Compressive behavior of circular concrete-filled steel tubular columns under freeze-thaw cycles

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Abstract: In this paper, the compressive behavior of circular concrete-filled steel tubular stub columns under freeze-thaw cycles was studied. Influence of different concrete grades and number of freeze-thaw cycles were investigated through experimental study. The results show that with the increase of the number of freeze-thaw cycle, the axial strength of specimens decreases regardless of concrete grade. The axial strength deterioration of the CFST specimens under different freeze-thaw cycles increases with the decrease of concrete strength, so does the reduction of enhancement factor. The ultimate displacement and the ductility indexes of CFST specimens are barely affected by freeze-thaw cycles. Based on the test results, a reduction factor k_{sr} for quantifying the strength reduction of CFST and a factor k_{csr} for considering the strength degradation of core concrete are proposed to predict the ultimate compressive strength of CFST stub column under freeze-thaw cycles.

Keywords: concrete-filled steel tube; axial behavior; stub columns; freeze-thaw; cold region; design standard

1. Introduction

Concrete-filled steel tube (CFST) has been widely used in different types of infrastructure across the world, due to its excellent structural performance, economic and constructional benefits [1-2]. Besides the short-term performance of CFST members, their long-term durability is also crucial in the design [3]. In high-latitude cold region, freeze-thaw environment would cause low-temperature failure of CFST columns due to the icing and migration of free water in core concrete, even with the protection of outer steel tube [4]. Therefore it is important to investigate the performance of CFST columns under freeze-thaw cycles and ensure the safety of CFST structures in high-latitude cold region.

A few experimental studies have been conducted to investigate the behavior of CFST columns under freeze-thaw cycles recently. Yang [5] carried out a rapid freeze-thaw test on steel tube confined concrete and plain concrete. The results showed that the strength degradation of steel tube confined concrete was less than that of plain concrete. Yang et al. [6] conducted a test on the behavior of CFST stub columns after being exposed to freeze-thaw cycles. The number of freeze-thaw cycles and the steel ratio were treated as key parameters. A simplified formula for predicting the ultimate strength CFST stub column under freeze-thaw cycles was developed based on the experimental results. Shen et al. [7] conducted an experimental study on the performance of CFST stub columns under freeze-thaw cycles. It should be mentioned that there was no base plate for the CFST column specimens during the test, in order to magnify the effect of freeze-thaw cycles.

The existing literature review shows that the studies on CFST columns under freeze-thaw cycles are still limited. More tests should be conducted to provide further experimental data that can be used to validate the further numerical and theoretical studies. In particular, currently, the strength of core concrete has not been treated as main parameter in relevant experimental study.

However, it is worth noting that there have been many studies on the behavior of plain concrete under the freeze-thaw cycles. Cao et al. [8-9] conducted a series of tests on constitutive relations of concrete and RC beams after freeze-thaw cycles. The results showed that with the increase of concrete grade, the effect of freezing-thawing cycles reduces. Tian et al. [10] conducted an experimental study on the dynamic damage mechanism of concrete under freeze-thaw cycles. The results showed that the dynamic ultimate compressive strength rises with the increasing of loading rates under the same freeze-thaw cycles. Duan et al. [11] performed an experimental research on the complete compressive stress-strain relationship for unconfined and confined concrete after exposure to freeze-thaw cycles. Analytical models for the stress–strain relationship of frozen-thawed unconfined and confined concrete were empirically developed respectively. Xu et al. [12] investigated the seismic performance of reinforced concrete columns after freeze–thaw cycles. Test results indicated that for the column specimens with the same level of frost damage, with the increase of axial compression ratio, the load carrying capacity and initial stiffness increased.

In this paper, the compressive behavior of circular concrete-filled steel tubular stub column under freeze-thaw cycles was studied. Both experimental and theoretical analyses were conducted, in which freeze-thaw cycles and concrete strength were both considered as main parameters. The specimens after freeze-thaw cycles were tested under axial compressive load. The freeze-thaw effect on concrete properties and axial performance of circular CFST stub column is discussed. Current design formulas for concrete-filled steel tubular stub columns are modified to consider the freeze-thaw effect.

2. Experimental program

2.1. Preparation of test specimens

In total 24 circular CFST specimens were fabricated and tested. The length-to-diameter ratio was 3 for all specimens to ensure these specimens can be classified to short column. Due to the limitation of test setup, all the steel tubes used in the specimens exert the same dimension as $L \times D = 270mm \times 90mm$, where L and D are the length and exterior diameter of the tubes respectively. Normally, the steel ratio of CFST is in the range of 4-10% in arch ribs of bridge and 8-10% in building structures. In that case, 2-mm-thickness tube with 10% steel ratio of CFST specimens are used during the tests. As shown in Table 1, the designation of specimens starts with concrete grade, followed by the number of freeze-thaw cycle and the number to distinguish specimens with same parameters.

Table 1 Parameters of specimens

No.	Concrete grade	$L \times D \times t$ /mm	N_C
S30-0-1/2	C30	$270 \times 90 \times 1.9$	0
S30-90-1/2			90
S30-180-1/2			180
S30-270-1/2			270
S40-0-1/2	C40		0

S40-90-1/2			90
S40-180-1/2			180
S40-270-1/2			270
S50-0-1/2	C50		0
S50-90-1/2			90
S50-180-1/2			180
S50-270-1/2			270

Note: t=the thickness of tube wall; N_C=freeze-thaw cycles

2.2. Material properties

The mixture proportions of concrete were: cement: 490kg/m^3; coarse aggregate: 955kg/m^3; fine aggregate: 575kg/m^3; water: 190kg/m^3. Type I Portland cement 42.5 R was used in the production of all specimens while super-plasticizer with 0.5% of the weight of cement was used to enhance the workability of the concrete.

150×150×150 mm cubes for testing concrete strength and 150×150×300 mm prisms for testing concrete Young's modulus were casted and cured in the same condition as the specimens. After testing, the average compressive strength of C30, C40 and C50 grade concrete were 37.2 MPa, 49.3 MPa and 56.1 MPa respectively while the Young's modulus of those were 3.03×10^4 MPa, 3.26×10^4 MPa and 3.37×10^4 MPa respectively. The yield strength f_y, ultimate strength f_u, Young's modulus E_s and ultimate strain ε_u of steel coupon were 359 MPa, 531 MPa, 2.01×10^5 MPa and 0.3 respectively.

