Some Recent Research on Concrete Filled Stainless Steel Tubular (CFSST) Structures

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Abstract

Concrete filled stainless steel tubular (CFSST) structures have attracted increasing research interests in recent years. This paper reviews some recent research on the behaviour of CFSST columns and joints at both ambient and elevated temperatures. The studies include tests of bond behaviour between the stainless steel tube and core concrete, the static behaviour of CFSST stub columns, slender columns, stainless steel-concrete-carbon steel double-skin tubular columns, and concrete filled bimetallic tubular columns, as well as the cyclic behaviour of CFSST beam-columns under combined axial and lateral cyclic loading. Fire test results of full-scale CFSST columns are presented together with finite element analysis results. The behaviour of composite joints with CFSST columns is also briefly reviewed in this paper.

Keywords

Concrete filled stainless steel tubes; static load; cyclic load; bond strength; fire performance; design approach.

1 Introduction

In recent years, the application of stainless steel in construction has attracted increasing interests among researchers and structural engineers. Compared with conventional carbon steel, stainless steel has several advantages, such as extremely high durability and corrosion resistance, easiness of maintenance and improved fire resistance. However, the high cost of stainless steel prevents its utilisation in real structural projects to some extent. To make more economical use of stainless steel, it is advisable to fill stainless steel hollow sections with concrete to form concrete filled stainless steel tubes (CFSST). Fig. 1 shows typical cross-sections of circular and square CFSST columns, where D is the diameter of the circular steel tube, and B is the width of the square steel tube. Since the material behaviour of stainless steel is different from that of conventional carbon steel, some recent studies have been carried out to investigate the behaviour of CFSST columns before any rational design guidelines can be developed for this type of innovative composite structure. This paper reviews the state-of-the-art of concrete filled stainless steel tubular columns and joints at ambient or elevated temperatures, especially the most recent developments in China and Australia.

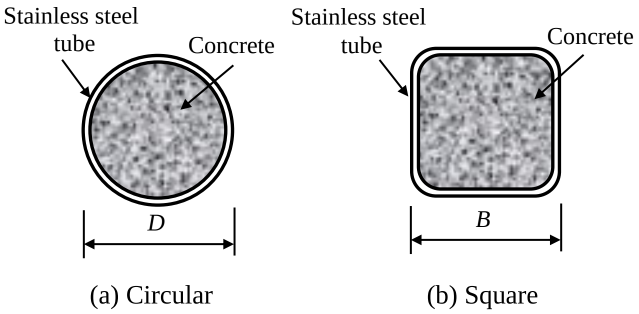


Fig. 1 Typical cross-sections of concrete filled stainless steel tubes.

2 Bond Behaviour of Concrete Filled Stainless Steel Tubes

Bond between the steel tube and core concrete could play a key role in the composite action between the two components. Sufficient bond strength is necessary to ensure the possible shear force transfer in a composite column. However, compared with the inner surface of a carbon steel tube, that of a stainless steel tube is generally smoother since it can be free of rust. This may lead to a reduction in the bond strength in stainless steel composite columns. In order to evaluate the influence of using stainless steel on the bond strength, Tao et al.[1] carried out push-out tests, where the main parameters were the cross-sectional dimension (120–600 mm), steel type (carbon and stainless steels), concrete type (normal, recycled aggregate and expansive concretes), concrete age (31–1176 days), and interface type (normal interface, interface with shear studs and interface with an internal ring). Before filling concrete, values of the average surface roughness (R_a) were measured for typical steel tubes. It was found that the R_a-value of a stainless steel tube was only about a half of that of the carbon steel counterpart. For this reason, the measured bond strengths between the stainless steel tube and concrete in CFSST columns decreased by 32% to 69% compared with the bond strengths in conventional CFST columns, as shown in Fig. 2. Therefore, the bond strength reduction in CFSST columns may need to be considered when there is a need of potential load transfer between the stainless steel tube and concrete. To enhance the bond strength, several approaches have been proposed in [1], such as welding internal ring(s) onto the inner surface of the steel tube, welding shear studs and using expansive concrete. Welding internal rings is the most effective method, followed by welding shear studs and adopting expansive concrete. However, further research is required to develop design guidelines to facilitate the use of these methods. Song et al.[2] carried out further tests to investigate the post-fire bond behaviour of CFSST columns, where the specimens were heated in the furnace to a target temperature of 800 °C before conducting the push-out test. They found that the bond strength of CFSST specimens was generally lower than that of the CFST specimens after fire. However, when the concrete age was relatively long (over six months), the influence of steel type on the bond strength was reduced due to the influence of concrete shrinkage.

