drift is loaded for the *j*-th time.

Figure 11 and Table 5 show the average secant stiffness corresponding to each lateral drift of each specimen from the start of loading at a 1/250 lateral drift to the end of loading. Figure 11 shows that all specimens essentially have a consistent trend of stiffness degradation: The stiffness

degradation is fast during the initial stage and slow during the later stage. The test results show that the horizontal stiffener has a small impact on the initial stiffness, whereas the vertical stiffener significantly improves the initial stiffness. Compared to specimen CFST-HS, specimen CSFT-3 has the following improvements: The initial stiffness improves by 19.53%. The core structure rebar slows the stiffness degradation. Compared to specimen CFST-VS, specimen CFST-SR has the following improvements: The initial stiffness improves by 6.42%. A comparison of specimen CFST-SR and specimen CFST-DT shows that the two have identical section steel ratios. For the double-skin circular CFST column, since the core concrete is under dual constraints, including an external steel tube and an internal steel tube, the section stiffness improves significantly. The stiffness degradation curve shows that the secant stiffness corresponding to each grade of lateral drift for specimen CFST-DT exceeds that for specimen CFST-SR. Specimen CFST-MC has the highest section steel ratio and the slowest stiffness degradation.



Figure 11. Stiffness degradation.

A, (rad)	<i>K</i> (10 <sup>4</sup> kN/m)						
	CFST-N	CFST-HS	CFST-VS	CFST-SR	CFST-DT	CFST-MC	
1/250	91.38	95.66	114.34	121.68	125.66	120.74	
1/200	80.49	76.44	83.25	96.71	107.64	120.51	
1/150	63.86	64.19	69.27	79.27	89.31	95.16	
1/100	48.86	50.12	53.61	59.40	66.06	71.55	
1/75	38.10	41.54	41.87	48.54	54.42	57.21	
1/50	26.22	29.92	28.35	32.65	38.45	43.18	
1/33	14.77	19.52	19.23	20.19	24.72	28.01	
1/25	7.47	13.41	13.27	14.74	16.33	19.76	

Table 5. Secant rigidity at each drift level.

#### 3.5. Ductility and Energy Dissipation

The displacement ductility coefficient  $\mu$  is calculated using  $\mu = \Delta_u / \Delta_y$ , where  $\Delta_u$  is the displacement when the bearing capacity decreases to 85% of the peak load, and  $\Delta_y$  is the displacement corresponding to the nominal yield load. The ductility coefficient is based on the average of values in the push and pull directions. When there are two loading cycles, the average of the ductility coefficients in the two cycles is used. The ductility coefficient of each specimen is listed in Table 4. Specimen CFST-N has the smallest ductility coefficient; however, it reaches 3.64. The average ductility coefficient of 6 specimens is 4.16, which means that the circular CFST column still has desirable deformation performance under a high axial load ratio. The results show that when the axial load ratio decreases, the ductility coefficient increases. It is also worth noting that when loading stops, the bearing capacities of specimens CFST-SR and CFST-MC are still above 85% of the peak load. Therefore, when calculating  $\Delta_u$ , the horizontal displacement values of the two specimens during the final loading cycle are used, and the ductility coefficient is actually larger than the calculated value in the table. A comparison of specimen CFST-SR and specimen CFST-VS shows that the deployment of core structure rebar in

the steel tube effectively improves the ductility of the specimen. This is because cement hydration heat produces significant thermal stress in concrete and cracks on the external surface of the concrete, which weakens the bonding between the steel tube and the concrete. Additionally, concrete contraction deformation reduces bonding. Previous research [8] has shown that strong bonding between the steel tube and the concrete results in superior structure ductility. The core structure rebar functions as thermal rebar, which enhances bonding between the steel tube and concrete, thus improving the ductility of the structure. Deployment of an internal circular steel tube improves the structure ductility coefficient by 18.96%. Table 4 shows that the elastoplastic lateral drift of each specimen always exceeds 1/31, which means that the structure has excellent plastic deformation capability.

