

Article

Research on Temperature Modes and Parameter-Related Responses of Long-Span Asymmetric Cable-Suspended Structures

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Abstract

Cable-suspended structures are important auxiliary structures for the construction of long-span arch bridges. Due to topographic constraints, the cable-suspended structure of Liuchehe Bridge adopts an asymmetric structure form with a main span of 736 m. Nevertheless, research focusing on the mechanical behavior of large-span asymmetric cable hoisting structures remains limited at present. Under unfavorable loads, including temperature and cable saddle friction, tower buckling failure may occur in cable hoisting structures as a result of overstress. In addition, inappropriate changes in physical parameters and temperature of the main cable will alter its sag and consequently compromise construction precision. For the sake of the safety of the cable hoisting structure, a temperature gradient experiment was conducted on the steel pipes of the prefabricated tower by virtue of a practical engineering project. The change rule of the measured point temperature was analyzed, a temperature gradient pattern for tower steel pipes was proposed, and the deficiencies of the specifications were compensated for. On this basis, the effects of variations in temperature, main cable weight, main cable elastic modulus, guy cable tension, and saddle friction resistance on the mechanical behavior of the cable-suspended structure were analyzed. According to the temperature tests on the tower steel pipes, the maximum radial temperature gradient of the steel pipe section reaches 15 °C, which is higher than the thermal gradient value stipulated in the codes. Moreover, the steel pipe stress under the thermal gradient model proposed in the current research is greater than that under the thermal gradient model in the codes. The steel tube stress under the temperature gradient model adopted in this study is 7.6 times that specified in the design code. Temperature and the elastic modulus of the main cable have a significant influence on the mid-span deformation of the main cable. For every 1 °C temperature variation, the vertical displacement at the main cable mid-span changes by 25 mm. During the construction of the main cable, the sag of the main cable should be adjusted according to the rule governing temperature's influence on the mid-span of the main cable to avoid elevation deviations of the main cable arising from temperature. Saddle frictional resistance exerts a notable effect on tower deformation, guy cable tension, and tower stress. At a friction coefficient of 0.3, the stress caused by friction in the steel tube at the tower bottom constitutes 35.1% of the total stress under the maximum design hoisting load. During construction, the free rotation of rollers at the saddle should be ensured to reduce the mechanical response of the structure. The findings of this study can provide a basis for the design and construction of long-span asymmetric cable-suspended structures.



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Keywords: asymmetric cable-suspended structure; the cable hoisting construction; fabricated tower; temperature test; parametric response

1. Introduction

Concrete-filled steel tube (CFST) arch bridges combine the advantages of steel tubes and concrete and have high load-bearing capacity and seismic performance [1–4]. CFST arch bridges take advantage of mountainous terrain to span valleys with smaller spans, demonstrating distinct economic benefits. In addition, CFST arch bridges resolve the horizontal force at the arch springings by means of tie rods, making them equally applicable to plain areas. Thanks to these advantages, China has built the Pingnan Third Bridge, which boasts the longest span in the world with a span length of 575 m [5].

Cable hoisting construction technology is an important construction method for CFST arch bridges [6]. A cable-suspended structure is composed of main cables, hoisting cables, traction cables, towers, and anchorage systems. With the characteristics of strong spanning capacity and large operating space, it is suitable for the erection of arch segments of CFST arch bridges in complex terrains, such as canyons and rivers. Aiming at long-span cable hoisting structures, Zhou et al. [7] took the Shuangbao Bridge as the engineering background, introduced the load test scheme of the kilometer-scale arch bridge cable hoisting system from the aspects of construction preparation, test lifting counterweight design, loading position, and test procedure and analyzed the load test data, providing a reference for the load test design of cable hoisting systems for similar arch bridges. Similarly, Ming et al. [8], based on the cable hoisting structure of the Shuangbao Bridge, introduced key technologies from the perspective of construction techniques, including the installation and tension adjustment of tower guy cables, as well as the installation and adjustment of load-bearing cables, traction cables, and hoisting cables, which provides a reference for the construction of kilometer-scale arch bridge cable hoisting structures. Li et al. [9] designed an ultra-long-span cable hoisting structure based on the Chishui River Hongjun Bridge with a main span of 1200 m and verified the mechanical performance of the structure under different hoisting loads by means of a simulation model, providing a reference for the design of ultra-long-span cable hoisting structures. Qing et al. [10] designed a double-layer towerless cable hoisting system for a deck-type concrete-filled steel tubular arch bridge with a main span of 220 m, adopting mountain-based anchorages with transversely movable main cables to install the left and right steel tubular arch ribs via lateral movement of the main cables. The double-layer towerless cable hoisting structure has significantly improved construction efficiency. Chang et al. [11] analyzed safety cases during the main cable erection of suspension bridges, investigating incidents such as the fracture and falling of haulage cables caused by opposing pulling in the haulage system and main cable detachment due to slippage at the cable clamp during erection, providing guidance for construction risk prevention and control of suspension bridges. Lu et al. [12] took a basket-handle arch bridge constructed by an asymmetric cable suspension as the engineering background, carried out an analysis on the influence of asymmetric construction on the bridge, and conducted tracking monitoring of the alignment and stress of key sections during the construction process to ensure reasonable force bearing and smooth alignment of the structure, thereby reducing the impact of asymmetric construction on the completed bridge alignment and internal force.

The rational design and reliability verification of the cable tower are key factors determining the cable hoisting construction of arch bridges. In this regard, Xu et al. [13] took the cable hoisting structure of the Yelang River Grand Bridge as the research object,

carried out static analysis on the cable tower structure by using ANSYS 19.2, verified the mechanical performance of the cable tower, and optimized the structural form of the tower frame. Yi et al. [14] carried out an analysis of the mechanical performance of the tower frame during different construction stages, used a parametric model to analyze the stiffness, strength and stability of the tower frame under parameters, such as different load levels and temperature, and pointed out that attention should be paid to the changes in counterweight load and sling cable tension during the construction of the tower frame. Sun et al. [15] constructed a three-dimensional model of the tower frame, analyzed the strength, rigidity, and stability of the tower frame during hoisting construction, and put forward suggestions that the cable tension value and the lateral displacement of the tower top should be closely monitored during construction, and the guy cable tension should be adjusted in a timely manner to prevent the tower frame from twisting. Zhang et al. [16] carried out the main tower design work based on the Longtan Yangtze River Bridge, considering loads such as vehicles, temperature, and wind, took the safety, stability, and economy of the main tower as the objectives, optimized the main tower structure, and provided a calculation basis for the design of similar high-rise structures in the future. Chen et al. [17] proposed a design scheme of a long-span cable hoisting system aiming at the basket-handle structural characteristics of a mid-bearing steel box arch bridge with a kilometer-level span, presented a tower structure serving dual purposes of main cable and buckle cable, and verified the structural safety by means of the finite element method. Li et al. [18] proposed a steel tube tower structure that can be used for both main cables and buckle cables simultaneously, aiming at solving the shortcomings of traditional separate towers, such as high construction cost and long construction period. This tower structure has the characteristics of a high steel utilization rate and convenient assembly, which can effectively shorten the construction period and reduce the construction cost.

