Experimental research on the CFST space intersecting connections

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ABSTRACT:

In this paper, a new tube-in-tube structural system is presented, in which the concrete filled steel tube(CFST) obliquely crossing lattice is used as structural outer tube in an ultrahigh-rise building with height of 432m. The detail of CFST spatial intersecting connection which satisfies the 'stronger connection' criteria is proposed. Four specimens of the connections are tested on a set of self-equilibrating steel loading equipment with the bearing capacity of 20000kN, with the purpose of obtaining the failure mode, ultimate capacity and collapse procedure. Before the test performed, a finite element analysis on the full range of test setup is conducted, which calculate the requirement of the lateral stiffness for the connections. As a result, the effect of the lateral deflection and necessity of design with good lateral confinement for the connections is verified.

KEYWORDS: concrete filled steel tube (CFST), obliquely crossing lattice, test, finite element analysis

1. INTRODUCTION

Recently, a new tube-in-tube structural system has become widely used in the world, with the term 'diagrid structures' or 'diagonal frame structures'. Diagrid structure system is characterized by vertical components which are not conventional columns but lattices that obliquely crossing at a certain angle, distinguished from the common portal frames. This system emerges both structurally efficient and architecturally aesthetic, which are exactly good for used in high rise buildings. In recent high rise buildings applying tube-in-tube structural systems, the Swiss Re Building in London, the Hearst Headquarters in New York, the Building of Qatar Ministry of Foreign Affairs in Doha, and the Guangzhou West Tower in China (with height of 432m) have used diagrid system as the outer tube and been favorably accepted by both the public and experts.

The obliquely crossing lattices in diagrid structural systems usually carry gravity loads as well as lateral forces owing to their triangulated configuration. Therefore the components will bear greater axial loads than the columns in common frames. As a composite structure that takes advantage of both concrete and steel, concrete filled steel tubular (CFST) structures can behave elastically under great axial loads due to the confinement effect provided by the steel tubes, hence are good alternatives of the obliquely crossing members.

CFST diagrid structures usually have spatial intersecting connections comprised of four obliquely crossing CFST columns. Figure1 shows that the cross-sectional area at the connection is reduced to that of each CFST column, so the connection details become the key link in the structural design. There appears to have been no systematic investigation of these details. This paper proposes a new type of connection detail and four 1/5.86-scale specimens were tested under monotonic axial loading. The deflection, stress, failure modes and capacity of the specimens are experimentally obtained and analyzed. In addition, the connections have been applied on the Guangzhou West Tower. Some design recommendations of the connections are proposed based on the previous analysis.

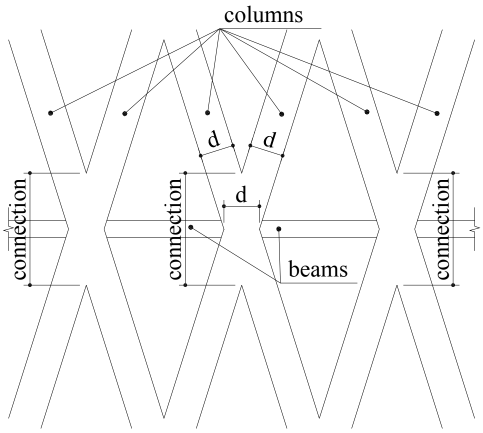


Figure 1 A typical connection in diagrid structures

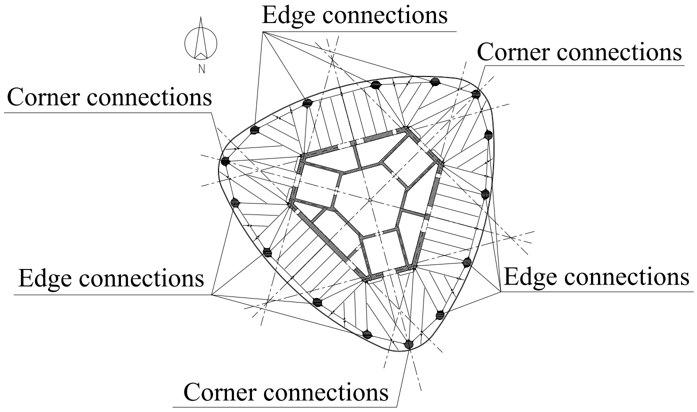


Figure 2 Corner and edge connections in the structure

2. EXPERIMENTAL INVESTIGATION

Four 1/5.86-scale specimens of CFST connection were tested under monotonic axial loading. The purpose of the study was (1) to investigate the failure modes of the connections, (2) to verify the ultimate capacity of the connection under axial loading, and (3) to measure the steel tube's strain distribution, the axial deformation and the lateral deflection of the specimens through the entire test procedure, (4) to validate whether the connections satisfy the 'stronger connection' criteria and (5) to assess the effectiveness of the connection in practical use.

