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Innovative design of the world's tallest electrical transmission towers

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Providing a new 500kV electricity supply to Zhoushan Islands off the east coast of China in 2018 involved building the world's two tallest transmission towers. The 380m typhoon-resistant towers enabled high-voltage cables to span 2.7km over the East China Sea. To reduce wind loads, the towers were made from a tubular steel filled with steel angle-reinforced, self-compacting concrete. This paper describes the design and detailing of the innovative composite members and joints specifically developed for the project. It aims to serve as a design reference for future long-span electrical transmission towers.

Notation

A_c	cross-sectional area of concrete
A_{si}	cross-sectional area of built-up latticed angles
A_{so}	cross-sectional area of steel tube
C_{si}	total circumference of the encased angles
D	tube diameter
D_b	tube diameter of the brace
E_c	elastic modulus of concrete
E_{si}	elastic modulus of steel tube
E_{so}	elastic modulus of encased angle
EA	initial stiffness of tensile member
e	load eccentricity
$F_{Built-up}$	design compressive strength of latticed built-up angles
F_{CFST}	design compressive strength of the concrete-filled steel tube (CFST) member
$F_{u,AXT}$	punching shear design strength for the connection under axial tension
$F_{u,ECT}$	punching shear design strength for the connection under eccentric tension
f_{ck}	characteristic compressive strength of concrete
f_u	ultimate strength of steel
f_{ui}	ultimate strength of steel angle
f_{uo}	ultimate strength of steel tube
f_y	yield strength of steel
f_{yi}	yield strength of steel angle
f_{yo}	yield strength of steel tube
L	column length
l_b	gusset plate length
M_p	ultimate flexural strength of the cross-section of embedded components
$M_{sc,u}$	ultimate flexural strength of the CFST member
M_u	flexural strength of reinforced CFST members
$M_{u,AXT}$	flexural strength of reinforced CFST members under axial tension
$M_{u,IB}$	punching shear design strength for the connection under in-plane bending
T	tube-wall thickness of the compressive and tensile members
t	tube-wall thickness of the chord

t_b	tube-wall thickness of the branch members
α_T	steel ratio, A_{so}/A_c
β	stress coefficient
β_{si}	stress coefficient of the angle
β_{so}	stress coefficient of the tube
γ	flexural strength index
γ_m	flexural strength index according to Han (2006)
η	stress coefficient
η_{si}	stress coefficient of encased angles
θ	brace angle
λ	slenderness ratio
ζ	confinement factor, $A_{so}f_{yo}/A_c f_{ck}$
ρ_{si}	reinforcement ratio, $A_{si}f_{yi}/A_c f_{ck}$
$\rho_{si,T}$	angle/tube ratio, $f_{yi}A_{si}/f_{yo}A_{so}$
ϕ_{si}	angle property factor
χ	ratio of reinforcement to confinement factor
ω	load eccentricity ratio of the CFST specimen, $2e/D$

1. Introduction

A 500kV electrical transmission project between Zhoushan Islands and the city of Ningbo on the eastern China mainland was initiated in 2007 to solve the problem of electricity supply shortages on the islands. The two-stage project was approved by the government of Zhejiang Province and managed by Zhejiang Electric Power Corporation and Zhoushan Electric Power Bureau. Design was by Zhejiang Electric Power Design Institute Co., Ltd and construction was by Zhejiang Power Transmission and Transformation Corporation.

The first stage was for an overhead transmission line across the Luotou channel of the East China Sea between Damao and Liangmao Islands, a distance of 2750m (Figure 1). This required the design and construction of two 370m tall transmission towers, which until 2018 were the tallest in the world (Figure 2(a)). Completed in 2010, the total cost of the crossing was approximately US\$85 million.

The second stage was for a submarine transmission line from Ningbo to Jintang Island, which was then carried 2656m over the Xihoumen channel to Cezi Island by means of two 380m tall

Figure 1. Plan of the 500 kV Zhoushan–mainland electrical transmission lines

Figure 3. Schematic view of the Xihoumen cable crossing completed in 2018

towers (Figure 2(b)) and three smaller anchorage towers (Figure 3). Construction started in December 2016, the towers were completed in October 2018 and the line was commissioned in December 2018. The cost of the Xihoumen crossing was US\$91 million.