The mechanical performance of the main cable also affects the safety of the structure. In this regard, Zhou et al. [19] proposed a simulation method and modeling solution steps for saddle pulleys, which were verified by engineering cases. Xiong et al. [20] investigated the main factors affecting the alignment accuracy of the main cable, analyzed the influences of parameters such as main cable temperature, tower pre-lifting value, steel wire elastic modulus, steel wire diameter, and steel beam weight on structural stress, and pointed out that temperature exerts a notable effect on the alignment of the main cable. Chen et al. [21] analyzed the effects of the main cable's elastic modulus, self-weight, temperature, and tower deviation on the main cable alignment and drew the conclusion that the temperature during construction and the main cable deformation exhibit a linear relationship. Hou et al. [22] studied the mechanical properties of super-long-span cable-stayed suspension cooperative system bridges, focusing on the main cable sag, a key parameter. They established four bridge schemes with a span of 2300 m but different main cable sags and analyzed the differences in static and dynamic properties of the structure under different schemes. It was concluded that with the increase of main cable sag, the vertical, horizontal, and longitudinal stiffness of the cable-stayed suspension cooperative system bridge all show a decreasing trend. Xu et al. [23] designed the Dongtingxi Yuanshui Extra Large Bridge with a main span of 800 m; central inclined buckle cables were reasonably arranged at the mid-span of the main bridge, which reduced the beam end displacement and improved the wind stability. An active protection system combining "wire wrapping + flexible wrapping tape" on the main cable surface with internal dehumidification of the main cable was adopted, which improved the durability of the main cable. Liu et al. [24] studied the adjustment method of the main cable force of suspension bridges, pointing out that the influence of cable force redistribution and saddle movement during the adjustment process can be ignored when adjusting the main cable force after the completion of main cable erection.

Lu et al. [25] comparatively analyzed the changes in structural alignment and internal force of suspension bridges by changing parameters such as stiffening girder self-weight, main cable elastic modulus, temperature, and shrinkage creep. They pointed out that changes in stiffening girder self-weight, main cable elastic modulus, temperature field, and shrinkage creep have a significant impact on the structural alignment and internal force of concrete self-anchored suspension bridges. The scholars mentioned above have conducted extensive research on cable hoisting structures. However, studies on the single-tower asymmetric cable hoisting structure form remain scarce. In addition, there is a lack of analysis on the multi-objective response of the structure to a single parameter.

Due to topographic constraints, asymmetric forms of cable hoisting structures are inevitable. Therefore, for the new type of tower cable hoisting structure, it is necessary to carry out temperature tests on tower steel tubes, clarify the temperature gradient mode of tower steel tubes, and lay a theoretical foundation for the determination of tower temperature gradient values. In addition, it is also necessary to carry out research on the influence of parameters such as temperature, structural parameters, cable wind force, and saddle friction on the force bearing of asymmetric cable hoisting structures, clarify the influence degree of unfavorable parameters on the structural force bearing, and provide a basis for the design and construction of asymmetric cable hoisting structures.

In view of this, this paper relies on the cable hoisting structure of the Liuche River Bridge to carry out a temperature gradient test on the assembled tower steel tubes of the long-span asymmetric cable hoisting structure. The variation law of the measured point temperature was analyzed, a temperature gradient model for tower steel tubes was proposed, and the deficiencies of the specifications were compensated for. On this basis, the influences of parameters such as temperature, main cable weight, main cable elastic modulus, guy cable tension, and saddle friction resistance on the structural mechanical response of the cable hoisting structure were analyzed.

2. Materials and Methods

The Liuche River Extra Large Bridge, the world's second-longest-span double continuous arch bridge, spans a V-shaped canyon. The main arch of the bridge is a concrete-filled steel tube variable-section truss arch, with a catenary adopted as the arch axis and an arch axis coefficient of 1.53. The span of the arch is 263.5 m, the rise is 48.552 m, and the rise-span ratio is 1/5.4. Figure 1 demonstrates the shape diagram of the bridge. The bridge is located in Panzhou City, Guizhou Province, which lies in a mountainous and canyon-rich area. Characterized by abundant sunlight, it has a subtropical climate with mild winters and cool summers. The annual maximum temperature is 33 °C, which occurs in July. The minimum temperature is 5.1 °C, occurring in January. The annual average temperature is 23 °C.

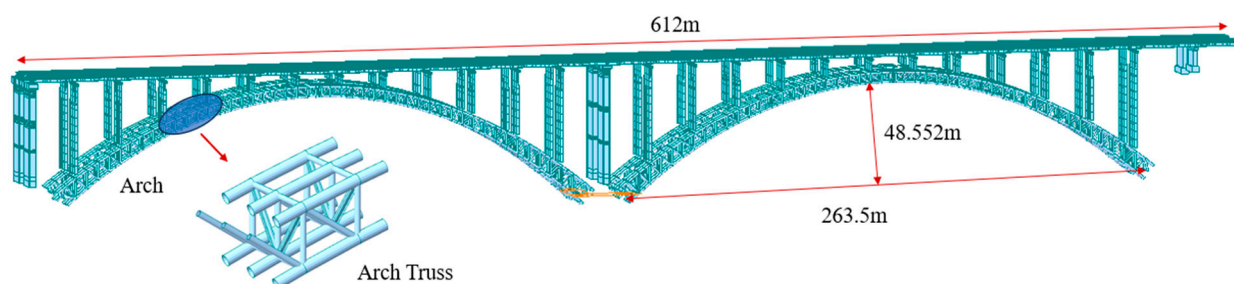


Figure 1. Bridge layout.

Since the bridge crosses a canyon and is restricted by terrain conditions, the arch segments of the bridge need to be transported to the installation positions via the cable

hoist. Figure 2 shows the structure of the cable hoist, which is composed of tower frames, saddles, main cables, and guy cables. The cable hoisting structure is a typical asymmetric structure, with towers only on the Anshun side, while the main cables are anchored on the Panzhou side via anchor blocks, and no side span is provided. Therefore, this project is selected as the research object of the present study.

