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Scale model experimental of a prestressed concrete wind turbine tower

Hongwang Ma*, Dongdong Zhang, Ze Ma and Qi Ma

Department of Civil Engineering, Shanghai Jiaotong University, Shanghai, 200240, China

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Abstract. As concrete wind-turbine towers are increasingly being used in wind-farm construction, there is a growing need to understand the behavior of concrete wind-turbine towers. In particular, experimental evaluations of concrete wind-turbine towers are necessary to demonstrate the dynamic characteristics and load-carrying capacity of such towers. This paper describes a model test of a prestressed concrete wind-turbine tower that examines the dynamic characteristics and load-carrying performance of the tower. Additionally, a numerical model is presented and used to verify the design approach. The test results indicate that the first natural frequency of the prestressed concrete wind turbine tower is 0.395 Hz which lies between frequencies 1P and 3P (0.25–0.51 Hz). The damper ratio is 3.3%. The maximum concrete compression stresses are less than the concrete design compression strength, the maximum tensile stresses are less than zero and the prestressed strand stresses are less than the design strength under both the serviceability and ultimate limit state loads. The maximum displacement of the tower top are 331 mm and 648 mm for the serviceability limit state and ultimate limit state, respectively, which is less than L/100 = 1000 mm. Compared with traditional tall wind-turbine steel towers, the prestressed concrete tower has better material damping properties, potential lower maintenance cost, and lower construction costs. Thus, the prestressed concrete wind-turbine tower could be an innovative engineering solution for multi-megawatt wind turbine towers, in particular those that are taller than 100 m.

Keywords: prestressed concrete wind turbine tower; model test; numerical simulation; dynamic characteristics; steel wind turbine tower

1. Introduction

Wind energy is one of the most commercially developed and rapidly growing renewable energy technologies in the world. Wind turbines have grown both in size and rated power, leading to a growth in the height, strength, and stiffness of their towers (Islam *et al.* 2013). Taller towers require larger investment as more building materials, more difficult fabrication, and more complex installation are needed (Quilligan *et al.* 2012). Given that 20% of the cost of a megawatt-scale horizontal axis wind turbine is in the tower, resulting in about 10% of the total cost of the energy, it is very important to find ways to examine wind turbine tower structures that are more cost-effective for multi-megawatt (MW) wind turbines (Marruth 2014).

Prestressed concrete has been a cost-effective choice for tower-like structures including bridge

*Corresponding author, Associate Professor, E-mail: hwma@sjtu.edu.cn

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piers, tall chimneys, and TV transmission towers. Compared with steel tube towers, the cost of materials for prestressed concrete towers is lower; they require lower maintenance, and have more fabrication flexibility and better dynamic characteristics as the design is less driven by fatigue. Prestressed concrete solutions have been utilized for wind turbine towers in recent years (Cajka 2013, Jairo *et al.* 2011, Kenna 2014).

Several conceptual or manufactured concrete tower systems have been developed worldwide. Most systems use precast post-tensioned construction methods with cross sections varying from round to polygonal. The tower sections are comprised of precast segments that may or may not be divided into sector panels. LaNier (2005) presents a study carried out by the National Renewable Energy Laboratory to investigate the feasibility of using wind turbines in low wind speed sites. The conceptual designs proposed by LaNier included a tubular steel tower, hybrid steel/concrete towers, and all-concrete towers for 1.5-, 3.6-, and 5.0-MW wind turbines. The design loads used for the towers were derived based on the WindPACT turbine rotor design study. For a 1.5-MW, 100-m tower the results indicate that the cost of a steel/concrete hybrid tower, an all-cast-in-place tower, and a tubular steel tower are all within 33% of each other. For the 3.6-MW and the 50-MW, 100-m towers, the cost of the all-cast-in-place concrete design is 68% that of the estimated cost of the tubular steel design.

Tricklebank and Halberstadt (2007) provided a similar estimate for wind turbines having tower heights of 70 m and 100 m for 2.0-MW and 4.5-MW wind turbines, respectively. They presented conceptual configurations for both onshore and offshore facilities, along with design philosophies and construction methodologies for concrete wind tower solutions. The study concluded that concrete towers have a potential for savings of up to 30% compared with steel towers for taller towers (70–120 m and higher) supporting larger turbines for onshore wind farms. Moreover, concrete towers have a potential for longer reliable operational lives (50–100 years) by allowing the re-use of the tower and foundation structure for large turbine re-fits. They could thereby provide significantly improved long-term financial returns and substantial improvements in sustainability for offshore wind farms.

