

PERFORMANCE ANALYSIS OF HIGH STEEL TUBE LATTICE SUPPORT SYSTEM IN TYPHOON AREA

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ABSTRACT

Research on safety of high steel tube lattice support systems in typhoon areas is still in the preliminary stage. The purpose of this paper is to study the overall buckling and overturning stability of the high steel tube lattice support systems in typhoon area. By constructing the spatial finite element model of the high steel tube lattice support system via MIDAS Civil, the optimal design of the steel tube lattice support system is carried out through the analysis of the main influencing parameters. The stability of steel pipe lattice support system is calculated theoretically, and the optimal design of steel pipe lattice support system is studied by finite element numerical method in Typhoon area. The calculation results show that Critical buckling load coefficient increases with the increase in diameter of the steel tube when the δ/d ratio of steel pipe structure is fixed. The critical load factor of the six-limb support system is slightly larger than that of the four-limb support system. When the transverse space of the support system is from 5 m to 7 m, stability increases rapidly. The best stability of the support system is obtained when the transverse space is approximately 7 m. The diagonal brace can significantly improve the stability of the steel tube lattice falsework.

KEYWORDS

High Steel Tube Lattice Support System, Stability, Finite Element Model, Wind Load

INTRODUCTION

The steel tube lattice support system is widely used in the construction of long-span bridges. However, when the support system reaches a certain height, the entire structure becomes flexible and the mechanical properties become complex. The existing research on mechanical analysis of the double limb lattice column and concrete-filled steel tube of lattice wind tower turbine [1-3] has been carried out. Some scholars also studied the mechanical properties of lattice columns by experimental and numerical methods [4-6]. Some achievements have also been made in the influence of bearing capacity of lattice columns [7] and optimization design of composite tower for large wind turbine systems [8]. Furthermore, the elastic buckling [9-10] and stability of lattice columns is also very important, some research results have also been obtained [11-16].

Under wind load, forced vibration occurs in the high lattice support system, leading to changes in surface pressure that result in vibration effects [17]. Wind load generally consists of mean wind and fluctuating wind loads. Mean wind load can be approximated to the static load acting on the structure, whereas fluctuating wind load causes the dynamic response of the structure [18-19]. The influence of the fluctuating wind load on the structure must be considered; moreover, the load is important to the safety of the flexible lattice support system in typhoon areas [20]. In this study, the influence of wind load on the bracing system of steel tube lattice formwork is

analysed, and the optimisation of the structural system is studied based on the actual project located in the typhoon area with a wind speed magnitude of 14m/s. This paper mainly studies the overall buckling and overturning stability of the high steel tube lattice support systems in typhoon area.

ESTABLISHMENT OF FINITE ELEMENT MODEL

Background

Dalian Island Bridge is located in the central area of Pingtan Strait and 7.85 km from the nearest point of the mainland. The area experiences strong winds all year round and waterways are flooded. The highway part of Dalian Island Bridge is divided into five units. The bridge span arrangement was a continuous beam with dimensions of $4 \times 40 \text{ m} + 4 \times 40 \text{ m} + 6 \times 40 \text{ m} + 5 \times 40 \text{ m} + 4 \times 32 \text{ m}$. The pier height ranged from 13 m to 50.5 m. The main beam adopted the cast-in-place bracket construction method, which used steel tube columns and the combined structural system of Bailey beam. The flat joint and diagonal bracings between mid-spans were connected by $\Phi 400 \times 8 \text{ mm}$ steel tubes. D9#–D19# columns utilised $\Phi 1,200 \times 14 \text{ mm}$ steel tubes. Mid-span flat joints adopted $\Phi 720 \times 14 \text{ mm}$ steel tubes. Diagonal bracings used $\Phi 400 \times 8 \text{ mm}$ steel tubes. In this study, the highest bracket D10#–D11# (61.702 m) segment was selected as the research object. Figure 1 and Figure 2 show the structure form of the bracket and D10–D11 Steel tube lattice support system, respectively.



