Mechanical performance of hexagonal multi-cell concrete-filled steel tubular (CFST) stub columns under axial compression

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ABSTRACT

Multi-cell concrete-filled steel tubular (CFST) column is a kind of composite structures developing from conventional CFST columns. The multi-cell CFST columns have greater cross-sectional dimensions and utilize internal webs to separate the inner concrete into smaller isolated cells. They have been used in super high-rise buildings recently as the main vertical load bearing members. However, existing research on CFST members is mainly focused on the conventional single-cell CFST members. To fill this research gap, this paper numerically investigates the mechanical performance of hexagonal multi-cell CFST stub columns under axial compression. A finite element analysis (FEA) model is initially established to simulate the mechanical performance of hexagonal multi-cell CFST columns. The FEA model is validated against existing experimental data. The mechanical performance of the multi-cell CFST columns are analysed, including the full-range load versus deformation relationships, the stress distributions of the main components and the distribution of contact stress on each concrete cell. A parametric study is then conducted to investigate the sensitivity of various geometric and material parameters on the compressive behaviour of multi-cell CFST columns. Finally, analytical formulae are derived to predict the axial compressive ultimate strength of hexagonal multi-cell CFST columns. The methods are found to be acceptable with reasonably good accuracy.

KEYWORDS

Concrete-filled steel tubular (CFST); hexagonal multi-cell; Axial compression; Finite element analysis (FEA); Full-range analysis; Ultimate strength

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Nomenclature

A_c	Cross-sectional area of concrete
A_{so}	Cross-sectional area of outer steel tube
A_{si}	Cross-sectional area of internal steel web
f_{ck}	Characteristic strength of concrete (=0.67f_{cu} for normal strength concrete)
f_{cu}	Cube strength of concrete
f_c'	Cylinder strength of concrete
f_y	Yield stress of steel plate
f_{yo}	Yield stress of outer steel tube
f_{yi}	Yield stress of internal steel web
$f_{y,b}$	Yield stress of steel bar
f_{scy}	Nominal average strength of the steel tube after filling with concrete
E_c	Young's modulus of concrete
E_s	Young's modulus of steel plate
$E_{s,b}$	Young's modulus of steel bar
$E_{sh,b}$	Hardening modulus of steel bar
t	Wall thickness of steel plate
t_o	Wall thickness of outer steel tube
t_i	Wall thickness of internal steel web
H	Height of stub column specimen
D	Overall depth of section
B	Overall width of section
b_i	Sub-panel plate width (i=1,2......)
α_o	steel ratio of outer steel tube ($\alpha_o=A_{so}/A_c$)
α_i	steel ratio of internal web ($\alpha_i=A_{so}/A_c$)
N	Axial load
N_u	Ultimate strength of composite sections
$N_{osc,u}$	Compressive force of single-cell CFST member
$N_{i,u}$	Compressive force of internal steel webs
$N_{b,u}$	Compressive force of rebar cages
$N_{u,test}$	Measured ultimate strength of composite sections
$N_{u,FEA}$	Predicted ultimate strength of composite sections by FEA model
$N_{u,Prop}$	Predicted ultimate strength of composite sections by simplified formulae
Δ	Axial deformation
ε	Average strain, $=\Delta/H$
ξ_0	Nominal confinement factor ($=\alpha_n f_{yo}/ f_{ck}$)
δ_c	Relative values of the load carried by concrete to its sectional capacity
δ_o	Relative values of the load carried by outer tube to its sectional capacity
δ_i	Relative values of the load carried by internal webs to its sectional capacity

1. Introduction

Concrete-filled steel tubular (CFST) members have been widely utilized in numerous engineering structures due to their advantages, including higher strength, better ductility and larger energy absorption capacities, when compared to conventional steel and reinforced concrete members [1]-[3]. It is noticed that, three typical CFST sections, i.e., concrete filled circular hollow section (CHS), concrete filled square hollow section (SHS) and concrete filled rectangular hollow section (RHS), have been extensively investigated and used in numerous engineering structures owing to their excellent structural behaviours and wide applicability [4]-[12]. Recently, CFST members with special-shaped cross-sections have been used in some super high-rise structures as the result of increasing demand in structural safety and architectural aesthetics, e.g., hexagonal sectional CFST column in Tianjin Goldin Finance 117 building (Qian et al. [13]) and octagonal sectional CFST column in China Zun (Xu et al. [14]).

However, as the dimensions of CFST cross-section increase, it could be much easier for the outer steel tube to buckle due to its larger width-to-thickness ratio. Moreover, the shrinkage and creep of mass concrete can adversely affect the composite action between the steel tube and the concrete. Thus, some enhancing measures, such as binding bars, vertical ribs, horizontal diaphragms, vertical webs and rebar cages, have been developed and applied in engineering practice. Existing design codes for composite structures also specify the construction details of CFST columns when the maximum dimension of CFST section exceeds a certain degree. For example, Chinese code JGJ 138–2016 [15] specifies that vertical webs should be utilized between the tube walls when the minimum side length exceeds 2000 mm for rectangular CFST, and vertical ribs or rebar cages should be utilized when the minimum side length exceeds 1500 mm.

