Rectangular concrete-filled steel tubular beam-columns using high-strength steel: Experiments and design

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A B S T R A C T

Concrete-filled steel tubular (CFT) columns and high-strength steel have been increasingly used in construction. However, the application of high-strength steel in CFT columns has not been permitted in many design codes. This paper reports an experimental investigation on the behavior of rectangular CFT beam-columns using high-strength steel. The influences of the in-fill concrete, eccentricity ratio and the width-to-thickness ratio on the resistance of the test columns are discussed. The contact behavior is studied using the finite element analysis.

1. Introduction

Concrete-filled steel tubular (CFT) columns have been used in many engineering structures such as bridges, high-rise buildings, residential structures and other industrial structures [1]. The CFT columns can provide high capacity, favorable ductility and large energy absorption capacity compared with the reinforced concrete structures. The strength of the steel and the concrete may be well used in CFT columns owing to the interaction effect. The interaction mechanism includes that the concrete increases the critical local buckling stress of steel tube, and the steel tube provides confinement to the in-fill concrete.

Previous researches on CFT columns focused on the square and circular sections. However, the rectangular CFT columns with different cross-section aspect ratios have their advantages over the square and circular CFT columns. They can provide different flexural resistance in two directions which may enable these members to be further beneficial in optimizing structural design, and rectangular CFT columns are also convenient for connection design which is superior to circular columns. Fig. 1(a) shows a typical application of square CFT columns in frame structures. The design moments are different in two directions. Fig. 1(b) illustrates a beam-column joint with the rigid connection in one direction to resist the major moment and the simple connection in another direction. Design of the structures may be optimized by using rectangular CFT columns in the above situation.

As the increase of the height of buildings, the column size is becoming much larger. To make the cross-section area of columns smaller, research interest in the behavior of CFT columns using high-strength steel (yield strength ≥ 460 MPa) has been increasing. Several experimental programs and numerical analysis had been conducted on rectangular CFT columns made from high-strength steel under axial load in recent years, for example, Sakino [2], Uy [3,4,5], Liu [6,7,8], Liew [9] and Aslani [10]. However, much less experimental programs have been found on rectangular CFT columns made from high-strength steel under combined axial compression and bending due to the complexity of the test setups. Liu [11,12], Varma [13,14], Fujimoto [15] and Uy [15,16] presented the test programs on rectangular (mainly square) CFT columns under combined axial compression and bending. The rectangular CFT columns may behave differently owing to the effects of aspect ratio and the high-strength steel which is susceptible to local buckling. The previous researches are not sufficient to investigate the behavior of rectangular CFT columns incorporating high-strength steel.
2. Design approaches of rectangular CFT columns

Several design codes cover the methods of calculating the capacity of rectangular CFT beam-columns. The representative design codes are EC4 [17], AISC 360 [18] and GB 50936 [19], etc. In China, there are also regional design specifications for building design including DB 29-57 [20]. It is worth noting that the design approaches adopted in the abovementioned codes/specifications are different from each other. EC4 [17] adopts the plastic stress distribution method. The method is based on the assumption of linear strains across the cross section and the plastic behavior. It assumes that the concrete has reached its crushing strength in compression on a rectangular stress block. AISC 360 [18] provides two different approaches (Method 1 and Method 2) to design rectangular CFT beam-columns. The Method 1 is based on the interaction equations of steel columns which provide a conservative assessment of the available strength of the columns for combined axial compression and bending. The Method 2 is based on developing interaction surfaces at the nominal strength level using the plastic stress distribution method. The Method 1 provides more conservative results than the Method 2. The design of noncompact and slender CFT columns is limited to the use of the Method 1. GB 50936 [19] assumes...
that the CFT columns are one uniform material and adopts the empirical equations to calculate the axial strength, flexural strength and the stiffness of CFT columns. The GB 50936 does not cover the design of rectangular CFT columns, so the rectangular CFT columns have to be converted to square CFT columns with the same cross-sectional area. DB 29-57 [20] ignores the interactions between steel and concrete because the confinement effect of rectangular CFT columns is not significant. The concrete was assumed to bear the axial load only, and the steel withstands the bending moment and part of the axial load. Hence, only the steel tubes are checked in the formulas when the columns are subjected to combined axial compression and bending. The bearing capacity of the columns subjected to combined axial compression and bending is given by the following simplified equations in DB 29-57:

\[
N \leq \phi_x f_y A_s + f_c A_c / C_{16}/C_{17} \quad (1)
\]

\[
\frac{N^2}{\phi_x A_s} + \frac{M_x}{W_x(1 - 0.8N/N_{EX})} \leq f \quad (2)
\]

\[
\alpha_s = \frac{f_y A_s}{f_c A_c + f_y A_s} \quad (3)
\]

\[
N_s = \alpha_s \cdot N \quad (4)
\]

In the equations, \( N \) is the design axial force; \( \phi_x \) is the reduction factor accounting for the length effect about \( x \) axis; \( f_y \) and \( f_c \) are the steel and the concrete strength, respectively; \( A_s \) and \( A_c \) are the cross-sectional areas of the steel and concrete, respectively; \( N_s \) is the design axial load sustained by the steel tube; \( M_x \) is the design bending moment about the \( x \) axis; \( W_x \) is the section modulus of the steel tube about \( x \) axis; \( N_{EX} \) is the Elastic critical normal force determining by \( \pi^2 E_s A_s / \lambda_x^2 \); \( E_s \) is the Young’s modulus of the steel; \( \lambda_x \) is the slenderness ratio of columns about \( x \) axis; \( A \) is the equivalent area of composite columns calculating by \( A_s + A_c \cdot f_c / f_y \); \( \alpha_s \) is the proportion of load sustained by the steel tubes.