2.3. Freeze-thaw test

Before axial compression test, all specimens should go through a freeze-thaw cycle test. A freeze-thaw test was conducted in accordance to Chinese standard GB/T 50082-2009 [13], since there is no test standard for CFST member under freeze-thaw cycles available. Due to the existence of outer steel tube, the core concrete was protected from the water; hence the requirements from GB/T 50082-2009 which requires that the plain concrete specimens should be soaked in the water for 4 days before under freeze-thaw cycles were not followed in this test [6].

The main technical parameters of the freeze-thaw test procedure are as follows:

(1). Every freeze–thaw cycle begins by decreasing the core temperature of the specimens from 5 to −18 °C followed by increasing it from −18 to 5 °C. The duration of each cycle is 4 hours. The time for thawing process should be more than 1/4 of one cycle duration.

(2). The lowest and highest temperatures at the center of the specimens should be in the range of −18±2 °C and 5±2 °C respectively. The center temperature of the specimens should not be lower than −20 °C or higher than 7 °C at any time.

(3). The time for decreasing the temperature from 3 to −16 °C should be more than half length of the freezing duration while the time for increasing the temperature from −16 to 3 °C should be more than half of the thawing duration.

(4). The temperature difference between the center and surface of a specimen should not be more than 28 °C at any time. The transition time of from freezing process to thawing process of every cycle should not be more than 10 min.

A rapid freeze-thaw testing equipment as shown in Fig. 1 was used to conduct the test. Antifreeze fluid in the container was used to perform the freeze-thaw cycle while the specimens were separately placed into the rubber boxes which were full of water. A counterpart specimen was adopted to monitor the center temperature of specimen. The quality loss and dynamic elastic modulus loss of specimen were not recorded since the core concrete was isolated from the water by outer steel tube. It should be mentioned that neither obvious deformation nor cracking was observed on the outer steel tube after the freeze-thaw cycles. This phenomenon may be explained by the fact that the water to cement ratio of concrete is relatively low in such small specimens [6]. The temperature of specimen center and antifreeze fluid within 24 hours is shown in Fig. 2.

2.4. Axial compression test

After freeze-thaw cycle test, all corroded specimens were axially loaded until failure using a 2000 kN hydraulic compression machine, as shown in Fig. 3(a). Four LVDTs were installed to record the axial deformation of the specimens. Four pairs of strain gauges were evenly spaced around the circumference of concrete-filled steel tubes as shown in Fig. 3(b).

3. Experimental results

3.1. Failure patterns of specimens

The representative failure pattern of the specimens is shown in Fig. 4. It can be seen that the cycle number and concrete grade have little effect on the failure pattern of circular CFST stub columns under freeze-thaw cycles. Outward buckling was observed at the outer steel tube of all specimens. After the test, the outer steel tube was removed to inspect the failure pattern of the core concrete. A diagonal shear crack and many micro cracks were observed at the core concrete. It seems that the crush of core concrete became severer with the increase of cycle numbers. The failure pattern of the core concrete was also hardly affected by the cycle number and concrete grade.

3.2. Load-displacement relationship curves

Fig. 5 shows the representative load-displacement curves of the specimens. It indicates that with the increase of the number of freeze-thaw cycle, the strength of specimens decreases regardless of concrete grade. Freeze-thaw cycle does not change the overall trend of the curves remarkably. With the increase of concrete grade, the reduction of axial strength after peak point becomes sharper.

Fig. 6 shows the relationship of ultimate strength N_u and ultimate displacement Δ_u of the specimens with freeze-thaw cycles. The ultimate strength of the specimens decreases linearly with the increase of freeze-thaw cycles regardless of concrete grade as shown in Fig. 6 (a). The strength deterioration of the CFST specimens under different freeze-thaw cycles increases with the decrease of concrete strength. Under 270 freeze-thaw cycles, the ultimate strength of the specimens with C30 grade concrete is reduced by 14% while that of the specimens using C40 and C50 grade concrete are only reduced by 8.3% and 7.9% respectively. This observation could be explained by the fact that larger water-cement ratio used in lower concrete strength would make core concrete more vulnerable to freeze-thaw cycles [14].

Fig. 6(b) implies that freeze-thaw cycle has little influence on the ultimate displacement of CFST column. In fact, the difference between the ultimate displacements of CFST columns under different freeze-thaw cycles is rather small. All the ultimate displacements of the tested specimen range from 4 mm to 5.2 mm. Normally, ultimate strain of concrete would decrease with the increase of freeze-thaw cycles [15]. The results in Fig. 6(b) indicate that the existence of outer steel tube would improve the reduction of deformation ability of core concrete due to freeze-thaw cycles.

The composite elastic modulus of CFST stub column is defined as follows [1]:

$$E_{SC} = \frac{0.4N_u}{(A_s + A_c)\varepsilon_{0.4}} \qquad (1)$$

where N_u is the ultimate strength of specimen; A_s and A_c are the area of steel tube and core concrete respectively; $\varepsilon_{0.4}$ is the strain corresponding to $0.4N_u$ in the ascending stage of load-displacement curve.

It can be seen from Fig. 6(c) that CFST stub column with higher concrete strength exhibits larger composite elastic modulus which generally decreases with the increase of freeze-thaw cycle regardless the concrete strength. This could be explained by the fact that the elastic modulus of core concrete would be degraded after freeze-thaw cycles.

3.3. Ductility of specimens

To quantify the influence of corrosion rate on the ductility of specimen, a ductility index λ is introduced as described in Eq. (2):

$$\lambda = \Delta_{0.85} / \Delta_u \qquad (2)$$

where $\Delta_{0.85}$ is the axial displacement of specimen when the applied load falls to 85% of the ultimate load after damage; Δ_u is the axial displacement of specimen when the ultimate strength is reached.