The original single cell of the CFST member is divided by the internal vertical webs into several isolated cells with concrete infilled, thus forming the enhanced CFST member. The confinement effect of steel tube on the isolated concrete cells in this enhanced CFST member could be different from the original one. For clarity, these new enhanced CFST members are called multi-cell CFST members herein. The contribution of internal webs to the compressive behaviour of multi-cell CFST could be complex and difficult to be predicted quantitatively because it is influenced by the cross-sectional configuration. Although existing specifications, such as JGJ 138-2016 [15], Eurocode 4 [16], ACI 318-14 [17], ANSI/AISC 360-16 [18], DBJ/T 13-51-2010 [19], AIJ-2008 [20], BS5400 [21], AS3600 [22] provide design methods for CFST members, especially for the member with square, rectangular and circle cross-sections, the feasibility of these methods for designing multi-cell sectional CFST members are unknown. Generally, the design approach for polygonal CFST members always adopted in the engineering practice neglects the concrete confinement and assumes that sufficient strains have developed in the steel and concrete for both to reach their yield strength. Although this approach is simple for application, it sometimes could lead to conservative design and economic inefficiency.

So far, existing studies on CFST members mostly focus on square, rectangular and circle cross-sections, while the researches on the behaviour of polygonal CFST members and multi-cell CFST members are limited. One of the multi-cell CFST cross-sections that recently gained significant attention is the mono-symmetric hexagonal shape which is used in Goldin Financial 117 Building in China. The schematic view of the hexagonal sectional multi-cell CFST member is shown in Fig. 1. It can be seen in Fig.1 that several vertical webs were placed into the original single cell to make a multi-cell cross-section. The multi-cell CFST cross sections were designed to be hexagonal in shape. This not only meets the requirements of bearing capacity, ductility, fire resistance and durability, but also increases the feasibility of the connection with beams [14].

As we all know, the performance of CFST member benefits from the composite action between the outer steel tube and core concrete. The confinement effect provided by the steel plate segments in the above hexagonal multi-cell CFST members may differ from that of the conventional CFST members due to the special configurations of the steel plates.

Previously, some research has been conducted on single-cell CFST members with hexagonal cross section. Xu et al. [14] experimentally and numerically investigated the compressive behaviour of dual-axisymmetric single-cell hexagonal CFST members. It is found that the concrete behaviour of dual-axisymmetric hexagonal CFST member is identical to that of the rectangular CFST member. Also, the ultimate compressive strength of dual-axisymmetric hexagonal CFST stub columns could be calculated accurately according to the equations of rectangular CFST columns specified in Eurocode 4 [16] and DBJ/T13-51-2010 [19]. Hassanein et al. [23] numerically investigated the compressive behaviour of regular-shaped single-cell hexagonal CFST members. It is found that the geometric internal angle (θ) as shown in Fig. 1(b) can act as an indicator of the confinement effect in a hexagonal CFST member.

The researches on multi-cell hexagonal CFST members are also available. Cao et al. [24] conducted compression tests on three 1/12 scale hexagonal multi-cell CFST columns based on the CFST columns of Goldin Finance 117 Tower. The test parameters included concrete strength and the existence of rebar cages. The load bearing capacity, stiffness, residual deformation and failure modes were analysed contrastively. It is found that the multi-cell CFST column with rebar cages being used has high ductility under compression. Also, Xu et al. [25] conducted compression tests on seven 1/20 scale hexagonal multi-cell CFST columns based on the CFST columns of Goldin Finance 117 Tower. The failure modes, load versus deformation curves and bearing capacity were analysed. The results show that concrete strength contributes most to the bearing capacity, while the existence of rebar cages have minor influence on the bearing capacity; increasing the wall thickness of the outer tube can not only enhance the bearing capacity, but also improve the ductility. A practical calculation formula for calculating the bearing capacity of stub multi-cell CFST columns under axial compression was proposed. However, the reliability of the formula needs to be further verified because the number of tests is limited. It should be noted that the above two investigations are both concerned with hexagonal multi-cell CFST columns with internal angle θ=135°, while researches on other hexagonal sectional multi-cell columns were very rare.

Thus, based on the existing experimental data, this paper numerically investigates the compressive behaviour of hexagonal multi-cell CFST members with internal angle θ=135°. Nonlinear finite element analysis (FEA) model is established and validated against existing test data. The validated FEA model is employed to conduct the mechanism analysis of hexagonal sectional multi-cell CFST stub columns under axial compression. Parametric study is conducted to investigate the compressive behaviour of such composite column with various geometric and material parameters. Finally, simplified analytical formulae for predicting the ultimate strength of hexagonal multi-cell CFST stub columns subjected to axial compression are derived.

2. Finite element analysis (FEA) model and validation

2.1 *General descriptions of the model*

The ABAQUS software package [26] is used in this paper to establish the FEA model of hexagonal multi-cell CFST stub columns under axial compression. The general static module was used. The schematic view of a typical FEA model established is shown in Fig. 2. The FEA model was composed of seven components, i.e., top endplate, outer steel tube, internal web, core concrete, vertical rib, rebar cage and bottom endplate. Solid elements (C3D8R) were used to model the concrete and the endplates, truss elements (T3D2) were adopted for the steel bar reinforced cages, while shell elements (S4R) were used for the outer steel tube, internal webs and vertical ribs. Displacement controlled loading was employed to achieve a better convergence. A mesh sensitivity study was performed to identify an appropriate mesh density which meets both accuracy and computational efficiency. Through this mesh-density analysis, it is found that the element size in longitudinal direction has a significant influence on the descending branch of the load versus deformation curve and the failure mode. Based on extensive FEA trials, element size of 25 mm is used to model steel tube and concrete in the longitudinal direction, and approximate global size of 15mm is taken to model steel tubes and the concrete in the hoop directions. The maximum aspect ratio limit of 0.1 was used within the cross section. The mesh is shown in Fig. 2.