The foresaid codes specify the width-to-thickness (\( h/t \)) ratio limitations and steel strength limitations for designing rectangular CFT beam-columns. EC4 [17] specifies a strict width-to-thickness ratio limitation \( h/t \leq 52/\sqrt{235/f_y} \) and the steel strength should not be \( > 460 \) MPa. The rectangular CFT columns are categorized as compact, noncompact and slender depending on the width-to-thickness ratio limitations (\( \lambda_p \), \( \lambda_r \) and \( \lambda_{max} \)) as specified in AISC 360 [18]. The specific values of the limitations are presented in AISC 360 Section II.4 [18]. The AISC 360 does not permit the use of high-strength steel in rectangular CFT columns as presented by Lai and Varma [21]. Chinese national standard GB 50936 [19] specifies that the width-to-thickness ratio should not exceed \( 60\sqrt{235/f_y} \) for compression members and \( 135\sqrt{235/f_y} \) for flexural members. DB 29-57 [20] stipulates the width-to-thickness ratio should

![Fig. 3. Illustration of test columns.](image)

### Table 1

<table>
<thead>
<tr>
<th>Coupon labels</th>
<th>Thickness (mm)</th>
<th>( f_y ) (MPa)</th>
<th>( f_u ) (MPa)</th>
<th>( E_s ) (MPa)</th>
<th>Yield ratio ( f_y / f_u )</th>
<th>( \Delta % )</th>
<th>Yield strain (( \mu ))</th>
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be smaller than \( \sqrt[235]{2235/f_y} \), for rectangular CFT columns subjected to axial compression or flexural load. Both GB 50936 and DB 29-57 limit the steel strength grade up to Q420 (nominal yield strength 420 MPa).

As discussed above, further experimental and theoretic investigations are needed to study the behavior of rectangular CFT beam-columns using high-strength steel. This study presents an experimental program and the analysis results on rectangular CFT beam-columns using high-strength steel. The available current design codes are evaluated by the test results. A new method is proposed to allow for the fully-plastic state of steel in bearing combined axial and bending load. The proposed method is verified by the test results and the design codes to be reasonably conservative. The steel strength in this study exceeds the limitation of AISC 360, GB 50936 and DB 29-57. This study can serve as the foundation of modifying the current standards to consider the effect of high-strength steel in rectangular CFT columns.

3. Experiment program

3.1. Material strength

To determine the stress-strain relationship of the Q460 steel plates in tension, three 20 mm wide coupons were produced from the original plates and tested under a 100 kN universal testing machine. The stress-strain curves of Q460 steel are shown in Fig. 2. The results for steel coupon tests are concluded in Table 1. \( f_y \) is the yield strength and \( f_u \) is the ultimate tensile strength. \( E_s \) is the Young's modulus and \( \Delta \% \) is the percentage elongation after fracture. The yield strength is 514.5 MPa in

### Table 2

Details of test specimens.

<table>
<thead>
<tr>
<th>Specimen no.</th>
<th>( f_y ) (N/mm²)</th>
<th>( f_u ) (N/mm²)</th>
<th>( \lambda )</th>
<th>( h ) (mm)</th>
<th>( b ) (mm)</th>
<th>( t ) (mm)</th>
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<th>( h/t )</th>
<th>( e_p ) (mm)</th>
<th>( N_u ) (kN)</th>
<th>( M_u ) (kN·m)</th>
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Fig. 4. Illustration of test setups.
average. The yield ratio reaches 0.8028 which is higher than that of mild carbon steel. The yield strains are also shown in Table 1 to determine the yielding of steel during the test. The backing plates for welds adopted the mild carbon steel with the yield strength 395 MPa. The bearing capacity of the backing plates is taken into account in the following analysis.

Two types of concrete (C40 and C50 grade concrete) were utilized in the specimens. To determine the compressive strength of concrete, six 150 mm cubes were cast and cured in environments similar to that of the test specimens. The standard cubic strength ($f_{cu}$) of the concrete is 55.3 MPa and 43.2 MPa, respectively.