Like the earlier 370 m tall towers, concrete-filled tubular steelwork was adopted for the 380 m tall tower structures based on comprehensive consideration of external actions, manufacturing, construction and design codes. However, some challenging technical problems needed to be solved during design, manufacture and construction.

This paper introduces the key techniques adopted and the innovative structures proposed for the towers. The structural performance and the design methods are also presented. Close collaboration between the structural designers and university researchers was of great importance during the design and construction processes to overcome the challenges and achieve a successful completion.

2. Structural selection

The two towers are identical and located at Jintang and Cezi Islands, respectively. The steel weight of each tower is 7280 t, making them the heaviest transmission towers in the world. The four main columns were constructed up to a height of 262 m using

Figure 2. One of the 370 m tall electrical transmission towers completed in 2010 (a) and one of the 380 m tall versions completed in 2018 (b)

concrete-filled steel tubes (CFSTs) with encased reinforcement lattices made from steel angles.

The maximum diameter and tube-wall thickness of the column tubes is 2300 mm and 28 mm, respectively. The tubes are made from Q345 steel (nominal yield strength of 345 MPa) and the 200 mm × 200 mm × 24 thickness angles are made from Q420 steel.

A total of 3200 m³ C50 (28 d strength of 50 MPa) self-compacting concrete fill was pumped into position in several lifts.

The towers are located in a typhoon-prone sea strait. The design wind speed was 41 m/s based on a report by the Meteorological Bureau of Zhejiang Province. Huang *et al.* (2012) performed experimental and numerical investigations into wind effects on the towers.

The preferable structure profile was a circular hollow section (CHS) tubular structure, which has a lower wind-load shape coefficient compared with a steel angle lattice structure. According to the preliminary design, adopting CHS for the main columns would have required a maximum diameter of 2300 mm to meet the requirements of external actions.

Furthermore, the tube-wall thickness would have had to be at least 32 mm for Q345 steel according to the Chinese code for design of steel structures (SAC, 2003a), in which the ratio of diameter (D) to thickness (T) is limited to $100(235/f_y)$.

Eurocode 3 (BSI, 2005a) has a stricter limit for the sectional slenderness ratio (D/T) than the Chinese code. The cross-classifications in Eurocode 3 (BSI, 2005a) for compressive members were catalogued as class 1, 2, 3 and 4, based on the cross-section's local buckling resistance. The D/T limit values for tubular sections were defined as $50(235/f_y)$, $70(235/f_y)$ and $90(235/f_y)$ for class 1–3 sections, respectively. Therefore, even if the cross-section had been designed as class 3, which may have used parts of plastic moment resistance due to local buckling; taking $D/T = 80(235/f_y)$ for example, the tube-wall thicknesses would have been larger than 40 mm for Q345 steel and approximately 50 mm for Q420 steel. It should be noted that this was only for the consideration of local buckling. Furthermore, the relatively thick tube wall would have introduced problems in both manufacturing and construction processes.

The design strength of thick steel plate was factored due to defects and manufacturing processes, which indicated that increasing the thickness of the tube walls might not be of much benefit to increase the ultimate capacity of members according to the Chinese code.

For instance, the design strengths for 32 mm and 35 mm thick steel plate (Q345) were 295 MPa and 265 MPa, respectively, which indicated that if the tube thickness increased from 32 mm to 35 mm, the compressive material strength ($f_y A_{so}$) of a 2200 mm CHS would decrease from 64 296 kN to 63 084 kN. Furthermore, increasing the tube-wall thickness would not be uneconomical but would also bring additional problems in structural performance.

In the Chinese code, steel with a yield strength (f_y) larger than 345 MPa and a ratio of yield strength to ultimate strength (f_y/f_u) larger than 0.8 should not be used in hot-rolled and cold-formed tubular structures. In addition, tube-wall thickness was advised to be no more than 25 mm for considerations of machinability, formability and ductile recovery after working processes to prevent lamellar tearing during plate manufacturing. Alternatively, special processing techniques could have mitigated lamellar tearing but it would have increased costs by approximately 40%.