Jorge(2012) presented another precast concrete tower concept developed by the Spanish company Inneo Torres for hub heights of 80–120 m that are suitable for onshore and offshore 1.5–4.5-MW wind turbines. The tower consists of a few large precast elements in the form of long narrow panels; by reducing the number of precast elements, the manufacturing system can achieve a production rate of two towers per week, similar to the erection rates of its tubular steel counterpart.

Ibrahim(2012) developed a prestressed concrete system that consists of vertical columns and horizontal panels with a triangular-shaped cross section that consists of three columns at each corner of the triangle, connected with commonly used precast concrete wall panels. Hub heights of 75 m and 100 m for a 3.6-MW wind turbine were analyzed and designed under dead, wind, and seismic loading. The research results show that the proposed system has potential as a system with a low initial cost, low maintenance cost, and fast simple erection method compared with steel tube systems.

Ma (2014) presented a prestressed concrete tower design employing a novel tower concept with a regular octagon cross section with interior ribs on each side. With this concept, a 100-m prestressed concrete tower system for a 5-MW turbine was optimized and designed under gravity, wind, and earthquake forces. The tower is made up of several precast segments. After assembly, each segment was post-tensioned at the bottom of the tower with enough force to maintain its stability.

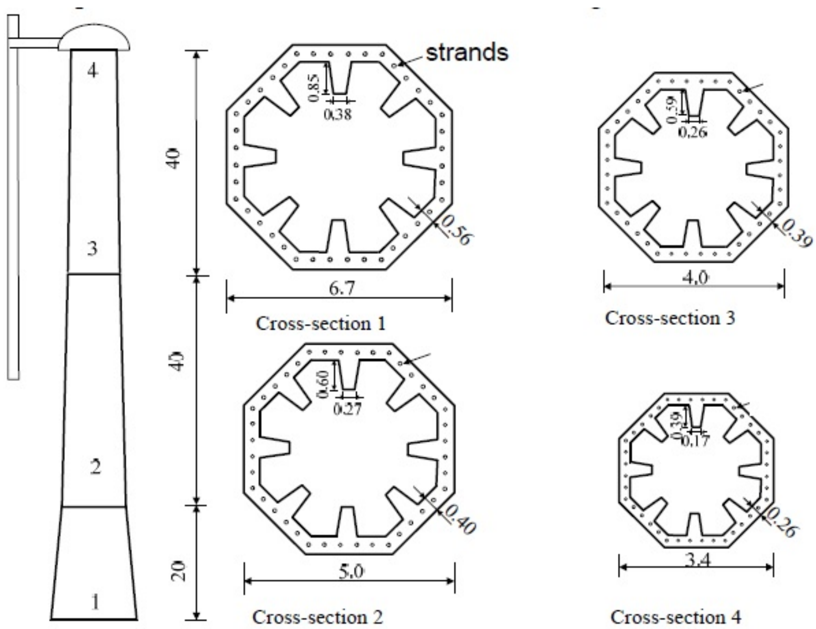


Fig. 1 Prototype prestressed concrete wind turbine tower (unit: m)

Eize (2009) of Advanced Tower Systems (ATS) offers a patented hybrid tower that combines a precast concrete segmented tower base with tube steel sections above it. The ATS tower is a unique solution for wind turbines of 1.5 MW and higher energy, at hub heights of 80 to 150 m. The first tower was built in Grevenbroich, Germany: a 133-m large hybrid tower supporting a Siemens 2.3-MW wind turbine. The system features easily transportable precast concrete sections forming a square or triangular cross section with rounded corners, and a tower installation time of less than a week. Several steel-concrete hybrid tower systems have already been constructed in Europe by Enercon of Germany, including the world's largest wind turbine E-126.

These research results show that concrete wind turbine towers are more economic than traditional steel tube towers for Multi-MW wind turbines. Most of the current research focuses on conceptual designs and numerical simulation, lacking adequate experiment verification. This paper describes experimental dynamic characteristics tests and static cycle load tests for the new prestressed concrete wind turbine towers presented by Ma (2014), to obtain the first frequency, damping ratio, and load capacity of the test tower and to verify the numerical model.

