

Article

Experimental Studies on Seismic Performance of UHPSFRC-Filled Square Steel Tubular Columns

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Abstract: The excellent seismic performance of concrete-filled steel tube (CFST) structures has been widely recognized, but there is a paucity of research on composite columns using UHPC with added steel fibers. This paper presents the experimental studies and numerical analyses with OpenSees on seismic performance of ultra-high performance steel fiber-reinforced concrete (UHPSFRC)-filled square steel tubular columns. Five half-scaled specimens of UHPSFRC-filled square steel tubular columns were tested under a combination of constant axial compression and cyclic horizontal load, with parameters of width-to-thickness ratio (28.5, 19.9 and 14.7) and axial compression ratio (0.133, 0.266 and 0.399) of the steel tube. With the decrease in width-to-thickness ratio, the maximum bending moment capacity increased by 33.5% and 15.3%, and the energy dissipation capacity and ductility increased, while the strength degradation and stiffness degradation reduced. With the increased axial compression ratio, the loading capacity increased from 55.3 to 70.2 kNm (26.94%). The results indicate that UHPSFRC-filled square steel tubular columns improve seismic performance by decreasing the axial compression ratio and by increasing the width-to-thickness ratio. When the width-to-thickness ratio was reduced, the steel tubular was able to provide higher lateral restraint to the internal UHPC; thus, seismic performance was improved. With the increase in the axial compression ratio, the second-order damage effect of the members was greatly affected, and it accelerated the plastic damage. A modified UHPSFRC model considering steel tubular constraints was adopted, and the nonlinear dynamic modeling of the column response using OpenSees led to good agreement with the tested response of the column under cyclic motion. The theoretical calculation model can better predict the bending capacity of the UHPSFRC-filled square steel tubular columns. However, the calculation formulas of initial stiffness and yield bending moment need further research.

Keywords: square steel tubular; ultra-high performance steel fiber reinforced concrete; seismic behavior; pseudo static; finite element



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1. Introduction

The UHPSFRC-filled square steel tubular columns combine steel tubular and UHPSFRC, resulting in a member that has beneficial qualities of both materials, which involve high tensile strength and ductility of steel members and high compressive strength and stiffness of concrete members. In ultra-high performance concrete filled steel tubular (UH-PCFST) members, the continuous restraint provided to the UHPSFRC core by the steel tubular enhances the UHPC's strength and ductility, and prevents the concrete from peeling, while the UHPC core delays local buckling of the steel tubular by preventing inward buckling; thus, it is expected to be widely used for building and bridge structures.

An effective measure to improve the ductility of concrete-filled square steel tubular composite columns is to add additional restraints in the concrete, such as steel fiber, spiral reinforcement or tie bars. Adding steel fiber into ultra-high performance concrete (UHPC) is an effective way to improve its mechanical performance such as ductility and tensile strength [1,2]. This is because the concrete crack pattern changes from crushed to the expansion of oblique shear cracks, and eventually leads to a significant increase in compressive strength and strain [3]. Experimental studies on the influence of steel fiber on improving mechanical performance of concrete revealed that the volume ratio of steel fiber should be larger than 1% [4]. As a new type of cementitious composite, ultra-high performance concrete (UHPC) with steel fiber exhibits excellent mechanical properties [5], which involve ultra-high compressive strength and higher tensile strength compared to high strength concrete (HSC) and plain concrete. Adding internal spiral reinforcement is an effective way to improve the cycling performance of the concrete-filled square steel tubular columns effectively [6]. Yan et al. [7] investigated the seismic performance of ultra-high strength reinforced concrete-filled high strength square steel pipe columns under a high axial pressure ratio. Experience shows that the local buckling of CFST columns in the plastic hinge zone have a great impact on their seismic performance [8]; thus, some experimental studies on strengthening the column foot with stirrup reinforcement [9], stiffener [10], and connectors [11] have also been carried out.

The seismic performance of UHPC-filled rectangular steel tubular columns has been tested by Cai et al. [12] with parameters of steel pipe thickness, compressive ratio, and width-to-thickness ratio, and the finite element models were established to simulate the performance of the specimens. Meanwhile, a test of concrete-filled high strength steel (HSS) tubular columns under cyclic loading was conducted by Wang et al. [13]. Wei et al. [14] experimentally investigated the recovery performance of UHPC-filled circular steel tubular composite columns under low circumferential cyclic reciprocating loads applied in the span. In addition, the seismic performance of concrete-filled other constraint materials was also studied and analyzed, such as corrugated steel tubular [15] and fiber-reinforced polymer (FRP) [16]. At present, CFST columns are mostly focused on regarding the study of axial pressure performance, and there is less research on its seismic performance. For different constraint materials, the axial pressure properties of the combination column are also different. Concrete-filled high strength steel (HSS) [17], stainless steel [7] or FRP [18] composite columns were tested on their axial pressure performance and different kinds of fiber [3,19], which had an important influence on the axial compression performance of the combined column. Chen et al. [20] compared the effects of combined columns of plain concrete and UHPC-filled steel tubes on their axial compression properties by tests. In addition, for CFST columns, temperature was one of the parameters of the study. Wang et al. [21] conducted a study on the post-earthquake fire resistance of square steel pipe concrete columns, and the test procedures included quasi-static tests and fire tests. Li et al. [22] carried out a finite element analysis of the bearing performance of the steel pipe concrete composite frame under the action of thermodynamic coupling, as well as the study of the compressive performance at low temperatures [23], which was suitable for engineering construction in cold areas.

Finite element simulation of CFST columns has also been greatly developed. Jin et al. [24] established a three-dimensional simulation method, in which the concrete inhomogeneity and the contact between concrete cores and steel pipes were considered, to study the structural behavior and the size effects of CFST columns under combined action of transverse shear and axial loads. Vinoth et al. [25] conducted a nonlinear analysis of square steel tube concrete columns and circular steel tube concrete columns with normal weight concrete, recycled aggregate concrete [26], and high-performance concrete. At present, finite element simulation studies have focused on monotonic loads [27–29] or reinforced concrete composite columns [30], but limited research on simulations on seismic behavior of UHPC-filled square steel tubular composite columns has been reported.

In summary, most of the current research focuses on the strengthening of the plastic hinge zone or the seismic performance study of normal concrete-filled steel tubular composite columns, and less on the seismic performance study of UHPSFRC-filled steel tubular columns. In this paper, five UHPSFRC-filled square steel tubular columns, in which the upper ends were free and bottom ends were fixed, were tested under combination of axial constant compressive force and cyclic horizontal loading to verify their seismic performance. In addition, finite element models using OpenSees were established for confined concrete to simulate the quasi-static behavior of the columns, and the results of finite element were in good agreement with the test, which verified the correctness of the modeling. At the same time, the established bearing capacity calculation equation can also predict the bending capacity of the UHPSFRC-filled square steel tubular column well.

2. Experiment of UHPSFRC-Filled Square Steel Tubular Columns

2.1. Test Program

UHPSFRC has more excellent mechanical properties than plain concrete, but it has not yet been widely used. The study on seismic performance of UHPSFRC-filled square steel tubular columns can check whether the formula in the specification is still suitable for UHPSFRC. The size of the specimens was designed according to Chinese codes, such as GB50936-2014 [31], CECS 159: 2004 [32] and CECS28:90 [33]. These Chinese codes state that for concrete-filled square steel tubular members, the edge length of the cross section should not be less than 168 mm, and the thickness of steel tubular should not be less than 4 mm. The coefficient of the confinement effect (θ) should be 0.3–3.0, and the length-to-width ratio should be lower than 20. The length and section width of the full scale specimens were 4000 and 240 mm. The section width of the column was relatively small because of the high compression capacity of the UHPSFRC. The thicknesses of the steel tubular were 8, 12 and 16 mm, and they all met the requirements of the specifications.

In order to improve the ductility of the CFST columns, steel fibers were blended in UHPC. When the fiber content increases from 0.5% to 3.5%, the compression strength, tensile strength and modulus of elasticity will increase significantly [34,35], and when the fiber volume ratio reaches 3%, UHPC can achieve satisfactory mechanical properties. In terms of construction, when the volume ratio of the steel fiber is greater than 3%, UHPC is difficult to mix, resulting in uneven internal distribution and poor workability behavior. Therefore, it was decided to mix 3% volume of steel fibers in UHPC.

In conclusion, to study the seismic performance of UHPSFRC-filled square steel tubular columns, five 1/2-scaled specimens had been tested under the combination of constant axial compression and cyclic horizontal loading, with the test parameters of axial compression ratio and the thickness-to-width ratio of the steel tube.

2.2. Details of Specimens

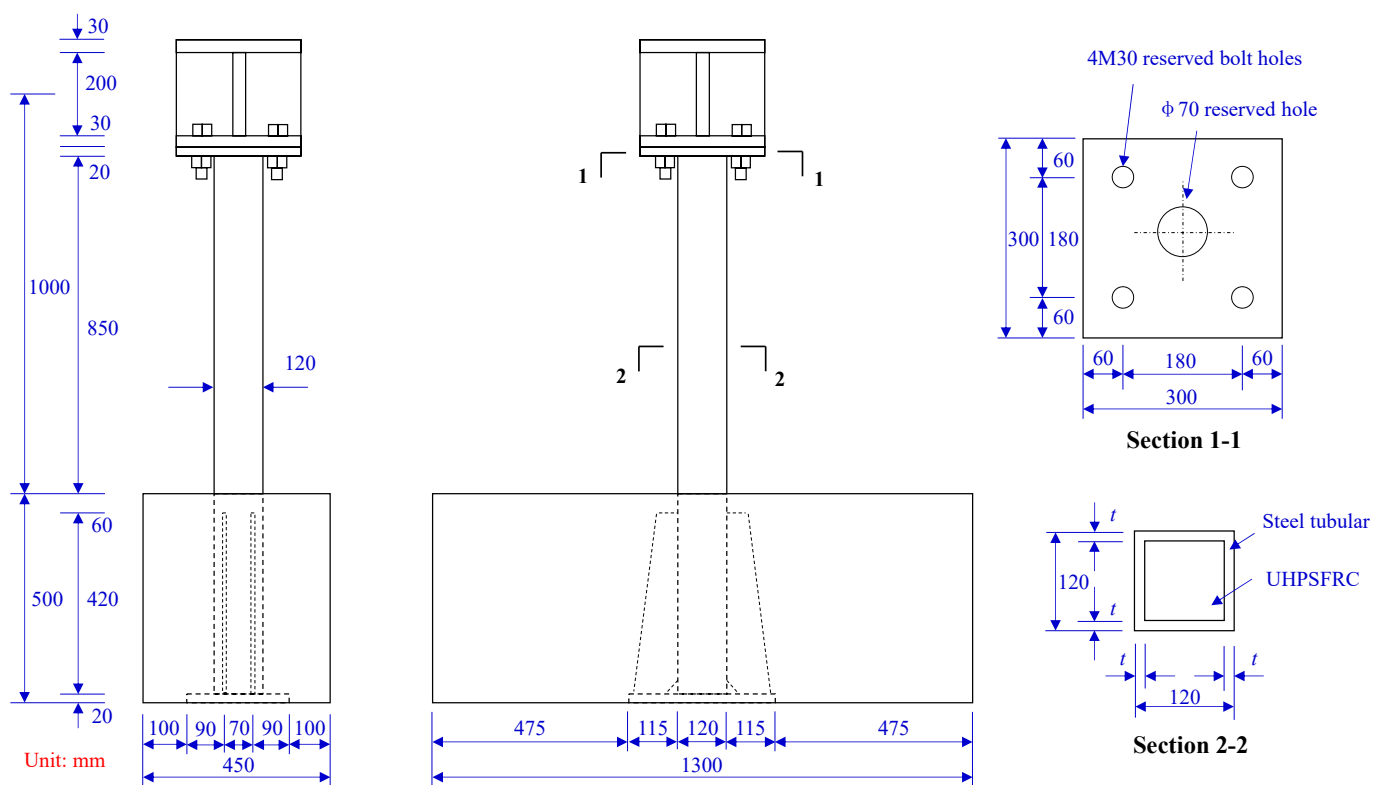
The types of materials selected and the size of the components according to the Chinese codes are shown in Table 1. The material properties parameters of Q235B and UHPC were obtained by material property testing. The material properties such as compressive strength (f_c) of 131.2 MPa, ultimate compression strength (f_{cu}) of 138.1 MPa, tensile strength (f_t) of 8.0 MPa, and elastic modulus (E_c) of 42.6 GPa were obtained by coupon tests. Design parameters and associated material properties are summarized in Table 1. Among the five specimens, specimen S1 was the control specimen, and the thickness of the steel tubular varied from 4 to 8 mm from S1 to S3, while the axial compression ratio ranged from 0.133 to 0.399, as shown in Table 1.