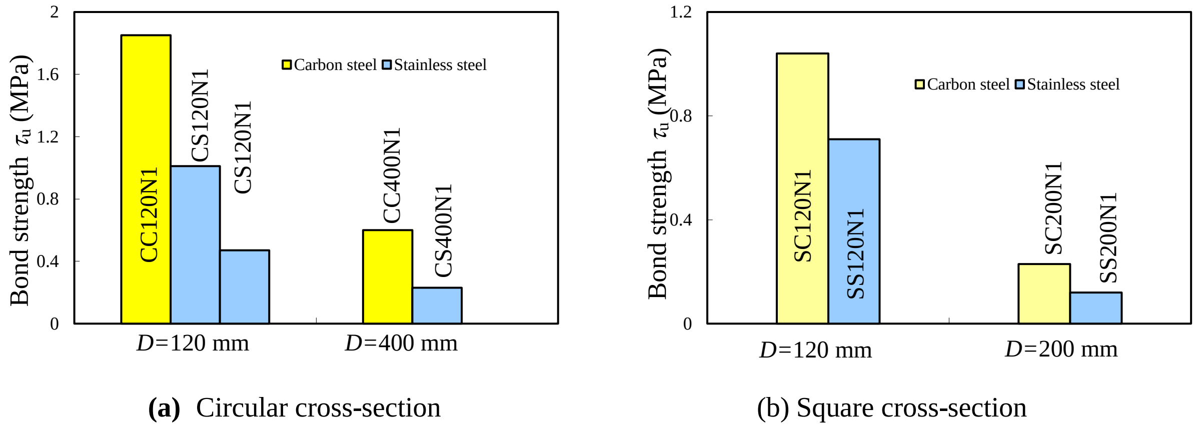


Fig. 2 Comparisons of bond strength between CFSST and CFST columns.

3 Static Behaviour of CFSST Columns

In order to comprehensively investigate the static behaviour of CFSST columns, an experimental study on 117 specimens was carried out, including 60 tests on short CFSST columns under axial compression or combined actions of compression and bending, 24 CFSST slender columns and 33 reference short empty stainless steel hollow sections[3]. The test results revealed that the failure modes of CFSST column are generally similar to those of conventional carbon steel CFST columns. However, due to the increased ductility, the stainless steel composite columns showed far higher capacity of axial deformation and larger amplitudes of local outward bulges.

Fig. 3 illustrates the measured typical axial load versus axial strain (N–ε) curves of the CFSST stub columns, where the axial load is normalised with respect to the maximum load N_{max}. Generally, the N–ε responses could be classified into three types, which depend mainly on the confinement of stainless steel tubes to concrete. If the confinement was strong enough, the N–ε relationship showed a strain-hardening response (Type A) with continuous strength increase from Point 1 to Point 2. As less confinement was provided, Type B curve had a strain-softening portion 1′2′ after reaching the first peak Point 1′. Because of the strong strain-hardening effect of stainless steel, the load increased once again to Point 3′ at the end of the test. Type C is the typical N–ε relationship with a strain-softening response which is very common for conventional carbon steel CFST stub columns. Generally, the residual strength of the "Type C" CFSST stub column was much higher than that of a carbon steel composite counterpart. It is evident that the stainless steel tube could provide better confinement for its core concrete at the late loading stage compared with the carbon steel tube in a CFST column.

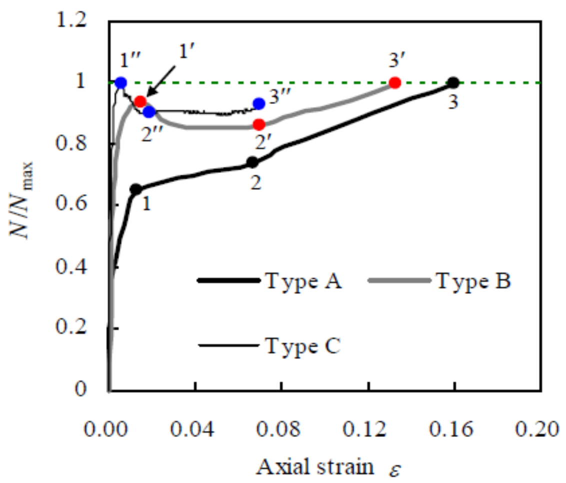


Fig. 3 Typical axial load (N) versus axial strain (ε) curves of CFSST stub columns.