2. Prototype prestressed wind turbine tower

A prototype prestressed wind turbine tower (Ma 2014) was defined to provide a realistic model for the modal test. The experiment was designed based on the conditions at the Eastern Sea Bridge Wind Farm, Shanghai, China. The wind specifications for the design are as follows: basic wind speed of 42.5 m/s, topographic specification A, and terrain roughness category D. The design uses a basic ground motion acceleration of 0.1 g. A 100-m tall prestressed concrete tower is designed to support a 5-MW wind turbine. The concrete used in the fabrication of the tower has a compression strength of 27.5 MPa, tensile strength of 2.34 MPa, and an elasticity modulus of 3.6×10^4 MPa. The basic plan of the prestressed concrete tower is shown in Fig. 1.

2.1 Test prestressed concrete tower and experimental setup

The prestressed concrete tower used in the test is related to the prototype prestressed concrete tower through the scale factor λ, equal to 1/15. To reproduce the scaled-down prototype prestressed concrete tower for the test, the model concrete tower was constructed by using the same materials and maintaining equal stresses. Thus, the test prestressed concrete tower length dimensions are related to the prototype prestressed concrete tower by the factor λ and the forces are related to by λ^2. The test tower consists of five parts. Table 1 lists the dimensions of each part, where t is the wall thickness of the tower, d is the diameter of the tower cross section, l_a and l_b are the rib height and rib width, respectively, and n is the number of prestressed strands. As the test tower is 6.6 m high, the experiment was difficult to perform in the laboratory; therefore, the substructuring technique (Souid 2009) was applied to the test tower, whereby the upper three parts of the structure are considered as the numerical substructure and the lower two parts are selected as the experimental substructure.

2.2 Mechanical properties of the materials

The self-compacting concrete was delivered from a ready-mix plant and all the casting was carried out in the Structural Engineering Laboratory at Shanghai Jiaotong University. The compressive strength, modulus of elasticity, Poisson's ratio, and tensile strength were obtained from the standard concrete cylinders on the date of the specimen test.

Table 1 Dimensions of the prestressed concrete test tower

Tower part		t (m)	d (m)	l_a (m)	l_b (m)	n	
Top part	Top	0.03	0.23	0.04	0.03	24	Numerical substructure
	Bottom	0.03	0.25	0.04	0.03	24	
Fourth part	Top	0.03	0.25	0.04	0.03	24	
	Bottom	0.03	0.27	0.04	0.03	24	
Third part	Top	0.03	0.27	0.04	0.03	24	
	Bottom	0.03	0.31	0.04	0.03	24	
Second part	Top	0.03	0.31	0.056	0.042	4	Experimental substructure
	Bottom	0.03	0.34	0.056	0.042	4	
Bottom part	top	0.03	0.34	0.056	0.042	4	
	Bottom	0.04	0.45	0.056	0.042	4	

Table 2 Mechanical properties of the materials

Material	f_c (MPa)	f_t (MPa)	E_c (GPa)	f_y (MPa)	E_s (GPa)	ν
Concrete	27.5	2.04	3.6*10^4	/	/	0.2
Prestressed strands	344.7	344.7	/	344.7	2.1*10^5	0.3

The mechanical properties of the steel were obtained from the tests performed on a sample of the bars (Table 2). A steel block was set on top of the tower to simulate the wind turbine mass.

2.3 Measuring instrumentation and experimental setup

Strain gauges were glued to the surface of the concrete test tower at three cross-section positions. Three linear variable differential transformers (LVDTs) were placed along the vertical direction at the bottom, middle, and top of the test tower to measure the linear displacement (Fig. 2). The prestressed concrete tower was anchored to the strong solid floor. The load was applied at the top of the test tower by an actuator attached to the reaction wall.