The dimensional details of the test specimens are given in Figure 1. The total length of the square steel tubular was 1330 mm, while the squared section was 120×120 mm with a thickness of 6 mm in the control specimen. An upper plate with an opening hole (70 mm in diameter) to fill the UHPSFRC was welded to the top face of the steel tube in advance. A stiffened I-beam was bolted to the upper plate and served as a connection. In the bottom of the composite column, there were two stiffening plates in each side.

Table 1. Design parameters and material properties.

Specimen	f_y (MPa)	f_u (MPa)	E_s (GPa)	T (mm)	t_s (mm)	A (%)	ζ	n	N (kN)
S1	393.4	549.5	206	6	6.04	23.5	0.71	0.266	633.6
S2	389.8	539.1	206	4	4.21	14.8	0.47	0.266	571.5
S3	391.1	546.7	206	8	8.14	33.1	1.01	0.266	698
S4	393.4	549.5	206	6	6.04	23.5	0.71	0.133	316.8
S5	393.4	549.5	206	6	6.04	23.5	0.71	0.399	950.5

Notes: f_c, f_{cu}, f_t , and E_c denote compressive strength, ultimate strength, tensile strength, and elastic modulus of UHPC, respectively; D is width of the steel tubular section; t and t_s are nominal thickness and actual thickness of the steel tubular, respectively; f_y, f_u , and E_s denote yield strength, ultimate strength, and elastic modulus of the steel tubular, respectively; α, ζ, n , and N are steel content, hoop coefficient, axial compression ratio and axial load, respectively.

**Figure 1.** Details and dimensions of specimens.

Fabrication procedures of test specimens are illustrated in Figure 2. The fabrication includes (1) Step I: Install the upper plate with reserved holes and bottom plate of the square steel tubular, and then, weld four stiffening plates. (2) Step II: Cast the foundation concrete of C40 with length, width and height equal to 1300, 450, and 500 mm, respectively, and then, cast the UHPFSRC to the square steel tubular through a reserved hole with 75 mm in diameter on the upper plate. (3) Step III: Bolt the stiffened I-beam to the upper plate.

2.3. Test Setup and Measurements

The test setup and arrangement of the instrumentation are shown in Figure 3. The upper end of the test specimen was hinged on the horizontal actuator, and the lower end was fixed to the ground by pressure beams and bolts. The 200-ton vertical actuator applies a constant axial load on the loading I-beam, while the 30-ton horizontal actuator applies a horizontal displacement with a maximum stroke of 80 cm. The distance between the center of the horizontal push-pull actuator and the top surface of the rectangular reinforced

concrete (RC) foundation was 1000 mm, which was considered as an effective height of the test specimens.

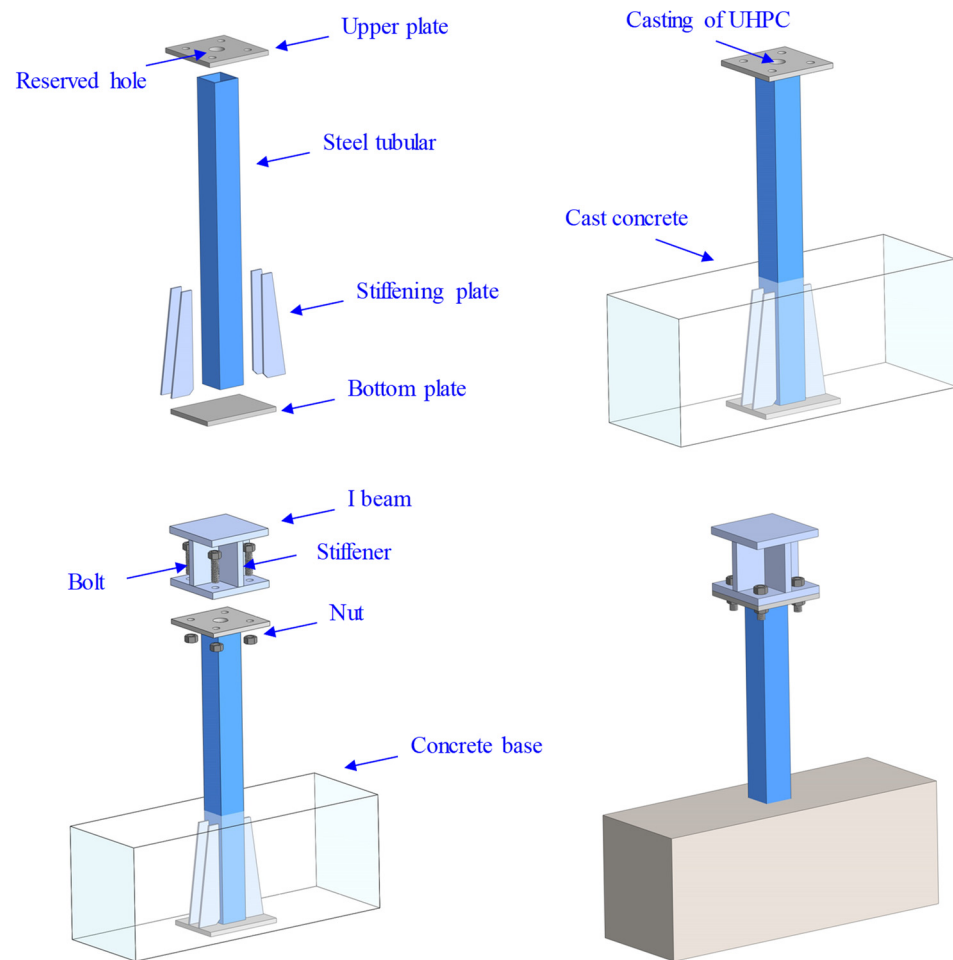


Figure 2. Fabrication procedures of UHPSFRC-filled square steel tubular columns.

The applied horizontal load was measured by the load cell attached to the horizontal actuator. In total, five LVDT were installed to measure the deformation of the column. LVDT D1 was installed to measure the horizontal displacement at the top of the column, while D2 and D3 were used to check the horizontal movement of the RC foundation. LVDT D4 and D5 were installed to measure the vertical displacement of both ends of the RC foundation to check if rotation of UHPSFRC-filled square steel tubular columns occurs during the test.

Linear strain gauges were installed to measure the strain at the bottom part of the steel tube, the arrangement of strain gauges are shown in Figure 3. The local diagram of Section A shows the three layers of longitudinal strain gauges arranged along the cross section within the region of height of the expected plastic hinge. As shown in Figure 3, the strain gauges were mounted at different sections (section 3-3, 4-4, and 5-5) to measure the distribution of strain along the column section height, in order to verify the assumption of plain section remains plain.

Figure 4 shows the actual test setup of the cyclic loading tests on the UHPSFRC-filled square steel tubular columns. The RC foundation was anchored to the foundation beam through pressure beams and anchor bolts to ensure that the bottom end of the column was in a fixed restraint, and the loading I-beam was hinged with the end of the horizontal actuator, while the rolling support was set between the reaction frame and vertical actuator to ensure the equal horizontal displacement between vertical actuator and column end.

Therefore, the axial pressure was uniformly transmitted to the loading end and UHPSFRC-filled square steel tubular columns.