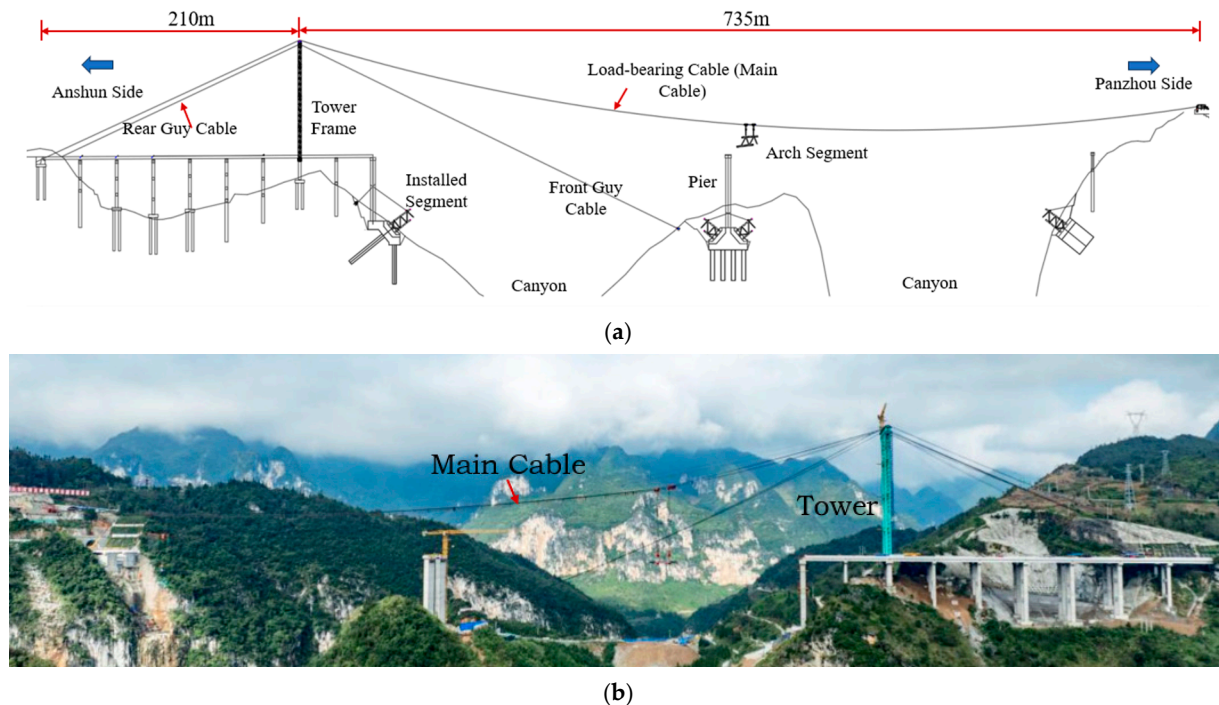


Figure 2. Cable hoist structure. (a) Layout of the cable hoisting structure. (b) Field view of the cable hoisting structure.

The tower functions to support the main cable and serves as the primary load-bearing component of the structure. With a height of 95 m, the tower is an overall portal-type structure, mainly composed of load-bearing columns, crossbeams, and diagonal braces. The tower is prefabricated, and different components are connected by bolts. The steel tubes of the tower are made of Q355 steel. The load-bearing columns are four-legged lattice steel tube trusses, arranged on the left and right sides of the tower. The transverse spacing between the load-bearing columns on both sides of the tower is 22 m. The vertical steel tubes of the load-bearing columns have a diameter of 820 mm and a wall thickness of 14 mm. Both the horizontal and diagonal steel tubes of the load-bearing columns have a diameter of 219 mm and a wall thickness of 8 mm. The crossbeams adopt a truss structure and are arranged in 3 layers along the height direction. Each crossbeam is composed of main chord tubes and connecting tubes. The main chord tubes have a diameter of 351 mm and a wall thickness of 10 mm, while the connecting tubes have a diameter of 219 mm and a wall thickness of 8 mm. Diagonal braces are arranged between the crossbeams and load-bearing columns, which effectively enhances the rigidity of the tower. A load-bearing beam is installed on the top of the tower to support the saddles. The load-bearing beam is composed of upper and lower layers of section steel and steel tubes in the middle. The section steel has a height of 800 mm, a width of 300 mm, a top and bottom plate thickness of 14 mm, and a web thickness of 26 mm. The middle steel tubes have a diameter of 820 mm and a wall thickness of 14 mm. The saddles are arranged on the top of the load-bearing beam. Figure 3 illustrates the tower structure. Table 1 lists the key numerical indices of the tower.

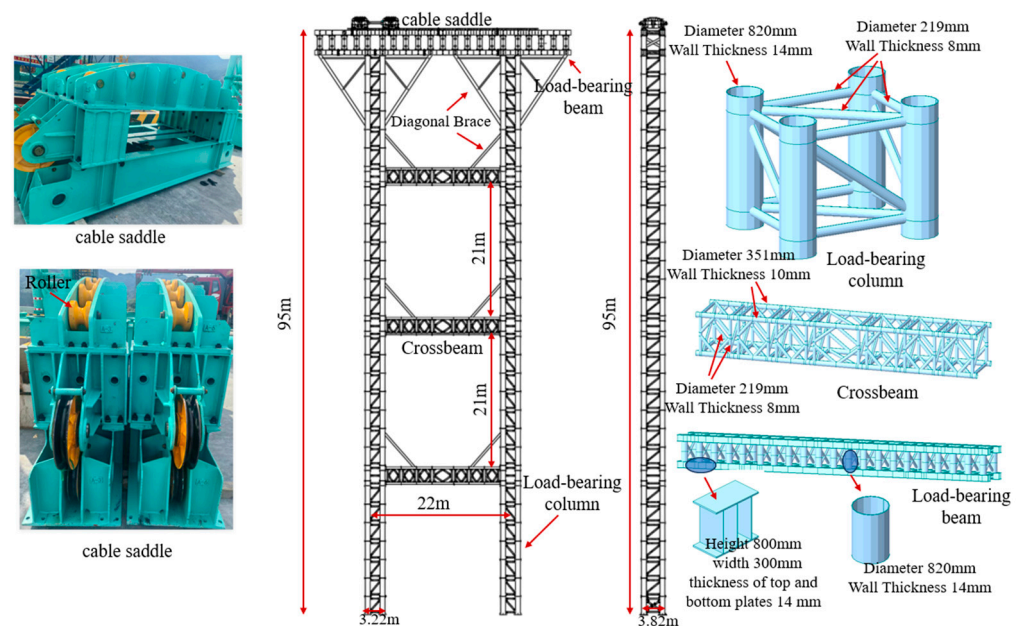


Figure 3. Tower structure.