For the tower columns designed using hollow sections, the tube-wall thickness would have had to be more than 40 mm to meet the requirements of both strength and stability. From a manufacturing perspective, a special machine for plate-bending would have been necessary to achieve sufficient accuracy. Moreover, special attention would have been needed to ensure the welding quality at joints.

2.1 Reinforced concrete infill solution

CFSTs have been renowned for their superior mechanical performance and excellent constructional properties for more than half a century (Han, 2006; Han *et al.*, 2014; Perea *et al.*, 2013; Uy, 2001; Wang *et al.*, 2018a, 2018b). They have been widely applied in building structures, bridges, power transmission towers and marine structures to utilise the advantages of both steel and concrete materials (Han *et al.*, 2014).

Therefore, the feasibility of applying CFST structures in the Zhoushan transmission towers was investigated. After structural calculation and comprehensive considerations on the issues related to manufacturing and construction, it was decided to use a CFST structure for the towers. The severe local buckling in slender sections and lamellar tearing due to the thick tube wall could be avoided by using infill concrete. The concrete significantly decreased the required tube-wall thickness and improve local buckling resistance.

To overcome the problems with using hollow sections, an innovative CFST member with encased angle lattices was proposed and successfully applied up to a height of 262 m in the 380 m tall towers (Figure 4). The encased angle lattices provided an economical solution to the insufficient steel ratio for the large-scale composite tubular structures, improved the composite effects between each component and thus increased the composite member strengths.

Furthermore, the reduced tube-wall thickness reduced the cost of cold-formed manufacturing and welding quality control. Meanwhile, the encased angle lattices could serve as inner scaffolding before casting concrete and therefore provide access for internal construction work. Due to the lack of design methodology in domestic and international design specifications, the mechanical performance of the proposed structure was extensively investigated to obtain the optimal design approaches (Chen *et al.*, 2017a, 2017b; Xu *et al.*, 2014, 2017c).

2.2 Joint design

Both gusset plate and CHS tube to CFST joints were adopted below 262 m, while spherical steel joints were applied for the upper steel truss structure (Figure 5). The tube splice in members of the largest diameter (2300 mm) is shown in Figure 6.

Although the infill concrete did not directly contribute to the strength of the composite joint, it changed the chord stiffness and thus altered the failure modes. Conservatively adopting the design approaches for pure steel CHS joints would not only have led to

Figure 4. Structural members and connections of the 380 m tall transmission tower: (a) overview; (b) tubular joints; (c) detail of double K joint; (d) detail of spatial joints. Concrete infill is used up to 262 m

Figure 5. Diagram of upper non-concrete-filled section of tower (a), conventional joint design (b), spherical joint Q1 design (c)

an uneconomical design but may also have caused errors in the overall structural analysis. Therefore, extensive investigation on the mechanical performance of the composite joints under various loading conditions was conducted. A design framework was established for practical engineering (Chen *et al.*, 2010; Xu *et al.*, 2015a, 2016a, 2016b, 2017a, 2017b).

Near the top end of the columns, approximately 87 m in Figure 5, a specially designed spherical joint was proposed to simplify the load-transfer mechanism from the branches to the main members (Figure 5(c)). The spherical joints reduced the possible initial eccentricity during manufacturing and installation. They could therefore achieve a better mechanical performance compared with a conventional welded-spatial CHS joint in Figure 5(b). The maximum diameter of the spherical joint was 2000 mm.

According to the Chinese technical specification for latticed shells (SAC, 2003b), the thickness of a joint should have been 45–80 mm, which could have brought great difficulties in manufacturing and construction. Therefore, inner stiffeners were adopted to improve the local buckling resistance of the shell and thus decrease the shell thickness to 24 mm. The arrangement of inner stiffeners is as shown in Figure 5(c). The numerical analysis showed that the design method in the Chinese specification could provide a satisfactory prediction for the hollow section stiffened spherical joint.