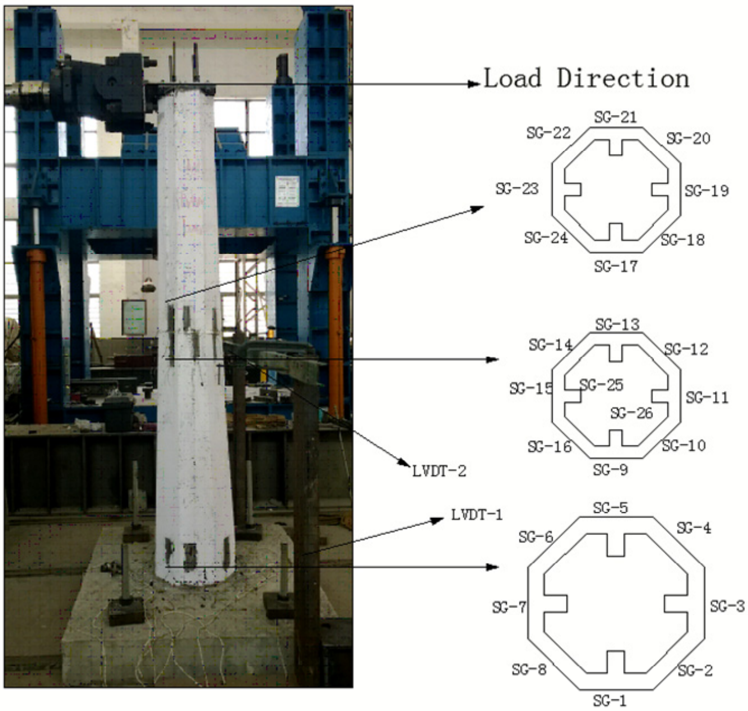


Fig. 2 Concrete test tower with instrument layout

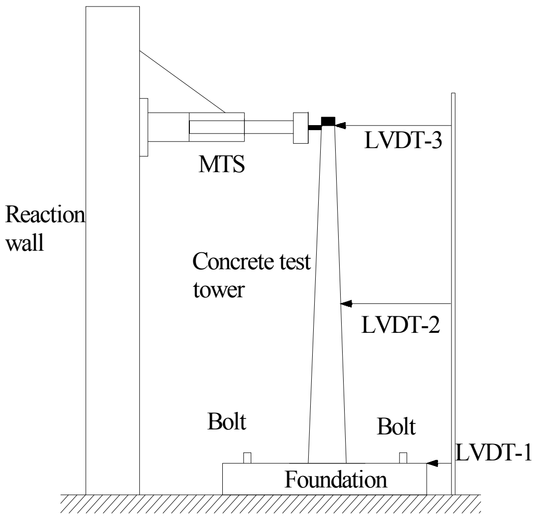


Fig. 3 Schematic drawing of test setup

2.4 Test procedures

After the test tower was anchored to the solid floor, and before it was connected to the MTS loading equipment, the dynamic characteristics were identified by an ambient vibration test. The natural frequency and damping ratio of the tower were derived from the structural response. The structural response was recorded by a uniaxial velocimeter which was placed on top of the tower. Then the tower was connected to the loading equipment and a 500-kN MTS universal testing machine was used to apply a horizontal load as shown in Fig. 3. The loading was controlled at a displacement rate of 0.04 mm/s. The loading history is shown in Fig. 4.

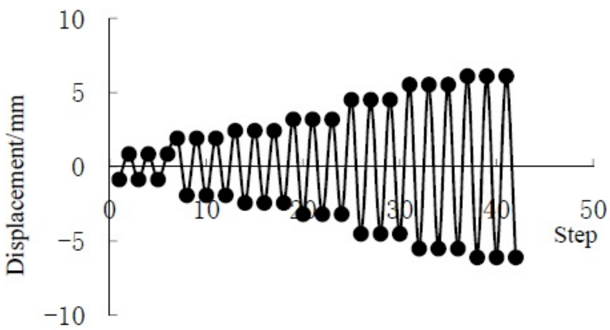


Fig. 4 Loading protocol

Fig. 5 Frequency time series

3. Experimental results

3.1 Dynamic characteristics

Ambient vibration tests were conducted on the tower after it was anchored to the floor. Figs. 5 and 6 present the measured velocity data and PSD spectra, respectively. The sharp peak at 24 Hz in Fig. 6 indicates that the first frequency of the tower is $f_0 = 24Hz$. The damping ratio is estimated using the half power bandwidth method. Figure 6 illustrates the procedure for the first horizontal mode. It is then assumed that half the total power dissipation in the first mode occurs in the frequency band between $f_1 = 23.2Hz$ and $f_2 = 24.8Hz$ where f_1 and f_2 are the frequencies corresponding to an amplitude of $f_0 / 2$. As shown in (Chopra 2001), the damping ratio ξ is approximately

$$\xi = \frac{f_2 - f_1}{2f_0} = \frac{24.8 - 23.2}{2 \times 24} = 3.3\% \tag{1}$$

3.2 Cyclic static test

The horizontal load–drift results are plotted in Fig. 7. Cracks in the concrete cause a progressive decrease in the stiffness. As the wind-turbine tower should not yield during its working life, the steel bars are not yielded during the test. The strain and displacement of each measuring point are plotted in Fig. 8. The strains at points SG-3, SG-7, SG-11, and SG-15 (see Fig. 2) show an obvious change under a top displacement load; these points are used to verify the numerical model.