Table 1. Key numerical indices of the tower (mm).

Item	Tower column steel tube	Horizontal tube of tower column	Inclined tube of tower column
Diameter	820	219	219
Wall thickness	14	8	8
Item	Chord tube of the cross beam	Connecting tube of the cross beam	Vertical steel tube between load-bearing beams
Diameter	351	219	820
Wall thickness	10	8	14

The mid-span of the main cables is 735 m, with two sets arranged at a spacing of 5 m. Each main cable consists of 10 steel wire ropes with a diameter of 60 mm, having an elastic modulus of 84,000 MPa and a unit weight of 105.6 kN/m³. Traveling carriages are installed on the main cables, enabling the transportation of arch segments to the specified locations, and the maximum design load capacity of the system is 160 t. Guy cables are arranged on the front and rear sides of the tower to enhance the rigidity and stability of the structure. The guy cables are composed of steel strands. There are two sets of guy cables on the front and rear, respectively, with each set containing 17 steel strands. The steel strands have a tensile strength of 1860 MPa, a unit weight of 78.5 kN/m³, and an elastic modulus of 195,000 MPa.

The research framework of this paper is shown in Figure 4.

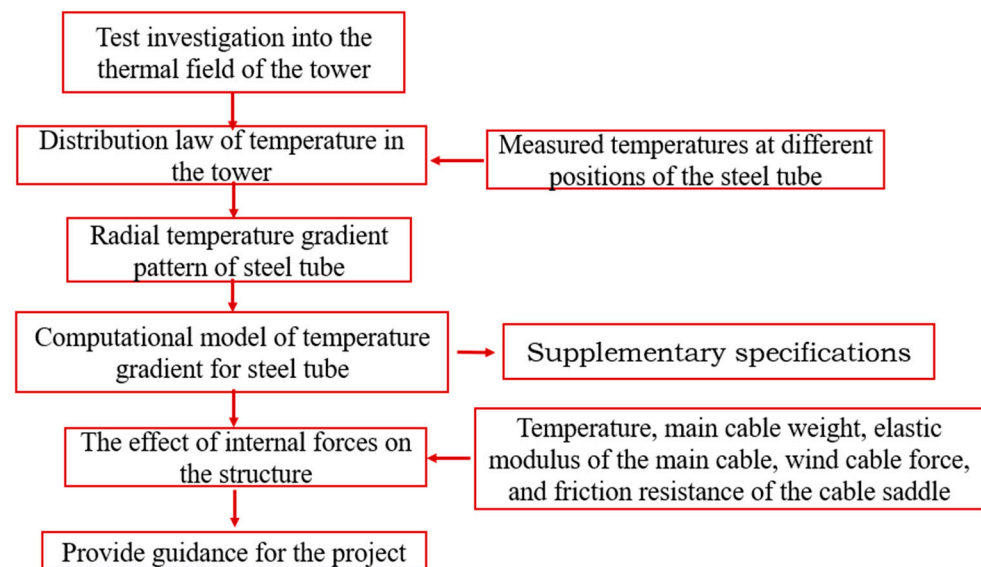


Figure 4. Research framework.

3. Temperature Test and Result Analysis

3.1. Temperature Test Layout

The tower is a steel tube truss structure with a light-colored coating, and the temperature test was carried out at a position 4.5 m displaced outward from the tower base. The tower is oriented from northwest to southeast. Four sensors were installed on the No. 1 tube of the right span, named R1, R2, R3, and R4. One sensor was arranged on No. 3 of the right span and named R5. Two sensors were installed on No. 1 and No. 3 of the left span, respectively, named L1 and L2. Figure 5 shows the sensor layout drawing. Table 2 shows the sensor layout.

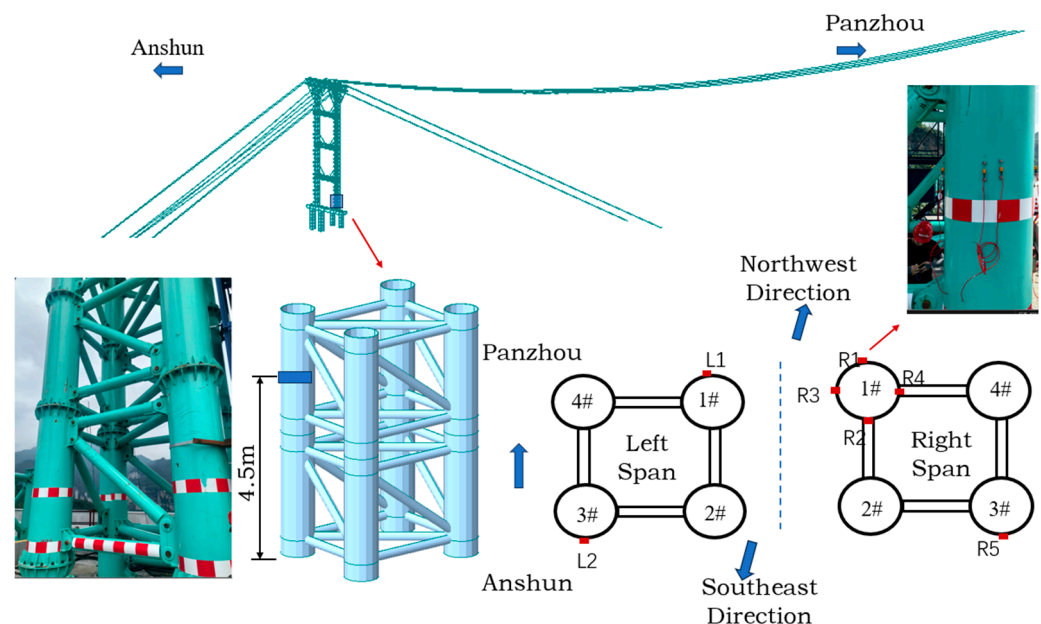


Figure 5. Temperature test measuring point layout.

Table 2. Sensor layout.