3. Structural investigations

3.1 CFSTs with angle lattice reinforcement

The experimental (Chen *et al.*, 2017a, 2017b; Xu *et al.*, 2014) and numerical (Xu *et al.*, 2017c, 2019) investigations on the compressive and tensile performance of CFST columns with an encased built-up lattice were conducted. The test specimens were scaled to one of the main members in the transmission tower, as shown in Figures 7(a) and 7(b). A total of 35 CFST test specimens with encased built-up angle lattices and their counterparts as the control groups were tested under axial compression (Xu *et al.*, 2014), axial tension (Chen *et al.*, 2017a; Xu *et al.*, 2019) and eccentric tension (Chen *et al.*, 2017b).

Figure 6. Flange splice for 2300 mm concrete-filled main members: cross-section (a) and plan (b)

The effects of built-up angle lattices on local buckling, global buckling and steel–concrete composite actions were experimentally investigated by 14 composite column specimens. The typical failure modes for specimens are shown in Figure 7(c). The discrepancy on the failure modes was found in the angle-reinforced specimen and its counterpart CFST (in 2500 mm length). The failure mode of the non-reinforced test specimen was shear failure of the inner concrete at the bottom, while the specimen with encased lattice angles failed at global buckling.

The inner angle lattice would effectively prevent the development of diagonal shear cracking in the concrete. The ultimate strength of each test specimen proved that the encased angle lattices could effectively enhance the compressive capacity and have little influence on the compressive stiffness and ductility. As shown in Figure 7(c), the angle lattices could work together with the surrounding concrete.

The design equations of the encased angle lattice CFST column are shown in Equations 1 to 3 (Xu *et al.*, 2017c). The proposed design approach was able to provide a reasonable and reliable prediction, achieving a satisfactory balance between accuracy and safety.

Figure 7. Test specimens of a CFST column with encased angle lattice (Chen *et al.*, 2017a, 2017b; Xu *et al.*, 2014, 2017c, 2019): (a) schematic view of compressive specimens; (b) schematic view of tensile specimens; (c) failure modes of compressive specimens

$$1. \quad \eta_{si} = e^{-1.425} \times \chi^{-1.081} = 0.241\chi^{-1.081}$$

when $\lambda \leq 20.0$,

$$2. \quad F_{u,AXT} = \begin{cases} F_{CFST} + (1 + 0.241\chi^{-1.081}) F_{Built-up} & 0.509 < \chi \leq 2.0 \\ F_{CFST} + 1.5 F_{Built-up} & 0.1 \leq \chi \leq 0.509 \end{cases}$$

when $\lambda > 20$,

$$3. \quad F_{u,AXT} = F_{CFST} = F_{Built-up}$$

where F_{CFST} is the design compressive strength of the CFST member calculated from AISC (2010) and $F_{Built-up}$ is the design compressive strength of latticed built-up angles calculated using AISC (2010).

A total of 28 large-scale test specimens, including CFST columns reinforced with steel bars or angle lattices and the control groups of steel and CFST columns, were tested under axial (Chen *et al.*, 2017a; Xu *et al.*, 2019) and eccentric tension (Chen *et al.*, 2017b). The details for each test specimen are shown in Figure 7. All test specimens failed at the yielding of outer tube and cracking of inner concrete.

Figure 8. Cracks in concrete cores from CFST specimens under axial tension without (a) and with (b, c) encased angle lattice (Chen *et al.*, 2017a, 2017b)

The crack patterns of inner concrete for the specimens under axial and eccentric loading, as shown in Figure 8, indicated that the maximum crack width and spacing developed in the inner concrete of encased angle CFST columns were significantly reduced compared to the CFST counterparts. The test results showed that the embedded reinforcement could work synergistically with the CFST columns and effectively contribute to the column strength through load transfer by the concrete. Therefore, the initial stiffness and tensile strength were significantly increased due to the presence of the angle and steel reinforcement.