2.2 *Material constitutive models*

2.2.1 *Material modelling of steel*

The elastic-plastic stress-strain relationships of structural steel proposed by Han et al. [27] are used for the outer steel tube, vertical ribs and internal webs. The relationships consist of five stages, i.e., elastic stage, elastic-plastic stage, plastic stage, hardening stage and fracture stage. And they are depicted in Fig.3(a), where f_p, f_y, and f_u are the proportional limit, yield, and ultimate strength of steel respectively, and ε_e=0.8f_y/E_s, ε_{e1}=1.5ε_e, ε_{e2}=10ε_{e1}, ε_{e3}=100ε_{e1}. The steel is assumed to have isotropic hardening behaviour, which means the yield stress changes in all stress directions as plastic strain occurs [26]. More details of the relationships can be found in [27]. The bi-linear stress-strain relationship proposed by Zhao et al. [28] is used for the longitudinal rebars and horizontal hooped rebars. This plasticity model considers the strain hardening effect. It is depicted in Fig.3(b), where $E_{s,b}$ and $E_{sh,b}$ are the Young's modulus and hardening modulus of rebars, respectively; and $\varepsilon_{sy,b}$=$f_{y,b}$/$E_{s,b}$ is the yield strain of rebars. $E_{sh,b}$ is taken as 0.01$E_{s,b}$.

The elastic modulus (E_s) and Poisson's ratio (ν_s) of steel plates and rebars are taken as 2×10^5 (N/mm^2) and 0.3, respectively. The measured values from tests were adopted when validating the FEA model.

2.2.2 *Material modelling of concrete*

For a multi-cell CFST column under axial compression, the lateral expansion of the concrete at each cell is confined by the surrounding steel plates. This confinement can increase the strength and ductility of concrete. This mechanism is often referred to as "composite action" between the steel and concrete [29]. It is believed that the confined concrete is in a triaxial stress state and the steel is in a biaxial state after interaction between the two components forms. FEA can consider the composite action by incorporating a rational and accurate concrete model to describe the behaviour of concrete under passive confinement.

Concrete Damage Plasticity (CDP) model is adopted for the concrete in the multi-cell hexagonal CFST column. This model adopts a unique yield function with non-associated flow and a Drucker-Prager hyperbolic flow potential function to describe the plasticity of concrete. The stress-strain relationships for the general three-dimensional multiaxial condition are given by the scalar damage elasticity equation. This damage plasticity model can be used to analyse the behaviour of concrete under multiaxial state, although it was initially developed for concrete under low confining pressure [26].

The CDP concrete model has been used in previous studies to investigate the behaviour of CFST column [29,30]. However, default values were used in [29] for many material parameters, such as the dilation angle (ψ), flow potential eccentricity (e_f), ratio of the compressive strength under biaxial loading to uniaxial compressive strength (f_{b0}/f_c') and the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian (K_c), which can give reasonable predictions for normal CFST columns. Thus, the same CDP model is adopted in this paper. The constant values of 30°, 0.1, 1.16 and 2/3 were used for ψ, e_f, f_{b0}/f_c and K_c in this study, respectively.

Moreover, the stress-strain relationships of confined concrete proposed by Han et al. [29] are employed to model the concrete in multi-cell CFST members. These uniaxial compressive stress-strain relationships were initially developed for the concrete in single CFST members, and a confinement factor was employed to reflect the confinement effect of the rectangular outer steel tube. These relationships were proposed for FEA analysis of conventional CFST members, and reasonably good agreements were obtained compared with experimental results. The confinement factor (ξ) defined by Han et al. [29] is as follows:

$$\xi = \alpha_n \ f_{yo} / \ f_{ck} \tag{1}$$

where α_n is the nominal steel ratio of CFST columns and is defined by $\alpha_{n=}\ A_{so}/A_c$. A_c is the cross-sectional area of concrete, A_{so} is the cross-sectional area of outer steel tube, f_{yo} is the yield stress of outer steel tube, and f_{ck} is the characteristic compression strength of concrete. The value of f_{ck} is equal to approximately 67% of the compressive cube strength of concrete (f_{cu}) for normal strength concrete.

Since the concrete in each multi-cell CFST cell is isolated due to the division of the internal webs, the confinement effect provided by the steel plates to each concrete cell could be different. Thus, the confinement factor was calculated individually for each concrete cell according to the geometric and material characteristic of its surrounding subpanel plates by Eq. (1).

The initial modulus of elasticity (E_c) and Poisson's ratio (v_c) of concrete are determined according to the recommendations in ACI 318-14 [17], given as $E_c = 4730\sqrt{f_c^{'}}$ and v_c=0.2, respectively. The measured values from tests were adopted when validating the FEA model.