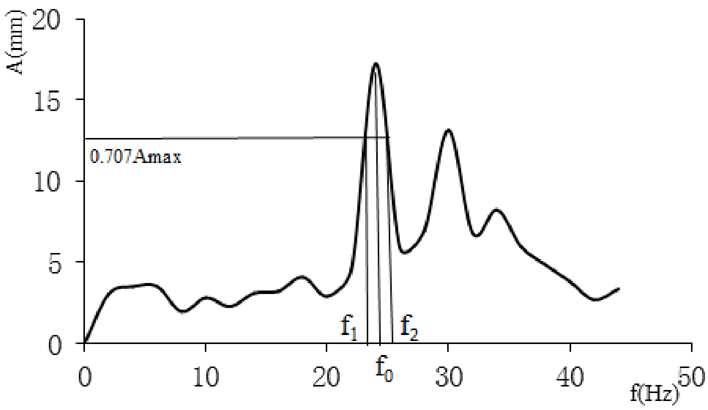


Fig. 6 Power density spectra showing the natural frequency of the structure

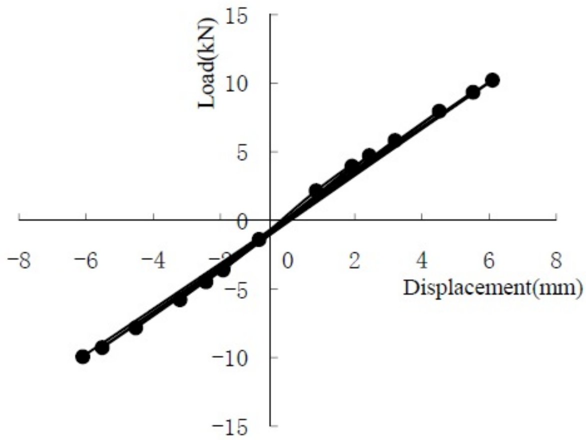


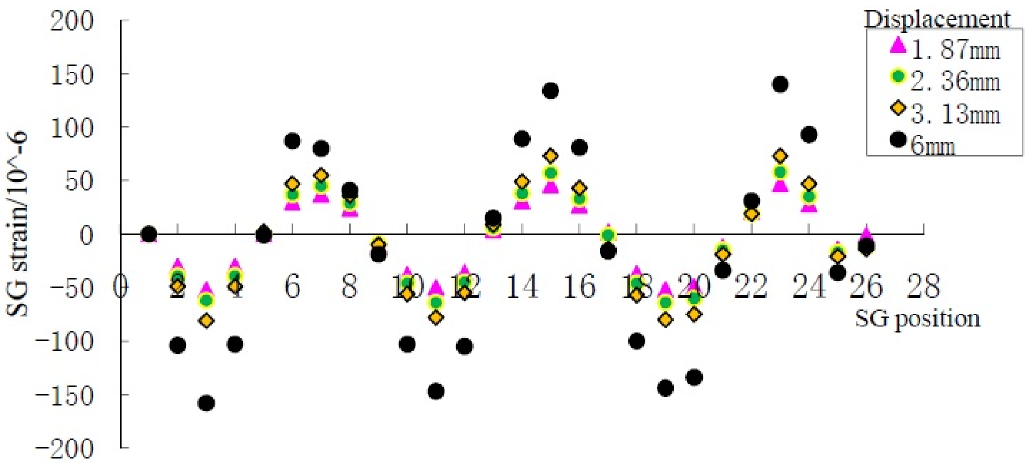
Fig. 7 Test tower top displacement verses top horizontal load