In order to examine the feasibility of using existing design codes to predict the ultimate strength of CFSST columns, the predictions from the Australian design code AS 5100[4], American code ANSI/AISC 360-05[5], Chinese code DBJ/T 13-51-2010[6] and Eurocode 4[7] were compared to the test results[3]. It was evident that all codes were conservative in predicting the load-carrying capacity of CFSST columns. For short columns under axial compression, AS 5100 gives the best predictions for circular columns whilst DBJ/T offers the closest predictions for square columns. Meanwhile, all codes underestimate the capacity by 47–67% for short columns under compression and bending and about 11.1–25.5% for slender columns, respectively.

The above studies focused on CFSST columns with regular circular or square sections shown in Fig. 1. Han et al.[8] proposed another innovative type of composite members with a stainless steel jacket, i.e., stainless steel-concrete-carbon steel double-skin tubular (DST) columns, as shown in Fig. 4, where b and d are dimensions of the inner carbon steel tube; B and D are dimensions of the outer stainless steel tube; t_{si} and t_{so} are the wall thicknesses of the inner and outer tubes, respectively. Compared with the conventional concrete filled double skin tubes, the use of the outer stainless steel tube will increase the corrosion resistance and aesthetics of the composite column. An experimental investigation was conducted on 80 specimens with different column types (straight, inclined and tapered) and cross-sectional types (circular, square, round-end rectangular and elliptical). The test results indicated that the stainless steel-concrete-carbon steel DST columns generally failed in a ductile manner with the outward local buckling of the outer stainless steel tubes and the inward buckling of the inner carbon steel tubes. It was evident that the inclined angle has a moderate influence on the load-carrying capacity of the inclined DST columns. However, the strength tended to decrease with the increase of the tapered angle for the tapered DST columns. A simplified model was also proposed in [8] for predicting the ultimate strength of stainless steel-concrete-carbon steel DST stub columns.

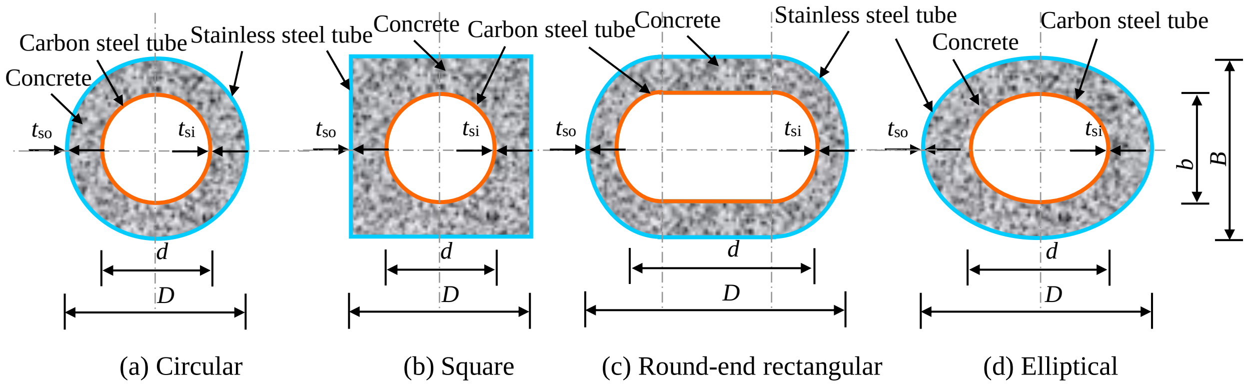


Fig. 4 Schematic view of stainless steel-concrete-carbon steel DST sections.

With the purpose of further utilising the advantages of stainless steel and compensating for its high cost, Ye et al.[9] proposed an innovative concrete-filled bimetallic tube (CFBT) construction, where the cross section of the bimetallic tube is composed of an outer layer made of stainless steel and an inner layer made of carbon steel. A serial of experimental and numerical investigations were conducted in [9, 10] on the behaviour of CFBT stub columns. It was found that the CFBT columns failed in a ductile manner, and the outer stainless steel tube layer could work well together with the inner carbon steel tube layer during the whole loading process. The two layers of the bimetallic tube generally buckled at same positions at failure. Three dimensional FE model was established to simulate circular CFBT stub columns subjected to axial compression[10]. A simplified model was then put forward to predict the ultimate strength of circular CFBT stub columns.