2.1. Test Specimens and Parameters

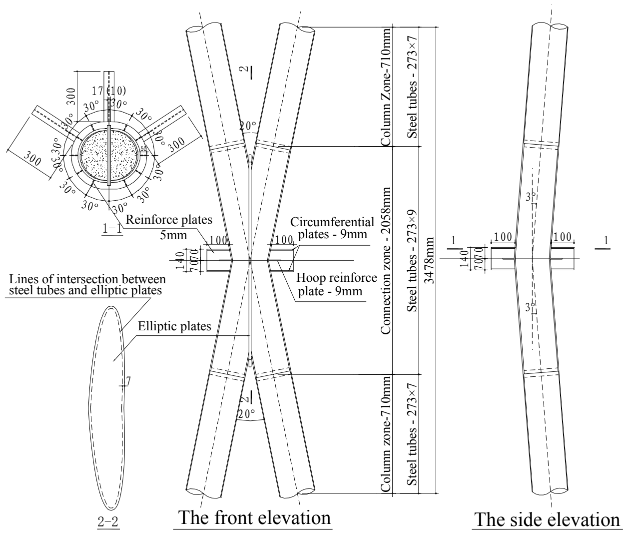
According to the design scheme of Guangzhou West Tower shown in Figure 2, two types of connections are used in the structure: corner connections and edge connections. Since the plane area of the outer tube varies with the floor and height, the four columns of both two connections are not only obliquely intersecting at an angle of 20° or 35° in the vertical plan, but also have an out-of-plan angle of 3° or 1° respectively. Furthermore, connections are divided by the depth of elliptic plate, the term 'thick connection' and 'thin connection' presents depth of 17mm and 10mm respectively. As the details of each specimen shown in Table 1 and Figure 3, the four columns were welded on an elliptic plate, then one hoop reinforce plate and two circumferential plates were welded outside the connection. In addition, several reinforce plates were welded between them. The specification of the connections are convenient to manufactured in the factories and facilitate the construction process.

Table 1 Number of specimens

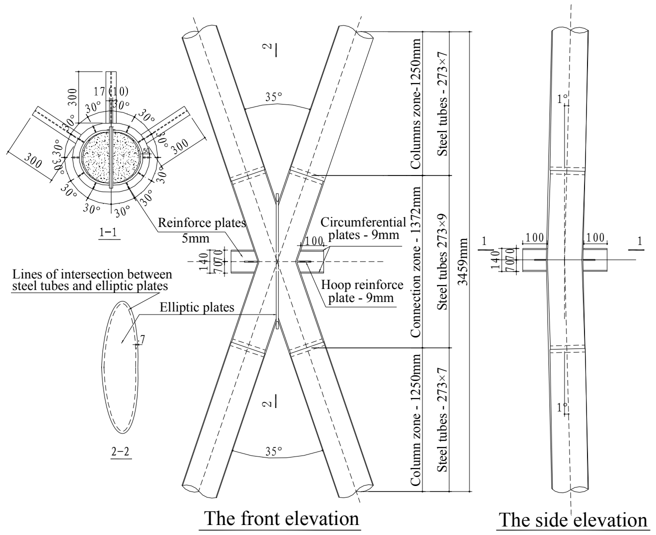
Depth of elliptic plate	corner connection $\alpha=20°, \beta=3°$	edge connection $=35°, \beta=1°$
17mm (thick connection)	A-1	B-1
10mm (thin connection)	A-2	B-2

Steel tubes and plates were manufactured using mild structural steel with a nominal yield stress of 345MPa. The material properties of the steel tubes and plates were obtained from tensile tests of coupons taken from each steel plate before manufacturing. The concrete grade in the central connection region of all specimens was C90, where the depths of steel tubes were reinforced to 9 mm. In addition, the concrete grade outside the connection region was C70 and depths of steel tubes were 7mm. Compression tests were carried out on three cube specimens (150mm in lengths) to determine the compressive strength of the concrete.