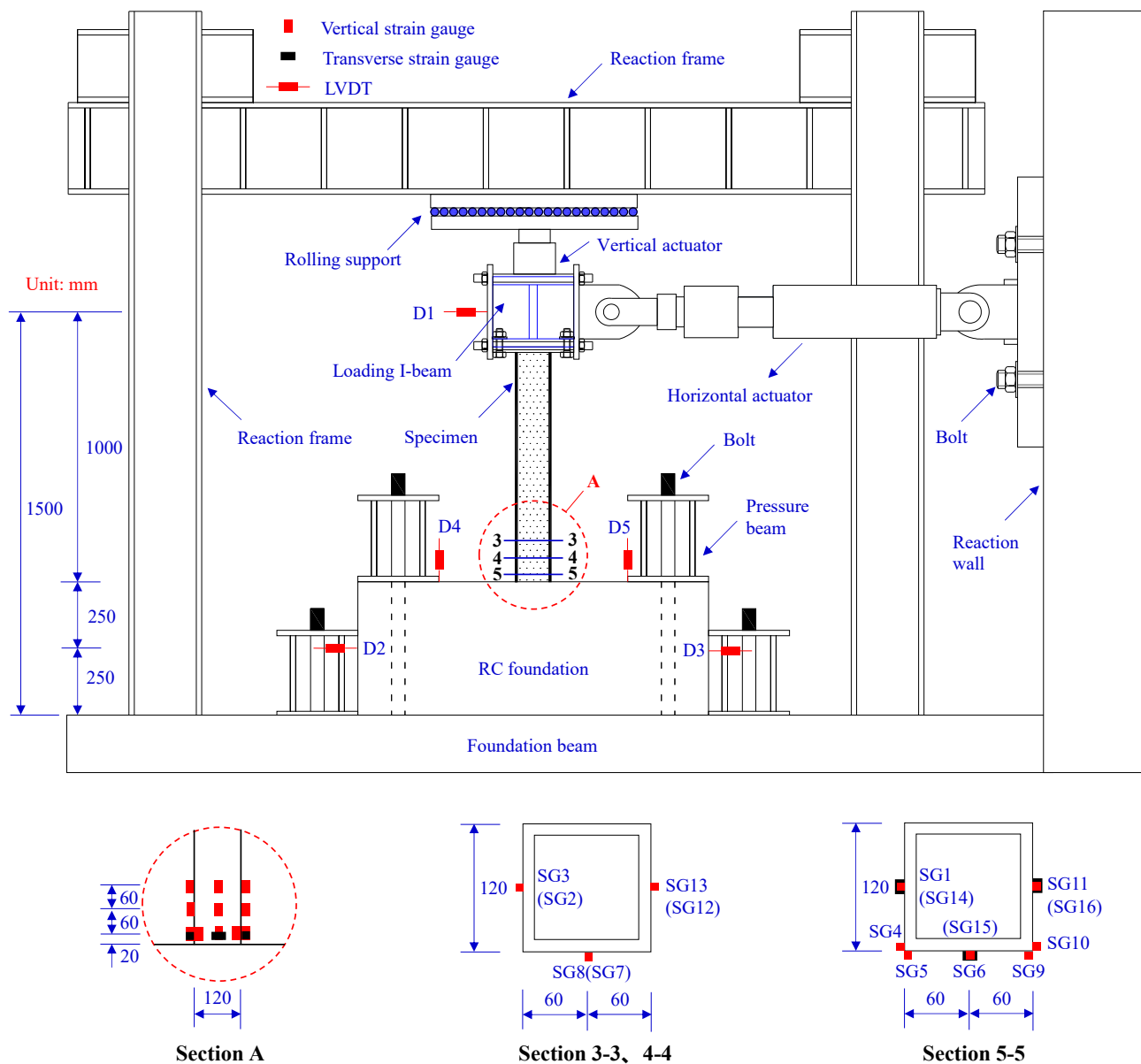


Figure 3. Test setup and instrumentation plan.

In order to eliminate the internal eccentricity of the specimen, the axial compression was first applied to 0.4~0.5 times of N and then uniformly unloaded to 0 before the target axial compression was applied. Then, the axial load was applied to the predetermined load two or three times and remained constant during the test.

The loading protocol with displacement-control is shown in Figure 5. Before the story drift ratio of 1%, only one cycle of full-reversed loading was applied, while two repeated was applied when the story drift ratio exceeded 1%. The test would be terminated until the specimens failed or the moment capacity reduced to 85% of the peak load. Horizontal thrust was defined as a positive loading in this test.

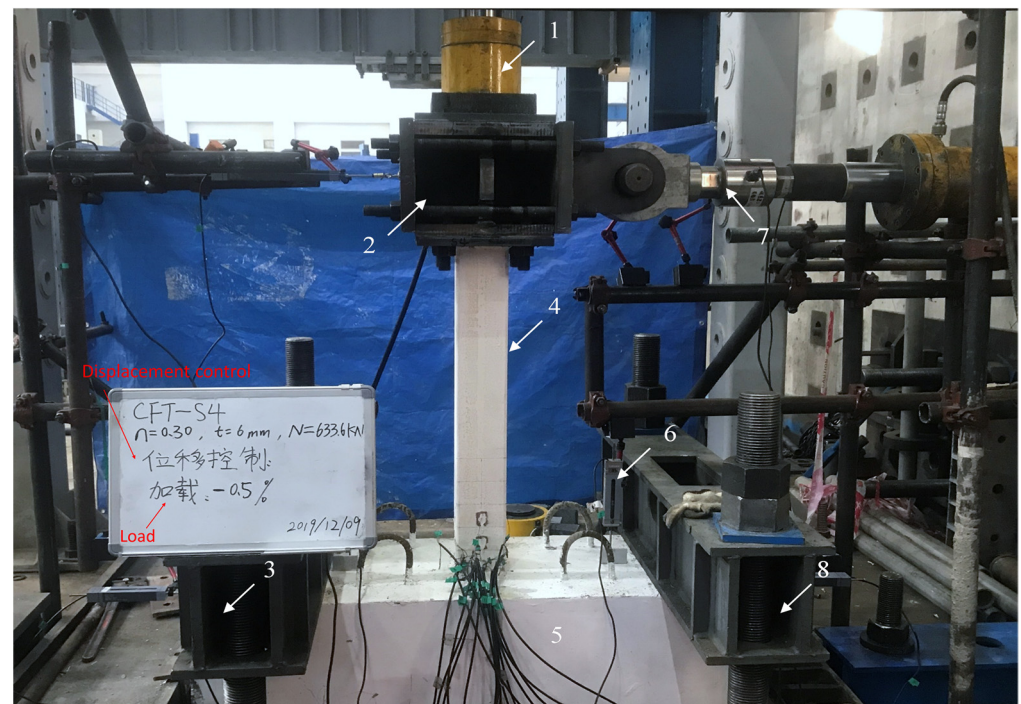


Figure 4. Setup of cyclic loading tests: (1) vertical actuator; (2) loading I-beam; (3) pressure beam; (4) specimen; (5) RC foundation; (6) LVDT; (7) horizontal actuator; (8) pressure beam.

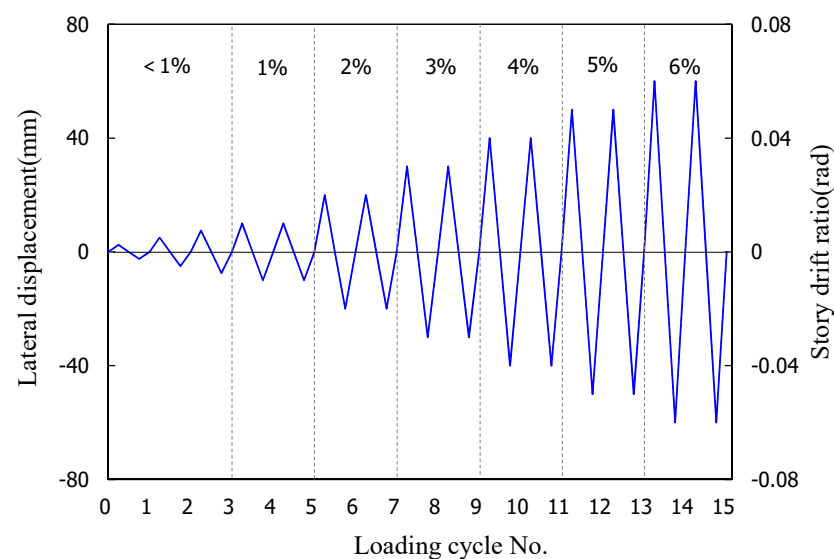


Figure 5. Horizontal loading acting on the column top.

3. Test Results and Discussion

3.1. Failure Modes

The failure modes of the UHPSFRC-filled square steel tubular columns after the test are shown in Figure 6. Significant local buckling of the steel tube at the lower part of the column (Figure 6a) was observed during the test, and concrete crushing (Figure 6b) at the root of the specimens was found after removing the part of the steel tube by flaming cut. At the beginning of the test, the surface of the square steel tubes had no obvious change in the elastic stage, and the stiffness was linear and stably increased. With the increase in horizontal displacement, the hoop and longitudinal strain of the steel tubes began to increase non-linearly, indicating that the steel tubes had entered the elastic-plastic

stage. However, due to the synergistic effect of the square steel tubular and UHPSFRC, the stiffness of the specimen had not yet decreased significantly during the followed loading.

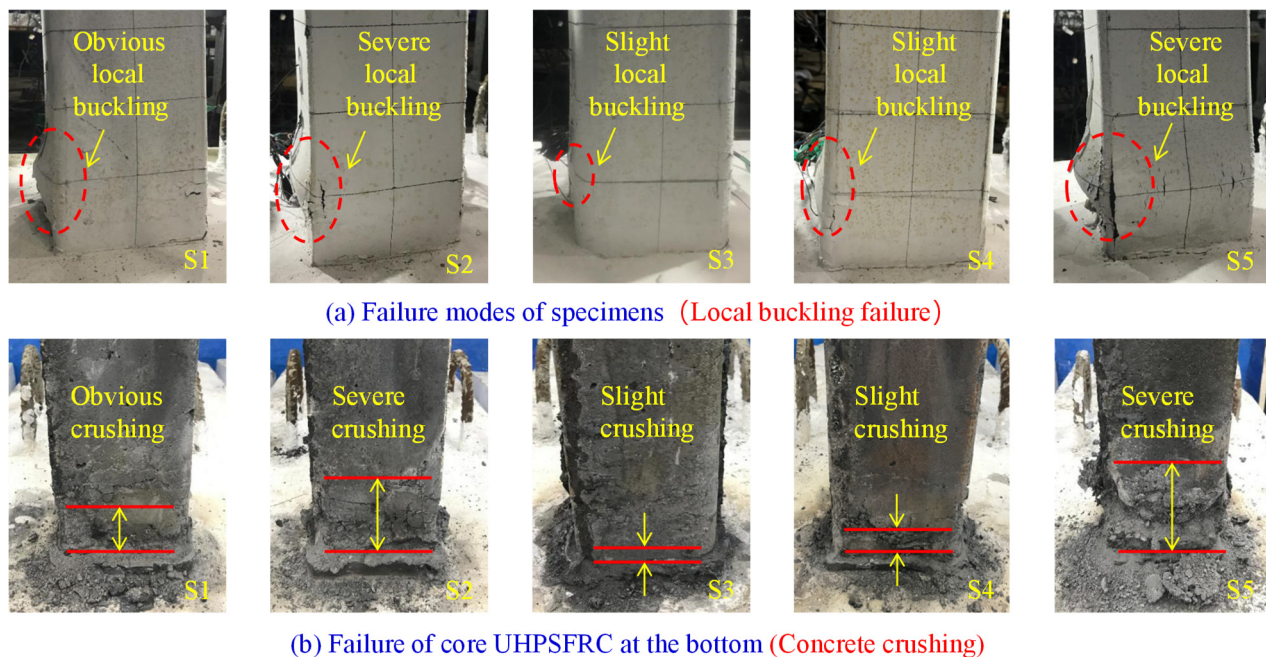


Figure 6. Failure modes of UHPSFRC-filled square steel tubular columns.

With the further increase in loading displacement, the interaction between the outer steel tubes and inner UHPSFRC further increased, while the concrete cracks continued to expand and extend, and the stiffness began to decrease continuously. Specimen S5 reached the peak load when the displacement angle was about 2%, S1~S3 was about 3%, and S4 was about 4%. When the peak load was reached, there was a loud “splitting” sound in the steel tubes, indicating that the steel fiber in the bottom column was broken and that the UHPSFRC was crushed locally.