Item	L1	L2	R1	R2
Position	Left span steel tube No. 1	Left span steel tube No. 3	Right span steel tube No. 1	Right span steel tube No. 1
Orientation	Panzhou side	Anshun side	Panzhou side	Panzhou side
Item	R3	R4	R5	
Position	Right span steel tube No. 1	Right span steel tube No. 1	Right span steel tube No. 3	
Orientation	Towards the left side	Towards the right side	Anshun side	

Surface-mounted sensors with the model JMZX-212HAT are adopted as temperature sensors. The temperature sensors have a measurement range of $-50\text{ }^{\circ}\text{C}$ to $120\text{ }^{\circ}\text{C}$, a test accuracy of $\pm 0.1\text{ }^{\circ}\text{C}$, and a sensitivity of $0.25\text{ }^{\circ}\text{C}$. The sensor bases are fixed onto the surface of the steel tube by welding. The monitored temperature is the surface temperature of the steel tube. To avoid the influence of solar radiation on the temperature measured by the sensors, a side-opening metal protective cover is installed outside the sensors after their installation. A comprehensive tester is adopted to automatically collect the temperature data from the sensors and upload the collected data to the data system. Figure 6 shows the on-site diagram of the temperature test.

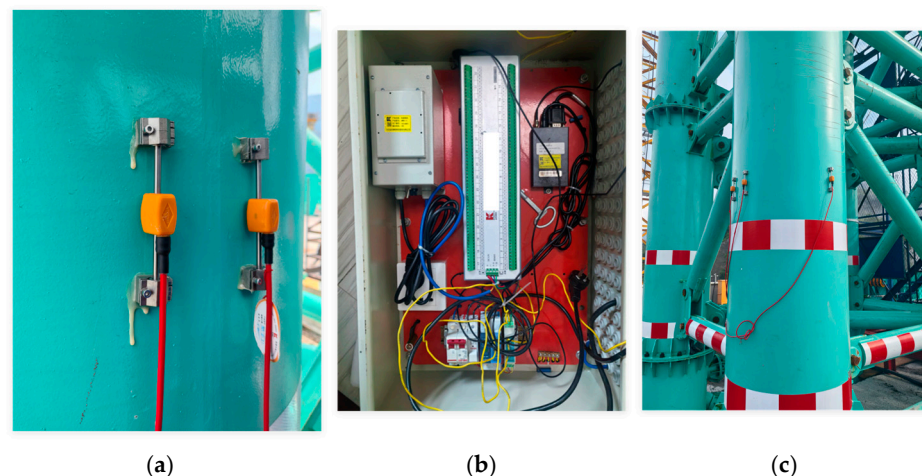


Figure 6. On-site temperature sensor layout. (a) Sensor. (b) Comprehensive tester. (c) Arrangement of sensors.

Considering the effects of solar radiation and temperature difference on the temperature of steel structures, typical weather conditions with strong solar radiation and large temperature difference during the construction stage were selected for the test. The temperature of the tower was continuously monitored from 12 to 28 July 2025. The temperature data were collected 24 h a day, starting from 0:00 a.m., with a sampling frequency of 60 min. The comprehensive tester can collect the temperature of multiple measuring points synchronously, avoiding the time deviation of the thermal measurement data among different monitoring points.

3.2. Analysis of the Temperature Test Results

The test was conducted on 18 July 2025, under clear weather conditions. The maximum atmospheric temperature was $31\text{ }^{\circ}\text{C}$, the minimum was $16\text{ }^{\circ}\text{C}$, and the southeast wind was at Level 1. Figure 7 is the variation diagram of measured temperature values of tower steel tubes.

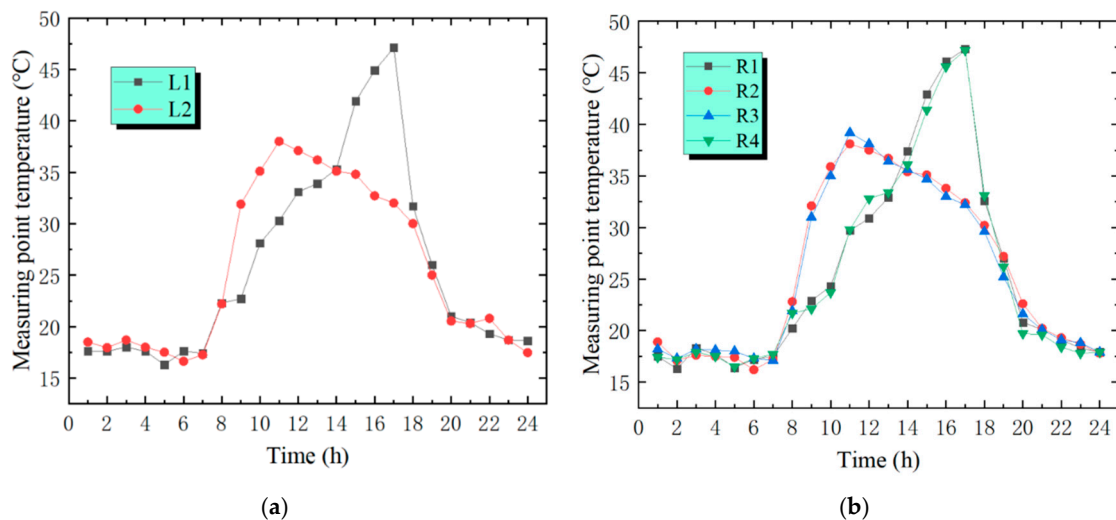


Figure 7. Temperature variation of tower steel tubes. (a) Left span. (b) Right span.

In Figure 7a, the temperatures of the monitoring points on the steel tubes show periodic variations. The temperature at measuring point L1 gradually increases before 17:00 and decreases gradually after 17:00. The maximum temperature at measuring point L1 is 47.1 °C, and the minimum temperature is 17.6 °C. The temperature at measuring point L2 gradually increases before 11:00 and decreases gradually after 11:00. The maximum temperature at measuring point L2 is 38 °C, and the minimum temperature is 17.5 °C. Before 11:00, the temperature at L2 is higher than that at L1, and the temperature rise rate at L2 is also faster than that at L1. The reason for this phenomenon is that before 11:00, the L2 measuring point is exposed to solar radiation, leading to a rapid temperature rise of the point. After 12:00, the sun no longer shines on the L2 measuring point but on the L1 measuring point, which causes the temperature of the L2 measuring point to drop while the temperature of the L1 measuring point continues to rise. In Figure 7b, the temperatures of the monitoring points on the steel tubes also show periodic variations. The temperatures at monitoring points R1 and R4 gradually increase before 17:00 and decrease gradually after 17:00. The highest temperatures at monitoring points R1 and R4 are 47.3 °C and 47.2 °C, respectively, and the minimum temperatures are 17.5 °C and 17.4 °C, respectively. The temperatures at measuring points R2 and R3 gradually increase before 11:00 and decrease gradually after 11:00. The maximum temperatures at measuring points R2 and R3 are 38.1 °C and 39.2 °C, respectively, and the minimum temperatures are 18.9 °C and 18.2 °C, respectively. Before 11:00, the temperatures of measuring points R2 and R3 are higher than those of measuring points R1 and R4, and the temperature rise rate is also faster. The reason for the temperature lag of the measuring points is related to solar radiation.