Fig. 7 shows the relationship between ductility index and freeze-thaw cycle. It can be seen that all the ductility indexes range from 1.3 to 2.0. The ductility of CFST stub columns without freeze-thaw cycles decreases with the increase of concrete grade. Similar to the ultimate displacement of specimen, the ductility indexes of CFST specimens are barely affected by freeze-thaw cycles, since the failure modes shown in Fig. 4 are also not affected by freeze-thaw cycles. It confirms the fact that the existence of outer steel tube would reduce the effect of freeze-thaw cycles on the ductility of core concrete.

3.4. Lateral deformation factor of steel tube

Fig. 8 shows the representative relationship between the ultimate strength and strain of the specimens after freeze-thaw cycles. In general the curves show similar trend regardless of the number of freeze-thaw cycle. Irregular influence of freeze-thaw cycles on the ultimate strain of specimens is observed.

Lateral deformation factor μ is defined as the ratio between transverse strain ε_t and longitudinal strain ε_l of steel tube. The lateral deformation factors of CFST columns under freeze-thaw cycles are illustrated in Fig. 9. It can be seen that the lateral deformation factors of CFST columns remain around 0.3 which equals to the Poisson ratio of the steel material before the axial load reached 75% of ultimate strength. After that, μ increases remarkably due to the confinement effect and the specimens yielded under compression. It can be seen that, the lateral deformation factors of specimens under normal condition are larger than those of specimens after freeze-thaw cycles. It indicates that the composite action between outer tube and core concrete are weakened under freeze-thaw cycles [6].

3.5. Confinement effect of specimens

The nominal strength N_0 of CFST column is defined as:

$$N_0 = f_y A_s + f_c A_c \qquad (3)$$

where f_y and f_c are the strength of steel and concrete respectively; A_s and A_c are the area of steel tube and core concrete respectively.

The confinement effect of CFST columns in this study is assessed by using the enhancement factor φ as described in Eq. (4):

$$\varphi = (N_u - N_0) / N_0 \qquad (4)$$

It should be mentioned that the concrete strength using in Eq. (3) should be reduced to consider the effect of freeze-thaw cycles. However the strength of core concrete in steel tube under freeze-thaw cycles is inconvenient to test, even though there are some prediction methods for the strength of plain concrete under freeze-thaw cycles [15]. The existence of outer steel tube would obviously reduce the influence of freeze-thaw cycles on the strength of core concrete. Therefore, the reduction of core concrete strength is not explicitly considered in Eq. (3) and would be covered by the enhancement factor φ.

Fig. 10 shows the relationship between φ and N_C. It can be seen that without freeze-thaw cycles, the enhancement factor deceases with the increase of concrete strength, due to the weakening of confinement effect. The enhancement factors of the specimens using C30, C40 and C50 grade concrete without freeze-thaw cycles are 0.33, 0.29 and 0.28 respectively. With the increase of the number of freeze-thaw cycles, the enhancement factor decreases regardless of concrete grade. Similar to the ultimate strength, the reduction of enhancement factor of the CFST specimens under freeze-thaw cycles increases with the decrease of concrete strength. Under 270 freeze-thaw cycles, the enhancement factor of the specimens using C30 grade concrete decreases to 0.14 while those using C40 and C50 grade concrete decrease to 0.18 and 0.17 respectively.

4. Analytical study of the ultimate strength

4.1. Reduction factor of ultimate strength

In Ref. [6], Eq. (5) was proposed based on the test results to predict the ultimate strength of CFST stub columns under freeze-thaw cycles:

$$N_{ue}(N_C) = (1 - 0.0005 N_C) N_{ue}(0) \qquad (5)$$

where N_C is the number of freeze-thaw cycle; $N_{ue}(N_C)$ and $N_{ue}(0)$ are the tested ultimate strength of CFST stub columns under and without freeze-thaw cycles respectively.

As shown in Table 2, the predicted values using Eq. (5) show good agreement with the tested values. It is worth noting that the difference between the predicted values and tested values increases with the increase of concrete grade. This is because the effect of concrete grade is not considered in Eq. (5). As mentioned above and in Ref. [14], the strength deterioration of the CFST specimens under different freeze-thaw cycles increases with the decrease of concrete grade. Hence the predicted values by using Eq. (5) become lower than the tested values with the increase of concrete grade.

Table 2 Comparison of tested values and predicted values

Specimen No.	Test value N_{ue}/kN	Average test value $\overline{N}_{ue}$ /kN	N_C	Predicted value of Eq. (5)/kN	Difference of Eq. (5)/%	Predicted value of Eq. (7)/kN	Difference of Eq. (7) /%
S30-0-1	538.6	530.4	0	--	--	--	--
S30-0-2	522.2				--		--
S30-90-1	500.1	500.1	90	506.5	+1.3	511.7	+2.3
S30-90-2	500.0				+1.3		+2.3
S30-180-1	486.4	484.9	180	482.6	-0.7	487.6	+0.2
S30-180-2	483.4				-0.1		+0.8
S30-270-1	463.8	465.2	270	458.8	-1.0	463.5	-0.1
S30-270-2	466.6				-1.6		-0.6
S40-0-1	614.1	615.7	0	--	--	--	--
S40-0-2	617.4				--		--
S40-90-1	590.7	592.8	90	588.0	-0.4	606.9	+2.7
S40-90-2	595.0				-1.1		+2.0
S40-180-1	583.8	582.5	180	560.3	-4.0	578.3	-0.9
S40-180-2	581.2				-3.6		-0.5
S40-270-1	563.0	565.5	270	532.6	-5.4	550.0	-2.3
S40-270-2	568.1				-6.3		-3.0
S50-0-1	659.1	656.1	0	--	--	--	--
S50-0-2	653.1				--		--
S50-90-1	647.1	645.6	90	626.6	-3.2	653.8	+1.0
S50-90-2	644.2				-2.7		+1.5
S50-180-1	637.8	638.7	180	597.0	-6.4	623.0	-2.3
S50-180-2	639.7				-6.7		-2.6
S50-270-1	606.7	603.6	270	567.5	-6.5	592.2	-2.3
S50-270-2	600.5				-5.5		-1.4

To tackle this problem, a reduction factor k_{sr} for quantifying the ultimate strength of CFST stub columns under freeze-thaw cycles which is the ratio between the ultimate strength of specimens under freeze-thaw cycles and that without freeze-thaw cycles is introduced as follows:

$$k_{sr} = N_{ue}(N_C) / N_{ue}(0) \qquad (6)$$

Fig. 11 shows the influence of N_C and concrete grade on k_{sr}. As shown in Fig. 10, the reduction factors of the specimens using different concrete grade decrease linearly with the increase of the numbers of freeze-thaw cycles. It is evident that lower concrete grade would result in a larger reduction on ultimate strength of CFST stub column, even the reduction factors of the specimens using C40 grade concrete and C50 grade concrete are close to each other under the same N_c of freeze-thaw cycles.