### 3.2. Design of the specimens

The columns were fabricated by four steel plates using butt weld as shown in Fig. 3(a). Four 30 × 60 mm backing plates were placed next to the corner for better welding quality. An internal vibrator was carefully applied to consolidate the concrete. Two cover plates were welded to the ends of the columns using fillet weld. Two steel bars were welded onto the cover plates to determine the loading location as illustrated in Fig. 3(b). Special attention was paid to the accurate centering and perpendicularity of the cover plates and the steel bars. The loading line was located in the middle of the two steel bars. The distance between the center line and the loading line was the eccentricity distance $e_0$, see Fig. 3(c). The different eccentricity distance was designed by changing the location of the steel bars.

The N-M interaction behaviors are affected by a series of parameters, namely, the material constitutive-curve, the aspect ratio, the width-to-thickness ratio and the slenderness of columns. In this test series, 23 specimens with different parameters were tested. The details of the specimens are shown in Table 2. The tests were labeled by a steel number-concrete strength-tube sections-eccentricity ratio convention. For example, specimen HS1C40SA-0.2 corresponds to the rectangular CFT column made of HS1 (Q460 grade steel) and C40 grade concrete with Section A and a nominal eccentricity ratio of 0.2. Section A (120 × 100 mm), section B (180 × 12 0mm) and section E (243 × 135 mm) were designed to evaluate the effect of different width-to-thickness ratio and aspect ratio. The length ($h$), width ($b$) and thickness ($t$) of the cross-sections are shown in Table 2. Eccentricity ratio was taken as $2e_0/h$ in this test, and $e_0$ is the eccentric distance which was measured before the test as shown in Table 2. $e$ is calculated using Eq. (3) including the strength of the backing plates in the calculation. The slenderness ratio ($\lambda$) is obtained using the following equations:

$$\lambda = \frac{l_e}{l_x} \tag{5}$$

$$i_x = \frac{l_x^2 + l_{xx} F_c / F_y}{A_x + A_c f_c / f_y} \tag{6}$$

In the equations, $\lambda$ is the slenderness ratio of columns about the principle $x$ axis, $l_e$ is the effective length about the principle $x$ axis, and it is defined as the distance between the tips of the edge, $l_{xx}$ and $l_{xx}$ are the second moment of areas of the steel section and the concrete section about principle $x$ axis, respectively. $E$ is the Young's modulus of concrete adopting the value specified in GB 50010 [22]. $A_x$ and $A_c$ are the sectional areas of steel and concrete, respectively.

### 3.3. The test setups

The maximum interaction strength of a beam-column in this study was reached by increasing both the axial and flexural loads
The columns were tested as pin-ended supported using the knife-edge which is shown in Fig. 4. Vertical load was applied through two pairs of knife-edges to each column before loading. Before the tests, white lines were painted on the surface of the steel tubes to highlight the local buckling behavior.

Five linear variable displacement transducers (LVDTs, LVDT1-LVDT5) along the specimen span were used to monitor the deflections of each column, see Fig. 5. Four LVDTs (LVDT6–LVDT9) were installed to measure the axial shortening of the columns. Four strain gages were attached to each flange (flange B and flange D) to measure the longitudinal and lateral strains. Six strain gages including one lateral strain gage and five longitudinal strain gages were utilized to measure the strains of web A at mid height cross-sections as shown in Fig. 5. Two strain gages were attached to web C as the comparison with the strains of web A.

A load interval of 50 kN was used and maintained for about 10 min for the convenience of recording. The local buckling and flexural buckling behaviors were also recorded in the experiments.

4. Experiment results and discussion

4.1. Experimental results

The following characteristics could be recorded during the test:

a. Yielding of compression flange
b. Concrete goes into tension
c. Concrete reaches failure strain ($\varepsilon_c$)
d. Yielding of tension flange
e. Local buckling of compression flange
f. Local buckling of web
g. Crack of steel tube.

Characteristics a, b, c and d were recorded using the longitudinal strain gages at the mid height cross-sections. No slip was assumed between the steel and concrete during the test because of the effect of the cover plates. Hence, the concrete stress close to the flange could be recorded using the strain gages on the steel tubes before local buckling. Characteristics b predicted that both the steel tube tension flange and the concrete went into tension. The concrete failure strain under compression ($\varepsilon_c$) is taken as 0.003 in characteristics c. Characteristics e, f and g are experimental observations.

Four specimens HS1C40SE-0.2, HS1C50SA-0.4, HS1C50SB-0.6 and HS1C0SA-0.4 are selected in the following analysis to evaluate the effect of different eccentricity ratio, width-to-thickness ratio, aspect ratio and infilled concrete.

The schematic diagram of curved columns under eccentric load is shown in Fig. 6. The axial force $N$ and the bending moment $N_0$ were increased proportionally. As a result of the deflection of the specimens,

![Fig. 7. Displacement and strain response of specimen HS1C40SE-0.2.](image)

![Fig. 8. Displacement and strain response of specimen HS1C50SA-0.4.](image)
during the test, the moment at the mid height of the columns was the sum of the moment $N_e$ and the secondary moment $N_δ$. The total bending moment $M_u$ corresponding to the ultimate load is calculated and listed in Table 2.