2.3 *Interactions and boundary conditions*

A surface-based interaction was used to simulate the contact behaviour between the steel tube cell and the core concrete. This interaction adopts a 'hard contact' in the normal direction which allows the separation of the interface in tension and no penetration of that in compression. The tangent contact can be simulated by the Coulomb friction model and a coulomb friction model with a friction coefficient of 0.6 in the tangential direction, which agrees with most test results [31]. Surface-based interaction was also employed between the endplates and the concrete. The 'shell to solid coupling' constraint was used between the endplates and all the shell edges, including the outer steel tube, the internal webs and the vertical ribs. In this coupling constraint, the displacements and rotations of the connected elements are identical during analysis. The steel tube, the vertical ribs and the internal webs were built in one part, hence the interactions between these parts were considered by the inherent method of sharing common nodes. The vertical ribs and rebar cages were embedded into the concrete, where the calculation of stiffness of the embedded elements will be carried out separately from the concrete elements. Moreover, the displacement of embedded elements will be compatible with the displacement of surrounding concrete elements.

The two endplates were coupled with two reference points respectively using the 'rigid body' constraint in ABAQUS. The boundary conditions were applied by the two reference points.

2.4 *Verifications of the FEA model*

To validate the FEA model, existing experimental studies conducted by other researchers on the axial compression of the hexagonal sectional multi-cell CFST stub columns were selected. The test data reported by Cao et al. [24] and Xu et al. [25] were adopted. Table 1 summarizes the information of the selected specimens, where H is the clear height of the column, t_s is the wall thickness of the steel plate, f_{cu} is the concrete cube strength, E_c is the elastic modulus of concrete, f_y is the yield strength of steel plate, $f_{y,b}$ is the yield strength of rebar, E_s is the elastic modulus of steel plate, and $E_{s,b}$ is the elastic modulus of steel bar.

Fig.4 shows the comparisons of the FEA-predicted and measured load (N) versus axial deformation (Δ) relationships. Table 1 also shows the tested and FEA-predicted ultimate strengths of the specimens. It can be seen from Table 1 and Fig.4 that the FEA model provides generally good predictions of the N-Δ relationships of hexagonal multi-cell CFST columns under axial compression. The predicted ultimate strength and the post-peak softening behaviour of the column are in good agreement with the experimental data. The mean value of $N_{u,FEA}$/ $N_{u,test}$ is 1.064 with a coefficient of variation (COV) of 0.043, where $N_{u,FEA}$ and $N_{u,test}$ are FEA-predicted and measured ultimate strength, respectively.

The ratio of the changed capacity of each component to its sectional capacity is also presented in Table 1. These values for concrete, outer steel tube and internal web are labelled as δ_c, δ_o and δ_i, respectively. The sectional capacity is calculated by multiplying the cross-sectional area of the component by the yield strength. The positive values represent increases while negative values represent decreases. It can be seen in Table 1 that the δ_c-value and δ_o-value change from 11.79% to 9.37% and from -14.26% to -17.33% respectively when the cube strength of concrete increases from 67.8 MPa to 84.1 MPa, while δ_i-value changes by only approximately 1%. This is probably because (1) the confinement effect decreases with the increase of concrete strength, which results in the decrease of δ_c-value (2) more severe transverse bulge occurs on the outer tube leading to the decrease of δ_o-value (3) the internal webs were encased in the concrete and restrained by the side concrete which results in minor change of the transverse stress.

3. Mechanism analysis

The behaviour of hexagonal multi-cell CFST stub columns under axial compression is analysed using the FEA model in this section.

3.1 *Failure modes*

Previously, Xu et al [14] has already compared the axial compressive behaviour of CFST stub columns of different single-cell cross-sectional shapes (square cross-section, hexagonal cross-section and circular section), including the effect of cross-sectional shape on the load (N) versus axial shortening (Δ/H) relationships and the stress distribution of core concrete. Results showed that the N-Δ/H curve of the hexagonal sectional CFST is similar to that of the rectangular CFST. Moreover, the stress concentration and contact stress are observed at corners of the hexagonal sectional CFST within limited area which is similar with that of the square sectional one, while the contact stress distributes uniformly along the circular cross section. It indicates that the confinement of the steel tube to the core concrete in the hexagonal CFST is similar to that in the rectangular one. Thus, only the axial compressive behaviour of single-cell hexagonal CFSTs and multi-cell hexagonal CFSTs are compared in the paper.

Fig.5 illustrates the schematic buckling modes within the cross section of the outer steel tubes in hexagonal CFST stub cross sections under axial compression, in which Fig.5 (a) has no internal steel webs (single-cell) and Fig.5 (b) has internal steel webs (multi-cell). These schematic failure modes were simplified based on the results from the FEA model. It can be seen from Fig.5 that all the outer steel plates bulge outwards within the cross section for both single-cell CFST columns and multi-cell CFST columns. This phenomenon tends to be more obvious when the steel ratio was smaller. Comparing the two failure modes, it can be found that the outer steel plates in multi-cell CFT columns buckled later than those in the single-cell CFST columns. And double half-wave buckling mode occurs at the outer steel plates which are intersected by the internal web, whilst single half-wave buckling occurs at the counterparts in the single-cell CFT columns. For multi-cell CFST columns, the internal webs significantly affect the overall local buckling failure mode. And no apparent buckling can be observed for the internal webs.