When specimens S1–S5 were loaded to the story drift ratio of 4%, 3%, 6%, 6% and 2%, respectively, the steel tubes on both sides of the bottom began to show slight local buckling. This was because the internal concrete provides sufficient support force to prevent internal local buckling of the steel tubular but does not have sufficient bonding capacity to prevent its local outward buckling. When the specimens S1, S2 and S5 were loaded to the story drift ratio of 6%, 4% and 3%, respectively, the steel tubes began to show obvious local buckling. When the specimens S2 and S5 were loaded to the story drift ratio of 6% and 4%, the steel tubes began to show serious local buckling. The white coating on the outer surface of the steel tube (especially the corner) was seriously wrinkled because the bottom concrete was crushed and could not provide sufficient support for the steel tubular. The bottom of the column was subjected to the largest bending moment; thus, it was also more likely to be damaged. The core UHPSFRC at the bottom of specimens S3 and S4 showed slight crushing, while specimen S1 showed obvious crushing, but specimen S5 had serious crushing.

3.2. Hysteretic Behavior

Figure 7 shows hysteretic curves of bending moment at the bottom section versus a story drift ratio relationship for the tested specimens, which shows that: (1) The hysteresis curves of each specimen were full, without obvious pinching, and exhibit good energy dissipation capacity, except that specimen S2 with the largest width-to-thickness ratio showed a small degree of pinch. (2) By comparing Figure 7a–c, it can be seen that the hysteresis curve of the specimen becomes fuller with the decrease in width-to-thickness ratio, which indicates that a specimen with a small width-to-thickness ratio can dissipate more seismic energy. After the peak load, the hysteresis curve of the specimen with a small

width-to-thickness ratio decreases more smoothly because of the small width-to-thickness ratio, which means that the constraint effect of the steel tubular on concrete improved with the increase in steel content ratio. (3) By comparing Figure 7a,d,e, it can be seen that with the increase in axial compression ratio, the hysteresis curve of the specimen was fuller, which indicates that a specimen with a larger axial compression ratio can dissipate more seismic energy. However, the bending moment displacement angle hysteresis curve of the specimen with a large axial compression ratio decreases more steeply after the peak bending moment. This was because the second-order damage effect of the members with a large axial compression ratio was greatly affected, and the further development of elastic–plastic damage after the yield of specimens was accelerated.

With the decrease in width-to-thickness ratio (14.8%, 23.5%, 33.1% for S2, S1, S3, respectively, corresponding to the thickness of the steel tubular of 4 mm, 6 mm and 8 mm), the mean values of the yield moment were 43.9, 59.2, and 65.3 kNm and the mean values of peak moment were 49.0, 65.4, 75.4 kNm, respectively. The yield moment increased by 34.8% and 10.3%, and likewise, the peak moment increased by 33.5% and 15.3%, respectively. Specimens S4, S1 and S5 were compared regarding the effect of the axial compression ratio (0.133, 0.266 and 0.399, respectively). The mean values of yield moment were 61.0, 59.2, and 51.25 kNm, decreasing by 3.0% and 13.4% with the increase in axial compression ratio. The mean values of peak moment for S4, S1 and S5 were 70.2, 65.4 and 55.3 kNm, respectively. When the axial compression ratio increased from 0.133 to 0.266, the peak moment decreased by 6.8%, and it reduced by 5.2% as the axial compression ratio continued to increase to 0.399.

The asymmetric behavior of the cyclic loading test curves was due to: (1) the initial imperfection of the specimens; (2) the initial deformation generated during the specimen setting; (3) the deviation and friction between the loading device and the specimen.

3.3. Skeleton Curves

Figure 8a shows the effects of the width-to-thickness ratio on the skeleton curves of the UHPSFRC-filled square steel tubular columns after the cyclic loading tests. The smaller the width-thickness ratio (larger thickness of steel tube), the greater the moment capacity of the specimen, and the slower the moment capacity decreases after passing the peak value. The specimens with larger width-thickness ratio developed greater peak bending moment, while the displacement corresponding to the peak bending moment was almost the same. In addition, three specimens with different width-to-thickness ratios performed well in lateral deformation and could reach a 6% displacement angle. This was mainly because the constraint effect coefficient of specimens with a small width-to-thickness ratio (large steel content ratio) was large, and the external steel tubular had a better constraint effect on the core UHPSFRC; thus, the fragility of UHPSFRC was improved.

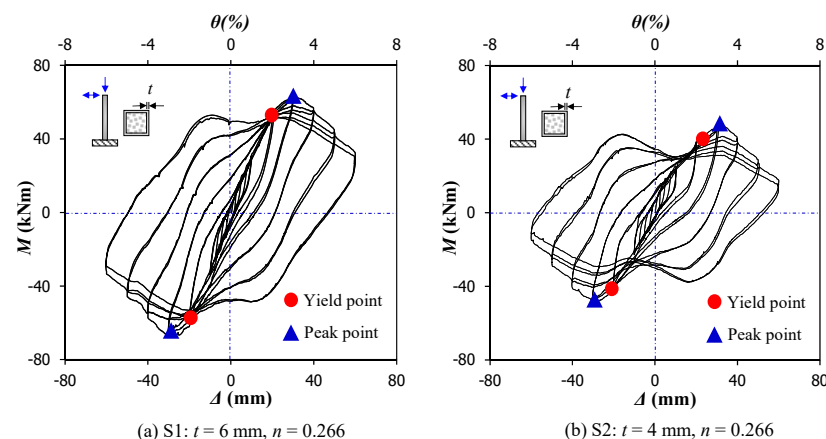


Figure 7. Cont.

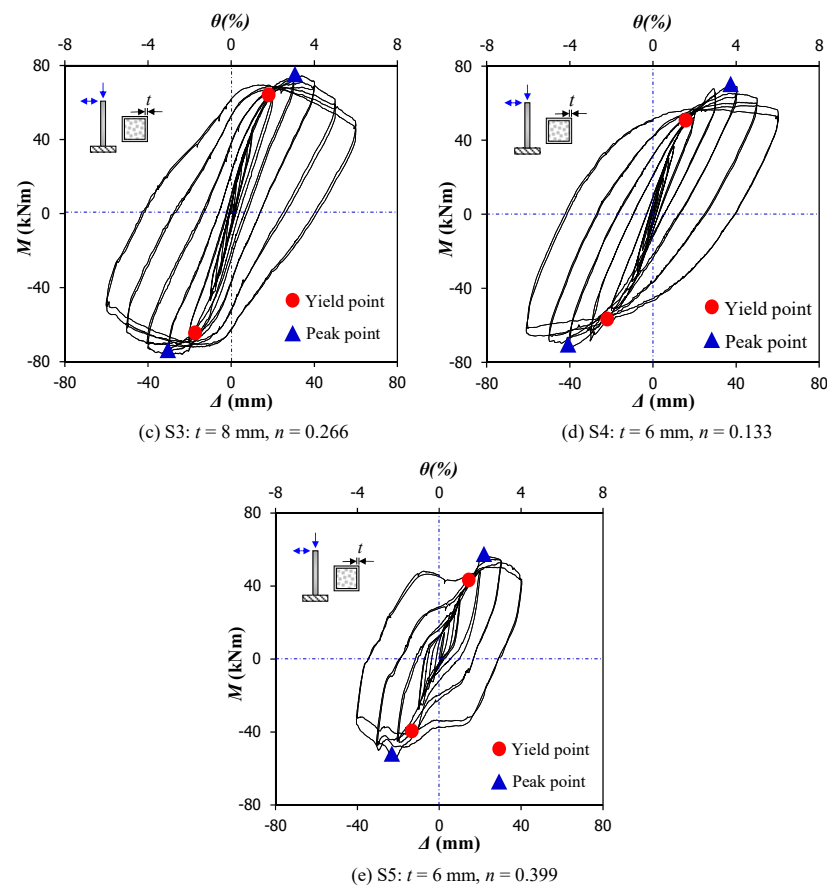


Figure 7. Hysteretic curves of specimens S1–S5.

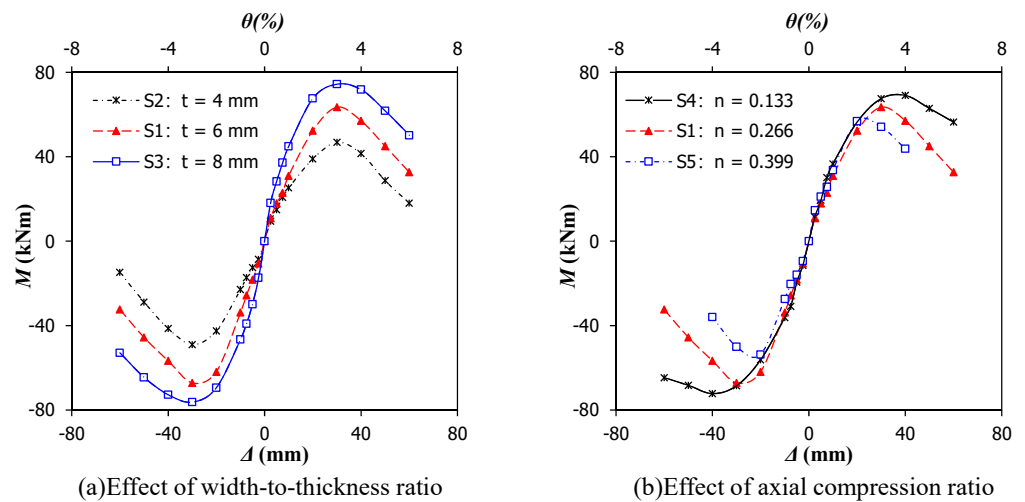


Figure 8. Comparison of skeleton curves.

The influence of the axial compression ratio on the skeleton curve is shown in Figure 8b, in which it can be seen that the larger the axial compression ratio, the smaller the moment capacity of the column and the faster the moment capacity decreases after passing the peak value. The main reason for this phenomenon was that the second-order damage effect of the specimens with large axial compression ratios had a great influence, which accelerated the development of the elastic–plastic and cumulative damage after the yield point. In addition, with the increase in axial compression ratio, the lateral displacement on the peak point of three specimens reduced gradually, and their ultimate lateral displacement had

also been reduced, which indicated that the increase in the axial compression ratio reduces the lateral deformation capacity of the composite columns.

3.4. Energy Dissipation Capacity

The capacity of energy dissipation is one of the most important indexes to evaluate the seismic performance of structures or structural components, and the equivalent viscous damping coefficient is most commonly used to represent energy dissipation capacity. Figure 9 shows the effect of the width-to-thickness ratio and axial compression ratio on the cumulative dissipated energy.