Fig. 1 - On-site bracket support system

(a)

(b)

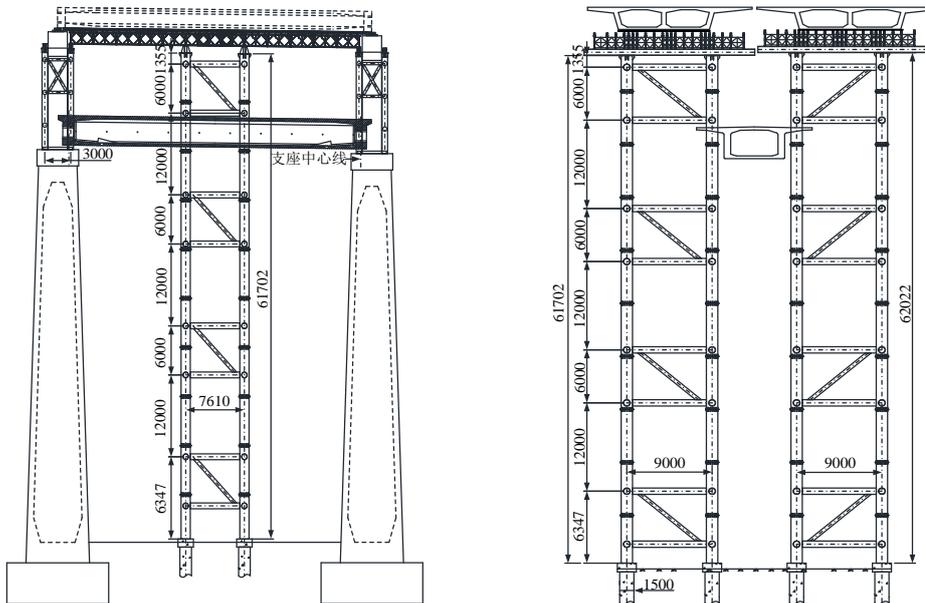


Fig. 2 - D10–D11 Steel tube lattice support system (a) longitudinal and (b) cross-section views

Finite element Model

MIDAS/Civil software was used to establish the finite element model for the high falsework. The model set the node at the connecting point of the member and connection of the foundation. Elements were divided based on the actual steel structure. The high steel tube lattice falsework was composed of 3,688 nodes and 6,993 elements. Bailey beams and steel tube lattice brackets were modelled as truss and beam elements, respectively. Figure 3 and Figure 4 present the falsework finite element discrete model and 3D finite element models of the falsework, respectively. The maximum wind load in the working state of the falsework was considered a grade 8 wind, and the wind speed was 20.7 m/s. The wind load of the nonworking state was considered in the 10-year typhoon period, and the wind speed was 45.4 m/ s.

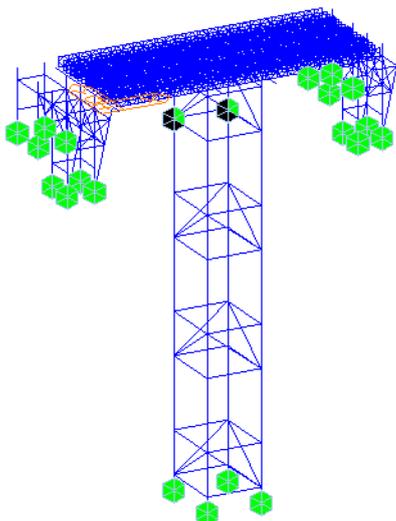


Fig. 3 - Falsework Finite Element Discrete Model

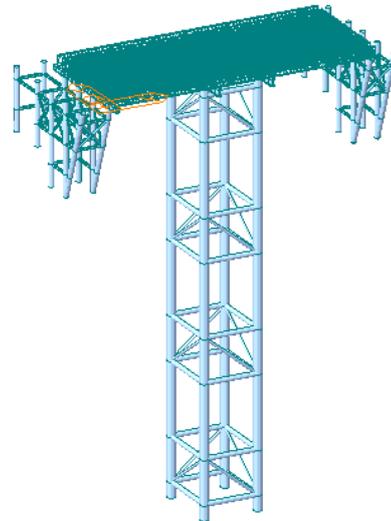


Fig. 4 - 3D Falsework Finite Element model

FORMWORK SUPPORT SYSTEM CALCULATION AND STABILITY ANALYSIS

Details of Formwork support on working stage conditions are shown in Table1.