Energy dissipation capability is another important index to evaluate the seismic performance of structures. In this paper, accumulated energy dissipation is used as an evaluation index. A CFST structure relies on steel tube plastic deformation, concrete micro-fracture development, and a bonding slip between the steel tube and concrete to dissipate absorbed energy. The method to calculate accumulated energy dissipation is to calculate the sum of areas surrounded by horizontal force-displacement hysteresis curves that correspond to previous grades of lateral drifts when the specimen reaches a certain grade of lateral drift. In the formula, E<sub>p</sub> is the accumulated energy dissipation when each specimen reaches its peak load, and Et is the accumulated energy dissipation when the loading on each specimen ends. The accumulated energy dissipation of each specimen is shown in Table 6 and Figure 12. Compared to specimen CFST-N, specimens CFST-HS through CFST-MC, with different structures, exhibit significantly improved energy dissipation capability. When the specimens reach their peak loads, the accumulated energy dissipation Ep improves by 44.2%, 51.5%, 59.8%, 96.8%, and 95.3%. The specimen energy dissipation  $E_t$  during the entire process improves by 3.1%, 6.8%, 13.7%, 28.4%, and 45.4%. The results show that the structure has a higher impact on E<sub>p</sub> than E<sub>t</sub>. Specimen CFST-SR and specimen CFST-DT have the same level of steel consumption. However, the  $E_p$  and  $E_t$  of specimen CFST-DT are 37% and 14.7% higher than those of specimen CFST-SR. The reason is that the embedded circular steel tube increases the contact area between the steel tube and concrete. Therefore, the structure dissipates more absorbed energy via the bonding slip effect. Specimen CFST-MC has the highest section steel ratio and the highest level of accumulated energy dissipation.

Specimens No.	$E_p$ (MN·mm)	<b>Relative Values</b>	$E_t$ (MN·mm)	<b>Relative Values</b>
CFST-N	188.943	1.000	950.806	1.000
CFST-HS	272.449	1.442	980.249	1.031
CFST-VS	286.272	1.515	1015.051	1.068
CFST-SR	301.263	1.598	1081.123	1.137
CFST-DT	371.930	1.968	1220.641	1.284
CFST-MC	369.072	1.953	1380.011	1.454

Table 6. Cumulative energy dissipation.



Figure 12. Cumulative energy dissipation of specimens.

## 3.6. Strain

Figure 13a illustrates the layout of the column root strain gauges for each specimen. Vertical strain gauge one and horizontal strain gauge two are deployed at 50 mm above the top surface of the base to measure specimen deformation. The strain variation patterns of the six specimens are essentially consistent. Figure 13b through Figure 13g show horizontal force-strain curves of typical specimens CFST-SR, CFST-DT, and CFST-MC.



**Figure 13.** Lateral load versus steel strain relations. (a)positions of strain gauges on the bottom surface of steel tube; (b) Strain1 (CFST-SR); (c) Strain2 (CFST-SR); (d) Strain1 (CFST-DT); (e) Strain2 (CFST-DT); (f) Strain1 (CFST-MC); and (g) Strain2 (CFST-MC).

16 of 21

The specimen horizontal force-strain curves show that at the initial stage of loading, the column roots are under alternating tension and compression in the vertical direction. As the lateral drift gradually increases, the specimen root is always in a compression state in the vertical direction. The vertical strains of specimen CFST-DT and specimen CFST-MC do not reach the yield strain in the tensile direction. Specimen CFST-SR has both a deployed horizontal and vertical stiffeners, which changes the stress state at the strain measurement point for the external steel tube. Therefore, this specimen reaches the yield strain in the tensile direction. The horizontal strains of all specimens are always in the tensile state and have reached the yield strain.