3.3 FEA modeling and numerical values

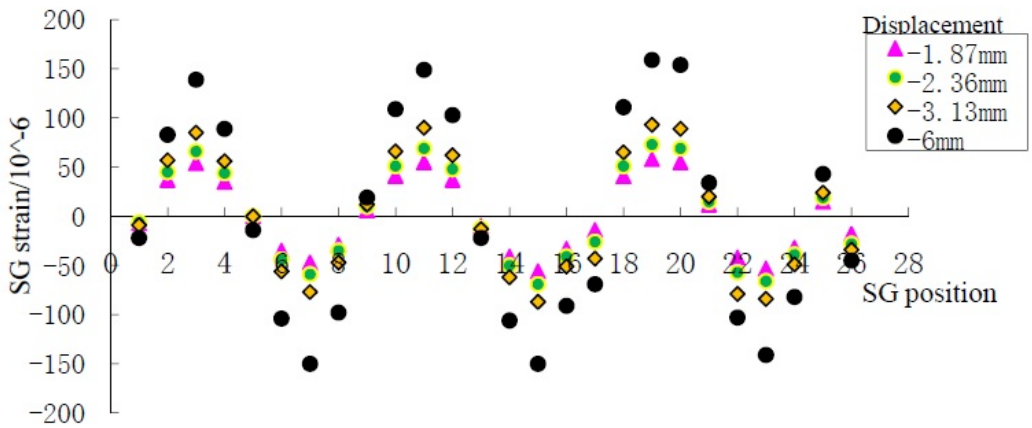
In the present study, a finite element model of the prestressed wind turbine concrete test tower was generated using the ABAQUS/Standard software. The concrete structure was modeled using 8-node reduced integration elements; the maximum element size for the concrete elements was around 50 mm. Two-node linear 3D truss elements were used to model the prestressed strands. The prestressing forces were introduced by the falling temperature method in the 3D truss elements representing the strands, which is an option available in ABAQUS/Standard. The prestressing strands were embedded into the solid concrete. The damaged-plasticity model for concrete was selected in this study. The dilation angel was taken as 15°, and default values were used for the rest of the parameters required by ABAQUS/Standard to define the damaged-plasticity model. These parameters are 1.16 for the ratio of the initial equibiaxial compressive yield stress to the initial uniaxial compressive yield stress and 0.667 for the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian (Abaqus 2006). The stress–strain relationship for concrete under uniaxial compression and uniaxial tension are based on the Code for the Design of Concrete Structures (GB 50010 2010).

The surface-to-surface contact available in ABAQUS/Standard was set between two top and bottom test tower parts. All the degrees of freedom were constrained on the bottom surface of the tower. The FM meshes along with the modeling characteristics are shown in Fig. 9.

The natural frequency calculated using the numerical model is 25.7 Hz. The difference between the numerical result and experimental result close to 7%, indicating a good agreement between the natural frequency obtained from the numerical and experimental results.



(a) Positive loading



(b) Reverse loading

Fig. 8 Strain recorded by strain gauges at the target displacement

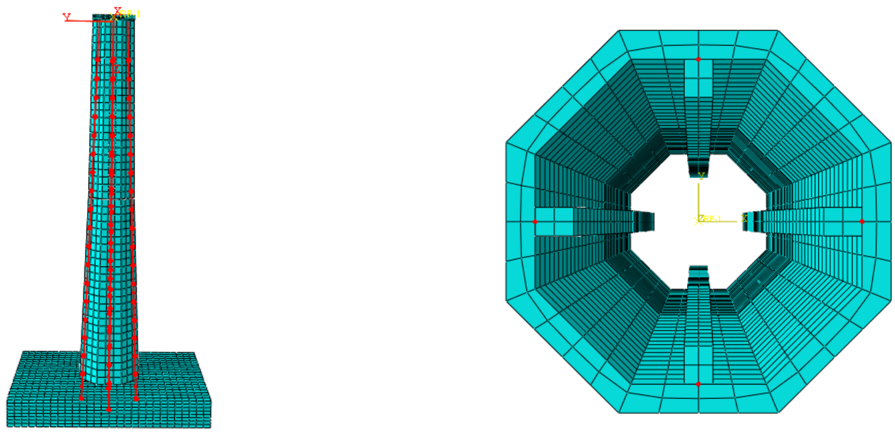


Fig. 9 Finite element mesh used in numerical simulation

To verify the numerical model, we selected some measured strains and LVCDs for comparison with the numerical value. Two target displacements, 1.87 mm and 2.13 mm, which approximately correspond to the serviceability limit state and the ultimate limit state, were selected.