A confinement factor (ξ) was defined and used to describe the passive confinement of steel tubes on their concrete infilled, i.e.,

$$\xi = \frac{A_s}{A_c} \cdot \frac{f_y}{f_{ck}} \tag{1}$$

where A_s and A_c are the cross-sectional areas of the steel and concrete, respectively; f_y is the yield stress of steel; and f_{ck} is the characteristic strength of the concrete. It was found that the core concrete tends to exhibit more ductile behaviour as ξ increases in a CFSST column[11, 12]. Fig. 5 shows a typical axial load (N/N_u) versus moment (M/M_u) interaction curve to predict the load-carrying capacity of CFSST columns, where N_u and M_u are the axial compressive capacity and bending moment capacity, respectively.

4 Cyclic Behaviour of CFSST Columns

Concrete filled stainless steel tubes also have good potential to be used in earthquake-prone zones. Information about the seismic performance of CFSST columns is of interest to structural engineers. Liao et al.[12] carried out a serial of tests on CFSST columns under constant axial load and cyclic lateral loading. The tested CFSST columns showed very plump lateral load versus lateral deflection (P–Δ) responses, indicating a high energy dissipation ability of the composite columns. Compared with conventional carbon steel CFST columns, the attained lateral displacements of CFSST columns are much higher whilst tensile fracture was less likely to occur after the application of the cyclic loading.

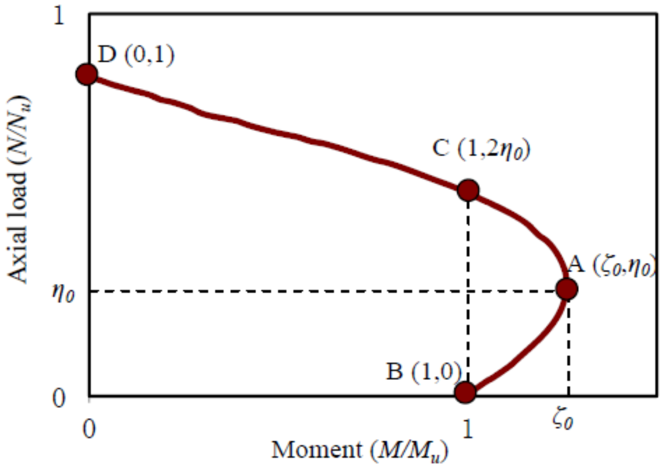
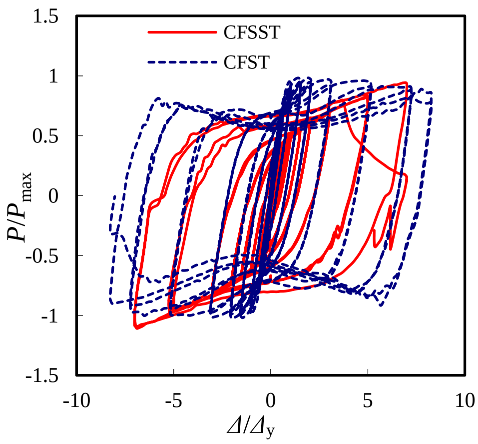
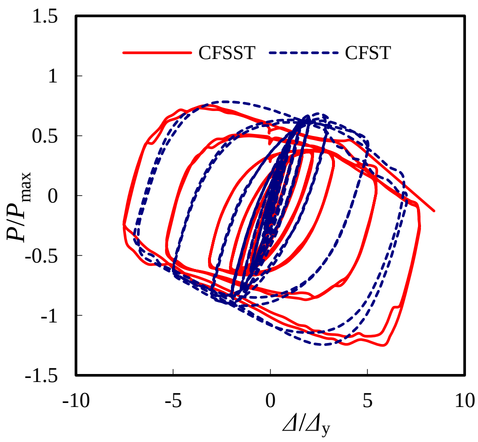


Fig. 5 Typical interaction curve of CFSST columns.