By synthesizing the data in Figure 7a,b, the temperatures of the measuring points facing northwest are higher than those facing southeast, and the period when the measuring point temperatures reach their peak is from 14:00 to 18:00. From 8:00 to 14:00, the temperatures of the measuring points facing southeast are higher than those facing northwest. From 14:00 to 18:00, the temperatures of the measuring points on the southeast side of the steel tubes are lower than those on the northwest side.

Figure 8 depicts the steel tube cross-section temperature difference pattern in accordance with the specifications. Figure 9 demonstrates the radial temperature difference pattern of steel tubes. It exhibits the time-dependent distribution pattern of temperature differences for the two measurement point pairs (R1–R2 and R3–R4) and makes a comparison with the temperature differences specified in the code.

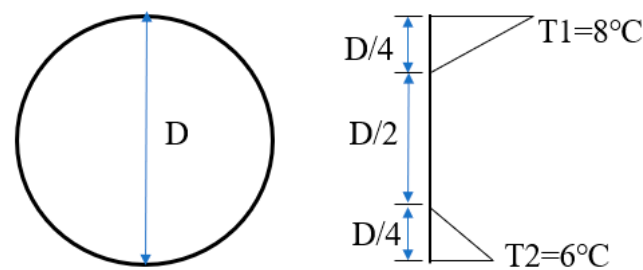


Figure 8. Radial temperature difference mode of steel tubes in the code.

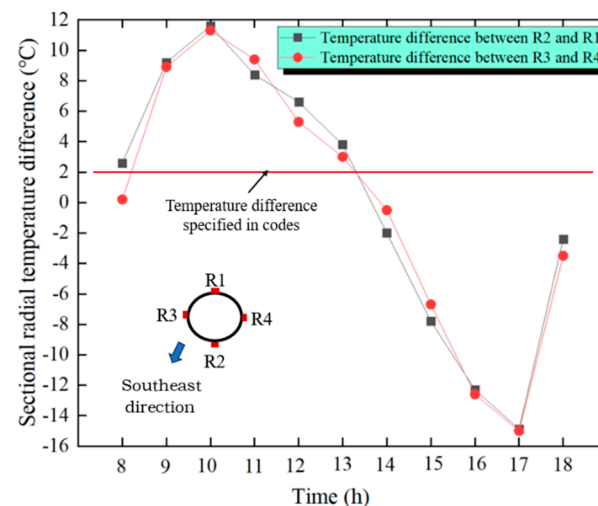


Figure 9. Radial temperature difference distribution mode of steel tube cross-sections.

In Figure 8, the radial temperature difference of steel tubes with light-colored coatings specified in the code [26] is 2 °C. In Figure 9, the radial temperature difference of the steel tube cross-section exhibits a sinusoidal function distribution regularity with time variation, and the temperature difference value changes continuously. The distribution characteristics of the temperature differences between measurement points R1–R2 and R3–R4 of the steel pipe are identical, and the cross-sectional temperature gradient peaks at 10:00 and 17:00. At 10:00, the temperature differences between measuring points R1 and R2, as well as between R3 and R4 of the steel tube, are 11.6 °C and 11.3 °C, respectively. At 17:00, the temperature differences between measuring points R1 and R2, as well as between R3 and R4 of the steel tube, are −14.9 °C and −15 °C, respectively. The absolute temperature differences of the two radial directions of the steel tube cross-section are 14.9 °C and 15 °C, respectively, with a small difference between them. Based on this, in Figure 10, the thermal gradient mode proposed in the present study is linear, and the maximum cross-sectional temperature difference is 15 °C. The temperature characteristic points are $T_1 = 15$ °C and $T_2 = 0$ °C. It should be noted that $T_1 = 15$ °C represents the maximum radial temperature gradient across the steel tube section during the continuous summer observation period, which is considerably larger than the temperature gradient specified for light-coated steel tubes in the code.

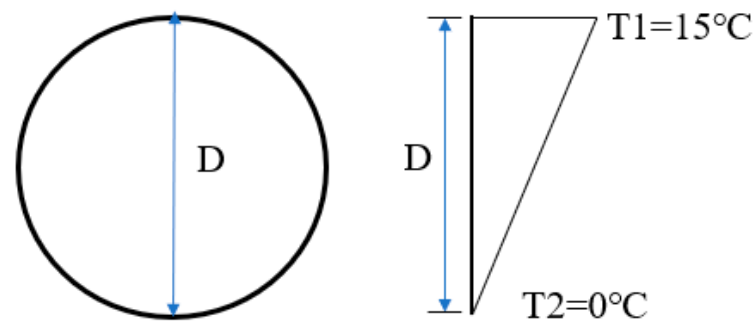


Figure 10. Radial temperature difference mode of steel tubes.

4. Analysis of the Influence of Key Parameters on Structural Mechanical Behavior

Midas Civil was adopted to develop a simulation model for the cable hoisting system of Liuche River Bridge [27–29], and the model consists of 4050 nodes and 5290 elements. The pylons and piers were simulated using beam elements, while the guy cables and main cables were simulated using cable elements. The main cable is divided into elements every 1 m. Fixed constraints were applied to the bottom of the piers. Rigid connections were adopted between the piers and pylons. Fixed constraints were imposed on the lower ends of the guy cables, while their upper ends were fixed to the pylons via rigid connections. A compression-only boundary condition was employed between the main cables and pylons to simulate the supporting effect of the cable saddles on the main cables. The compression-only boundary ensures the supporting effect of the cable saddle on the main cable and does not restrict the sliding deformation of the main cable. The load cases considered include the self-weight of the structure, as well as the weight of the trolley, traction cables, and hoisting cables acting on the main cables. The traction cables and hoisting cables are simulated as uniformly distributed loads applied to the main cable. The weight of the traveling trolley is simulated as a nodal load applied to the corresponding main cable node, and the movement of the trolley can be simulated by adjusting the position of the nodal load. The main cable adopts the unstressed length to achieve passive stress behavior. The unstressed length of the main cable is mainly determined as follows: under the no-load condition of the cable hoist, a nonlinear analysis is carried out, and an initial cable force is assigned to the main cable elements according to the designed sag of the main cable for model calculation. The cable force of the main cable elements is adjusted iteratively until the mid-span sag matches the designed sag, at which point the unstressed length of the main cable elements is extracted. The guy cable forces are modeled using external forces, aiming to ensure the tower deviates to the side span at no load and to the main span at maximum design hoisting load, with its deformation complying with specification limits. Figure 11 shows the simulation model of the cable hoisting structure.