## 4. Finite Element Analysis

In this paper, ABAQUS (ABAQUS Inc., Pawtucket, RI, USA) finite element software is used for numerical simulation analysis. The steel tube and stiffener are based on an S4R shell element, and the concrete is based on a C3D8R solid element. The base is defined as a rigid body. A cubic rigid body with a 528 mm side length and a 20 mm thickness is created at the top of the steel tube column to apply a uniform vertical load to the specimen. The length of foundation is 1800 mm, width 900 mm, and height 600 mm. The height of the column is 1270 mm and the outer diameter of the tube is 508 mm. As specimen failure occurs at the column base, the top is essentially in an elastic state during the test. Therefore, the column top structure is simplified in the model. The specimen boundary conditions are as follows: the translational and rotational degrees of freedom of the base in three directions are fixed, and a 32.93 kN/m<sup>2</sup> uniform vertical load is applied to the pressure bearing plate at the top to achieve a constant resultant force of 9180 kN during the test. Cyclic horizontal displacement is applied to the column top. The displacement load history is the same as in the test. The finite element model has the same stress mechanism as the test.

Steel and rebar are based on an elasticity enhancement model, as shown in Figure 14a. The stress-strain relationship of the hardening section is simplified as an oblique line. The yield load and yield strain are based on actual measurements. The test results of material properties indicated that  $E_{s}' \approx 0.01 E_{s}$ . The Poisson ratio is set to 0.3, and the strengthening type is kinematic hardening. The core concrete is based on the stress-strain relational model proposed by Han [24], which considers the constraint effect coefficient, as shown in Figure 14b,c. Figure 14b shows the compression constitutive relation of circular steel tube core concrete. Figure 14c shows the tensile constitutive relation of concrete. The Poisson ratio of concrete is set to 0.2. In ABAQUS, the nominal stress  $\sigma_{norm}$  and nominal strain  $\varepsilon_{norm}$  should be converted to real stress  $\sigma_{true}$  and real strain  $\varepsilon_{true}$  via formulae  $\sigma_{true} = (1 + \varepsilon_{norm}) \sigma_{norm}$  and  $\varepsilon_{true} = \ln(1 + \varepsilon_{norm})$  [25]. The concrete plasticity parameters are based on the concrete plastic damage model provided by ABAQUS [26]. The relevant parameters are shown in Table 7.



**Figure 14.** Constitutive relationship of materials. (**a**) steel and rebar; (**b**) compression of concrete; and (**c**) tension of concrete.

Table 7. Parameters of CDP.

Dilation Angle	Eccentricity	fb0/fc0	К	Viscosity Parameter
30	0.1	1.16	0.6667	0.005

The degradation of the elastic stiffness is characterized by two damage variables,  $d_t$  and  $d_c$ , which are assumed to be functions of the plastic strains. As shown in Formulas (3) and (4):

$$d_t = 1 - \frac{\sigma_t}{E_0(\varepsilon_t - \widetilde{\varepsilon}_t^{pl})} \tag{3}$$

$$d_c = 1 - \frac{\sigma_c}{E_0(\varepsilon_c - \widetilde{\varepsilon}_c^{pl})} \tag{4}$$

where the subscripts *t* and *c* refer to tension and compression, respectively;  $\tilde{\varepsilon}_t^{pl}$  and  $\tilde{\varepsilon}_c^{pl}$  are the equivalent plastic strains.  $E_0$  is the initial modulus of the material. Stiffness degradation variable curves are shown in Figures 15a and 15b.  $\tilde{\varepsilon}_t^{ck}$  is the cracking strain and  $\tilde{\varepsilon}_c^{in}$  is the inelastic strain.



Figure 15. Damage variable curves. (a) d<sub>c</sub>; and (b) d<sub>t</sub>.

The contact between the steel tube and concrete is defined as the surface-to-surface contact. Normal behavior is based on hard contact; the friction formula for the tangential behavior is based on the penalty function, and the friction coefficient is 0.4. The stiffener and steel tube are bonded and constrained. The part that overlaps with concrete is buried. Rebar is based on a truss element and is buried in concrete. The meshing for the steel structure in the circular CFST column specimen is shown in Figure 16. Overall, the meshing of the specimen model is shown in Figure 17.