3.2 ***Full-range load versus deformation relationships***

In this section, a numerical example is employed to study the full-range compressive behaviour of hexagonal multi-cell CFST stub column. The basic parameters used in calculations are taken from the specimen CC-5 in Xu et al. [25]. The parameters of this specimen are listed in Table 1. The reason of choosing this particular specimen is that the cross section of this specimen consists of all the components of multi-cell CFST cross section without any other enhancing measures used. The cross section is composed of outer steel tube, internal steel webs and concrete. The full-range loading process herein includes two stages, i.e., an increasing stage from the initial loading to the attainment of the ultimate strength, and a descending stage from the ultimate strength to the failure when the load drops to 85% of the peak load. The predicted load (N) versus average strain (ε) relationship of this specimen is shown in Fig.6. And the loads carried by the concrete (N_c), the outer steel tube (N_{so}) and the internal steel webs (N_{si}) are also presented. This N-ε relationship is characterized by four stages, i.e., OA, AB, BC and CD.

Stage 1: Elastic stage (OA). Point A represents the yielding point of the specimen, when both the outer steel tube and internal steel webs start to yield. In stage OA, the values of N_c, N_{so} and N_{si} increase linearly, indicating elastic behaviour of each component. The load is distributed among all components according to their compressive stiffness.

Stage 2: Elasto-plastic stage (AB). The member attains its ultimate strength at point B. The interaction between the concrete core and outer steel tube (internal steel webs) formed at this stage. The outer steel tube and internal steel webs are subjected to transverse tension and longitudinal compression, which results in a plastic development and slight decrease of N_{so} and N_{si}. The concrete core is restrained by the outer steel tube and internal steel webs which results in the continuous increase of N_c. At point B, N_c also reaches its peak value. It is found that the buckling of steel plate initially occurred when the ultimate load carrying capacity was attained because the yield strength of outer steel tube was achieved. This is possibly because the local buckling of outer steel plate segments was effectively delayed by the internal webs.

Stage 3: Plastic stage (BC). Point C denotes the point when the load decreases to 0.85 times of the peak load, and this state is taken as the failure of the column. After point B, the curve enters the descending stage when N_c declines due to the crush of core concrete and N_{so} start to decrease slightly due to the occurrence of the outward local bucking. However, N_{si} keeps stable mainly because approximate equal restraints were applied by the concrete on two sides of the internal steel webs. This can be further confirmed by the similar stress development of the two sides (side 1 and side 2) of the inner steel webs in Fig.7 and the contact stress distribution in Fig.9.

Stage 4: Post-failure stage (CD). Point D corresponds to an axial shortening nominal strain of 0.01. After point C, N_c continues to decrease due to the crush of core concrete while N_{so} and N_{si} keep steady. For this particular specimen, N_r equals to 0.612 $N_{u,FEA}$.

3.3 *Stress distributions*

To better understand the performance and interactions of different parts of the multi-cell CFST column, the predicted stress developments were obtained and discussed in this section. The specimen CC-5 reported by Xu et al. [25] was selected and modelled. The stress (σ) versus average strain (ε) relationships of steel plate segments at different locations within the mid-height cross section are shown in Fig. 7. The longitudinal stress (σ_l) and transverse stress (σ_t) at the two sides of steel plate are all depicted, and they are labelled by $\sigma_{l\text{-}1}$, $\sigma_{l\text{-}2}$, $\sigma_{t\text{-}1}$, $\sigma_{t\text{-}2}$, respectively. In Fig.7, the longitudinal stress (σ_l) and transverse stress (σ_t) are shown in black and blue lines respectively, and the stress on two sides of the steel plates are shown in straight and dashed lines respectively. The longitudinal stress (σ_l) and the contact stress (p) distribution of the individual concrete at typical points (corresponding to Fig.6) are further compared in Fig. 8 and Fig.9, respectively. The stress distribution is analysed based on the stage division in Fig. 6.

Stage 1: Elastic stage (OA). It can be seen from Fig. 7 (a)~(j) that, the longitudinal stress reaches the yield point when ε is approximately 1800 $\mu\varepsilon$, which is consistent with the yield status in Fig.6 (point A). The longitudinal stress (σ_l) at each location exhibits an initial linear increase, during which the transverse stress (σ_t) is close to zero. Two reasons may account for this: firstly, the lateral deformation of the outer steel tube is greater than that of the concrete since the Poisson's ratio of the former is higher than the latter; secondly, the confinement of the adjacent concrete on the internal steel webs restrains the in-plane transverse deformation in the internal steel webs, as shown in Fig.9(a). It can be seen in Fig.8 (a) that the longitudinal stress of concrete σ_l uniformly distributes, ranging from 0.70 f_c' to 0.90 f_c' across the cross-section. Localized stress concentration occurs in corners, where the σ_l reaches 0.90 f_c' to 1.14 f_c'.