The design equations and tension–moment relationship, considering composite actions among components, were proposed. The design equations for the encased angle CFST members under axial (Chen *et al.*, 2017a; Xu *et al.*, 2019) and eccentric tension (Chen *et al.*, 2017b) are summarised in Equations 4–8. It should be noted that more experimental and numerical investigations were also needed to verify the proposed design equations and provide the safe, reliable and effective design framework to guide the practical design.

3.1.1 Initial stiffness

Axial tension, $(EA)_{AXT}$

$$4. \quad (EA)_{AXT} = E_{so} = A_{so} + 0.1 (E_c A_c + E_{st} A_{st})$$

Eccentric tension, $(EA)_{ECT}$

$$5. \quad (EA)_{ECT} = (1 + \omega^2) \times (EA)_{AXT}$$

where AXT refers to axial tension loading and ECT refers to eccentric tension loading.

3.1.2 Design strength

Axial tension, $F_{u,AXT}$

$$6. \quad F_{u,AXT} = \beta_{so} f_{yo} A_{so} + \beta_{si} f_{yi} A_{si}$$

Eccentric tension, $F_{u,ECT}$

$$7. \quad \left(\frac{F_{u,ECT}}{F_{u,AXT}} \right)^2 + \left(\frac{e \times F_{u,ECT}}{M_u} \right) \leq 1.0$$

where M_u is the flexural strength of reinforced CFST members, which can be calculated as follows

$$8. \quad M_u = M_{sc,u} = M_p$$

$$9. \quad M_{sc,u} = \gamma_m W_{sc} f_{scy}$$

β_{so} and β_{si} are the stress coefficients of the tube and the angles, respectively, which can be calculated as follows

$$10. \quad \beta_{so} = 1.1 - 0.4\alpha_T$$

$$11. \quad \beta_{si} = \begin{cases} 0.85 & 0.14 \leq \phi_{si} \leq 0.95 \\ \frac{0.828}{\sqrt{\phi_{si}}} & 0.95 \leq \phi_{si} \leq 100.0 \end{cases}$$

where $\gamma_m = 1.1 + 0.48 \ln(\xi + 0.1)$; $W_{sc} = \pi D^3/32$; $f_{scy} = (1.14 + 1.02\xi) f_{ck}$; $\phi_{si} = (A_{si} / (C_{si} \times L)) \times (\rho_{si,T} / \alpha_T)$; $\rho_{si,T}$ is the angle–tube ratio; $\rho_{si,T} = f_{yi} A_{si} / f_{yo} A_{so}$.

3.2 Joints

Three groups of large-scale CFST longitudinal gusset plate joints were tested under axial tension, eccentric tension and in-plane bending, respectively (Xu *et al.*, 2016a). Test specimens failed in tube wall punching shear, with the typical failure mode shown in Figure 8. The deformation limit, 3% of tube diameter, in the design guideline CIDECT-1 (Wardenier *et al.*, 2008) for pure steel joints was found not to govern the ultimate state of the composite joints, which had been verified by both experimental (Xu *et al.*, 2016a) and numerical (Xu *et al.*, 2017a) investigations.

The inner concrete increased the radius stiffness of the chord member, thus mitigating the tube-wall flexibility. In the design specification of Eurocode 3 (BSI, 2005b), the design strength of hollow section joints was determined by the lower value between the plastification strength and punching shear strength (without concrete). However, these guidelines could fail to predict the failure mode accurately and underestimate the strengths of concrete-filled joints.

The fracture mechanism-based numerical analyses (Xu *et al.*, 2107a) were performed to investigate the punching shear mechanism of the composite joints. Based on experimental, numerical and theoretical investigations, the design methodology and design equations were proposed as shown in Figure 9 and Table 1, respectively. They provided accurate and reliable predictions for composite joints under various loading conditions.

Experimental, numerical and theoretical investigations were conducted to derive the mechanical performance of CHS to CFST joints under tension and in-plane bending. In total, 24 large-scale joint specimens were tested (Chen *et al.*, 2010; Xu *et al.*, 2015a, 2017b), including T, Y, K and KT types of joints, under static and cyclic loadings of tension and in-plane bending.