Fig. 6 compares the P–Δ hysteretic responses of CFSST and CFST columns, where the confinement factors (ξ) of them were selected to be close. Since the sizes of the columns were not exactly the same, the lateral load P and lateral deflection Δ were normalised with respect to the maximum load P_{max} and yield displacement Δ_y, respectively, to have a meaningful comparison. Both types of composite columns showed very plump hysteretic hoops without significant pinching effect. In general, the CFSST column displayed better energy dissipation ability under cyclic loading as compared with the CFST column. The axial load level was a crucial factor affecting the cyclic behaviour of composite columns. If a column was subjected to a large axial compressive load, it was the core concrete rather than the steel that determined the performance of the composite members. Thus, the difference between the CFST and CFSST columns were generally negligible in terms of P–Δ hysteretic curves when a high axial load level of 0.6 was applied, as shown in Fig. 6(b). However, at a small axial load level (n=0.02), the performance of a column was more significantly affected by the tensile behaviour of the steel. Therefore, the CFSST column with n of 0.02 exhibited more obvious hardening characteristic compared with the carbon steel counterpart, as shown in Fig. 6(a). After reaching the peak load, the lateral load of the CFST column tended to decrease gradually, whereas that of the CFSST column still experienced significant hardening even under large lateral deformation. Moreover, a slight pinching effect was observed for the CFST column at a late loading stage, but that was not found for the CFSST column. This phenomenon is mainly attributed to the significant strain hardening effect of the stainless steel.



(a) n=0.02



(b) n=0.6

Fig. 6 Comparison of lateral load versus lateral deflection hysteretic curves between CFSST and CFST columns.

The American Specification ANSI/AISC 360-10[13], Chinese code DBJ/T 13-51-2010[6], and Eurocode 4[7] were employed to predict the ultimate strength and flexural stiffness of the tested specimens[12]. It is evident that all the three codes underestimated the load-carrying capacities of CFSST columns under combined axial force and bending moment. The EC4 method gave the closest predictions for both circular and square CFSST columns. In predicting the flexural stiffness, the standard deviations are large for all design approaches because the dependency of the axial load is not accounted for. The AISC-360-10 and DBJ/T specifications optimistically predict the initial section flexural stiffness for n=0.02, whereas the EC4 prediction is conservative. For higher axial load levels, the initial section flexural stiffness values are conservatively predicted by all the three specifications. For serviceability-level section flexural stiffness, the DBJ/T method gives the closest predictions for both the circular and square CFSST specimens.

5 Fire Performance of CFSST Columns

Fire performance of concrete filled stainless steel tubes should be considered in structural design. Han et al.[14] conducted an experimental investigation on five full-scale concrete filled stainless steel tubular (CFSST) columns in standard fire test conditions, where the length of the CFSST columns (L) was 3600 mm and the largest cross-sectional dimension was 630 mm. Grade S30408 (EN 1.4301 or AISI 304) austenitic stainless steel was used to manufacture the outer steel tubes which was infilled with self-consolidating concrete. The load level ($n_f=N_F/N_u$, where N_F is the applied axial compression load; N_u is the load-bearing capacity of the column at ambient temperature) of the tested CFSST specimen ranged from 0.15 to 0.45. The tests were carried out in a column furnace in Tianjin Fire Research Institute, China. The temperatures in the furnace were controlled in accordance with ISO 834 standard fire curve. Fig. 7 presents the failure modes of the specimens after exposed to fire. For square CFSST specimens, local buckling appeared along the whole column length, where weld fracture of the stainless steel tube was observed at the place where severe local buckling occurred. For CFSST specimens with circular sections, local buckling of the steel tube was also observed, whereas the mode of buckling was different from that of the square section. Only one major elephant foot bulge was found at the mid-height of the column for the circular specimens. After removing the outer tube, it was evident that the core concrete and the stainless steel tube separated from each other at places where the local buckling had occurred.

A FE model was established in [14] to simulate the structural behaviour of CFSST columns in fire, and also to compare the fire performance between CFSST and CFST columns. It was clear that the fire resistance of the CFSST column is much higher than its CFST counterpart. In a typical calculating example, the fire resistance (R) increases from 48 to 82 min when stainless steel is used to replace carbon steel in a CFST column, mainly due to the superior material properties of stainless steel at elevated temperatures. A series of parametric studies were also performed by using the FE model, and two design tables (Tables 1 and 2) were then proposed to predict the fire resistance of CFSST columns, where the slenderness ratio λ is determined as $4L/D$ for circular columns and $2\sqrt{3}\,L/B$ for square columns.