A new formula Eq. (7) considering the influence of both the number of freeze-thaw cycle and concrete grade is developed based on Eq. (5) and experimental results. Two principles were followed in developing Eq. (7): (a) the factor increases with the increase of concrete grade; (b) when concrete grade is C30, Eq. (7) equals to Eq. (5).

$$k_{sr} = (1 - 0.0005N_C)(1 + (f_c - 30) / 700) \qquad (7)$$

where f_c stands for the compressive strength of concrete.

The predicted ultimate strengths of the specimens under freeze-thaw cycles by using Eq. (5) are listed in Table 2. Fig. 12 shows the comparison of the predicted values and tested values using Eq. (5) and Eq. (7) respectively. It can be seen that the difference between the tested values and predicted values by using Eq. (5) ranges in $\pm$7% which is acceptable. However the difference shows an ascending trend with the increase of concrete grade. On the contrary, the difference between the tested values and predicted values by using Eq. (7) is reduced to ±3% and no obvious ascending trend is observed in the comparison.

4.2. Modified design formula for axial strength

Based on the experimental results, a simplified design formula is proposed to predict the theoretical ultimate strength $N_{up}(N_C)$ of CFST stub columns under freeze-thaw cycles, by considering the number of freeze-thaw cycle and concrete grade, as follows:

$$N_{up}(N_C, f_c) = k_{sr} N_{up}(0, f_c) = (1 - 0.0005N_C)(1 + (f_c - 30)/700)N_{up}(0, f_c) \tag{8}$$

The predicted ultimate strength $N_{up}(0)$ of CFST stub columns without freeze-thaw cycles could be obtained by using design standards, such as Eq. (9) in GB50936 [16], Eq. (10) in AIJ [17], Eq. (11) in Eurocode 4 [18] and Eq. (12) in AISC360-10 [19].

$$N_{up}^{CN}(0, f_c) = (A_c + A_s)(1.212 + (\frac{0.176 f_y}{213} + 0.974)\xi + (\frac{-0.104 f_c}{14.4} + 0.031)\xi^2) f_c \tag{9}$$

$$N_{up}^{\mathrm{JP}}(0, f_c) = A_c f_c + 1.27 f_y A_s \tag{10}$$

$$N_{up}^{\mathrm{EU}}(0, f_c) = 0.85 A_c f_c \left[1 + 4.9(t/\mathrm{D})(f_y / 0.85 f_c)\right] + 0.75 f_y A_s \tag{11}$$

$$N_{up}^{\mathrm{US}}(0, f_c) = 0.658^{f_y/\left[\pi^2 E_s/(L/r)^2\right]}(f_y A_s + 0.85 f_c A_c) \tag{12}$$

where ξ is the confinement factor of CFST columns.

Fig. 13 shows the comparisons between the tested and predicted ultimate strength of CFST columns under freeze-thaw cycles. It can be seen from Fig. 12 that the predicted values by using the proposed formula and the formulas in Chinese and EU standards matches well with the tested values. The deviation in Fig. 12 (a) and (c) is in the range of 5%-6%. The predicted values from US standard without considering confinement effect [20] are significantly larger than the tested values while 45% of deviation is observed in Fig. 12 (d). The predicted values from Japanese standard seem also conservative since 25% of deviation is observed in Fig. 12 (b). More tests should be conducted in the future to improve the proposed formula.

4.3. Strength degradation factor for core concrete

It is obvious that the performance degradation of CFST stub column under freeze-thaw cycles is due to the material property degradation of core concrete under freeze-thaw cycles. In Ref. [21], a degradation formula for concrete strength under freeze-thaw cycles was proposed as described by Eq. (13). It should be mentioned that Eq. (13) was proposed in Ref. [21] only based on only one type of concrete strength which was 55.5 MPa.

$$f_{cp} = e^{-0.0018 N_C} f_c \tag{13}$$

where f_{cp} is the compressive strength of concrete under freeze-thaw cycles.

Since the formula from EU standard matches the best with the tested values as shown in Fig. 13, Eq. (11) and Eq. (13) are adopted to predict the axial strength of the specimens. As shown in Fig. 14 (a), the deviation between the tested values and predicted values increases with the increase of concrete strength. With similar concrete strength in the specimen with C50 grade concrete, the predicted values are lower than the tested values by 15%. It indicates that the degradation of core concrete is reduced due to the protection of outer steel tube. In that case, a degradation factor k_{csr} for core concrete strength in CFST under freeze-thaw cycles is proposed to consider the influence of concrete strength and protection of outer steel tube as follows:

$$f_{cp} = k_{csr} f_c = e^{\kappa N_C} f_c \qquad (14)$$

where N_C is the number of freeze-thaw cycle; $\kappa = 5 \times 10^{-5} f_c - 0.0034$.

Fig. 14(b) shows the comparison of the tested values and predicted values using Eq. (14). It can be seen that the deviation is in the range of ±4%. It indicates that the proposed factor for the strength degradation of core concrete could be used to predict the ultimate strength of CFST stub columns under freeze-thaw cycles. Fig. 15 shows the strength degradation of core concrete based on Eq. (14). It can be seen that the number of freeze-thaw cycle has more influence on the relatively lower grade concrete. This influence would decrease with the increase of the number of freeze-thaw cycle.