The discrepancies between pure steel and CFST joints on both stress concentration factor distributions and maximum values were observed from tests (Chen *et al.*, 2010; Xu *et al.*, 2015b). The design recommendations (API, 2014; DNV, 2005; Wardenier *et al.*, 2008) would give a higher stress concentration factor value, leading to conservative predictions on the fatigue life. The typical failure mode, as shown in Figure 9, was punching shear fracture of the tube wall near the weld toe.

The analogous investigation methodology in the plate joint investigation, as shown in Figure 9, was also adopted (Xu *et al.*, 2015a, 2016b, 2017b) to obtain the capacity design equations for the CFST–CHS joints (Figure 10). The design equations (Xu *et al.*, 2016b, 2017b) are summarised in Table 2.

4. Conclusions

The 380m tall Zhoushan towers, like their 370m tall predecessors, demonstrate successful practical application of innovative CFST members with encased angle lattices and corresponding joints.

The CFST members achieved and provided advantages in mechanical performance, manufacturability and constructability compared with hollow steel sections in the long-span electrical transmission towers.

The encased angle lattices worked effectively with the CFST member, provided a substantial complementary steel area, and significantly improved the strength of a thin-walled CFST member. Construction work also benefited from the encased lattices.

Figure 9. Schematic diagram of investigations on CFST–gusset plate connections

Table 1. Proposed design capacity of CFST–plate connections (Xu *et al.*, 2017a)

Loading type		Punching shear design strength	Validity range
$\beta = t_b/D$ $\gamma = D/(2t)$ $\eta = l_b/D$	$F_{u,AXT} = (0.9l_b + 1.1t_b) t f_u$	$0.02 \leq \beta \leq 0.06$ $20.0 \leq \gamma \leq 66.7$ $0.75 \leq \eta \leq 2.50$
$\beta = t_b/D$ $\gamma = D/(2t)$ $\eta = l_b/D$	$M_{u,AXT} = (0.31l_b + 0.49t_b) l_b t f_u$	$0.02 \leq \beta \leq 0.08$ $18.0 \leq \gamma \leq 66.7$ $0.85 \leq \eta \leq 3.33$
$\beta = t_b/D$ $\gamma = D/(2t)$ $\eta = l_b/D$	$\frac{F_{u,ECT}}{F_{u,AXT}} + \frac{e \times F_{u,ECT}}{M_{u,IB}} \leq 1.0$ $\frac{F_{u,ECT}}{F_{u,AXT}} \geq 0 \text{ and } \frac{e \times F_{u,ECT}}{M_{u,IB}} \geq 0$	$0.02 \leq \beta \leq 0.06$ $20.0 \leq \gamma \leq 66.7$ $0.85 \leq \eta \leq 2.50$

Figure 10. Schematic diagram of investigations on CFST–CHS connections

The composite joints and spherical joints provided excellent mechanical performance and good load-transfer mechanism within the connecting regions. Extensive investigations and close collaboration between industrial designers and university

researchers were essential to establish the design framework and ensure successful design and construction.

All key techniques adopted in the structural design aimed to achieve safe, reliable and sustainable infrastructure. Moreover, these innovative techniques helped to save up to US\$5 million in constructing the transmission towers compared to a conventional pure steel design.

The techniques adopted at Zhoushan can improve the performance and constructability of future large-span electrical transmission towers worldwide.

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Table 2. Proposed design capacity of CFST–CHS connections (Xu *et al.*, 2016b, 2017b)

Loading type		Punching shear design strength	Validity range
$\beta = D_b/D$ $\gamma = D/(2t)$	$F_{u,AXT} \times \sin\theta = \left(1.74 + \left(\frac{D_b}{D}\right)^3\right) f_u D_b t$	$30^\circ \leq \theta \leq 60^\circ$ $0.20 \leq \beta \leq 0.80$ $25.0 \leq \gamma \leq 50.0$
$\beta = D_b/D$ $\gamma = D/(2t)$	$M_{u,IB} = \left(0.79 + 0.56 \left(\frac{D_b}{D}\right)^3\right) f_u D_b^2 t$	$0.20 \leq \beta \leq 0.60$ $30.0 \leq \gamma \leq 75.0$

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