Stage 2: Elasto-plastic stage (AB). The column reaches the ultimate strength at point B, then the longitudinal stress of steel plates begins to decrease and transverse stress begins to increase due to the interaction between the steel plates and concretes. It can be found in Fig.7 (b) and (d) that the stress of each side of the steel plates develops considerable difference at these two positions due to the influence of local buckling within the cross section. By contrast, the stress of each side of the steel plates develops similarly at the other positions because minor local buckling occurs. It is found in Fig.8 (b) that the maximal longitudinal stress of concrete reaches 1.30 f_c', and it appears at the corners because of the non-uniform confinement provided by the surrounding steel plates. It can also be found in Fig.9 (b) that the contact stress on the two sides of the internal steel webs still distributes symmetrically. The contact press starts to increase due to the transverse expansion of the core concrete during stage AB.

Stage 3: Plastic stage (BC). Comparing Fig.8 (b) and (c), it can be found that the stress of concrete close to the outer steel tube decrease due to the concrete crush and the local buckling of outer steel tube. By contrast, the stress of concrete at the outer right-angle corners still increases to a maximum of 2.30 f_c', while that at the inner right-angle corner increases to about 1.80 f_c'. The contact pressure on the sides of the internal steel webs still distributes symmetrically and begins to decrease due to the failure of the concrete, as shown in Fig. 9 (c).

Stage 4: Post-failure stage (CD). Both the longitudinal stress and contact pressure of concrete decrease continuously due to the failure of concrete and the development of local buckling at the outer steel tube, as shown in Fig.8(d) and Fig.9 (d), respectively.

It can be found from Fig.9 that the contact stress tends to be greater near the corners than in-between the corners during the whole loading process, which is similar to that of the single-cell hexagonal CFST under compression [14]. Moreover, the contact stress distributes symmetrically on the two sides of the internal steel webs during the whole process. It should be noted that the contact stress is obviously greater at the position where the outer steel plates intersect with the internal webs. This is because the internal webs can effectively restrain the out-of-plane deformation of the outer steel plates at this position during the axial compression testing.

4. Parametric study

Parametric study is conducted on the compression behaviour of the hexagonal multi-cell CFST columns based on specimen CC-5 in Xu et al. [25]. The aims of the parametric study are to extend the parameters which were not covered by the limited tests, and to provide data to derive a formula for predicting the ultimate strengths of hexagonal multi-cell CFST columns. All specimens have identical overall sectional dimensions to specimen CC-5. The basic parameters used in the calculations are: t_o=3mm, t_i=3mm, f_{yo}=345MPa, f_{yi}=345MPa, f_{cu}=40MPa, $E_c = 4730\sqrt{f_c^{'}}$, where $f_c^{'}$ is the cylinder strength of the concrete. The overall width of the hexagonal cross-sections are B = 262 mm and D = 562 mm, with an internal angle being θ =135°. The height of all specimens, H, is 1000 mm. The height of all specimens were designed to be less than three times the overall width of hexagonal section to avoid the effects of overall buckling (SAA [32], Zhao and Hancock [33], Han [34]). The fixed-fixed boundary conditions are assumed. The bottom end of the column is fixed, while the top end is only axially moveable. The load is applied on the top end. The investigated parameters include concrete strength (f_{cu}), internal steel webs (f_{yi}), yield stress of outer tube (f_{yo}), wall thickness of internal steel webs (t_i) and wall thickness of outer steel tube (t_o). The values of the parameters are determined based on construction practice. The ranges of the parameters are: f_{cu}=40~120MPa, f_{yi}=235~490MPa, f_{yo}=235~490MPa, t_i=1~5mm (α_i: 0.92%~4.60%), and t_o=1~5mm

(α_o: 1.22%~6.09%).

The effects of various parameters on the *N-Δ* relationships are presented in Fig.10. It can be noticed that f_{cu} has minor effect on the initial stiffness of the columns (Fig.10 (a)). The main differences in curves are found in post-yield stage, where the ductility decreases as f_{cu} increases. The ultimate strength of the columns increases as the value of f_{cu} increases. Moreover, the *N-Δ* relationships of normal strength concrete (C40 and C60) decreases gradually to the residual strength in the descending stage. By contrast, the *N-Δ* relationships of high strength concrete (C80, C100 and C120) decrease sharply after the peak load. This is probably because the confinement effect decrease with the increase of concrete strength. The influence of other parameters shows similar trends. From Figs.10 (b)-(e), it can be noticed that the ultimate strength and residual strength increase as these parameters increase. The parameters of f_{yo} and f_{yi} have minor influence on the initial stiffness, probably because the strength of steel has minor influence on its modulus of elasticity. By contrast, the parameters of t_i and t_o have moderate effect on the initial stiffness. The initial stiffness shows an increasing trend when either t_i or t_o increases.

The effects of various parameters on the ultimate strengths of hexagonal multi-cell CFST columns under axial compression are presented in Fig.11, where the linear regression are also presented. It can be seen from Fig.11 (a)-(e) that the ultimate strengths increase linearly as the values of the parameters increase. It is found that f_{cu} shows the most significant influence on the ultimate strength (Fig.11(a)), since the concrete contributes most to the axial load bearing capacity. Comparing Fig.11(b) and (d), it is found that increasing either f_{yi} or t_i can increase the ultimate strength. However, it is more economic to use higher steel grades (to increase f_{yi}) than to increase t_i to achieve a higher ultimate strength. The same phenomenon can also be observed for outer steel tube as seen in Fig.11(c) and (e). Comparing Fig.11(d) and (e), it is found that increasing the wall thickness of outer steel tube (t_o) is more effective in increasing the ultimate strength than the internal steel webs. The main reason is that the cross-sectional area of the outer steel tube is greater than that of the internal steel webs. Another possible reason is that the local buckling of the outer steel tube is less likely to occur when t_o increases (the width-to-thickness ratio decreases), which provides stronger concrete confinement.