The test strain values and displacement values and the numerical strain values and displacements are listed in Tables 2 and 3, respectively. The maximum difference between the numerical strain and experimental strain (Table 3) is 11.2% and the minimum difference is 1.0%. The difference between the numerical displacement and experimental displacement (Table 2) of LVCD2 is within 7.4% at all the loading stages. Thus, the numerical model is reliable for calculating the dynamic characteristics and static loading response.

3.4 Prototype tower response based on experimental data

We constructed the finite element model of the whole prestressed concrete test tower including the experimental substructure and numerical substructure. The horizontal load is applied at the top of the tower; the material properties, boundary conditions, and load procedure are the same as for the experimental substructure model. The horizontal loads were determined through target displacements of 1.87 mm and 3.13 mm which correspond approximately to the serviceability limit state and ultimate limit state of the tower. The calculated results for the prototype tower response based on experimental data are listed in Table 4.

Table 2 Displacement values from experiment and numerical simulation

Target Displacement (mm)	1.87			3.13		
	Test (mm)	Numerical (mm)	Difference (%)	Test (mm)	Numerical (mm)	Difference (%)
LVCD-2	0.66	0.63	4.5	1.055	1.01	4.3
LVCD-3	1.87	2.37	26.7	3.13	3.33	6.4

Table 3 Stress values from experiment and numerical simulation

Target Displacement (mm)	1.87			3.13		
	Test (MPa)	Numerical (MPa)	Difference (%)	Test (MPa)	Numerical (MPa)	Difference (%)
SG-3	-4.33	-4.78	10.4	-5.61	-6.05	7.8
SG-7	-0.95	-0.69	27.4	-0.37	-0.63	70.3
SG-11	-6.54	-6.41	1.9	-7.80	-7.72	1.0
SG-15	-3.00	-2.26	31.3	-2.04	-0.83	59.3
SG-19	-6.48	-6.95	7.3	-7.55	-8.25	9.3
SG-23	-2.31	-2.26	2.2	-1.07	-1.12	4.7

Table 4 Prototype tower response based on experimental data

	Model	f_1 (Hz)	Target displacement (mm)		Stress (MPa)							
					1.87 mm				3.13 mm			
			1.87	3.13	SG-3	SG-7	SG-11	SG-15	SG-3	SG-7	SG-11	SG-15
①	Numerical experimental substructure	25.7	2.37	2.73	-4.8	-0.7	-6.4	-2.1	-6.1	-0.34	-6.9	-1.6
②	Numerical whole test tower	6.35	28	34.5	-10.2	-2.4	-19.6	-4.0	-13.5	-1.43	-21.4	-2.2
③	Ratio ②/①	0.247	11.8	12.6	2.1	3.4	3.1	1.9	2.2	4.2	3.1	1.4
④	Experimental substructure	24	1.87	2.36	-4.3	-0.95	-6.54	-3.0	-5.6	-0.6	-6.93	-2.58
⑤	Experimental whole test tower ④×③	5.93	22	29.87	-9.03	-3.23	-20.3	-5.7	-10.1	-2.52	-21.5	-3.6
⑥	Similar constants	1/15	15		1							
⑦	Prototype Tower ⑤×⑥	0.395	331	648.4	-9.03	-3.23	-20.3	-5.7	-10.1	-2.52	-21.5	-3.6

Table 5 Wind turbine loads at the tower top

Top loads	Mx (kN.m)	My (kN.m)	Mz (kN.m)	Fx (kN)	Fy (kN)	Fz (kN)
P1	4336	16458	-4103	891	128	3502
P2	-7407	10282	-3348	373	-343	3089

As shown in Table 4, the first natural frequency of the prototype prestressed concrete wind turbine tower is 0.395 Hz, between the 1P and 3P frequencies. The top displacement is 331 mm, less than 1000 mm under the serviceability limit state. The maximum compression stress is 21.5 Mpa, less than the concrete compression strength, and the maximum tension stress is less than zero. These figures comply with the design standards for wind turbines for both the serviceability limit state and ultimate limit state. Thus, the prototype prestressed concrete wind turbine tower meets the design requirements for the experimental design load case.

4. Comprehensive comparison between prestressed concrete tower and steel tubular tower

The case study considered here is defined using a 5-MW wind turbine at a hub height of 100 m (Ma 2014), with the parameters described in section 2. The design parameters of the prototype prestressed concrete wind turbine tower are listed in Table 1. The wind turbine loads at the top of the tower are listed in Table 5.