**Figure 16.** Finite element mesh of steel structure. (a) CFST-N; (b) CFST-HS; (c) CFST-VS; (d) CFST-SR; (e) CFST-DT; and (f) CFST-MC.



Figure 17. Finite element mesh of whole specimen.

capacity is significantly improved.

Simulation results for hysteresis curves of the 6 circular CFST column specimens are shown in Figure 7. Figures 18 and 19 show the Mises stress contour plot of outer steel tube and the minimum principal stress contour plot of core concrete when the lateral drift reaches 1/16.7. The failure mode is essentially consistent with the test results. As can be seen from Figure 18a, compared with other specimens, specimen CFST-N has the most severe buckling at the bottom of column, and the maximum Mises stress reaches 461 MPa, which is close to the failure load. The steel ratio of CFST-MC increased by 4.42% compared with that of CFST-N, while the Mises stress of steel tube decreased by 17.9%. This is because the internal and external steel tubes are connected as a whole by welding steel plates, and work together to enhance the local stability of the external steel tubes. The stiffeners can significantly alleviate the buckling of the column root, and make the steel tube enter the yield area wider, which can make full use of the properties of steel. It can be seen from Figure 19 that when the lateral drift reaches 1/16.7, the concrete in the steel tube is under compression on both sides of the loading direction. With the increase of section steel ratio, the absolute value of the minimum principal stress of concrete is stronger. It can be seen that the minimum principal stress of concrete in some areas is greater than the peak load input in the concrete constitutive

model, which is because the concrete in the steel tube is under tri-axial stress state, and the bearing



Figure 18. Mises stress conctour plot of outer steel tube. (a) CFST-N; (b) CFST-HS; (c) CFST-VS; (d) CFST-SR; (e) CFST-DT; and (f) CFST-MC.



Figure 19. Min principal stress contour plot of concrete. (a) CFST-N; (b) CFST-HS; (c) CFST-VS; (d) CFST-SR; (e) CFST-DT; and (f) CFST-MC.

The peak bearing capacities of the 6 circular CFST columns are calculated with the finite element model and are compared with the test results, as shown in Table 8. The error between the calculated results and measured results is within 5%.

Specimens No.	F <sub>test</sub> (kN)	F <sub>simulate</sub> (kN)	$F_{simulate}/F_{test}$
CFST-N	979.9	985.5	1.01
CFST-HS	1049.0	1068.6	1.02
CFST-VS	1151.7	1161.9	1.01
CFST-SR	1153.0	1214.6	1.05
CFST-DT	1303.4	1312.4	1.01
CFST-MC	1422.5	1447.6	1.02

Table 8. Comparison between tested and simulated values of bearing capacity.

#### 5. Conclusions

Quasi-static tests are performed for each specimen under identical axial forces. Based on the test results, the failure mode, hysteresis characteristics, bearing capacity, stiffness degradation, ductility, and energy dissipation of each specimen are analyzed. Additionally, a finite element model is created to verify the test results. The conclusions of the study are as follows:

(1) When the lateral drift reaches 1/25, the bearing capacity of specimen CFST-N declines rapidly. At the end of loading, the bearing capacity decreases to 62.8% of the peak load. The column root has the most severe buckling deformation. Specimen CFST-MC has the fullest hysteresis curve, in which the skeleton curve of the declining section is gentle and the seismic performance is stable. This is mainly

because with the increase of section steel ratio, the constraint effect on core concrete is continuously improved, and the stability of members is significantly improved.