For the maximum design hoisting load state of the structure, the traveling trolley is positioned at mid-span, and the design hoisting load is 160 t. The combined loads consist of the structure's dead weight, guy cable forces, main cable internal forces, and the masses of the traction cable, hoisting cable, trolley, and suspended load. On this basis, the effect of individual single-parameter variations on the structural mechanical behavior was analyzed. The parameters analyzed include temperature, elastic modulus of the main cable, unit weight of the main cable, friction resistance of the cable saddle, and guy cable force. The temperature variation ranges from $-20\text{ }^{\circ}\text{C}$ to $20\text{ }^{\circ}\text{C}$, with the reference state being the structure without temperature effects. The elastic modulus varies from -10% to 10% , with a reference value of $84,000\text{ MPa}$. The unit weight of the main cable ranges from -10% to 10% , with a reference value of 105.6 kN/m^3 . The friction resistance of the

cable saddle is analyzed in the range of 0.03 to 0.3 times; according to relevant research [30] and engineering tests, the maximum friction coefficient of the cable saddle is taken as 0.3. The cable saddle friction is obtained by multiplying the horizontal component of the main cable force at the saddle by the friction coefficient, with a reference value of 0, meaning no friction is considered, and the main cable deforms freely. The guy cable force varies from -10% to 10% , with the baseline values of the rear and front guy cable forces being 750 kN and 535 kN, respectively. Figure 12 shows the analysis positions of structural mechanical responses. Among these, the stress measurement positions of the pylon steel tubes are consistent with the sensor layout positions.

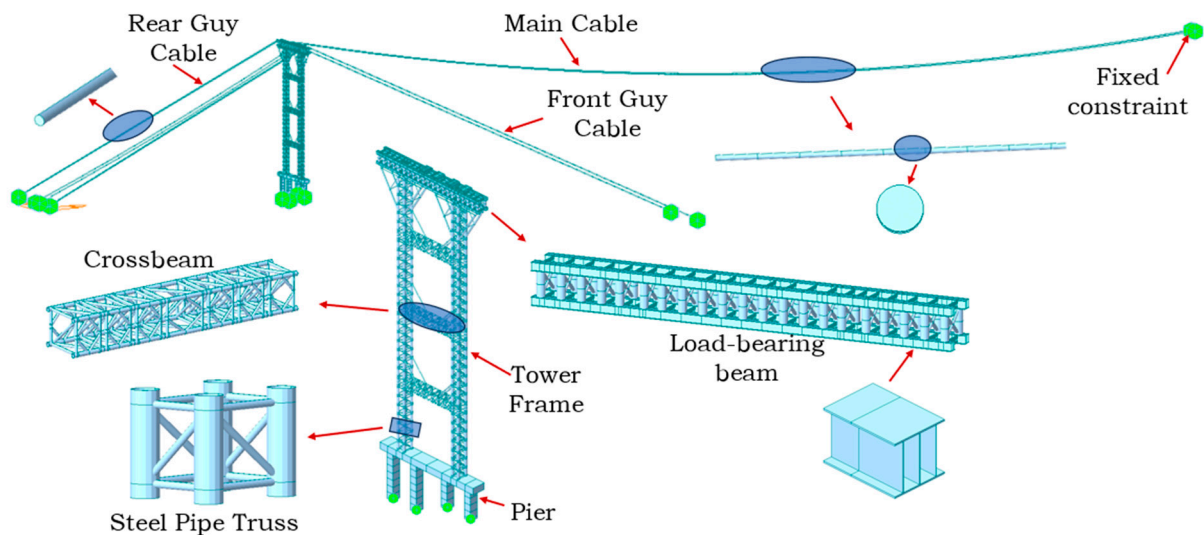


Figure 11. Simulation model.

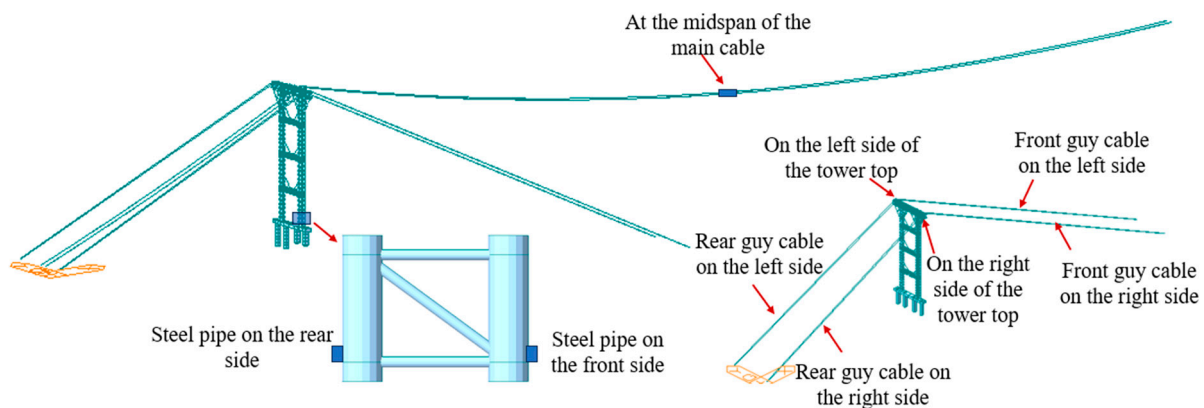


Figure 12. Analyze location.

The thermal gradient pattern proposed in Section 3 of the present study was applied to the simulation model, and the stress of the steel tubes at the bottom of the pylons was obtained and compared with the code specifications. Figure 13 shows the stress comparison of the pylon steel tubes under the two temperature gradient models.

In Figure 13, the thermal gradient induces compressive stress in the steel tubes on the front side of the pylons and tensile stress in those on the rear side. Under the thermal gradient pattern devised in the present study, the stresses in the front and rear steel tubes are -19.7 MPa and 19.8 MPa, respectively. However, under the thermal gradient model defined in the code, the stresses in the front and rear steel tubes are -2.6 MPa and 2.6 MPa, respectively. The steel tube stresses obtained under the temperature gradient model devised in the present study are much higher than those under the code-specified temperature gradient model.