Tab. 1 - Details on working stage conditions

Condition	Description	Load combination	Purpose
C-I	Shoring tower completed, prior to rebar installation, nonworking stage	$0.9 \times (② + ③ + ⑦)$	Structure and overturning stability verification
C-II	Concrete casting, prior to stretch prestressing, nonworking stage	$1.2 (① + ②) + 0.9 \times 1.4 (③ + ④ + ⑤ + ⑦)$	Shoring tower stability verification
C-III	Concrete casting completed, shoring towers are now load bearing	$1.2 (① + ②) + 1.4 (③ + ⑦)$	Shoring tower robustness verification
C-IV	Normal working stage	$① + ②$	Shoring tower stiffness verification

In Table 1:

- ① Reinforced concrete bulk density of main beam: 26 kN/m³.
- ② Formwork deadweight: 3.0 kN/m².
- ③ Construction crew, materials and construction machines and tool load:
 - a. For shoring column calculations, load was uniformly distributed at 2 kN/m².
 - b. For horizontal and diagonal brace calculation, load as uniformly distributed at 1.5 kN/m².
 - c. Props and other accessory structures were at 1 kN/m².
- ④ Vibration load: Load generated by mixing concrete was 2 kN/m².
- ⑤ Impact load: Impact generated when pouring and casting concrete was 2 kN/m².
- ⑥ Dead weight of falsework system: Bailey beam and steel lattice column were 78.5 kN/m³.
- ⑦ Static wind load: Static wind load was calculated according to 'Load Code for the Design of Building Structures' as follows:

$$w_k = \beta_z \mu_z \mu_s w_0 \quad (1)$$

where:

- w_k —standard value of wind load (kN/m²)
- β_z —wind vibration coefficient at height z is 1.0
- μ_s —wind load shape factor, column and Bailey beams are considered and calculated as round tubes and truss beams, respectively
- μ_z —wind pressure height coefficient is 2.12
- w_0 —basic wind pressure (kN/m²), $w_0 = v^2 / 1600$

Formwork Support System Design and Calculation

Internal force and stress of bailey beam, cross beam, column and bracket connection system (horizontal and oblique brace) were calculated. Under the most unfavourable circumstances, the maximum axial force of the main chord in Bailey beam, vertical bar and oblique rod were -390.16 , -192.13 and 139.15 kN, respectively. The maximum stress of the vertical column, the maximum displacement of the falsework, the maximum shear force of the connection joints and the maximum stress were -126.81 MPa, 21.46 mm, -150.41 kN and -84.14 MPa, respectively. According to design code of steel structures, the design value for the upper and lower chords of Bailey beams, vertical columns and diagonal bracings were 510 , 193 and 156 kN, respectively. The design compressive strength of grade Q235 steel was 195 MPa. Therefore, actual load as lower than designed bearing load of the structure, and strength of the structure fulfilled the load-bearing requirements.

Stability Analysis

The lattice column at mid-span bears not only the bending moment. My caused by the asymmetry of the bridge construction but also the bending moment M_x caused by the longitudinal wind load. The lattice column was the crucial part under the eccentric compression state, causing the column to be under the state of bidirectional bending. The stability was related to all three variables N , M_x and M_y . Lattice column falsework section property and internal force calculation results are shown in Table 2 and Table 3.

Tab. 2 - Lattice column falsework section property

A (cm ²)	W (cm ³)	I (cm ⁴)	EA (kN)
522	15,288	1,834,563	1,075,320

Tab. 3 - Lattice column falsework internal force calculation results

Condition	Axle force, N (kN)	Bending moment, M_x (kN·m)	Bending moment, M_y (kN·m)
C-I	1,063	29	102
C-II	4,067	94	148

(1) Working stage condition C-I

At condition C-I, the falsework was completed but under the nonworking state. Overall analysis on buckling and stability was conducted. The results are as follows:

a. Buckling Stability

Slenderness ratio: $\lambda_x = \lambda_y = L/i = 10000/420 = 23.8$, based on type B cross section and stability coefficient $\varphi_x = \varphi_y = 0.957$.