Specimen HS1C40SE-0.2 had a width-to-thickness ratio 42.63 and an aspect ratio 1.8. As shown in Fig. 7(a), the compression flange B of specimen HS1C40SE-0.2 yielded at 71% of the ultimate load. The concrete reached failure strain at 84% of the ultimate load. The load was still increasing after the buckling of steel tube webs. The stiffness dropped after the yielding of steel compression flange. The concrete went into tension during the descending stage and after the steel tube cracked. The steel tube flange D didn't develop yielding in the compressive or tensile stage. The load of specimen HS1C40SE-0.2 presented a fast drop which was similar to the specimen subjected to axial load.

The strains of web A showed a linear relationship along the cross-section as illustrated in Fig. 8(b). The tensile flange D converted compressive strain into tensile strain before the peak load and developed tensile yielding at the descending stage of loading as shown in Fig. 8(c).

The width-to-thickness ratio and the aspect ratio were respective 31.58 and 41.63 for specimen HS1C50SB-0.6. Fig. 9(a) illustrated the load-mid height deflection and load-axial deformation curves of specimen HS1C50SB-0.6. The concrete at the tensile flange presented tensile strains at the first of loading. Fig. 9(b) shows that the strains of web A were increasing linearly as the load rose before the peak load. The strains at the —60 mm distance from the central axis remained to be approximate zero as a balance of the compression under axial load and the tension under bending. As shown in Fig. 9(c), the flange D was under tension during the full loading history which differs from the columns with eccentricity ratio 0.2 and 0.4. The yielding of steel tube tension flange D occurred at the ultimate load which was earlier than the specimen HS1C50SA-0.4 as foresaid.

Specimen HS1C0SA-0.4 was the hollow steel columns with a width-to-thickness ratio 21.05 and an aspect ratio 1.2. The compression flange B developed yielding before the tension flange D going into tension which differs from that of specimen HS1C50SA-0.4. The concrete at the tensile flange went into tension before the yielding of the steel tube compression flange which was contrary to the specimen HS1C40SE-0.2. The strains of the web A showed a linear relationship along the cross-section as illustrated in Fig. 8(b). The tensile flange D converted compressive strain into tensile strain before the peak load and developed tensile yielding at the descending stage of loading as shown in Fig. 8(c).

The width-to-thickness ratio and the aspect ratio were respective 31.58 and 41.63 for specimen HS1C50SB-0.6. Fig. 9(a) illustrated the load-mid height deflection and load-axial deformation curves of specimen HS1C50SB-0.6. The concrete at the tensile flange presented tensile strains at the first of loading. Fig. 9(b) shows that the strains of web A were increasing linearly as the load rose before the peak load. The strain at the —60 mm distance from the central axis remained to be approximate zero as a balance of the compression under axial load and the tension under bending. As shown in Fig. 9(c), the flange D was under tension during the full loading history which differs from the columns with eccentricity ratio 0.2 and 0.4. The yielding of steel tube tension flange D occurred at the ultimate load which was earlier than the specimen HS1C50SA-0.4 as foresaid.

Specimen HS1C0SA-0.4 was the hollow steel columns with a width-to-thickness ratio 21.05 and an aspect ratio 1.2. The compression flange B developed yielding before the tension flange D going into tension which differs from that of specimen HS1C50SA-0.4. The strains of web A yielded at 0.8$N_u$ while HS1C50SA-0.4 did not yield until the ultimate load $N_u$. The strains of the tension flange D went into tension at 1013kN as shown in Fig. 10(c).
4.2. Failure behaviors

The rectangular CFT columns showed the in-plane bending failure, see Fig. 10. The buckling and the crack of the steel tube were in the same location along the column length (see Fig. 11). The concrete was crushed near the mid height of the columns followed by the local buckling of steel tube with the increase of the eccentric load as shown in Fig. 12(a)–(b). The local buckling of steel tubes located at or near the mid height of the columns corresponding to the location of the initial imperfection. The CFT columns generally cracked at the corner due to the failure of welding, see Fig. 12(c). After removing the steel tube away, the obvious crush of concrete was observed at local buckling of steel tube with the increase of the eccentric load as shown in Fig. 12(a)–(b). The local buckling of steel tubes located at or near the mid height of the columns corresponding to the location of the initial imperfection. The CFT columns generally cracked at the corner due to the failure of welding, see Fig. 12(c). After removing the steel tube away, the obvious crush of concrete was observed at
the mid height of the columns as shown in Fig. 12(d). The inward and outward local buckling of hollow steel columns were observed as illustrated in Fig. 12(e).

4.3. Influence of in-fill concrete

The in-fill concrete has influence on both the failure behavior and the strength of the columns. The columns HS1C0SE-0.4, HS1C40SE-0.4 and HS1C50SE-0.4 are compared with each other to evaluate the influence of concrete as shown in Fig. 13. The in-fill concrete had little influence on the stiffness of the beam-columns until the yielding of the steel tube. The concrete can increase the bearing capacity of the beam-columns with the width-to-thickness ratio 42.63 and eccentricity ratio 0.4 by almost 25%. For columns with width-to-thickness ratio smaller than 42.63, the concrete contribution to the bearing capacity may be <25%. The yielding of compression flange B was delayed by the in-fill concrete as shown in Fig. 13(b). The other interesting observation is that the load corresponding to the local buckling of steel tube webs was increased by the concrete as characterized by point f, see Fig. 13(b). The concrete strength does not have much influence on the strains during loading whereas it increases the ultimate load $N_u$ by 4.7% when the concrete strength grade changes from C40 to C50.