5. Conclusions

This paper aims to study the compressive behavior of circular concrete-filled steel tubular stub columns under freeze-thaw cycles. The specimens after freeze-thaw cycles were tested under axial compressive load. The experimental phenomena and results are discussed in detail and compared using current design standards. The following conclusions can be drawn:

1). It can be seen that with the increase of the number of freeze-thaw cycle, the strength of specimens decreases regardless of concrete grade. The reduction of axial strength after peak point becomes sharper with the increase of concrete grade.

2). The strength deterioration of the CFST specimens under different freeze-thaw cycles increases with the decrease of concrete strength, so does the reduction of enhancement factor, due to the fact that larger water-cement ratio which is used in lower concrete strength would make core concrete more vulnerable to freeze-thaw cycles

3). The lateral deformation factors of specimens under normal condition are larger than those of specimens after freeze-thaw cycles which indicates that the composite action between outer tube and core concrete is weakened under freeze-thaw cycles.

4). By considering the effects of the number of freeze-thaw cycles and concrete grade, a reduction factor k_{sr} for quantifying the ultimate strength of CFST stub columns under freeze-thaw cycles is proposed based on the experimental results.

5). A factor k_{csr} for considering the strength degradation of core concrete under freeze-thaw cycles is proposed which could be used to predict the ultimate strength of CFST stub column under freeze-thaw cycles.

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Conflict of interest

None

Data availability

The raw/processed data required to reproduce these findings cannot be shared at this time as the data also forms part of an ongoing study.

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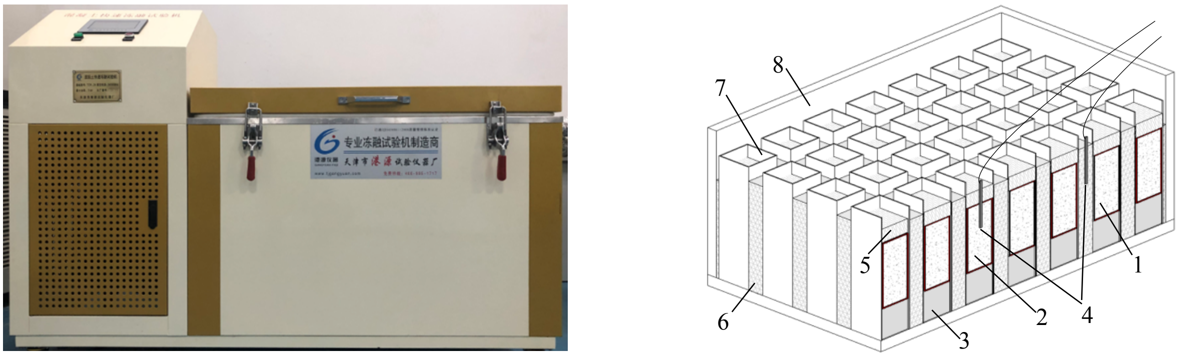


(a) Appearance of equipment (b) Schematic diagram of equipment and specimens

Fig. 1 Rapid freeze-thaw testing equipment

(Note: 1-Specimen; 2-Specimen for temperature monitoring; 3-concrete cube; 4-Thermocouple; 5-Water; 6-Antifreeze fluid; 7-Ruber box; 8-Container)