If $E = 206,000,000$ kN/m², then $A = 0.00522$ m², $\lambda_x = 23.8$, then N'_{Ex} can be calculated as follows:

$$N'_{Ex} = \frac{\pi^2 EA}{1.1\lambda_x^2} = \frac{\pi^2 \times 206000000 \times 0.00522}{1.1 \times 23.8^2} = 17016 \text{ kN}$$

$$\sigma_1 = \frac{N}{\varphi_x A} + \frac{\beta_{mx} M_x}{\gamma_x W_x (1 - 0.8 \frac{N}{N'_{Ex}})} + \eta \frac{\beta_{ty} M_y}{\varphi_{by} W_y} = 27.7 \text{ MPa} < f = 195 \text{ MPa}$$

$$\sigma_2 = \frac{N}{\varphi_y A} + \frac{\beta_{my} M_y}{\gamma_y W_y (1 - 0.8 \frac{N}{N'_{Ey}})} + \eta \frac{\beta_{tx} M_x}{\varphi_{bx} W_x} = (21.3 + 1.3 + 6.1) MPa = 28.7 MPa < f = 195 MPa$$

Therefore, the falsework is in good stability under this working stage condition.

b. Anti-overturning Stability:

Overturning stability of the shoring tower under nonworking stage was analysed according to the following equation:

$$k = \frac{M_k}{M_q}$$

$$M_k = 6000 \times 30 / 2 = 90000 kN \cdot m$$

$$M_q = M_{formwork} + M_{stent} = 124 \times 60 \times 1.28 + 162 \times 30 \times 1.28 = 15744 kN \cdot m$$

$$k = \frac{90000}{15744} = 5.7 > 1.5$$

The calculated overturning stability coefficient was 5.7, which was significantly larger than that required in the design code. Therefore, overturning stability was high.

(2) Condition C-II

At condition C-II, the beams on top of the falsework completed casting, but prestressing was not stretched and under the nonworking state. The calculation process is as follows:

$$\sigma_1 = \frac{N}{\varphi_x A} + \frac{\beta_{mx} M_x}{\gamma_x W_x (1 - 0.8 \frac{N}{N'_{Ex}})} + \eta \frac{\beta_{ty} M_y}{\varphi_{by} W_y} = 94.8 MPa < f = 195 MPa$$

$$\sigma_2 = \frac{N}{\varphi_y A} + \frac{\beta_{my} M_y}{\gamma_y W_y (1 - 0.8 \frac{N}{N'_{Ey}})} + \eta \frac{\beta_{tx} M_x}{\varphi_{bx} W_x} = 96.4 MPa < f = 195 MPa$$

Stability under condition C-II met the requirement.

OPTIMAL DESIGN OF STEEL TUBE LATTICE FALSEWORK

Influence of Steel Tube Diameter

The falsework column was assumed to have identical cross sections. The force line of the column was consistent with the longitudinal axis of the cross section, and the material was completely uniform and elastic. The critical load of the column could be obtained by using the Euler critical force formula.

The ratio of the wall thickness δ to the diameter d of the steel tube was considered the control parameter. By solving the buckling stability coefficient of columns with different diameter and thickness values, the optimum diameter and thickness were obtained, as shown in Figure 5.

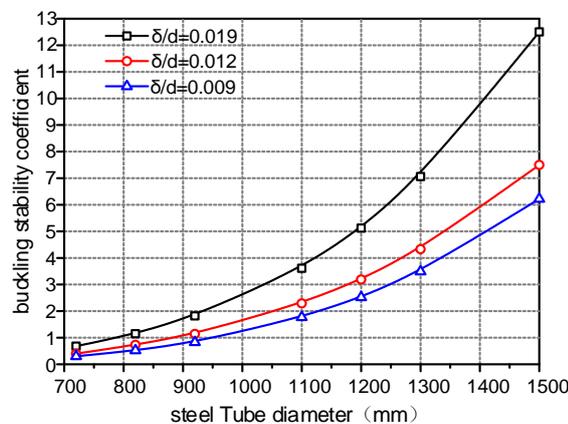


Fig. 5 - Comparison of the different δ/d ratios and buckling coefficients

Figure 5 shows that the critical buckling load coefficient increases at an exponential rate as the diameter of the steel tube increases when δ/d ratio is fixed. When the diameter of the steel tube was constant, the critical load coefficient of the structure increased with the increase in wall thickness. The enlarged diameter of steel pipe increases the change in the critical load coefficient. Therefore, the cross-section size of the structure was the main factor affecting the stability of falsework.

Influence of the number of limbs

According to the design drawings, the falsework was constructed with $\Phi 1,200$ steel tube vertical column, $\Phi 720$ horizontal bracing and $\Phi 400$ diagonal bracing. The influence of steel column number on the stability was studied by establishing the finite element model of single-limb, double-limb, four-limbs and six-limbs falseworks. The buckling diagrams of the different types of steel tube lattice column models are shown in Figure 6. Critical load coefficients of the different types of falseworks are shown in Table 4.