4.4. Influence of eccentricity ratio

The influence of eccentricity ratio is evaluated by comparing the specimens HS1C40SA-0.2, HS1C40SA-0.4 and HS1C40SA-0.6. The load-axial deformation curves are shown in Fig. 14(a). The load corresponding to the characteristics a, b and c decreases with the increase of the eccentricity ratio as shown in Fig. 14(b). As eccentricity ratio increases from 0.2 to 0.6, the ultimate load decreases from 1440kN to 1110kN, and the bending moment $M_u$ increases from 30.74 kN·m to 50.61 kN·m as shown in Fig. 14(c). The load during the test dropped faster for the specimen with higher eccentricity ratio as shown in Fig. 14(a).

4.5. The evaluation of test results as comparison to DB 29-57

The design axial load $N_c$ and the bending moment $M_c$ of the columns can be calculated using Eqs. (1)–(4). The ratios $N_u/N_c$ and $M_u/M_c$ are compared with the theoretical values from DB 29-57.
Fig. 15. Variation of $N_u/N_c$ and $M_u/M_c$ with varying width-to-thickness ratio.

Fig. 16. FE model of the test specimens.
Mu/Mc are utilized to evaluate the design equations and the test results with the varying test parameters as shown in Fig. 15. DB 29-57 [20] is conservative for designing rectangular CFT beam-columns using high-strength steel as all $N_u/N_c$ and $M_u/M_c$ are much greater than one. The mean values of $N_u/N_c$ are 1.18, 1.27 and 1.34 for test columns with eccentricity ratio 0.2, 0.4 and 0.6, respectively, which indicates that DB 29-57 may be more conservative for rectangular CFT columns with higher eccentricity ratio.

As shown in Fig. 15(e), the $N_u/N_c$ decrease with the increase of width-to-thickness ratio for test columns with eccentricity ratio 0.6, whereas for columns with eccentricity ratio 0.2 and 0.4 as shown in Fig. 15(a) and Fig. 15(c), the width-to-thickness ratio does not have much influence on $N_u/N_c$. All $M_u/M_c$ decrease as the width-to-thickness ratio increases from 21.05 to 42.63 as shown in Fig. 15. The increase of the width-to-thickness ratio means that the $\alpha_s$ decreases at the same time. It indicates that the DB 29-57 is less conservative for rectangular CFT beam-columns with higher width-to-thickness ratio and smaller $\alpha_s$.

4.6. The nonlinear FE analysis

The finite element (FE) analysis was performed to investigate the contact behavior of rectangular CFT columns. The software package ABAQUS was used to simulate the experiments. A general view of the model and the cross-sections were shown in Fig. 16. The top and the bottom loading line were restrained against all degrees of freedom except for the rotation around the Y axis to model the pinned ends of the test columns. The nodes in the middle symmetry surface of the column were limited to the displacement in the XOZ plane. The element types and the mesh were shown in Fig. 16(b).

The simplified bilinear model without strain-hardening was used to simulate the Q460 steel and the backing plates as presented by Du et al. [23]. The stress-strain curves of concrete proposed by Han [24] was used to simulate the in-fill concrete. The damaged plastic model was utilized to simulate the concrete. In the damaged elasticity model, the key parameters (the dilation angle ($\psi$), the flow potential eccentricity ($e_f$), etc.) were set according to Du et al. [23].

An eigenvalue buckling analysis was first carried out to provide the lowest buckling mode to be used as the shape of the initial imperfection in the following nonlinear analysis. A surface-to-surface interaction with a hard contact model in the normal direction and a Coulomb friction model in the tangential direction was used to simulate the interface between steel tube and the concrete. The friction factor was taken as 0.25 recommended by Ellobody and Young [25].

The load-axial deformation and load-deflection curves obtained from the FE analysis match well with the test results as shown in Fig. 17. The comparison results of other specimens are not presented.
herein being similar to Fig. 17. \( N_u/N_{FE} \) and \( M_u/M_{FE} \) are calculated to calibrate the results of FE analysis with the test, where \( N_{FE} \) and \( M_{FE} \) are the ultimate axial load and the corresponding total moment obtained from the FE, respectively. The mean values of \( N_u/N_{FE} \) and \( M_u/M_{FE} \) are respectively 0.98 and 1.05 which indicate the FE analysis results are accurate in predicting the capacity.