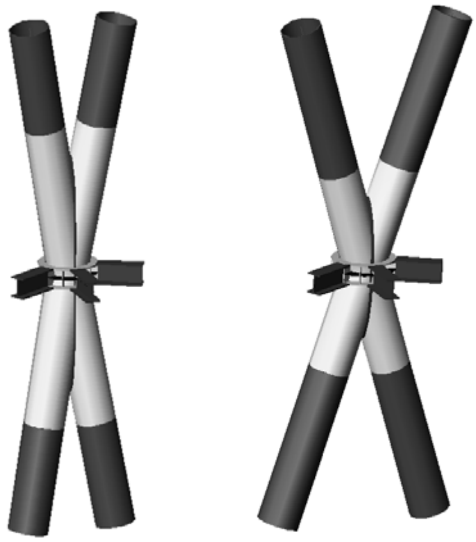
The test parameters were (1) the separation angle between left and right columns: 20° and 35° while the angle between columns and vertical direction were 3° and 1°; (2) the depth of elliptic plate: 17mm and 10mm. Before the test, two steel sleeve barrels with 10 mm wall thickness were welded on the loading end while a 20mm plate was welded on the fixed end.



(a) Corner connection specimens (A-1, A-2)



(b) Edge connection specimens(B-1, B-2)



(c) 3D schematic diagram of two connections

Figure 3 Specimens details

Figure 4 Test setup

2.2. Test procedure

Two sets of loading systems including a 10000kN hydraulic jack and a related sensing unit were used to apply the axial load on each column in turn. Due to the columns intersected obliquely, each of the systems had a support comprised of two steel plates welded on three rib stiffeners so that the hydraulic jacks could apply axial loads on the specimens. In addition, one self-equilibrating steel loading equipment with horizontal capacity of 20000kN and vertical capacity of 2000kN was designed to bear the counterforce generated by two jacks and three steel pull rods (Figure 4). Those pull rods were set to simulate the lateral constraints provided by main girders and ring beams.

A detailed instrumentation procedure was used to measure the strains and deformations. The steel tubes in the connections and the columns were instrumented with 84 strain gauges and 20 dial gauges connected to a computer data acquisition system to record their values. Any one of three conditions concluded the test: (1) The ultimate capacity decreased to 85% of the maximum, (2) the maximum deformation reached 200mm, or (3) obvious features of failure occurred on the specimens.

3 A FINITE ELEMENT ANALYSIS SIMULATION OF THE TEST SETUP

Before the test procedure, a simulation using finite element analysis (FEA) of the entire test setup was conducted to determine the diameter of the three steel pull rods (Figure 5). As mentioned in section 2.2, the rods provided significant lateral stiffness to the connections. In addition, the loading frame screwing the pull rods would have bending deflection, further led to affect the lateral stiffness provided by the pull rods. To obtain a more accurate experimental result, an elastic simulation considering the stiffness of both concrete and steel tubes is essential.

In this FEA model, the tangent modulus of concrete C70 and C90 were 37.0GPa and 39.0GPa respectively, which are defined by Chinese code for design of concrete structures (GB50010-2002). Meanwhile, the tangent modulus of steel was set to 206GPa which was measured before test. One of the pull rods was vertical, while the others had an horizontal angle of 28.5° . The loading equipment was pinned at four corners. The diameter of pull rods were calculated by comparing the axial forces to the prototype structures, shown in Table 2. As a result, the eccentric ratio e of prototype structures and FEA results was consistent.

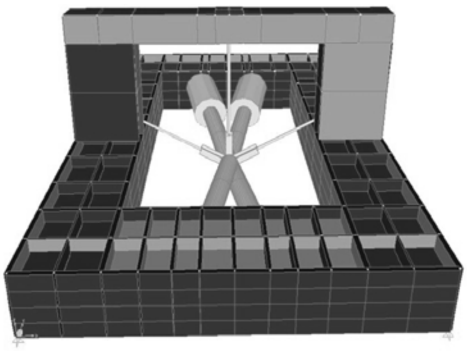


Figure 5 FEA model diagram

Table 2 Comparison of internal forces of the prototype structure and FEA results

connection	model	vertical rods (mm)	other rods (mm)	diameter of steel tubes d (m)	axial forces N (kN)	bending moment M (kN·m)	$e=\frac{M}{N\bullet d}$
corner	prototype	/	/	1.6	-87889.1	11085.2	0.079
	FEA model	2 ϕ 48	2 ϕ 42	0.273	-5001.4	106.0	0.078
edge	prototype	/	/	1.6	-80292.3	5729.0	0.045
	FEA model	1 ϕ 42	2 ϕ 42	0.273	-5000.3	60.7	0.044

4 EXPERIMENTAL RESULTS

4.1. Failure modes

The pertinent measurements and observations extracted from the experiment included failure modes, load-steel strain and load-end deflection results.

a) bulging at connection zone b) Lüders slip lines in part of tubes c) bulging at the support end