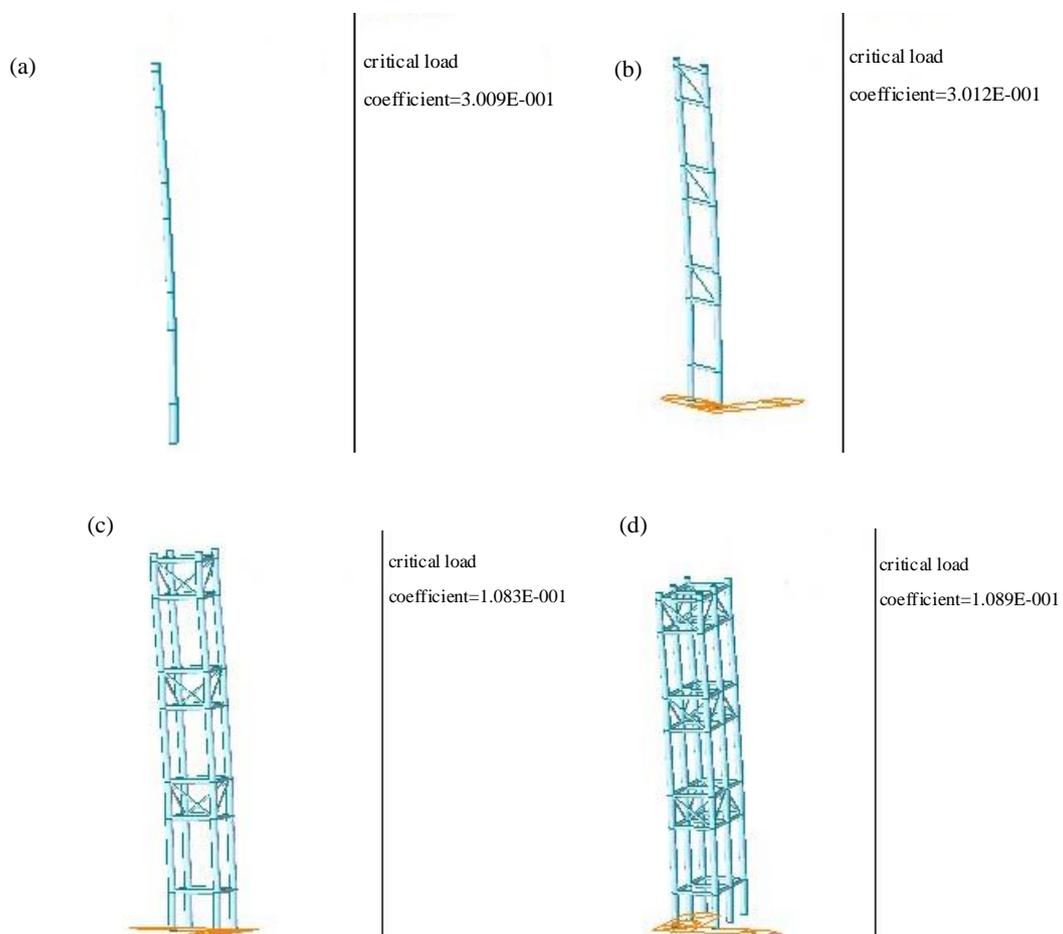


Fig. 6 - buckling diagrams of the different types of steel tube lattice column models

Notes:(a) Single limb; (b) Double limbs; (c) Four limbs; (d) Six limbs

Tab. 4 - Critical load coefficient of the different types of falseworks

Limb(s)	Critical Load Coefficient	Model Diagram	Configuration
Single	0.30	(a)	—
Double	0.31	(b)	9 m spacing
Four	10.83	(c)	9 m transverse, 7.61 m longitudinal
Six	10.89	(d)	2 × 9 m transverse, 7.61 m longitudinal

Figure 6 results of buckling calculations and data from Table 4 show the critical load coefficient calculated value of single- and double-limb structures were close, thereby the result indicated that stability of the double-limb structure was almost similar to that of the single-limb structure. However, when the number of limbs increased to four, the critical load coefficient also increased significantly. The stability of the lattice column improved at this time as the resistance to external load also increased. When the number of limbs increased to six, the critical load coefficient increased to less than 4 limbs, thereby indicating that the four-limb steel tubular column was the most cost-effective option.