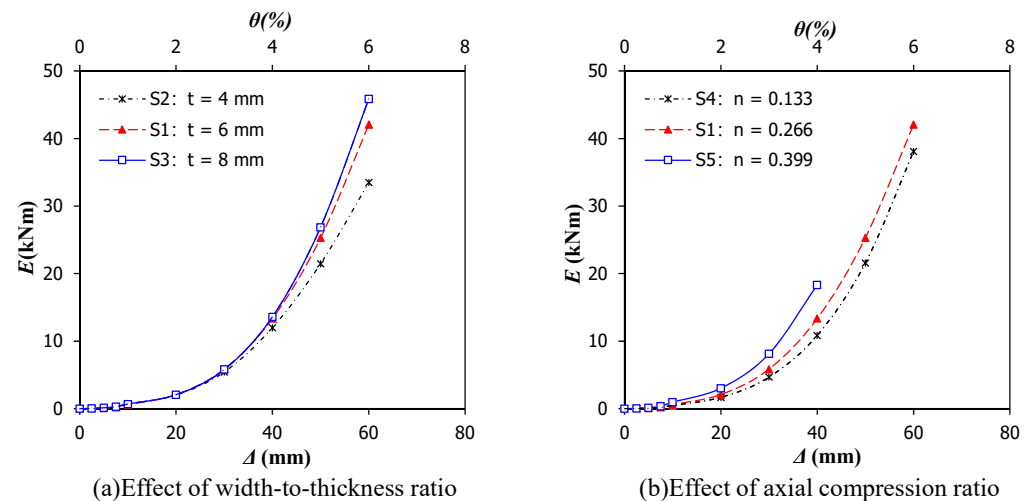


Figure 9. Comparison of cumulative dissipated energy.

Figure 9a,b shows the comparison curves of the cumulative dissipated energy for the effects of the width-to-thickness ratio and axial compression ratio, respectively. It can be seen in Figure 9a that the capacity of energy dissipation improved gradually and grew faster and faster with the decrease in the width-to-thickness ratio, which varied from 28.5 to 19.9, and then to 14.7. The final cumulative dissipated energy of specimen S1 and S3 increased by 25.6% and 36.9%, respectively, compared with specimen S2, which indicates that with the decrease in the width-to-thickness ratio (the increase in thickness of the steel tubular), the final cumulative energy dissipation capacity of the UHPSFRC-filled square steel tubular columns increases significantly, and the energy dissipation rate increases significantly.

Only the cumulative dissipated energy of specimens with an axial compression ratio at the first 4% displacement angle was compared in Figure 9b because the test for specimen S5 had been terminated due to excessive deformation when its displacement angle reached 4%. The specimen with a bigger axial compression ratio (such as S5) had superior energy dissipation capability but worse lateral deformation capability. When the lateral displacement reached a 4% displacement angle (40 mm), it could not continue to sustain the lateral load. The ultimate cumulative dissipated energy increased with the increase in the axial compression ratio. Compared with specimen S4, the final cumulative dissipated energy of specimens S1 and S5 increased by 23.1% and 68.8%, respectively, which shows that under the same loading displacement angle, increasing the axial compression ratio can not only improve the energy dissipation rate, but can also improve the final cumulative energy dissipation capacity of the HPSFRC-filled square steel tubular columns.

Figure 10 plots the dissipated energy at each single cycle for each specimen, and the effects of the width-to-thickness ratio and axial compression ratio were compared. No energy dissipation occurred before the first five loading circles until the drift ratio reached 2%, and after yielding occurred at the bottom of the columns, all specimens showed good energy dissipation, which means that they had entered the plastic working stage. Every specimen could dissipate more energy with the increase in loading cycle, especially at cycle 14 and 15. However, they needed to be unloaded because of the excessive lateral

displacement. As shown in Figure 10a,b respectively, reducing the width-to-thickness ratio (increase in steel tubular thickness) or increasing the axial compression ratio, was found lead to greater single-cycle energy dissipation capacity of specimens.

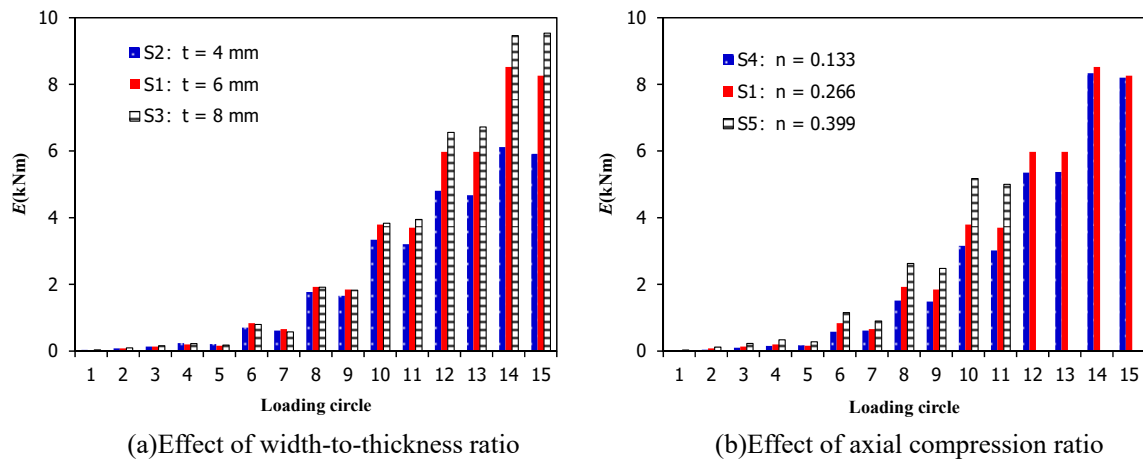


Figure 10. Comparison of single-cycle energy dissipation capacity.

Figure 11 shows variation curves of equivalent viscous damping coefficients (ζ_e) of UHPFSRC-filled square steel tubular columns under different amplitudes of the story drift ratio. It can be seen that ζ_e of each specimen decreased slightly at the beginning of loading and then increased with the increase in loading displacement angle, and the rate of improvement gradually accelerated. With the increase in the width-to-thickness ratio and axial compression ratio, the equivalent viscous damping coefficients were also larger. In addition, ζ_e of possessive specimens at the time of destruction were 0.28, 0.33, 0.32, 0.35, 0.28, while ζ_e of the normal reinforced concrete (RC) column was 0.1~0.2, which was a better indication that UHPFSRC-filled square steel columns have better energy dissipation capacity than ordinary RC columns.

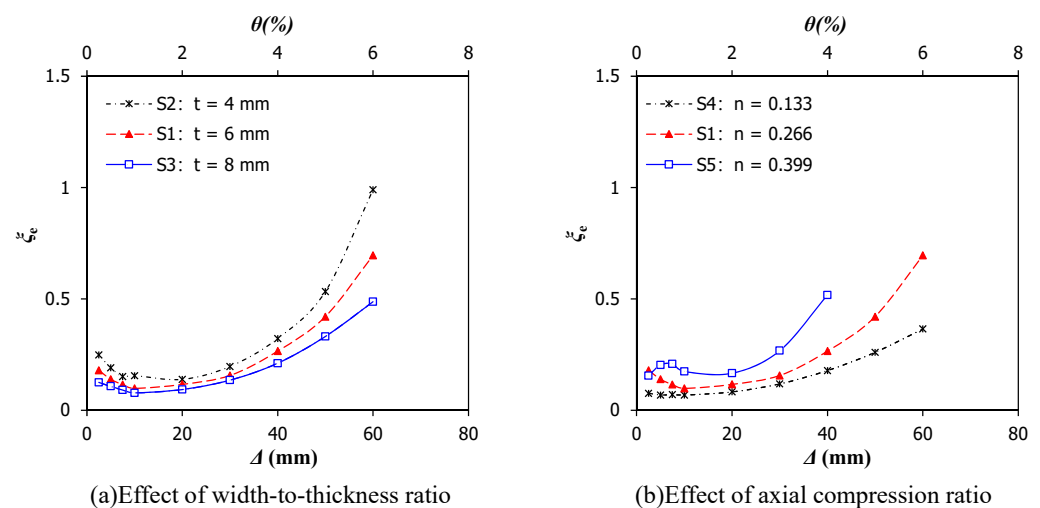


Figure 11. Comparison of equivalent viscous damping coefficients.

3.5. Strength Degradation

The strength degradation coefficient (λ_{ji}) was used to describe the strength degradation of specimens, and λ_{ji} means the strength degradation value of the i -stage cycle under j -stage loading. The effects of different factors on strength degradation are shown in Figure 12.

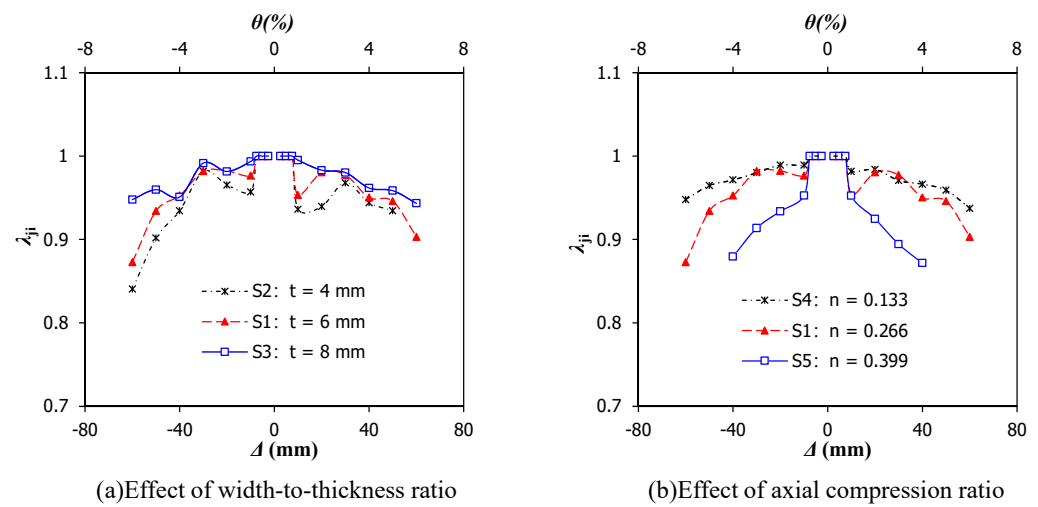


Figure 12. Comparison of strength degradation.

The strength degradation coefficients of specimens S2, S1 and S3 decrease in turn, indicating that the smaller the width-to-thickness ratio (the bigger the thickness of the steel tubular), the smaller the λ_{ji} , as shown in Figure 12a, and the more stable the mechanical performance. With the decrease in the axial compression ratio of specimens S5, S1 and S4, the same strength degradation performance was also shown: the strength degradation coefficient decreased in turn, and the strength degradation was smaller. It can be seen from Figure 12b that the axial compression ratio of specimen S5 was the largest; thus, strength degradation was the most obvious.

3.6. Rigidity Degradation

The secant stiffness (K) was used to describe the stiffness degradation of the specimens, as shown in Figure 13. The degree of stiffness degradation becomes more serious as the loading displacement angle increases, which is because of the Bauschinger effect of steel, and the damage accumulation of the specimen reduces the moment capacity of the specimen.