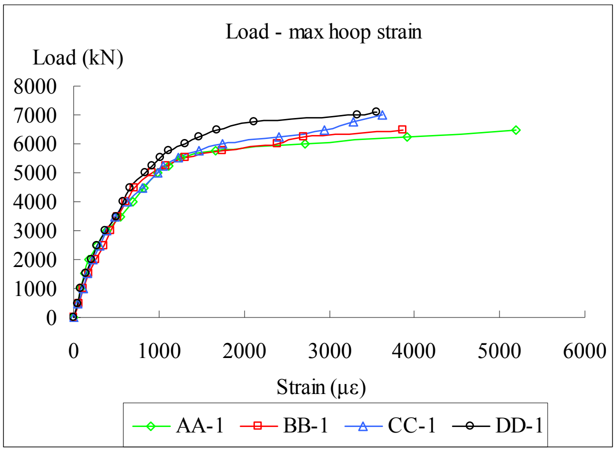
Figure 6 Failure modes

It was observed that the failure modes could be divided into two types. For the specimens with a separation angle equal to 20°, the failure was due to the tube bulging within the connection zone (Figure 6a). It produced some Lüders slip lines (Figure 6b) along the surface of the tubes. In addition, bending in-plane occurred in some part of the specimens. For the specimens with a separation angle equal to 35°, the failure was due to the tube bulging outside of the connection zone. It also produced some Lüders slip lines along the surface of tube near both the support ends and loading ends (Figure 6c).

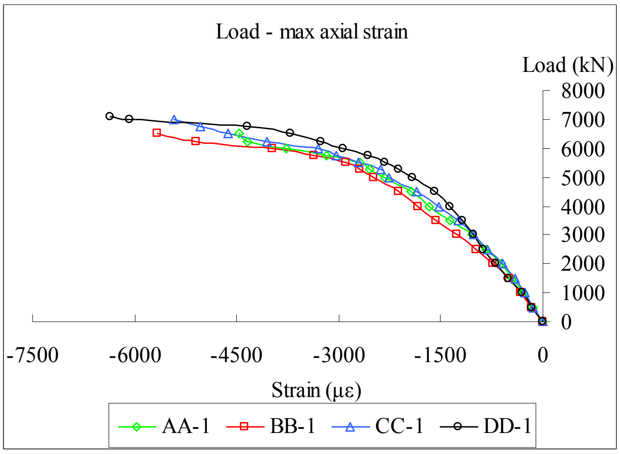
4.2. Load-steel strain and load-deflection

The curves which illustrated the relationship between axial load and maximum hoop strain, maximum axial strain, axial deflection and lateral deflection of the specimens are illustrated in Figure 7a, b, c, d respectively.

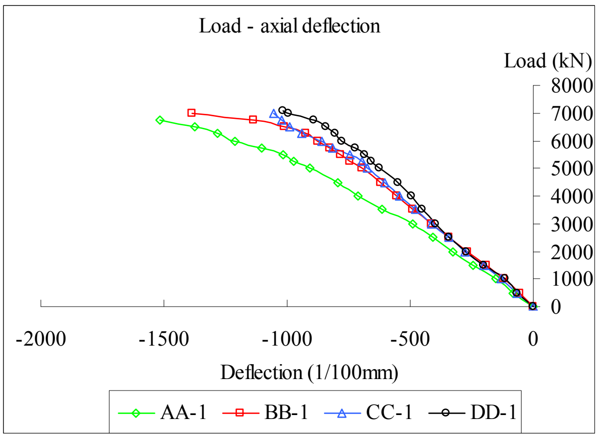
The curves of Figure 7a and b illustrate the point of steel yielding, the maximum load and the ductility of the element. It can be seen that the trend of the curves are corresponds well with those of short CFST columns in previous research conducted in China and Japan. The four significant events for each specimen during testing are as follows: (1) the steel tube was compressed in the axial direction while tensioned in the circular direction, and the axial strains increased faster than the hoop strains, (2) the axial and hoop strains continued increasing linearly until the axial strains reached 2000με, or the axial loads were below 4500kN~5000kN, (3) subsequently, the strains increased abruptly with a non-linear trend, which indicates that the steel tubes yield under a axial compression and a hoop tension stress, (4) although the tubes were in yield phase, the load could be increased until it reached 6800kN ~ 7200kN due to the stress redistribution which occurred between the concrete and tubes. Some Lüders slide lines occurred along the surface of the tubes.