The interaction of the steel and the concrete is not consistent during the loading which could be predicted using FE analysis. The initial contact could be guaranteed if the concrete is filled with good compaction. Therefore, the initial gap and contact pressure between the steel and the concrete are set to be zero as shown in Fig. 18(a) and (b). As the eccentric load increases, the steel and the concrete tend to be separated because the Poisson’s ratio of steel is higher than that of the concrete. Fig. 18(c) presents the gap between the steel and the concrete at ultimate load. Most part of the interaction surface separates except for the corner between Web C and Flange B. The corresponding contact pressure is relatively small as illustrated in Fig. 18(d). The expansion of the concrete becomes significant as the increasing of the concrete strain which makes the steel and the concrete contact again. When the average vertical strain \( \varepsilon = 0.01 \), the gap at the corners is filled and the maximum contact pressure reaches 4.564 MPa as shown in Fig. 18(e) and (f). The middle of Web C and Flange D remain separated from the concrete due to the effect of local buckling. It indicates that the contact

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**Fig. 18.** The contact behavior of Specimen HS1C40SA-0.2.

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**Fig. 19.** The variation of \( \alpha_s \) with the aspect ratio and steel strength.

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**Fig. 20.** Variations in stress and strain in a beam column of symmetrical cross section.
between the steel and the concrete is weak and the confinement effect could be ignored in rectangular CFT columns.

5. Proposed N-M interaction approach

Lai and Varma [21] developed FE models and proposed the revisions to the AISC 360 equations for designing noncompact and slender CFT beam-columns. Liang [26,27] presented a performance-based analysis (PBA) technique based on fiber element formulations for the nonlinear analysis and design of thin-walled CFT beam-columns with local buckling effects. Patel and Liang [28,29] developed and verified a numerical model of uniaxially loaded high strength thin-walled rectangular CFT columns. Choi and Kim [30] proposed simplified strength equations that can be conveniently used to establish a P–M interaction curve of square CFT columns with concrete strength up to 100 MPa. Kwon [31] proposed a simple formula for the axial and flexural strength of CFT columns to account for the local buckling effects and the enhanced strength of concrete. For rectangular CFT columns using high-strength steel, the steel tubes can take much larger load than the in-fill concrete, which was less investigated in the literature. Based on the analysis of the test results, a new approach to developing the N-M interaction curves is proposed in this study.

5.1. The basic assumption

The cross-sectional areas of steel ($A_s$) and concrete ($A_c$) may be expressed by width ($b$), length ($h$) and thickness ($t$) of the steel tubes with Eqs. (7)–(8). The aspect ratio ($\beta$) is calculated by dividing the $h$ by $b$. The width-to-thickness ratio is taken as the maximum value $72\sqrt{235/f_y}$, as specified in DB 29-57. Substituting the Eqs. (7)–(10) into Eq. (3), the $\alpha_c$ can be obtained by Eq. (11):

$$A_s = 2(b + h)t - 4t^2$$  \hfill (7)

$$A_c = (b - 2t) \cdot (h - 2t)$$  \hfill (8)

$$\beta = h/b$$  \hfill (9)

$$h/t = 72\sqrt{235/f_y}$$  \hfill (10)

$$\alpha_c = \frac{72(1 + \beta)\sqrt{235/f_y/\beta - 2} \cdot (f_y/f_c)}{72(1 + \beta)\sqrt{235/f_y/\beta - 2} \cdot (f_y/f_c) + \frac{36\sqrt{235/f_y/\beta - 1}}{36\sqrt{235/f_y/\beta - 1}}}$$  \hfill (11)

Taking the concrete strength as 50 MPa, the value of $\alpha_c$ is illustrated in Fig. 19 with the varying aspect ratio and the steel strength. As shown, an increasing load may be sustained by the steel tubes with the increase of both aspect ratio and the steel strength. For example, >60% of the load may be sustained by the steel tubes when $f_y = 460$ MPa. The proportion $\alpha_c$ could be larger if the width-to-thickness ratio is smaller than the limitation in practical engineering design.

The approach adopted by DB 29-57 [20] assumes that the steel tubes bear the total bending moment and part of the axial load. The behavior of rectangular CFT columns is similar to that of bare steel columns according to the above assumption. This assumption is suitable for designing rectangular CFT columns with high-strength steel and high value of aspect ratio which would result in high value $\alpha_c$. The $\alpha_c$ of the rectangular CFT columns in this study is higher than 0.7. Hence, the assumption that total bending moment is taken by the steel is reasonably conservative.

Eqs. (1)–(4) adopt the design formulas which are utilized for designing the bare steel structures. These formulas are based on the elastic bending theory as shown in Fig. 20(a). The stress and strain are proportional to the distance from the neutral axis, and the extreme fiber stress is allowed to develop the yield stress $f_y$. For rectangular CFT columns, the steel tubes can develop the yield stress $f_y$ over the whole cross section illustrating the fully-plastic state as shown in Fig. 20(b). The fully-plastic state of steel tubes is adopted in the plastic stress distribution method [17,18]. With regard to this, the design approach adopted by DB 29-57 is conservative in designing rectangular CFT columns. Based on the fully-plastic assumption, the proposed design formulas to establish the N-M curves are derived in this study.