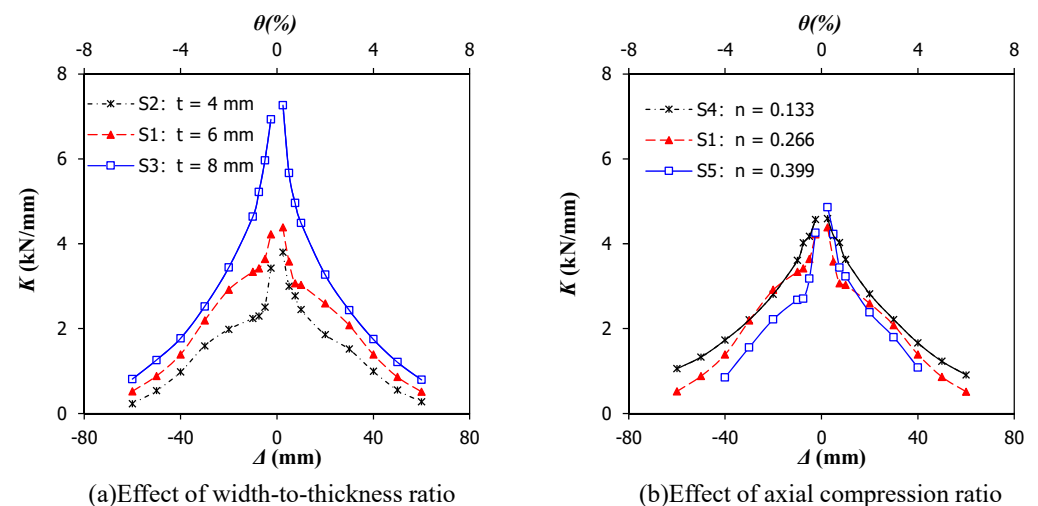


Figure 13. Comparison of rigidity degradation.

The stiffness degradation curve of Figure 13a shows that the degradation of specimen S2 was the largest, followed by S1, and the degradation of S3 was the most gentle. This shows that the smaller the width-to-thickness ratio, the smaller the stiffness degradation degree of the UHPC column specimen.

Comparing the stiffness degradation curves in Figure 13b, it can be seen that the stiffness degradation degree of specimen S5 was the largest, followed by S1, and S4 was the most gentle, indicating that the greater the axial compression ratio, the greater the stiffness degradation degree of UHPSFRC-filled square steel tubular columns, and the more unstable the mechanical performance.

3.7. Ductility and Deformation Behavior

Ductility can fully reflect the ability in inelastic deformation of structures and components. It is more meaningful to use dimensionless ductility parameters for comparison of specimens with different properties; thus, ultimate drift ratio (μ_Δ) was used to evaluate ductility of specimens in this paper.

Table 2 shows the yield bending moment (M_y), peak bending moment (M_m) and ultimate bending moment (M_u) of all specimens, and the corresponding yield displacement (Δ_y), peak displacement (Δ_m) and ultimate displacement (Δ_u) of the five. In addition, there were also the initial stiffness (K_0) and μ_Δ of all UHPSFRC-filled square steel tubular columns. It can be seen that the axial compression ratio and width-to-thickness ratio (thickness of steel tubular) have important effects on the peak bending moment and ductility of specimens.

Table 2. Summary of test results.

Test Specimen	Load Direction	Δ_y (mm)	M_y (kNm)	Δ_m (mm)	M_m (kNm)	Δ_u (mm)	M_u (kNm)	K_0 (kN/mm)	μ_Δ (%)
S1	+	23.3	56.1	30.1	63.5	42.6	54	4.39	4.11
	−	21.1	62.6	30.7	67.2	39.6	57.1	4.22	
S2	+	21.3	43.4	32.1	48.9	41.3	39.9	3.8	4.05
	−	22.9	44.4	29.7	49	39.6	41.7	3.42	
S3	+	18.6	64.6	30.9	74.4	48.6	63.3	7.26	4.91
	−	18.5	66	30.4	76.3	49.6	64.8	6.93	
S4	+	23.1	59.8	40.2	68.2	56.5	58.7	4.59	5.83
	−	24.9	62.2	40.1	72.2	60.0 *	63.6 *	4.57	
S5	+	17.2	50.4	20.2	56.9	35.6	48.4	5.86	3.44
	−	19.4	52.1	20.9	53.7	33.1	45.6	3.76	

Notes: * represents the ultimate displacement and bending moment corresponding to the displacement angle of 6% when the ultimate load does not fall below 85% and the ultimate bending moment takes the displacement and bending moment corresponding to loading to the last level. Δ_y and M_y denote the horizontal displacement and the bending moment of yield point, respectively; Δ_m and M_m denote the horizontal displacement and the bending moment of peak point, respectively; Δ_u and M_u denote the horizontal displacement and the bending moment of ultimate point, respectively; K_0 is initial stiffness of specimen and μ_Δ is ultimate drift ratio.

As shown in Figure 14a, the peak bending moment (M_m) and ultimate drift ratio (μ_Δ) of specimens increase with the decrease in the width-to-thickness ratio (increase in thickness of steel tubular). Compared with specimen S2, the peak bending moments of specimens S1 and S3 increased by 33.5% and 53.9%, respectively, and the ultimate drift ratios increased by 1.5% and 21.2%, respectively, which both indicate that the smaller the width-to-thickness ratio, the higher the peak bending moment. If the width-to-thickness ratio was too small (the thickness of the steel tubular was too large), the moment capacity will not be greatly improved, but it will cause economic waste. Only when the width-to-thickness ratio was small (the thickness of steel tubular was large) and the UHPSFRC was effectively restrained by the square steel tubular did the ultimate drift ratio of specimens increase significantly. Therefore, it was suggested that the width-to-thickness ratio (thickness of steel tubular) of the UHPSFRC-filled square steel tubular column should be limited to a reasonable range to ensure that the steel tubular had sufficient constraints on UHPSFRC as well as good economy.

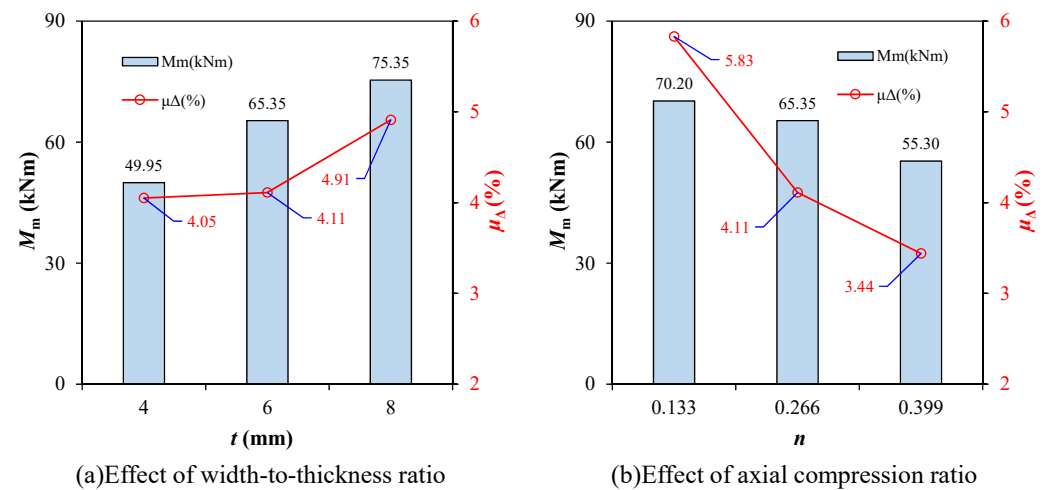


Figure 14. Comparison of peak bending moment and rigidity degradation.

Figure 14b shows that the peak bending moment and ultimate drift ratio of the specimens decreased with the increase in axial compression ratio. Compared with specimen S4, the peak bending moments of specimens S1 and S5 decreased by 6.9% and 21.2%, respectively, and the limit drift ratios decreased by 29.5% and 41.0%, respectively. Increasing the axial compression ratio will significantly damage the ductility of UHPC columns. Although the increase in axial compression ratio will lead to a decrease in peak bending moment, the increase in axial compression ratio in a small range has no obvious effect on the decrease in peak bending moment. When the axial compression ratio was larger than 0.4, the peak moment capacity of UHPSFRC-filled square steel tubular columns decreased significantly. Therefore, a wide range of parameters on concrete-filled steel tube columns was required to control the axial compression ratio within a reasonable range.

4. Finite Element Analysis

In this paper, open source finite element software “The Open System for Earthquake Engineering Simulation” (OpenSees) was used to model the UHPSFRC-filled square steel tubular column and simulate its hysteresis performance under the combination of constant axial load and cyclic horizontal load. OpenSees is a well-recognized and powerful tool on simulating the seismic behavior of structural components, such as RC wall [36], column [37] and joint [38], as well as framed structures [39].

The finite model used the “nonlinearBeamColumn” element that was based on the flexibility method. For the number of elements, Wang et al. [40] considered that excellent simulation results would be obtained by using only one element when elastic–plastic analysis of the cantilever column was conducted. For the number of integration points, the study by Neuenhofer et al. [41] showed that ideal accuracy can be achieved with a reasonable number of integration points of three to four. Li et al. [42] studied the modeling method based on the flexibility method for circular concrete-filled steel tube columns under cyclic reciprocal loading and suggested that three to five integration points should be taken for each element. To ensure the accuracy of the results, the number of integration points was chosen as five. The mesh width of the steel tubular and core concrete was taken to be similar to the thickness of the steel tubular to obtain a uniform meshing model; thus, 20 mm was chosen in this paper.

To summarize, only one “nonlinearBeamColumn” element was used in the model, which was assigned five integration points along the column length, and the section was divided into 20×20 microfiber elements, as shown in Figure 15.

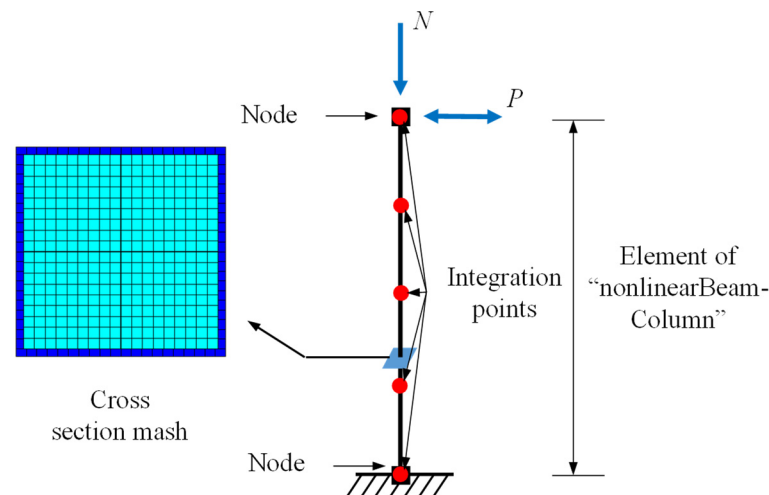


Figure 15. Schematic diagram of finite element model.