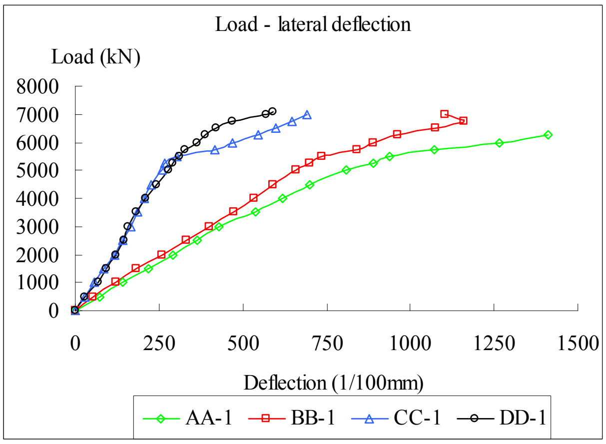
a) Axial load-maximum hoop strain



b) Axial load-maximum axial strain



c) Axial load-axial deflection



d) Axial load-lateral deflection

Figure 7 Load-steel strain and load-deflection curves

The curves of Figure 7c illustrate that the axial deformation increased linearly until the load reached 7000kN, so that the axial stiffness of the connections is applicable to tall buildings. In addition, it can be seen that the lateral deflection increased extremely fast after the tube stress achieves yield stress, when the load reached 5500kN. It shows that the out-plane lateral stiffness of the connections is not as good as the axial stiffness and lateral braces are necessary.

4.3. Comparison of the test results

4.3.1 Results and analysis of bearing capacity

Table 3 compares the design values and test results of the ultimate axial load at each single column of the connections. The actual axial capacities of the connections were 3.57~4.47 times the maximum design values, so that the design strengths of the connections were reliable.

Table 3 Comparison of bearing capacity of the specimens

Specimen	A-1	A-2	B-1	B-2
Elastic limit load (kN)	4500	4500	5250	5750
Bearing capacity load (kN)	6750	7000	7000	7100
Corresponding prototype load (kN)	231792	240377	240377	243811
Designed prototype structure load standard combination values (kN)	64863	64863	54556	54556
Safe coefficient	3.57	3.71	4.41	4.47

4.3.2 Corner connections and edge connections

Figure 6 shows that the corner connections bulged in the connection zone while the edge connections bulged in the column zone. The difference appeared in failure modes can be interpreted as: (1) the bearing capacity of the corner connections are lower than that of the edge connections, (2) the axial loads applied on the corner connections are larger than that on the edge connection due to the in-plane separation angles, (3) the bending moment applied on the corner connection are larger than that on the edge connection due to the out-plane separation angles. Corner separation connection needs to be reinforced appropriately to adhere to the seismic design principle “stronger connection, weaker components”, which stipulate that the capacity of the connection or joints must be larger than that of the components connect to them.

4.3.3 Thick connections and thin connections

Figure 7 and Table 2 show that the capacity of the thick corner connection is slightly higher than that of the thin corner connections. On the other hand, the capacity of the thick edge connection and the thin edge connection are almost equal.

4.4. The effect of the lateral deflection

As mentioned in section 2.2 and previous analysis, the three steel rods which simulate tie effect of beams regulated the lateral deflection, provided the lateral stiffness for the connections. According to the test results, the lateral deflections of corner connections are larger than that of edge connections, resulting in the bearing capacity decrease. Hereby, it can be concluded that increment of the lateral deflection will lead to the reduction of the bearing capacity. Moreover, assuming that those steel rods or beams are not designed in the test or the prototype structure, the lateral deflection will increase nonlinear rapidly on account of out-plane component of the axial force and the P-Δ effect, finally failure due to bending moment. Therefore, the obliquely crossing connections need to design with good lateral confinement.

5 CONCLUSIONS

The failure modes and bearing capacity of four connections were obtained from experimental results, which offer the basis of the structural design. The bearing capacity of both connection details can meet the values required by structural design. In fact, the ratios of the actual axial capacities to the design values of the connections were from 3.57 to 4.47. The separation angle of the connections will result in different failure modes. Corner separation connection needs to be reinforced appropriately to adhere to the seismic design principle "stronger connection, weaker components" which was defined by current codes. Experimental results can be substantiated to be correct and usable by comparing to the FEA results. Finally, the obliquely crossing connections are verified necessary to design with good lateral confinement.

The behavior of the CFST spatial intersecting connections in one specific ultrahigh-rise building were studied under axial monotonic compression loads. The experimental program and numerical analysis was conducted on connections with two specific detail and two specific separation angles. Therefore, the conclusions may or may not be valid for other connections. Further research is also still needed on CFST oblique crossing connections with different details used in the field.

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