(2) The horizontal stiffener has a significant constraining effect on the steel tube column surface external deformation, and it enhances the confining effect on concrete. The vertical stiffener improves the section bending resistance capacity and initial stiffness. The core structure rebar has a small impact on the bearing capacity, but it effectively improves the structure ductility. The embedded circular steel tube has a substantial constraining effect on concrete in the core area, and it significantly improves the bearing capacity, ductility coefficient, and accumulated energy dissipation of the structure. Specimen CFST-DT has the same steel consumption as specimen CFST-SR; however, compared to specimen CFST-SR, the bearing capacity and accumulated energy dissipation at peak load improve by 13.13% and 37%, respectively. This is because the double-skin steel tube increases the contact area between steel tube and concrete. The members can dissipate more energy through bond slip, so the accumulated energy dissipation is significantly increased. Compared to specimen CFST-N, specimen CFST-MC has the following improvements: The section steel ratio increases by 4.42%, and the yield load and peak load improve by 47.25% and 42.72%, respectively.

(3) The ductility coefficient of specimen CFST-MC was 24.2% higher than that of specimen CFST-N. When the section steel ratio increases, the ductility coefficient increases, and the stiffness degradation slows significantly. Various constructional measures significantly improve the accumulated energy dissipation when the structure reaches peak load.

(4) A finite element model is created to perform a numerical simulation analysis for each specimen. The simulation results and test results match well. It can be seen from the finite element simulation results that the stiffeners can significantly reduce the buckling at the bottom of the column. The bearing capacity of the concrete inside the steel tube is increased greatly due to the constraint effect of the steel tube.

Author Contributions: L.Z. performed the experiments and analyzed the data and wrote the paper. W.C. proposed the topic of this study. H.G., Y.Z., Z.Y., and Y.S. participated in the manuscript preparation.

**Acknowledgments:** This work was supported by National Key Research and Development Plan of China (2017YFC 0703304) and National Natural Science Foundation of China grant funded by the Chinese government. (No. 51878015). The support is greatly appreciated.

**Conflicts of Interest:** The authors declare that there is no conflict of interests regarding the publication of this paper.

#### References

- 1. Ding, J.M.; Wu, H.L.; Zhao, X. Current situation and discussion of structural design for super high-rise buildings above 250 m in China. *J. Build. Struc.* **2014**, *35*, 1–7. (In Chinese)
- 2. Webb, J.; Peyton, J.J. Composite concrete filled steel tube columns. In *Second National Structural Engineering Conference 1990: Preprints of Papers;* Institution of Engineers, Australia: Barton, Australia, 1990; pp. 181–185.
- 3. Young, B.K.; Sung, W.P. Resistance of circular concrete-filled tubular sections to combined axial compression and bending. *Thin-Walled Struct.* **2017**, *111*, 93–102.
- 4. Michel, B.; Julia, M. Seismic design of concrete-filled circular steel bridge piers. J. Bridge Eng. 2004, 9, 24–34.
- 5. Jiho, M.; Dawn, E.L.; Charles, W.R.; Hak-Eun, L. Strength of circular concrete-filled tubes with and without internal reinforcement under combined loading. *J. Struct. Eng.* **2013**, *139*, 04013012.
- 6. Li, X.P.; Lu, X.L.; Guo, S.C. Seismic behavior of CFRT columns under cyclic loading I: experimental study. *Earthq. Eng. Eng. Vib.* **2005**, *25*, 95–103. (In Chinese)
- Ahmed, E.; Atorod, A. Behavior and strength of circular concrete-filled tube columns. J. Constr. Steel Res. 2002, 58, 1567–1591.
- 8. Amir, F.; Frank, S.Q.; Sami, R. Concrete-Filled steel tubes subjected to axial compression and lateral cyclic loads. *J. Struct. Eng.* **2004**, *130*, 631–640.
- 9. Gajalakshmi, P.; Helena, H.J. Behaviour of concrete-filled steel columns subjected to lateral cyclic loading. *J. Constr. Steel Res.* **2012**, 55–63. [CrossRef]