The constitutive model of steel and concrete was provided by OpenSees. The bilinear kinematic hardening “Steel01” material was used for steel, of which the stress–strain curve under cyclic loading was composed of an elastic section and strengthened section. The main input parameters were steel yield strength (f_y), initial elastic tangent (E_0), and strain-hardening ratio (ratio between post-yield tangent and initial elastic tangent, b), which was taken as 0.01 in this paper.

The core concrete UHPSFRC adopts the “Concrete02” material, which considers the tensile stress–strain relationship with linear softening and the influence of steel tubular constraints. The main input parameters were as follows: compressive strength (f_{cc}) and strain at compressive strength (ϵ_{cc}), crushing stress (f_{cu}) and strain crushing stress (ϵ_{cu}), tensile strength (f_t) and tensile softening stiffness (E_{ts}), namely the slope of the linear tension softening branch. The constitutive curve is shown in Figure 16.

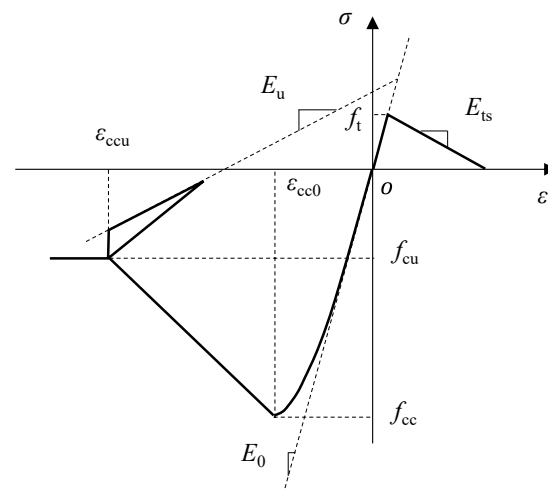


Figure 16. The “Concrete02” constitutive model in OpenSees.

The required parameters were calculated by the formula proposed by Cai et al. [12] considering the constraint effect of the steel tubular to UHPSFRC and the strengthening effect of steel fiber, and Formula (4) was used to correct the influence of the section shape. The calculation formula is as follows:

$$\frac{f_{cc}}{f_c} = f_c + m f_r \quad (1)$$

$$\varepsilon_{cc} = \varepsilon_0 \left[1 + 5 \left(\frac{f_{cc}}{f_c} - 1 \right) \right] \quad (2)$$

$$f_t = 0.31 \sqrt{f_{cc}} \quad (3)$$

Among them, m was taken as 4.0 according to the study by Susantha [43], and the equivalent diameter of the square section is:

$$D_e = 2D / \sqrt{\pi} \quad (4)$$

where, f_{cc} is the peak compressive stress, f_c is the cube compressive strength of concrete, f_t is the confining pressure [44], ε_{cc} is the peak compression strain, and ε_0 ($= 0.0022$) is the peak compression strain of UHPSFRC without horizontal constraint. The failure stress was defined as 0.5 times the peak compressive stress, and the remaining parameters were calculated in Reference [12]. Table 3 shows the values for the parameters for the UHPSFRC and steel in different models.

Table 3. Table of values for the parameters of the principal structure model.

UHPC							Q355B				
t	f_c	f_{cc}	ε_{cc0}	f_{cu}	ε_{ccu}	λ	f_t	E_{ts}	f_y	E_0	b
(mm)	(MPa)	(MPa)		(MPa)			(MPa)	(MPa)	(MPa)	(GPa)	
4	131.2	229.1	0.0173	114.6	0.0181	0.1	4.7	1196.8	389.8	206	0.01
6	131.2	284.2	0.0250	142.1	0.0258	0.1	5.2	1333.0	393.4	206	0.01
8	131.2	340.8	0.0329	170.4	0.0337	0.1	5.7	1459.8	391.1	206	0.01

Notes: f_c is compressive strength of the concrete cylinder; f_{cc} and ε_{cc0} denote the peak stress and strain of UHPC under compression; f_{cu} and ε_{ccu} denote the ultimate stress and strain under compression; λ is the ratio between the unloading slope at ε_{ccu} and initial slope; f_t is the tensile strength under tension; E_{ts} is the tension softening stiffness, which is the slope of the linear tension softening branch; f_y is the yield strength of steel; E_0 is the elastic modulus; b is the strain-hardening ratio, which was the ratio between the post-yield tangent and initial elastic tangent.

Figure 17 shows the results of the comparison of the finite element simulation and test skeleton curve. It can be seen from Figure 17 that the moment capacity of numerical simulation was slightly larger than that of the test, which was due to the high ultimate failure stress of the numerical model. The peak moment capacity deviations of the five models were 3.3%, 0.6%, 3.2%, 12.4% and 6.0%, and the mean square deviation was only 0.0016, meaning the discreteness was small. The initial stiffness of the hysteresis curve was relatively large, which was caused by the fact that the small bond-slip model between the outer steel tubular and the inner concrete after deformation was not considered, and the bottom of the column model was fixed and its stiffness was infinite, which was different from the actual test specimen. The simulation results of the finite element model under horizontal cyclic loading were in good agreement with the test results. The numerical simulation hysteretic curve was fuller than the experimental results, which show that the energy dissipation capacity was more ideal. The stiffness and the characteristic bending moment of the curve were almost consistent with the test results, except for the specimens with a high axial compression ratio. This was because the local bulging and reduction in the stability of the columns in OpenSees was difficult to capture with “BeamColumn” elements [45], especially in the state of a high axial compressive ratio. In general, the accuracy of the numerical simulation results was high, indicating that the established model can accurately simulate the hysteresis behavior of UHPSFRC-filled square steel tubular columns in the test.

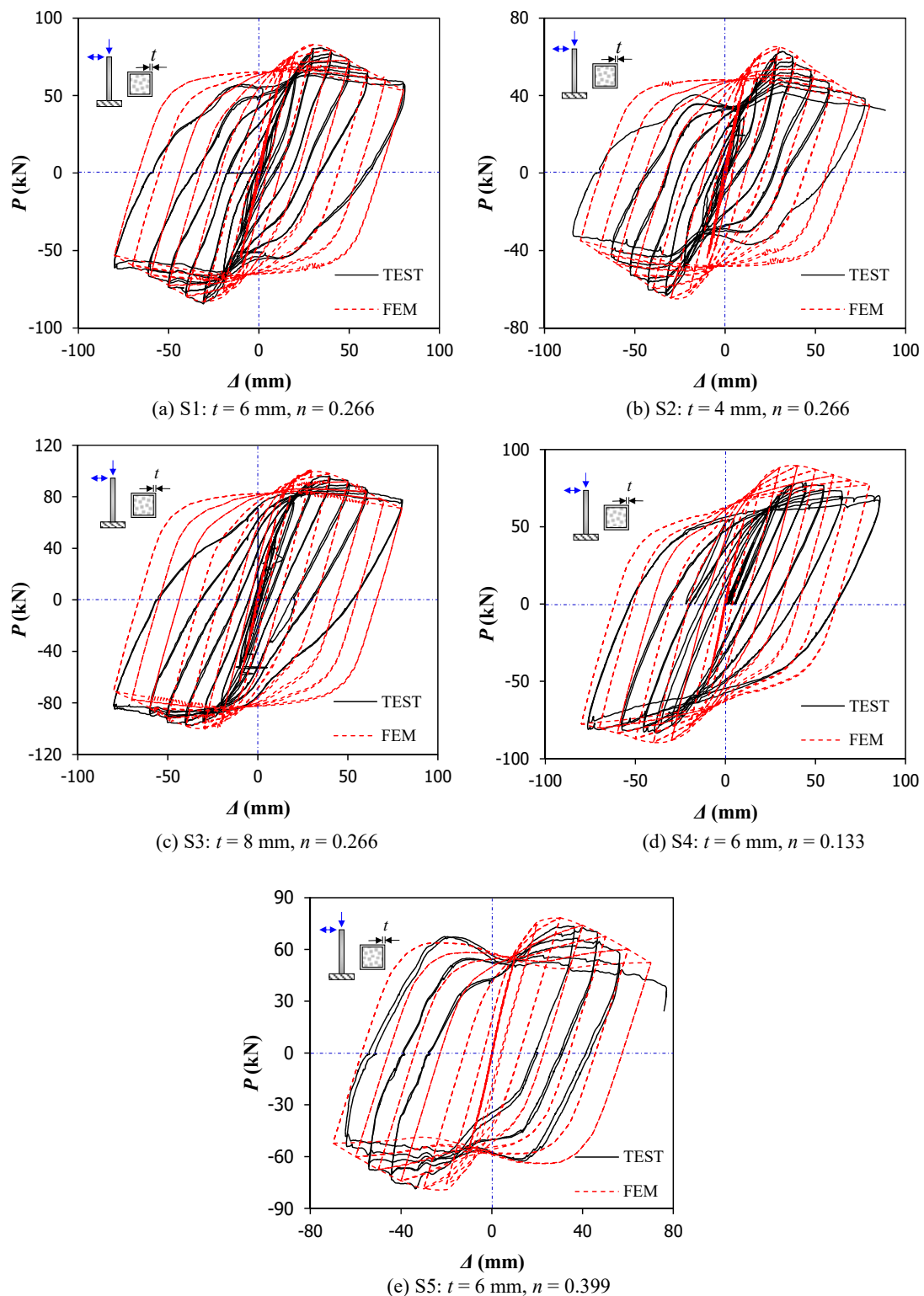


Figure 17. Comparison of rigidity degradation for FEM and test.

5. Discussion

The failure mode of the five UHPSFRC-filled square steel tubular columns was local buckling failure. The theoretical calculation of the bearing capacity of the compression bending can be calculated from the section bending analysis. The UHPSFRC constitutive curve was measured by compression tests, as shown in Figure 18. The assumptions for the calculation: (1) under the ultimate compression state, the UHPSFRC strain at the edge

reached the ultimate compression strain (ϵ_{cu}); (2) the cross-sectional steel tubular has yield; (3) the cross-sectional strain distribution follows the assumption of a plain section remaining plain. The schematic diagram of the cross-section calculation is shown in Figure 19.

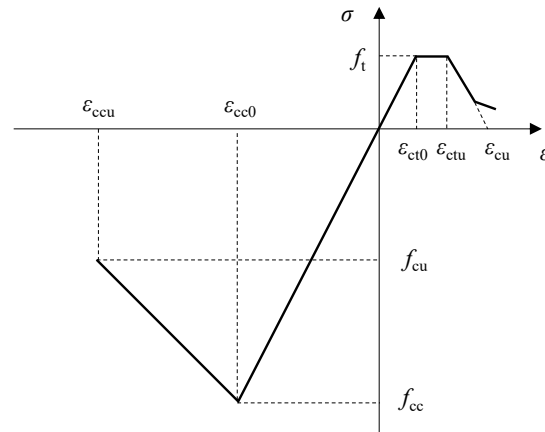


Figure 18. UHPSFRC constitutive curve tested by the compression test.