5.2. The proposed approach and the validation

The typical stress blocks of fully-plastic state for determining the strength of rectangular CFT columns subjected to combined axial compression and bending are shown in Fig. 21. Only the stress...
blocks of steel are used in the calculation. The resistance of backing plates is included in the formulas for accuracy purpose. Three kinds of neutral axis locations should be taken into account in the proposed approach as shown in Fig. 21.

For $0 \leq e_c < h/2 - t - h_b$ as illustrated in Fig. 21(a), the design axial load and bending moment can be calculated by Eqs. (12)–(14):

$$N_s = 4f_y e t$$  \hspace{1cm} \text{(12)}
In the formulas, $\epsilon_s$ is the distance from the neutral axis to the center line of the cross section; $M$ is the design bending moment around the principal axis.

For $h/2 - t - h_b \leq \epsilon_s < h/2 - t$ as shown in Fig. 21(b), the design axial load and bending moment can be calculated by Eqs. (15)–(17):

$$N = \frac{4f_y \epsilon_t t}{\alpha_s}$$  \hspace{1cm} (13)

$$M = 2f_y t \left[ b (h - t)/2 + (h/2 - t)^2 - \epsilon_s^2 \right] + 2f_y b_2 b (h - 2t - h_b)$$ \hspace{1cm} (14)

$$N_s = 4f_y \epsilon_t t + 4f_y b_2 (\epsilon_s - h/2 + t + h_b)$$ \hspace{1cm} (15)

$$N = \frac{4f_y \epsilon_t t + 4f_y b_2 (\epsilon_s - h/2 + t + h_b)}{\alpha_s}$$ \hspace{1cm} (16)

Fig. 24. Comparison of proposed N-M interaction curves with test results Fujimoto [15].
\[ M = 2f_y t \left[ b(h - t)/2 + (h/2 - t)^2 - e_s^2 \right] + f_y b_y b_0(2h - 2t)(h/2 - e_s - t) \]  
\[ N = f_y \left[ 2(h + b)t - 4t^2 - b(h - 2e_s) \right] + 4f_y b_y b_0 \frac{e_s}{\alpha_s} \]  
\[ N = f_y b \left( h^2 / 4 - e_s^2 \right) \]

For \( h/2 - t \leq e_s, sh/2a \) as shown in Fig. 21(c), the design axial load and bending moment can be calculated by Eqs. (18)-(20):

\[ N = f_y [2(h + b)t - 4t^2 - b(h - 2e_s)] + 4f_y b_y b_0 \]  
\[ N = f_y [2(h + b)t - 4t^2 - b(h - 2e_s)] + 4f_y b_y b_0 \frac{e_s}{\alpha_s} \]  
\[ M = f_y b \left( h^2 / 4 - e_s^2 \right) \]

By changing \( e_s \) from zero to \( h/2 \), the N-M curves can be developed with the corresponding M and N. The proposed design approach should be modified to account for the length effect relative to the test results. Axial strength in the strength interaction curves is reduced by using the reduction factor \( \varphi_s \) as specified in DB 29-57 [20].

The proposed N-M curves for the test columns in this study are established and provided in Fig. 22. To evaluate the predictions of the test results and the proposed method, the N-M curves calculated in accordance with EC4 [17], AISC 360 (Method 2) [18], GB 50936 [19] and DB 29-57 [20] are utilized to calibrate the experimental results and the proposed N-M interaction approach. The experimental N-M relationships during the loading are plotted as shown in Fig. 22. The green points stand for the ultimate compressive load and the corresponding bending moment which characterize the bearing capacity of the columns.

EC4 is less conservative than the three design codes and the proposed design approach as shown in Fig. 22. For specimens with width-to-thickness ratio 42.63, the EC4 is unsafe in prediction as shown in Fig. 22(c) and (f). AISC 360 interaction curves provide conservative results except for the specimen HS1C50SE-0.6 with width-to-thickness ratio 42.63. The conclusion can be drawn that the larger the steel section takes load, the more conservative the strength prediction by the EC4 and AISC 360 interaction curves could be. GB 50936 interaction curves are much more conservative than the others. With the increase of the width-to-thickness ratio, the GB 50936 interaction curves are getting closer to the test results. The DB 29-57 interaction curves are rather conservative owing to the absence of the concrete strength and the plastic strength of steel.

As shown in Fig. 22, predictions by the proposed N-M interaction curves are more accurate compared with the test results than the predictions from DB 29-57. The proposed N-M interaction curves are more conservative than EC4 and AISC 360 interaction curves for the test columns.

As discussed in Section 1, many tests had been presented by the other researchers. A test database including 105 test data from literatures [3,11,12,13,15,16,20] was used to validate the proposed N-M interaction approach with respect to different steel strength, \( \alpha_s \) and width-to-thickness ratio. The database covers test data with steel strength ranging from 242 MPa to 834 MPa, \( \alpha_s \) ranging from 0.28 to 0.87, and width-to-thickness ratio ranging from 18.7 to 102.