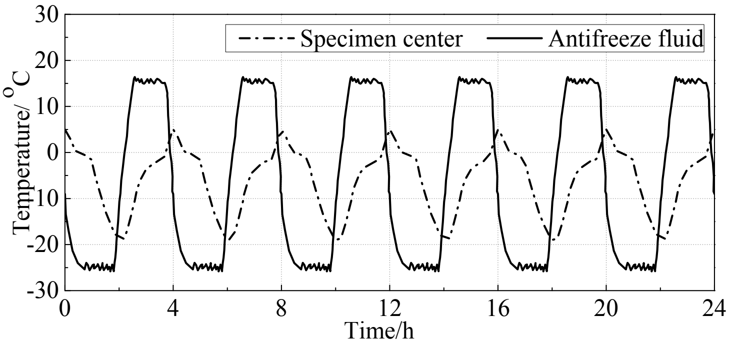
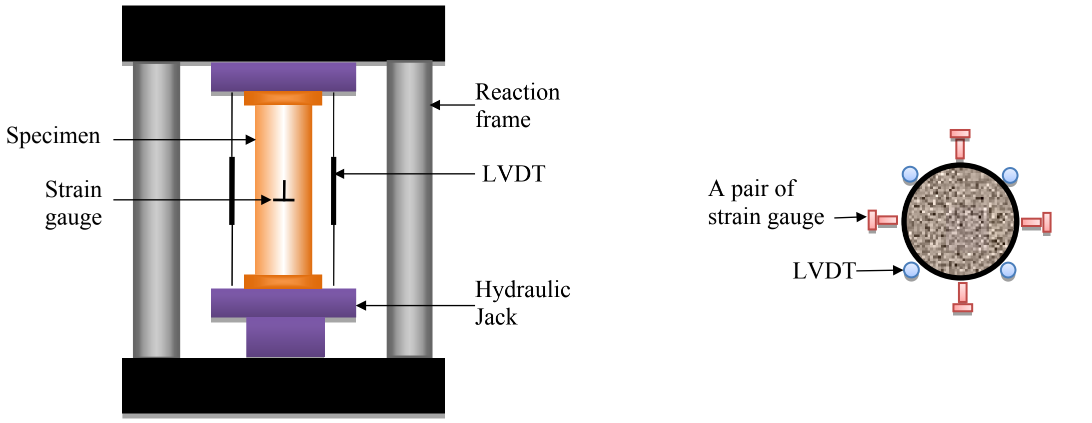
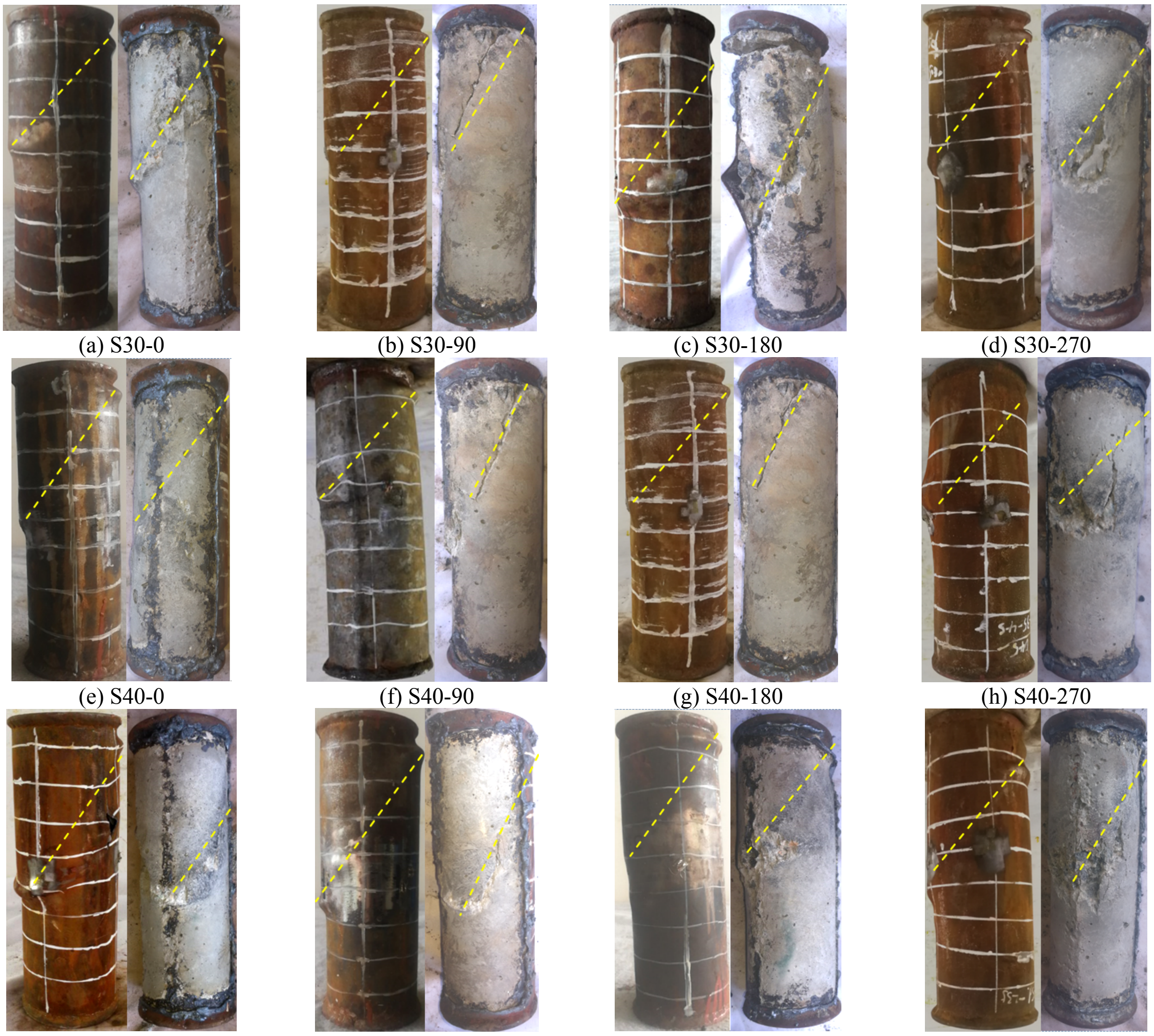