- 10. Konstantinos, A.S.; Kazuhiro, H.; Ryosuke, N.; Hiroyuki, I.; Masayoshi, N. Experimental behavior of concrete-filled steel tube columns using ultrahigh-strength steel. *J. Struct. Eng.* **2016**, *142*, 04016057.
- 11. Duarte, A.P.C.; Silva, B.A.; Silvestre, N.; Brito, J.; Júlio, E.; Castro, J.M. Experimental study on short rubberized concrete-filled steel tubes under cyclic loading. *Compos. Struct.* **2016**, 394–404. [CrossRef]
- 12. Liao, F.Y.; Han, L.H.; Tao, Z.; Rasmussen, K. Experimental behavior of concrete-filled stainless steel tubular columns under cyclic lateral loading. *J. Struct. Eng.* **2017**, *143*, 04016219. [CrossRef]
- 13. Idris, Y.; Ozbakkaloglu, T. Behavior of square fiber reinforced polymer-high-strength concrete-steel double-skin tubular columns uner combined axial compression and reversed-cyclic lateral loading. *Eng. Struct.* **2016**, *118*, 307–319. [CrossRef]
- 14. Abdelkarim, O.I.; ElGawady, M.A.; Gheni, A.; Anumolu, S.; Abdulazeez, M. Seismic performance of innovative hollow-core FRP–concrete–steel bridge columns. *J. Bridge Eng.* **2016**, *22*, 04016120. [CrossRef]
- Abdelkarim, O.I.; ElGawady, M.A.; Anumolu, S.; Gheni, A.; Sanders, G.E. Behavior of Hollow-Core FRP-Concrete-Steel Columns under Static Cyclic Flexural Loading. *J. Struct. Eng.* 2017, 144, 04017188. [CrossRef]
- Teng, J.G.; Yu, T.; Wong, Y.L.; Dong, S.L. Hybrid FRP concrete-steel tubular columns: Concept and behavior. Constr. Build. Mater. 2007, 21, 846–854. [CrossRef]
- Teng, J.G.; Yu, T.; Wong, Y.L. Behavior of hybrid FRP concrete-steel double-skin tubular columns. In Proceedings of the 2nd International Conference on FRP Composites in Civil Engineering, Adelaide, Australia, 8–10 December 2004; Taylor & Francis: Abingdon, UK, 2004; pp. 811–818.
- 18. Yang, Y.; Wang, Y.; Fu, F. Effect of reinforcement stiffeners on square concrete-filled steel tubular columns subjected to axial compressive load. *Thin-Walled Struct.* **2014**, *82*, 132–144. [CrossRef]
- 19. Yang, Y.; Yang, H.; Zhang, S. Compressive behavior of T-shaped concrete filled steel tubular columns. *Int. J. Steel Struct.* **2010**, *10*, 419–430. [CrossRef]
- 20. Ye, L.P. Design of Concrete Structures; China Architecture & Building Press: Beijing, China, 2012. (In Chinese)
- 21. China Standards Publication. *Technical Code for Concrete Filled Steel Tubular Structures*; GB50936-2014; China Architecture&Buildling Press: Beijing, China, 2014. (In Chinese)
- 22. Qian, J.R.; Li, N.B.; Ji, X.D.; Zhao, Z.Z. Experimental study on the seismic behavior of high strength concrete filled double-tube columns. *Earthq. Eng. Eng. Vib.* **2014**, *13*, 47–57. [CrossRef]
- 23. Park, R. Evaluation of ductility of structures and structural assemblages from laboratory testing. *Bull. N. Z. Natl. Soc. Earthq. Eng.* **1989**, *22*, 155–166.
- 24. Han, L.H. Steel Tube Concrete Structure: Theory and Practice; Science Press: Beijing, China, 2007. (In Chinese)
- 25. Hibbitt, Karlsson & Sorensen. *ABAQUS/Standard User's Manual, Version 6.4*; Hibbitt, Karlsson & Sorensen Inc.: Rawtucket, RI, USA, 2004.
- 26. Shi, Y.P. Detailed Annotation of Abaqus Finite Element Analysis; Mechanical Industry Press: Beijing, China, 2005. (In Chinese)



© 2018 by the authors. Licensee MDPI, Basel, Switzerland. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC BY) license (http://creativecommons.org/licenses/by/4.0/).