The effects of various parameters on the deformation patterns of hexagonal multi-cell CFST columns at ultimate point are presented in Fig.12. For clarity, the deformation scaling factor is set as 10. Comparing the five parameters investigated, it is found the wall thickness of outer steel tube (t_o) has a significant influence on the failure modes (Fig.12 (e)), while the others have a minor influence on the failure modes, as shown in Fig.12 (a)-(d). Local buckling is observed when the wall thickness of outer steel tube (t_o) is less than 3 mm, as shown in Fig.12 (e). By contrast, global outward bulge is observed at the mid-height when t_o exceeds 3mm.

The effects of wall thickness of internal steel webs (t_i) on the ultimate strength of the column are illustrated in Fig.11 (d), where t_o=5mm represents the case of global outward bulge failure mode and t_o=3mm represents the case of local buckling failure mode, respectively. It can be seen in Fig.11 (d) that the ultimate strength shows increasing trends as t_i increases in both cases. The increasing rate of ultimate strength is approximately equal to the yield strength of inner steel webs for global outward bulge, while that is slightly lower for local buckling failure mode. This is because the confinement effect on the core concrete is enhanced when t_o increases. As can be seen in Fig.11(e), the increasing rate of ultimate strength in the case of t_i=5mm is approximately equal to that of t_i=0mm. It seems that t_i has minor effect on the confinement effect. It should be noted that Xu et al. [14] found that the ultimate compressive strength of dual-axisymmetric hexagonal single-cell CFST stub columns could be calculated accurately according to the equations of rectangular CFST columns in Eurocode 4 [16] and DBJ/T13-51-2010 [19]. Hence, the simplified equations presented in DBJ/T13-51-2010

[19] are selected temporarily to calculate the ultimate strength of axial compressive dual-axisymmetric hexagonal CFST stub columns with t_i=0 mm.

The effect of width-to-thickness ratios of outer steel plates on the behaviour of the hexagonal multi-cell CFST stub columns under axial compression is presented in Fig.11(f). The variations of width-to-thickness ratio were attained by changing the wall thickness of outer steel tube while keeping other parameters constant. Since the outer steel tube is made up of several piece differing in width (b_i), the b_i/t_o of each piece was calculated and the maximum $(b_i/t_o)_{max}$ is taken as the investigated parameter. It can be noticed in Fig.11(f) that the ultimate strength of the hexagonal multi-cell CFST stub columns significantly increases with the decrease of $(b_i/t_o)_{max}$ when it is not exceed 46.3. This value is a critical value relating to the failure mode. When $(b_i/t_o)_{max}$ is not exceed 46.3, global outward bulge failure mode is likely to occur. Otherwise, the longitudinal local buckling failure mode is likely to occur. It illustrates that the effects of $(b_i/t_o)_{max}$ on the ultimate axial strength is obvious in the case of the global outward bulge failure mode. When $(b_i/t_o)_{max}$ exceeds the critical value, the ultimate axial strength increases slowly with the decrease of $(b_i/t_o)_{max}$. Also, it can be found that the critical value is approximately equal to the slenderness limit of $52\sqrt{235/f_y}$ specified in Eurocode 4 [16].

By comparison, it is unlikely that local buckling could occur in the internal steel webs since the internal steel webs are encased in the concrete. It is expected that the width-to-thickness ratio of internal steel webs has minor effect on the mechanical behaviour of the hexagonal multi-cell CFST columns. Hence, the influence of width-to-thickness ratio on the ultimate strength was not studied. It is certain that the thickness of the internal steel webs affects the ultimate strength and ductility of the columns to some extent. But according to the design code for steel structures, the internal steel webs cannot be too thin in thickness to facilitate welding and construction.

5. Discussions on the ultimate load

Based on the above analysis, It can be assumed that the load carried by the multi-cell CFST stub column at peak point can be divided by two parts, one part is the load carried by a single-cell CFST column which is composed by the outer hexagonal steel tube and the concrete, and the other part is the load carried by the internal webs. Thus, the ultimate strength ($N_{u,prop}$) of hexagonal multi-cell stub column can be obtained by using superposition method as follows:

$$N_{u,prop} = N_{osc,u} + N_{i,u} + N_{b,u} \tag{2}$$

Where, $N_{osc,u}$ is the load carried by the hexagonal single-cell CFST column, $N_{i,u} = f_{yi}A_{si}$ is the load carried by the internal steel webs and $N_{b,u} = f_{yb}A_{sb}$ is the load carried by rebar cage, if any.