Tao et al.[15] also carried out tests on CFSST columns in fire and after fire exposure. The sectional size of the tested specimens was 200 mm and the load level ranged from 0.28 to 0.48. A total of 12 specimens were tested, including 6 CFSST columns in fire and another 6 post-fire CFSST columns. A three-dimensional FE model was also developed in [15] by introducing the measured initial imperfections and load eccentricities. It once again confirms that the fire resistance of carbon steel composite columns is much lower than that of their CFSST counterparts, as shown in Fig. 8, highlighting the benefits of using stainless steel. Meanwhile, it was found that the post-fire strength of circular CFSST specimens was not sensitive to fire exposure, whilst a strength loss up to 36% was observed for square specimens.

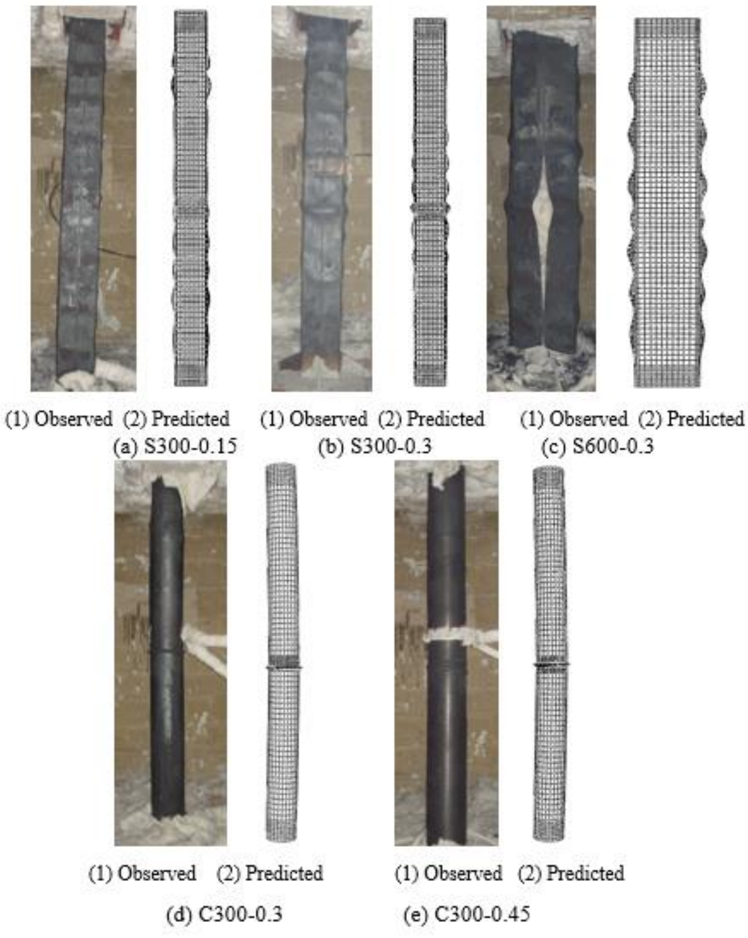


Fig. 7 Failure modes of specimens in full scale fire tests.

Table 1 Design fire resistance (*R*) of circular CFSST columns (unit: min)

Slenderness ratio λ	Sectional dimension *D* (mm)	Load level n_F 0.3	0.4	0.5	0.6
20	300	110	70	45	25
	600	170	95	65	40
	900	230	130	85	50
40	300	80	55	35	25
	600	125	75	50	35
	900	150	90	60	45
60	300	65	45	30	20
	600	80	50	40	30
	900	100	60	50	40

Table 2 Design fire resistance (*R*) of square CFSST columns (unit: min)

Slenderness ratio λ	Sectional dimension *B* (mm)	Load level n_F 0.3	0.4	0.5	0.6
	300	110	70	45	25
17.3	600	170	95	60	35
	900	220	120	75	45
	300	95	65	45	25
34.6	600	145	80	55	35
	900	160	95	65	45
	300	75	50	35	20
52	600	95	65	45	30
	900	125	85	60	40