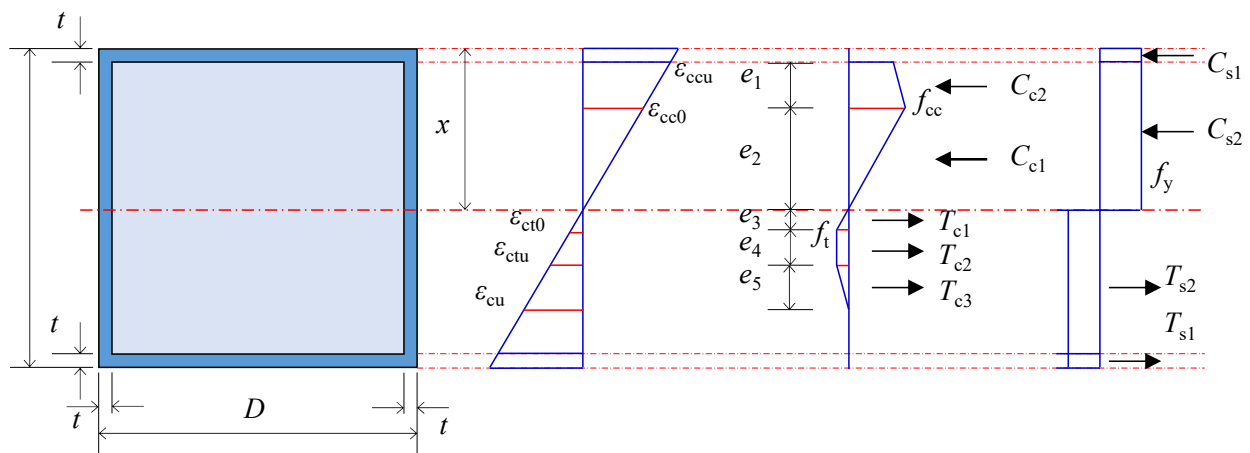


Figure 19. Schematic diagram of the cross-section calculation.

It can be known from the vertical force balance that

$$N = C_{s1} + C_{s2} + C_{c1} + C_{c2} - T_{s1} - T_{s2} - T_{c1} - T_{c2} - T_{c3} \quad (5)$$

where N is the applied external compression load; C_{s1} and C_{s2} are the vertical loads of the steel tubular in the pressure zone; T_{s1} and T_{s2} are the vertical loads of the steel tubular in the tensile zone; C_{c1} and C_{c2} are the vertical loads of UHPSFRC in the pressure zone; T_{c1} , T_{c2} and T_{c3} are the vertical loads of UHPSFRC in the tensile zone. C_{s1} , C_{s2} , T_{s1} , T_{s2} , C_{c1} , C_{c2} , T_{c1} , T_{c2} , and T_{c3} are determined by the following formulas:

$$C_{s1} = T_{s1} = tDf_y \quad (6)$$

$$C_{s2} = 2tf_y(x - t) \quad (7)$$

$$T_{s2} = 2tf_y(D - x - t) \quad (8)$$

$$C_{c1} = (D - 2t)e_2f_{cc}/2 \quad (9)$$

$$C_{c2} = e_1(D - 2t)(f_{cc} + f_{cu})/2 \quad (10)$$

$$T_{c1} = (D - 2t)e_3f_t/2 \quad (11)$$

$$T_{c2} = (D - 2t)e_4f_t \quad (12)$$

$$T_{c3} = (D - 2t)e_5f_t/2 \quad (13)$$

where, e_1, e_2 was the height of UHPSFRC in different areas of the pressure zone, and e_3, e_4 and e_5 were the heights of UHPSFRC in different regions of the tensile zone, as shown in Figure 19. e_1, e_2, e_3, e_4 and e_5 were calculated by the following formulas:

$$e_1 = x - t - e_2 \quad (14)$$

$$e_2 = (x - t)\varepsilon_{cc0}/\varepsilon_{ccu} \quad (15)$$

$$e_3 = e_2\varepsilon_{ct0}/\varepsilon_{cc0} \quad (16)$$

$$e_4 = e_2\varepsilon_{ctu}/\varepsilon_{cc0} - e_3 \quad (17)$$

$$e_5 = e_2\varepsilon_{cu}/\varepsilon_{cc0} - e_3 - e_4 \quad (18)$$

Among them, x is the height of neutral axis; $\varepsilon_{cc0}, \varepsilon_{ccu}$ are the strains on the core UHPSFRC compressive stress–strain curve; $\varepsilon_{ct0}, \varepsilon_{ctu}$ and ε_{cu} are the strains on the core UHPSFRC tensile stress–strain curve. They were calculated by the following formulas:

$$\varepsilon_{cc0} = f_{cc}/E_c \quad (19)$$

$$\varepsilon_{ct0} = f_t/E_c \quad (20)$$

where, E_c and E_s are the UHPSFRC and steel pipe modulus of elasticity, respectively.

The height of the neutral shaft x can be calculated from the formula of vertical force balance and by taking the bending moment of the neutral shaft of the column section:

$$\begin{aligned} M_{mT} = & N(0.5D - x) + C_{s1}(x - 0.5t) + 0.5C_{s2}(x - t) \\ & + \frac{2}{3}C_{c1}e_2 + C_{c2}\left(e_2 + \frac{f_{cu} + 2f_{cc}}{3(f_{cu} + f_{cc})}e_1\right) + T_{s1}(D - x - 0.5t) \\ & + 0.5T_{s2}(D - x - t) + \frac{2}{3}T_{c1}e_3 + T_{c2}(e_3 + 0.5e_4) + T_{c3}(e_3 + e_4 + \frac{e_5}{3}) \end{aligned} \quad (21)$$

In the formula, M_{mT} is the maximum bending moment capacity of the bottom section of the column. The yield bending moment (M_{yT}) can be calculated from the following equation [46]:

$$M_{yT} = 0.6M_{mT} \quad (22)$$

The initial stiffness K_{0T} can be calculated by Equation (23):

$$K_{0T} = 3K_e/H^3 \quad (23)$$

where H is the effective length of the specimen. K_e is the elastic stiffness of the column and was calculated by Equation (24).

$$K_e = E_s I_s + 0.2E_c I_c \quad (24)$$

The comparison between the analytical results and the test results for the five tested specimens are summarized in Table 4.

As can be seen from Table 4, the ratio of the calculated maximum bending moment (M_m) to the experimental value was 0.96, and the covariance was 0.07, which was in good agreement. The ratio of yield bending moment (M_y) and the calculated initial stiffness (K_0) of the specimen to the test value were 0.84 and 0.65, respectively, which was conservative compared to the experimental values. Therefore, the theoretical calculation model established in this paper can better predict the bending capacity of the UHPSFRC-filled square steel tubular columns. However, the calculation formulas of initial stiffness and yield bending moment need to be further corrected.

Table 4. Comparison of theoretical results and test results.

Test Specimen	Load Direction	M_y (kNm)	M_{yT} (kNm)	M_{yT}/M_y	M_m (kNm)	M_{mT} (kNm)	M_{mT}/M_m	K_0 (kN/mm)	K_{0T} (kN/mm)	K_{0T}/K_0
S1	+	56.1	36.7	0.65	63.5	61.2	0.96	4.39	3.98	0.91
	−	62.6	36.7	0.59	67.2	61.2	0.91	4.22	3.98	0.94
S2	+	43.4	29.8	0.69	48.9	49.7	1.02	3.90	2.99	0.79
	−	44.4	29.8	0.67	49.0	49.7	1.01	3.42	2.99	0.87
S3	+	64.6	43.4	0.67	74.4	72.3	0.97	7.26	4.99	0.69
	−	66.0	43.4	0.66	76.3	72.3	0.95	6.93	4.99	0.72
S4	+	59.8	35.9	0.60	68.2	59.9	0.88	4.59	3.98	0.87
	−	62.2	35.9	0.58	72.2	59.9	0.83	4.57	3.98	0.87
S5	+	50.4	34.4	0.68	56.9	57.4	1.01	5.86	3.98	0.68
	−	52.1	34.4	0.66	53.9	57.4	1.06	3.76	3.98	1.06
Mean				0.65			0.96			0.84
Cov				0.04			0.07			0.12

Notes: M_y and M_{yT} are the experimental and theoretical calculations of the bending moment of yield point, respectively; M_m and M_{mT} are the experimental and theoretical calculations of the bending moment of peak point, respectively; K_0 and K_{0T} are the experimental and theoretical calculations of the initial stiffness, respectively.

6. Conclusions

In this paper, the quasi-static test was carried out on the ultra-high performance concrete-filled square steel tubular columns with steel fiber to study their ultimate strength and seismic performance, and numerical analysis was also carried out. From these studies, the following conclusions can be drawn:

1. The UHPSFRC-filled square steel tubular columns showed local buckling failure, and their hysteresis curves were full and have good seismic performance.
2. When the width-to-thickness ratio decreased from 28.5 to 19.9 and 14.7 (the thickness of the steel tubular increased from 4 to 6 and 8 mm), the moment capacity of the UHPSFRC-filled square steel tubular columns increased significantly, and the ductility increased slightly first and then increased obviously, which slowed down the degradation of the stiffness and strength.
3. When the axial compression ratio increased from 0.133 to 0.266 and 0.399, the moment capacity of UHPSFRC-filled square steel tubular columns decreased slightly at first and then decreased significantly, and the ductility decreased obviously, which accelerated the degradation of stiffness and strength.
4. The finite element model established with the UHPFSRC constitutive model considering external steel tubular constraint can simulate the hysteresis behavior of UHPSFRC-filled square steel tubular columns well, and the skeleton curve was in good agreement with the experimental results.
5. When the width-to-thickness ratio (the thickness of steel tubular) and axial compression ratio were within a reasonable range, the UHPSFRC-filled square steel tubular column had good energy dissipation capacity and plastic deformation capacity, and it shows good ductility under earthquake action, which can better meet the seismic design requirements of seismic fortification areas versus RC columns.
6. UHPSFRC-filled steel tubular columns exhibit good energy dissipation capability and ductility under earthquake. In the future, new materials can be developed in composite columns, which expected to be researched to determine optimal size for practical engineering applications. In addition, the finite element models can provide a reference and recommendation for subsequent research, but they also need to be developed on method to simulate local flexion and bugling. The proposed theoretical calculation model can well predict the flexural capacity of UHPSFRC-filled square steel tubular columns, but the calculation formulas of yield bending moment and initial stiffness need further research.

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