Influence of Segment length and Height

When the height of the falsework was high, the high falsework was usually divided into segments with lengths of 9, 12, 15 and 18 m. The critical load coefficient at heights 30, 40, 50, 60, 70 and 80 m was analysed, and the critical load coefficient in the first three orders buckling of falsework was obtained. The critical load coefficient of different segment Lengths of falsework is shown in Table 5. Figure 7 and Figure 8 show the first-order and second-order critical load coefficient results at different segment height.

Tab.5 - Critical load coefficient of Different Segment Lengths

Segment Length (m)	Falsework Height (m)	first-order buckling	Second -order buckling	Third -order buckling
9	30	8.08	8.57	11.61
	40	7.89	8.28	11.60
	50	7.70	7.69	11.59
	60	7.47	7.63	11.59
	70	7.17	7.30	11.59
	80	6.74	7.02	11.58
12	30	6.77	7.23	10.88
	40	6.61	6.99	10.87
	50	6.45	6.73	10.86
	60	6.28	6.46	10.86
	70	6.08	6.18	10.86
	80	5.79	5.93	10.85
15	30	5.72	6.14	10.39
	40	5.60	5.94	10.37
	50	5.47	5.72	9.46
	60	5.33	5.51	10.36
	70	5.18	5.28	9.66
	80	4.99	5.07	9.80
18	30	4.56	4.92	10.02
	40	4.77	5.06	10.01
	50	4.67	4.90	9.35
	60	4.55	4.71	6.62
	70	4.44	4.55	8.36
	80	4.31	4.34	7.43

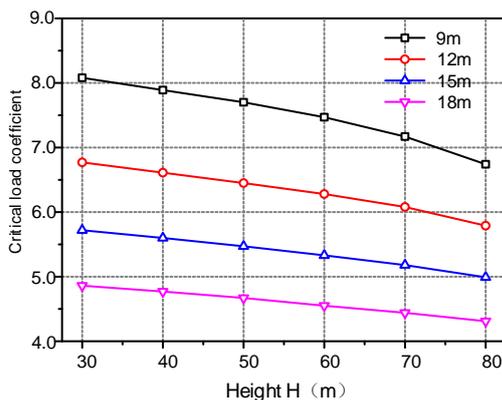


Fig. 7 - First-order critical load coefficient

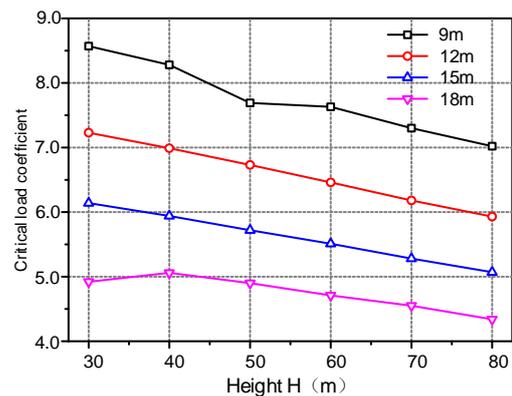


Fig. 8 - Second-order critical Load coefficient

The data from Table 5 show that the critical coefficient of the third-order load is significantly larger than that of the first two-order load at the different segment height, thereby indicating that the structure can hardly reach its third stage of instability. Failure mainly occurred in the first two stages of loading. Figure 7 and Figure 8 illustrate that the critical load coefficient decreases as falsework height increases, thereby indicating poor stability. However, critical load coefficient was guaranteed to exceed 4.0, which satisfied the basic stability requirement when structure segment was not more than 18 m. By contrast, the critical load coefficient decreases drastically when falsework is more than 70 m high. Therefore, in the falsework design, the reasonable segment length and total height of the falsework could be selected based on the different critical coefficients.

Influence of Transverse and longitudinal Spacing

The longitudinal and transverse spacing of a steel tube column was an important parameter that reflected the support area. To study the effect of longitudinal and transverse distances on stability, falsework with segment length and height of 12 and 60 m were used, respectively. With varying transverse and longitudinal spacing, critical load coefficients were calculated. Critical Load coefficient of different spacing of falsework is shown in Table 6.