Fig. 2 Freeze-thaw cycles within 24 hours



(a) Test setup (b) Measure instruments

Fig.3 Test setup and measure instruments

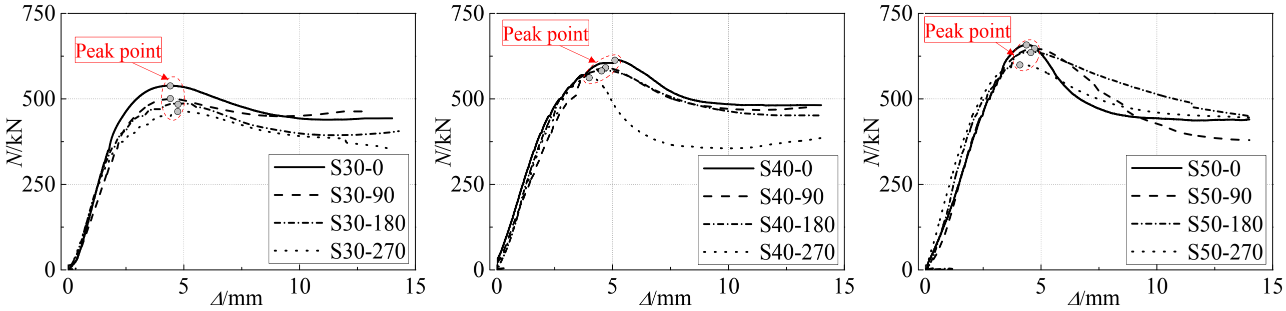


(a) S30-0 (b) S30-90 (c) S30-180 (d) S30-270

(e) S40-0 (f) S40-90 (g) S40-180 (h) S40-270

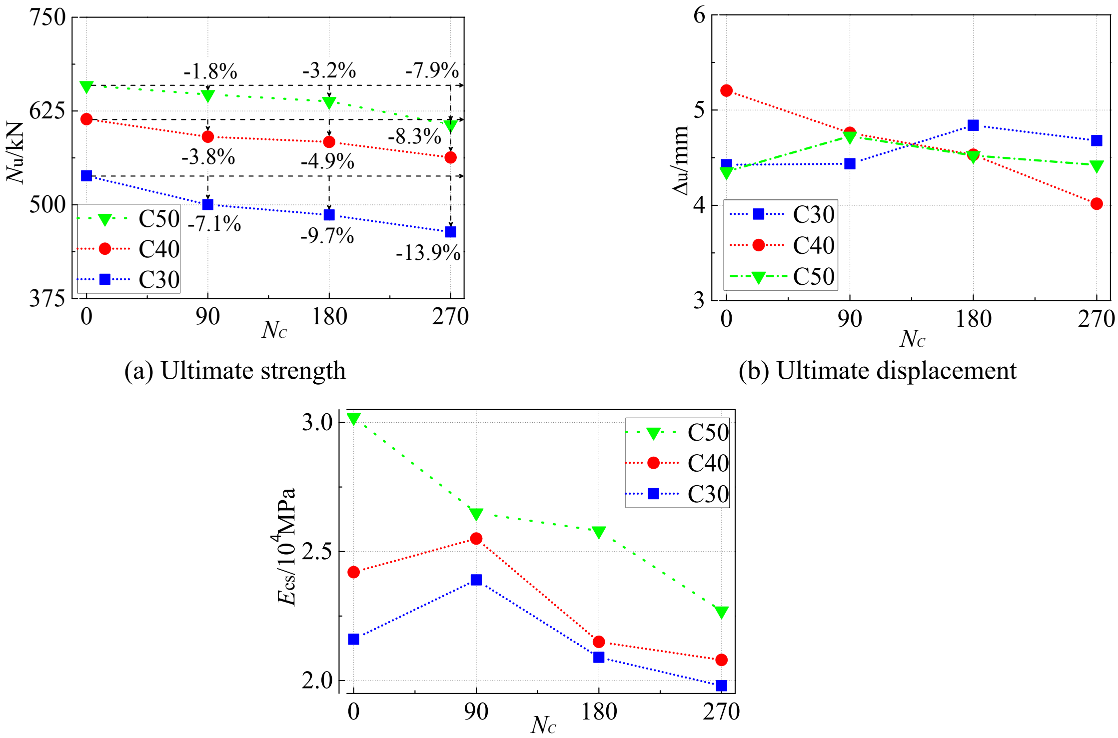
(i) S50-0 (j) S50-90 (k) S50-180 (l) S50-270

Fig. 4 Failure patterns of specimens



(a) C30 specimens (b) C40 specimens (b) C50 specimens

Fig. 5 Load-displacement curves of CFST columns



(a) Ultimate strength

(b) Ultimate displacement

(c) Elastic modulus

Fig. 6 Performance comparison of specimens after freeze-thaw cycles

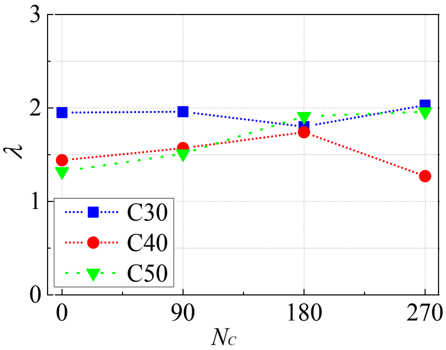
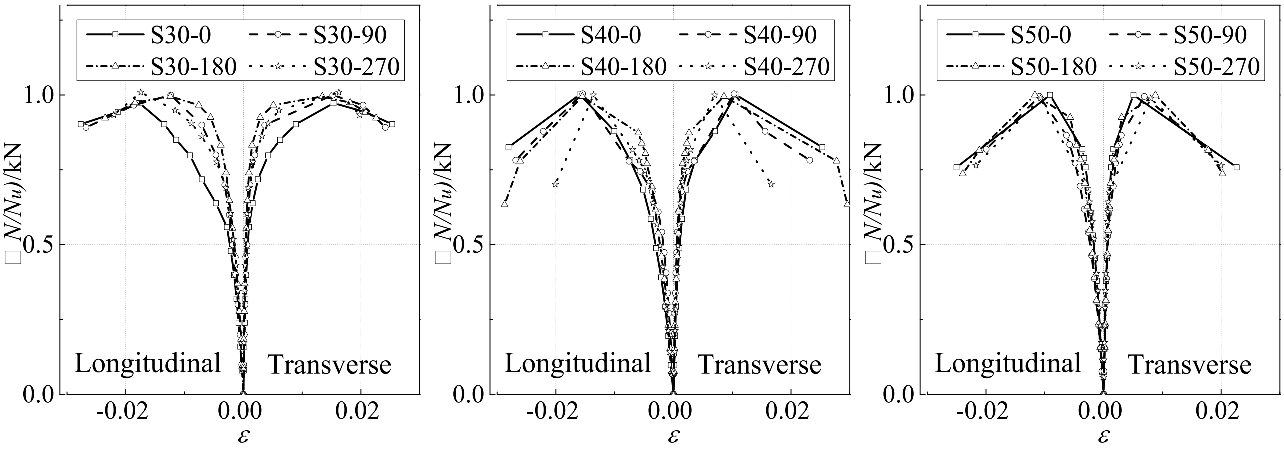
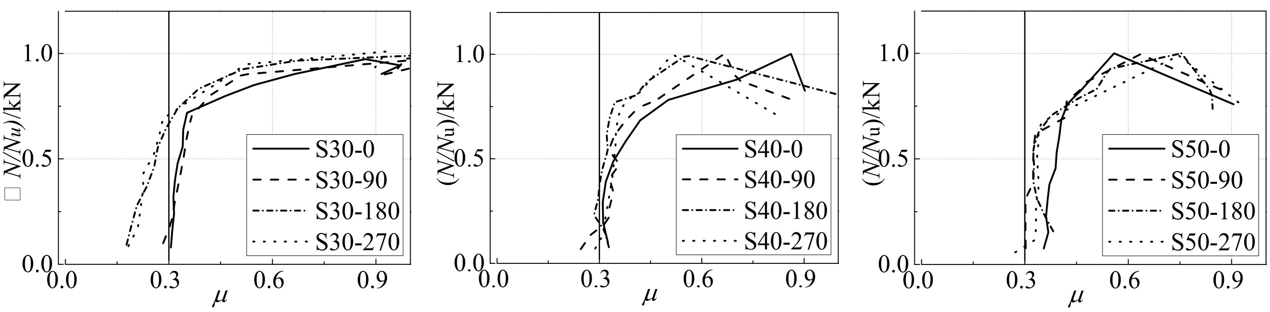