Considering one serviceability limit state and one ultimate limit state, the load combination is calculated by Eqs. (2) and (3), respectively.

Service limit state (DLC1)

$$S_{SL} = 1.0S_{DL} + 1.0S_{TWL2} + 1.0S_{EQ2} + 0.2S_{WL2} \tag{2}$$

Ultimate limit state (DLC2)

$$S_{UL} = 1.2S_{DL} + 1.35S_{TWL1} + 1.3S_{EQ1} + 0.2 \times 1.4S_{WL1} \tag{3}$$

Here S_{DL}, S_{TWL}, S_{EQ}, and S_{WL} are the dead load, wind turbine load, earthquake load, and wind load, respectively.

The direct wind load on the tower and the earthquake load are calculated according to the load code for the design of building structures (GB 5009 2012) and the Code for Seismic Design of Buildings (GB 50011 2010), respectively. The strength and stability design checks for the steel tubular tower were made according to the “stress design” methodology defined in the code for load combinations EW, EO, and earthquake. The simplified procedure according to the Code for Design of Steel Structures (GB 50017 2011) was used to check the fatigue limit state for the steel tower. Table 6 lists the final dimension of the steel tower.

Table 6 Dimension of the 100-m height steel tubular tower

Steel tower segment	Thickness [mm]	Diameter [mm]	Vertical position [m]
Top	22.2	3400	100
Middle	34.9	4900	49
Bottom	36.5	6500	0

Table 7 Design results for the prestressed concrete tower and the steel tower

Criteria		Prestressed concrete tower	Steel tower
Natural frequency [Hz]		0.395	0.28
Damping ratio [%]		3.3	1–2
Gravity [T]		1944.0	391.0
Materials cost [ten thousand Yuan]		118.9	213.5
Top displacement [mm]	DLC-1	0.364	0.418
	DLC-2	0.792	0.968
Maximum concrete Compression stre	DLC-1	22.3	96.9
[MPa]	DLC-2	25.6	172.1
Maximum concrete Tensile stress	DLC-1	0	96.76
[Mpa]	DLC-2	0	173.3
Maximum steel stress [MPa]	DLC-1	1087	\
	DLC-2	1125	\

The main design parameters of the two tower types for the two load cases are summarized in Table 7. The prestressed concrete wind turbine tower has higher natural frequency and damping ratio than the steel tower and therefore higher tolerance to dynamic loads. Consequently, the steel tower will experience greater deflections and vibrations than the prestressed concrete tower. Because concrete raw materials are much cheaper than steel, the cost of the prestressed concrete wind turbine is 44% lower than the steel wind turbine tower. Compared with the steel tower, the maintenance costs of a prestressed concrete tower would be much lower. Therefore, the prestressed concrete tower is a cost-effective option for tall wind turbine towers, in particular those taller than about 100 m.

5. Conclusions

In this study we investigated the dynamic characteristics and responses of a new prestressed concrete wind-turbine tower subjected to cyclic loading. The following findings and conclusions were drawn from the research:

- Based on the experimental results, the first natural frequency of the prestressed concrete wind turbine tower is 0.395 Hz, which lies between frequencies 1P and 3P (0.25–0.51 Hz). The damper ratio is 0.33. This indicates that the prestressed concrete tower frequency does not coincide with the excitation frequencies of the dominant forces acting on the tower; hence, the tower is not at risk of entering a state of resonance. The high damping ratio also reduces the dynamic response and fatigue damaged.
- Under the serviceability limit state and the ultimate limit state selected in the test, the maximum concrete compression stresses are less than the concrete design compression strength; the maximum tensile stresses are less than zero, and the prestressed strand stresses are less than the design strength. The maximum displacements at the top of the tower were 331 and 648 mm based on the serviceability limit and ultimate limit states, respectively, which is less than L/100 = 1000 mm.
- A 3D numerical model of the prestressed concrete wind turbine tower was established and verified by the experimental test data. The results indicate that this model is a useful tool for simulating prestressed concrete wind-turbine tower responses under different design loads.
- The prestressed concrete tower has good material damping properties, potential low maintenance costs, and has lower construction costs compared to tall steel wind turbine towers; it is therefore a cost-effective option, particularly for towers taller than 100 m.

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