Tab. 6 - Critical Load coefficient of different spacing

Transverse and Longitudinal spacing	Critical Load Coefficient
5 m × 5 m	5.66
6 m × 6 m	6.20
7 m × 7 m	6.39
8 m × 8 m	6.38
9 m × 9 m	6.26
10 m × 10 m	6.09

Figure 9 shows that spacing and critical load coefficient were set as the x- and y-axes based on the values in Table 6.

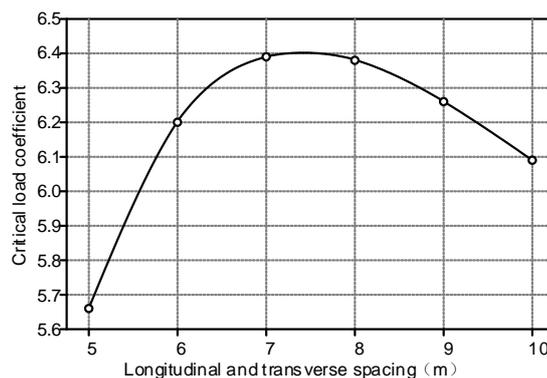


Fig. 9 - Transverse spacing versus critical load coefficient graph

Figure 9 shows that the relationship between the critical load coefficient and the transverse spacing is a parabolic curve. When the latticed column spacing was less than 6 m, the change was rapid and exponential. The rate of increase in the critical load coefficient decreased when the latticed column spacing was 6–7 m. The critical load coefficient reached the peak value at 7 m. The curve reached a plateau immediately after transverse spacing reached 7 m. Therefore, the falsework obtained the most stable and safest condition at spacing of around 7 m.

Influence of Diagonal Bracing

For lattice columns, lateral bracing was essential to maintain its stability, whereas the effect of diagonal bracing must still be determined. Finite element software was used to simulate the influence of braces on the stability of the lattice support. The first three stages of instability modes for structures with and without diagonal braces are shown in Figure 10.

Figure 10 shows that the critical load coefficient of the first-stage load is 6.28 and 4.91 with and without diagonal brace respectively.