The representative comparison results are shown in Figs. 23–25. Liu [12] presented the test results using 495 MPa high-strength steel as shown in Fig. 23. The results indicate that the behaviors of CFT columns didn’t show significant difference with respect to similar \( \alpha_s \) and different width-to-thickness ratio. The test results of CFT columns (S1, S2, S5, S6, S9, S10, S11) is close to the predictions of EC4, and the capacity of the others (S3, S4, S7, S8, S12, L4) are lower than the predictions of the EC4. AISC 360 is unsafe in the design of S3, S4 and S8. The predictions of GB 50936, DB 29-57 and the proposed N-M interaction approach are safe in the design of all test columns. Fujimoto [15] reported the test data of CFT columns using both high-strength steel (618 MPa and 834 MPa) and normal strength steel (262 MPa) as presented in Fig. 24. The conclusion can be drawn that the proposed N-M curves are getting more conservative with the decrease of \( \alpha_s \) compared with EC4 and AISC 360. This is reasonable because the decrease of \( \alpha_s \) denotes the increase of width-to-thickness ratio and the weak confinement effect of steel to concrete; hence, EC4 and AISC 360 is not conservative when \( \alpha_s \) is getting lower as shown in Fig. 24(g)–(i). The results of ER8-D-4-40, ER8-D-4-60, ER6-D-4-47 and ER6-D-4-23 are unsafe.
compared with EC4 and AISC 360 although $\alpha_s$ is relatively high because the width-to-thickness ratio has exceeded the maximum limitations specified in the design codes. Fig. 24(g)-(i) indicates that the proposed N-M interaction curves are more conservative with respect to CFT columns with normal strength steel and low $\alpha_s$. Fig. 25 shows another comparison results from test presented by Qu [20] which utilize normal strength steel (242 MPa and 295 MPa). As shown, the proposed N-M interaction curves shows good agreement with the test results.

To evaluate the feasibility of the design methods in the use of CFT columns, the safe proportion of design has been calculated and shown in Fig. 26. 70% test data using normal strength steel is expected to be safe in design for EC4, whereas only 35% design is safe incorporating high-strength steel. AISC 360, GB 50936, DB 29-57 and the proposed N-M interaction curves can guarantee the safe design of CFT columns using normal strength steel. However, only 70% and 80% design is safe incorporating the use of high-strength steel for AISC 360 and GB 50936, respectively. DB 29-57 is safe for design of all the test CFT columns with very conservative results. The proposed N-M interaction approach can ensure safe design of 95% CFT columns using high-strength steel.

The conclusion can be drawn that the proposed N-M interaction curves can conservatively predict the strength in comparison with the test results. Compared with EC4 and AISC 360, the proposed approach is conservative for rectangular CFT columns using high-strength steel. The approach is more accurate in comparison to GB 50936 and it is more accurate than the predictions of DB 29-57. Regardless of the $\alpha_s$ and the width-to-thickness ratio, the proposed N-M interaction approach for rectangular CFT columns using high-strength steel is verified to be reasonably conservative. The proposed N-M curves can also provide conservative predictions in the design of rectangular CFT columns using normal strength steel.

6. Conclusions

This study investigates the behavior of rectangular CFT beam-columns experimentally and theoretically. The previous researches and the current design codes on the design of rectangular CFT columns using high-strength steel are reviewed at first. An experimental program with extensive rectangular CFT beam-columns using Q460 grade steel is reported. The influence of in-fill concrete and the eccentricity ratio was analyzed. The effect of width-to-thickness ratio and $\alpha_s$ was evaluated in comparison with DB 29-57. A new method was proposed for the design of rectangular CFT columns using high-strength steel. The following conclusions can be drawn with the scope of this study:

1. The in-fill concrete can delay the yielding and buckling of the steel tubes. The bearing capacity of the rectangular CFT columns increases by at most 25% in this study.

2. DB 29-57 is conservative in calculating the resistance of the rectangular CFT columns using high-strength steel. For higher width-to-thickness ratio and smaller $\alpha_s$, DB 29-57 is less conservative in prediction.

3. An increasing load may be taken by the steel tubes with the increase of aspect ratio and the steel strength. The assumption that the in-fill concrete takes only the axial load and the steel tubes bear a portion axial load and the total bending moment is reasonable for rectangular CFT columns using high-strength steel. The steel tubes are assumed to develop the yield stress $f_y$ over the whole cross section for rectangular CFT columns using high-strength steel.

4. A new N-M interaction approach is proposed to design the strength of rectangular CFT beam-columns using high-strength steel. The approach is verified by the test results and the available current design codes. This approach is more conservative for columns with high width-to-thickness ratio compared with EC4 and AISC 360, and it is more accurate than the predictions of GB 50936 and DB 29-57. Regardless of the $\alpha_s$ and the width-to-thickness ratio, the proposed N-M interaction approach for rectangular CFT columns using high-strength steel is verified to be reasonably conservative. The proposed N-M curves can also provide conservative predictions in the design of rectangular CFT columns using normal strength steel.

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