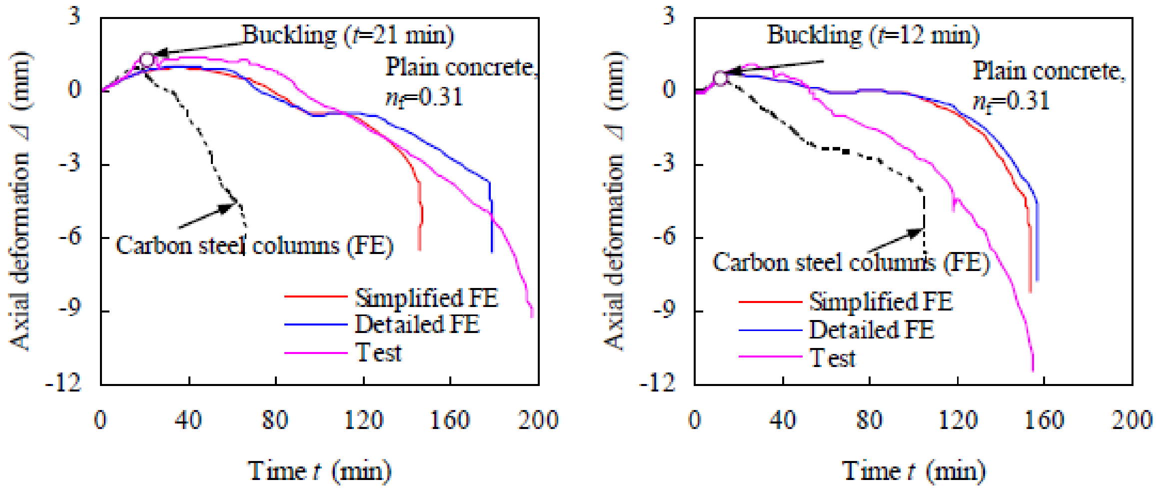


Fig. 8 Axial deformation (*Δ*) versus time (*t*) curves for fire-resistance test specimens.

6 CFSST Column-to-beam Joints

If CFSST columns are applied in real buildings, reliable beam to column joints are crucial for transferring beams' loads to columns and ensuring structural safety. Tao et al.[16] carried out tests on seven full-scale joints with blind bolted connections to CFSST columns. In the panel zone, binding bars were used in three joint specimens with square columns to tie the opposite surfaces of the steel tube together. This aimed to increase the integrity of the panel zone. It was evident that the application of binding bars in the connection tended to increase the joint stiffness and strength, and reduce the separation between the steel tube and concrete. By adding binding bars, the ultimate hogging moment capacity was improved by 10.7%, whilst the initial stiffness was improved by 62.5%. According to Eurocode 3[17], the blind bolted joint without composite slab could be classified as nominally pinned joint. However, in the presence of the floor slab, the joint nearly reached its full strength and the stiffness was also significantly increased close to the limit for rigid sway frames. In general, the adoption of stainless steel or carbon steel for the column had very minor influence on the joint behaviour. Song et al.[18] conducted fire tests on eight full-scale blind bolted joints with CFSST columns in the standard fire condition. The test results indicated that the blind bolted joint demonstrated very good performance in fire, and no bolt shank fracture or bolt pull-out failure was observed in the test. When the steel beam was protected, the fire resistance times of the blind bolted joints with CFSST columns ranged from 72 to 90 min. Meanwhile, it was found that the presence of the binding bars or the type of the steel tube (carbon or stainless steel) had only moderate influence on the fire resistance.

7 Concluding Remarks

This paper reviewed some recent research on concrete filled stainless steel tubes (CFSST). From the previous studies, it could be concluded that CFSST columns generally show improved ductility, higher energy dissipation ability, and superior fire performance compared with conventional concrete filled steel tubes (CFST) with carbon steel. However, the bond strength between the steel tube and core concrete of a CFSST is 32–69% lower than that of a CFST due to the smoother surface of the stainless steel. This issue should be considered if a possible load transfer occurs between the steel tube and concrete via bond. If required, suitable measures can be taken to increase the bond strength, such as welding internal rings and shear studs.

The existing codes for CFST columns with carbon steel, such as AS5100, AISC, DBJ/T and EC4, all underestimate the load-carry capacity of CFSST columns, mainly due to the fact that the strain hardening characteristic of stainless steel has not been beneficially considered. Further research needs to be carried out to provide more comprehensive design approaches for concrete filled stainless steel tubes. The feasibility of connecting CFSST columns to carbon steel beam by blind bolts has been examined, and the results showed that the composite joint exhibited favourable performance both at ambient temperature and in fire.

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