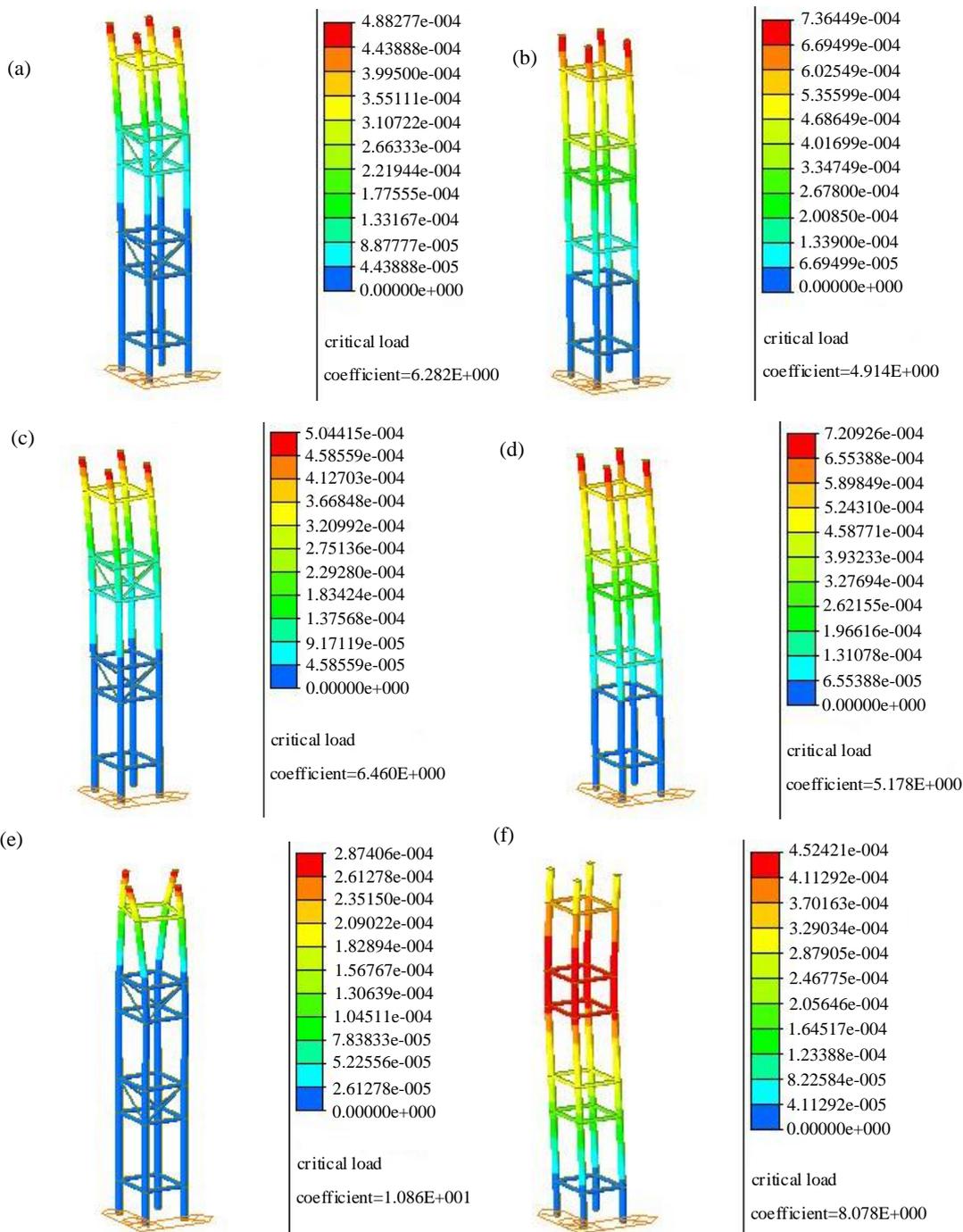


Fig. 10 - Stability of Shoring Tower with and without Diagonal Brace (a) First Stage with Diagonal Brace;(b) First Stage without Diagonal Brace; (c) Second Stage with Diagonal Brace;(d) Second Stage without Diagonal Brace; (e) Third Stage with Diagonal Brace; (f) Third Stage without Diagonal Brace

Figure 10 shows that the diagonal brace increased the critical load coefficient by 27.9%. The critical load coefficient of the second stage was 6.46 and 5.18 with and without diagonal brace, respectively. An increase of 24.7% in the critical load coefficient was not a significant improvement

compared with that of the first stage. The 34.4% improvement in critical load coefficient of the third stage was 10.86 and 8.08 with and without diagonal brace, respectively. Therefore, diagonal bracing significantly improved stability, especially during structure failure under higher orders of loading. Diagonal bracings should be considered an essential component in the design of high steel tube lattice support systems.

CONCLUSION

The finite element model of steel tube lattice column formwork support system in typhoon construction sites is established to ensure that the strength of the support system can meet the design requirements under the most unfavourable load conditions. The falsework-latticed column is simplified as a bidirectional bending member, and the stability of the falsework is calculated. The results show that the falsework meets the stability requirements. The influence of steel tube diameter, limb number, segment length, height, spacing and the presence of oblique braces on the critical load coefficient of the support structure is studied, and the optimal design size of the falsework is obtained. This study can provide a reference for similar types of falsework construction. The main results are as follows:

- (1) When the ratio of wall thickness δ to diameter d of latticed column is fixed, the critical load coefficient increases rapidly with the increase in steel tube diameter d . As δ/d increases, the stability of the falsework is enhanced.
- (2) Four-limb high steel tube lattice column is more stable than double- or single-limb structures, but the difference is not much compared with the bearing capacity of the six-limb structure. Therefore, four-limb high steel tube lattice column is the most cost-effective solution.
- (3) When the total height of the lattice column is constant, the small segment length enhances the stability of the structure, but the total height should not exceed 70 m. When the total height is more than 70 m, the stability of the falsework structure is poor. Considering the economy of the material used for the latticed column, the maximum length of the segment should not exceed 3 times the length of the joint system to ensure the stability of the structure effectively.
- (4) When the longitudinal and transverse spacing of the latticed column falsework is 5–7 m, the stability of the falsework increases rapidly and reaches the peak at around 7 m. When the longitudinal and transverse spacing is greater than 7 m, the stability of the falsework decreases gradually.
- (5) The critical load coefficient of the latticed column is 27.9% higher than that of the non-inclined brace, and the stability of latticed column is significantly improved by using the oblique brace.

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