Xu et al. [14] pointed out that the confinement effect and the stress distribution of the hexagonal CFST column with internal angle θ=135° are similar to those of the rectangular one. And they found that the methods of calculating the sectional strength of rectangular CFST column proposed in DBJ/T13-51-2010 [19] and EC4 [16] give reasonably good predictions when used to predict the strength of hexagonal CFST column. In the Chinese code DBL/J1351-2010 [19], the CFST column is assumed to be made of one type of material with a nominal yield strength factor of f_{scy}, and a confinement factor (ξ) is used to describe the composite action between the steel tube and concrete. To consider the concrete confinement, the method in DBJ/T13-51-2010 [19] is adopted herein to calculate the sectional strength of hexagonal single-cell hexagonal CFST. The formulae are as follows:

$$N_{osc,u} = f_{scy}(A_{so} + A_c) \tag{3}$$

$$f_{scy} = (1.18 + 0.85\xi_0) f_{ck} \tag{4}$$

$$\xi_0 = A_{so} f_{yo} / A_c f_{ck} \tag{5}$$

Where, f_{scy} is the nominal average strength of a square or rectangular steel tube section with concrete infilled, in MPa; f_{ck} is the characteristic compressive strength of concrete, in MPa. The value of f_{ck} is approximately equal to 67% of the cube strength of concrete (f_{cu}) for normal strength concrete. ξ_0 is the sectional confinement factor.

To verify the accuracy of Eq. (2), the ultimate strength of hexagonal multi-cell CFST stub columns obtained from the FEA model in the parametric study are used. The ultimate strengths are also calculated with the simplified formulae. The mean value of $N_{u,prop}$/ $N_{u,FEA}$ is 0.902 with a corresponding COV of 0.034, where $N_{u,prop}$ and $N_{u,FEA}$ are the formula-calculated and FEA predicted values, respectively. Moreover, available test data from Cao et al. [24] and Xu et al. [25] are also compared with the formula-calculated ones, as shown in Table 1. The mean value of $N_{u,prop}$/ $N_{u,test}$ is 0.959 with a COV of 0.033, where $N_{u,test}$ is the ultimate strength obtain in tests.

The comparisons between the experimental results and the calculated results predicted by other existing design standards using the plastic strength approach have also been presented in Fig.13, such as Eurocode 4 [16], ACI 318-14 [17], ANSI/AISC 360-16 [18], AIJ-2008 [20], BS5400 [21] and AS3600 [22]. It should be noted that Eurocode 4 [16], ACI-318-14 [17], AS3600 [22], AIJ-2008 [20] and BS5400 [21] use the similar formula for calculating the ultimate axial capacity of the rectangular sectional CFST stub columns, in which the confinement effect between the steel tube and concrete is ignored and a factor of 0.85 is employed for concrete strength. The above five equations have minor difference with the equation presented by ANSI/AISC 360-16 [18], in which the longitudinal reinforcement is converted to concrete by multiplying its area with the value of E_s/E_c.

It can be seen in Fig.13 that the predicted results which are calculated using the plastic strength approach from Eurocode 4 [16], ACI 318-14 [17], ANSI/AISC 360-16 [18], AIJ-2008 [20], BS5400 [21] and AS3600 [22] are conservative mainly because they ignore the confinement effect between the steel tube and concrete. By contrast, Chinese code DBL/J13-51-2010 [19] accounts for the concrete confinement and gives the most accurate results.

In general, Eq.(2) which based on DBL/J1351-2010 [19] gives reasonably good predictions when compared with numerical and experimental results. It seems that the proposed method can well predict the ultimate strength of the hexagonal multi-cell CFST stub columns.

6. Conclusions

This paper numerically investigates the performance of hexagonal multi-cell CFST stub columns under axial compression. The following conclusions can be drawn within the limitations of this study:

(1) A simplified FEA model of hexagonal multi-cell CFST stub columns under axial compression is established and validated against existing experimental data. In general, the FEA model can predict the behaviour of hexagonal multi-cell CFST stub columns subjected to axial compression with reasonably good accuracy.

(2) A mechanism analysis including full-range analysis and stress distribution are performed using the validated FEA model. It is found that the contact stress between the outer steel plate the concrete tends to be greater near the corners than in-between the corners during the whole loading process, which is similar to that of the single-cell hexagonal CFST. The contact stress on each side of the internal steel webs distributes symmetrically during the whole process.

(3) A parametric study is conducted to investigate the sensitivity of various geometric and material parameters on the compressive behaviour of the hexagonal multi-cell CFST stub columns. It is found that the ultimate strength increases linearly as the values of the investigated parameters increase. It is found that the concrete strength shows the most significant influence on the ultimate strength, since the concrete contributes most to the axial load bearing capacity. Increasing the

concrete strength can be the most effective method to increase the ultimate strength. Increasing the wall thickness of outer steel tube is more effective in increasing the ultimate strength than the internal steel webs. Increasing the wall thickness of internal steel webs can effectively increase the ultimate strength, while it seems that t_i has minor effect on the confinement effect on the core concrete.

(4) A simplified formula for calculating the ultimate strength of axially loaded hexagonal multi-cell CFST stub columns is proposed based on the methods of calculating the sectional strength of rectangular CFST column proposed in DBJ/T13-51-2010 [19]. The contributions of the single-cell hexagonal CFST column and the inner steel elements have been taken into consideration individually in the formula. The proposed formula can provide reasonably good predictions when compared with numerical and experimental results.

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