Fig. 7 Relationship between ductility index and N_C



(a) C30 specimens (b) C40 specimens (b) C50 specimens

Fig. 8 Load-strain relationship of specimens



(a) C30 specimens (b) C40 specimens (b) C50 specimens

Fig. 9 Lateral deformation factor of specimens

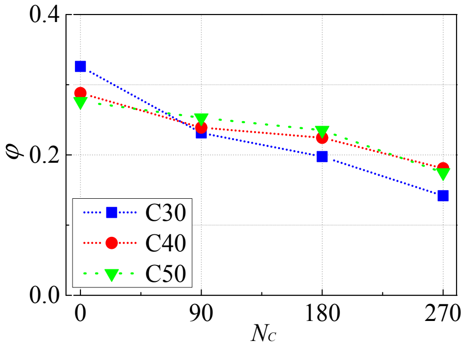


Fig. 10 Relationship between φ and N_C

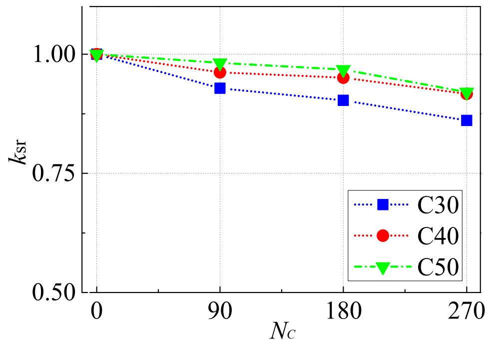
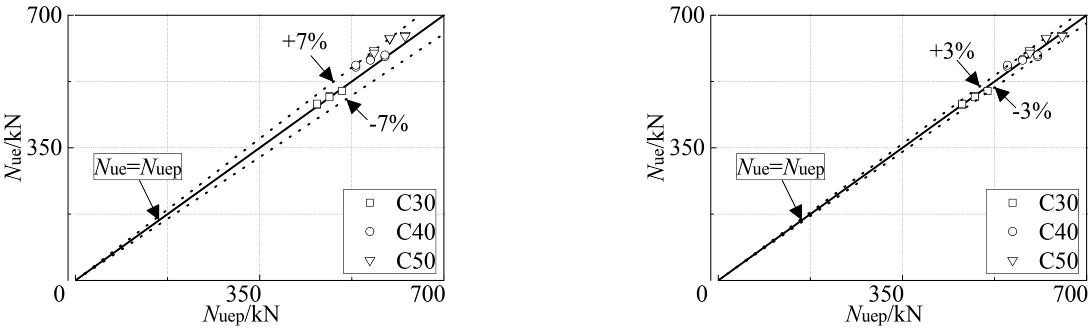


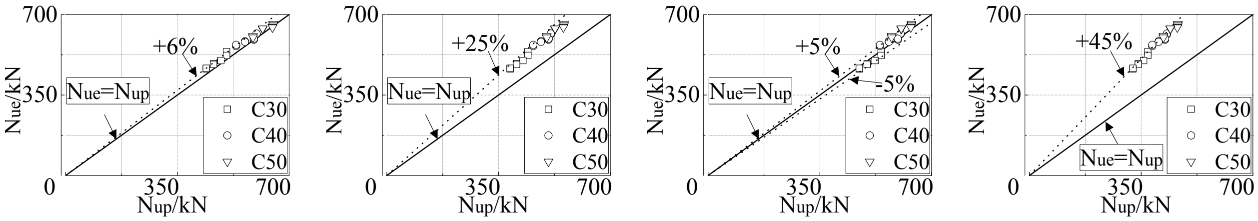
Fig. 11 Influence of Nc and concrete grade on k_{sr}



(a) Based on Eq. (5) (b) Based on Eq. (7)

Fig. 12 Comparison between tested values and predicted values by using k_{sr}

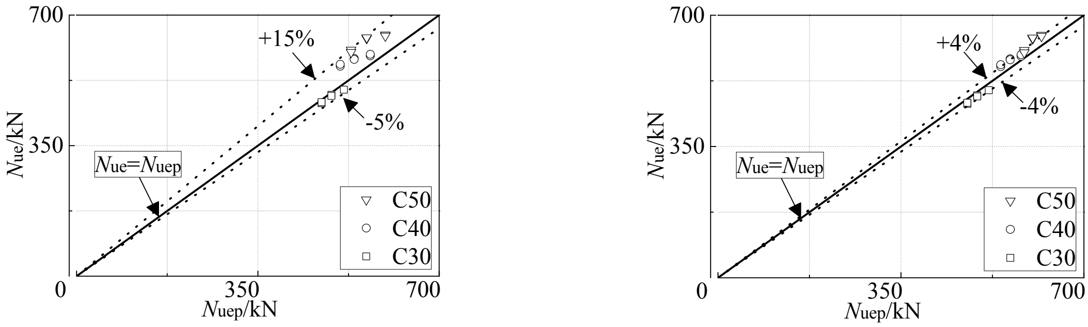
Note: N_{ue} for tested ultimate strength and N_{uep} for predicted ultimate strength based on experimental values



(a) GB50936 (b) AIJ (c) Eurocode 4 (d) AISC360-10

Note: N_{ue} for tested ultimate strength and N_{up} for predicted ultimate strength

Fig. 13 Comparisons between tested and predicted ultimate strength of CFST columns



(a) Based on Eq. (13) (b) Based on Eq. (14)

Fig. 14 Comparison between tested values and predicted values considering concrete strength reduction

Note: N_{ue} for tested ultimate strength and N_{uep} for predicted ultimate strength based on experimental values

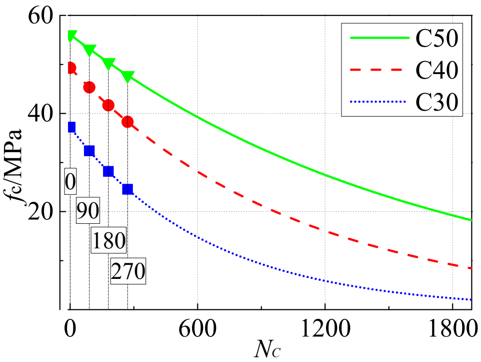


Fig. 15 Strength degradation of core concrete