Figure 14 shows the impact of key parameters on the vertical deformation of main cables.

In Figure 14a, the main cables deform upward when the temperature decreases and downward when the temperature increases, with the temperature variation exerting the most significant influence on the mid-span of the main cables. When the temperature drops by 5 °C, 10 °C, 15 °C, and 20 °C, the variation values of vertical deformation at the mid-span of the main cables are 126 mm, 253 mm, 379 mm, and 506 mm, respectively. For every 1 °C change in temperature, the vertical deformation at the mid-span of the main cables changes by 380 mm. For every 1 °C change in temperature, the vertical deformation at the mid-span of the main cables changes by 25 mm. In Figure 14b, the mid-span of the main cables deforms downward when the elastic modulus of the main cables decreases and deforms upward when the elastic modulus increases. When the elastic modulus of the main cables changes by 10%, the maximum variation of the deformation at the mid-span of the main cables reaches 1689 mm. In Figure 14c, the mid-span of the main cables deforms upward when the unit weight of the main cables decreases and deforms downward when the unit weight increases. When the unit weight of the main cables changes by 10%, the maximum variation of the deformation at the mid-span of the main cables is 424 mm. In Figure 14d, the mid-span of the main cables deforms downward as the saddle friction resistance increases. When the friction coefficient is 0.3, the downward deformation at the mid-span of the main cables reaches 268 mm. In Figure 14e, the mid-span of the main cables deforms upward when the tension of the rear guy cables decreases. When the tension of the rear guy cables changes by 10%, the variation of the deformation at the mid-span of the main cables is 34 mm.

Overall, temperature and the elastic modulus of the main cables have a significant influence on the mid-span deformation of the main cables, while the influence of guy cable tension is the smallest. Therefore, the machining accuracy should be strictly controlled during the fabrication of the main cables to avoid deviations in the elastic modulus. During the erection of main cables, the sag of the main cables should be adjusted according to the influence of temperature on the mid-span of the main cables to avoid deviations in the elevation of the main cables caused by temperature.

Figure 15 reveals the influence of key parameters on the horizontal deformation of pylons.

In Figure 15a, the pylons deflect horizontally toward the side-span side when the temperature decreases and toward the main-span side when the temperature increases. The horizontal deformation on the right side of the pylon top is larger than that at the left side, while the impact of temperature variation on the horizontal deformation of pylons is relatively minor. When the temperature drops by 20 °C, the variation of horizontal deformation at the pylon top reaches 19.5 mm. In Figure 15b, the pylons deflect horizontally toward the side-span side when the elastic modulus of the main cables decreases. The variation of the elastic modulus of the main cables exerts a relatively minor influence on the horizontal deformation of the pylons. When the elastic modulus of the main cables decreases and increases by 10%, the variation values of horizontal deformation at the pylon top are 23 mm and 24 mm, respectively. In Figure 15c, the pylons deflect horizontally toward the side-span side when the unit weight of the main cables decreases. When the unit weight of the main cables decreases and increases by 10%, the variation values of horizontal deformation at the pylon top are 112 mm and 117 mm, respectively. In Figure 15d, the pylons deflect horizontally toward the main-span side when the saddle friction resistance increases. The saddle friction resistance exerts a considerable influence on the horizontal deformation of the pylons. When the friction coefficient is 0.3, the horizontal deformation at the pylon top reaches 584 mm. In Figure 15e, the pylons deflect horizontally toward the main-span side when the tension of the rear guy cables decreases. The variation of guy

cable tension exerts a considerable influence on the horizontal deformation of the pylons. When the tension of the rear guy cables decreases and increases by 10%, the variation values of horizontal deformation at the pylon top are 551 mm and 539 mm, respectively.

Overall, the saddle friction resistance plays the most crucial role in the horizontal deformation of the pylons, followed by the guy cable tension, while the temperature has the least influence.

Figure 16 is the impact of key parameters on guy cable tension.

In Figure 16a, the tension of the guy cables increases when the temperature decreases and decreases when the temperature increases. When the temperature changes by 20 °C, the maximum variation of the guy cable tension reaches 60 kN. In Figure 16b, when the elastic modulus of the main cables decreases, the tension of the rear guy cables decreases while the tension of the front guy cables increases. When the elastic modulus of the main cables decreases or increases by 10%, the maximum variation values of the tension of the front and rear guy cables are 1.43 kN and 4.91 kN, respectively. In Figure 16c, when the unit weight of the main cables decreases, the tension of the rear guy cables on the left span increases, while the tension of the front guy cables on the left span decreases. The influence law of the main cable unit weight on the guy cable tension of the right span is opposite to that of the left span. When the unit weight of the main cables decreases or increases by 10%, the maximum variation values of the tension of the front and rear guy cables are 3.2 kN and 11.3 kN, respectively. In Figure 16d, when the saddle friction resistance increases, the tension of the rear guy cables increases while the tension of the front guy cables decreases. The saddle friction resistance exerts a considerable influence on the guy cable tension, and its influence on the guy cable tension of the right span is greater than that of the left span. When the friction coefficient is 0.3, the variation of the tension of the rear guy cables on the right span reaches 1013 kN.

Overall, the saddle friction resistance plays the most crucial role in the guy cable tension, while the influence of other parameters is relatively minor.

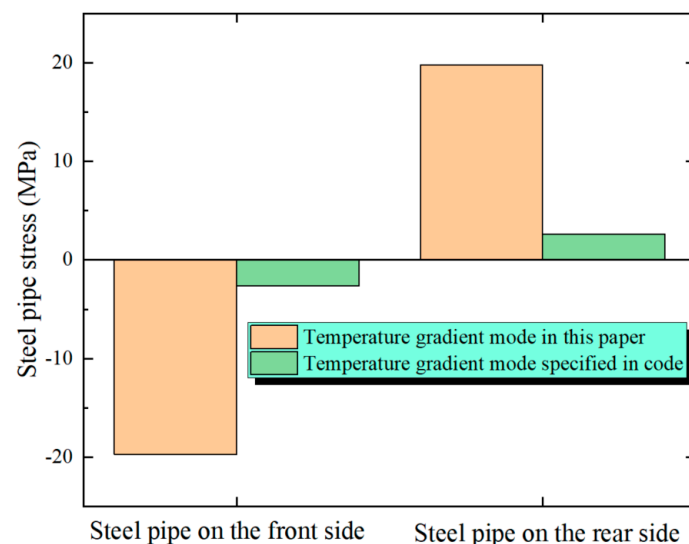


Figure 13. Stress comparison of pylon steel tubes.

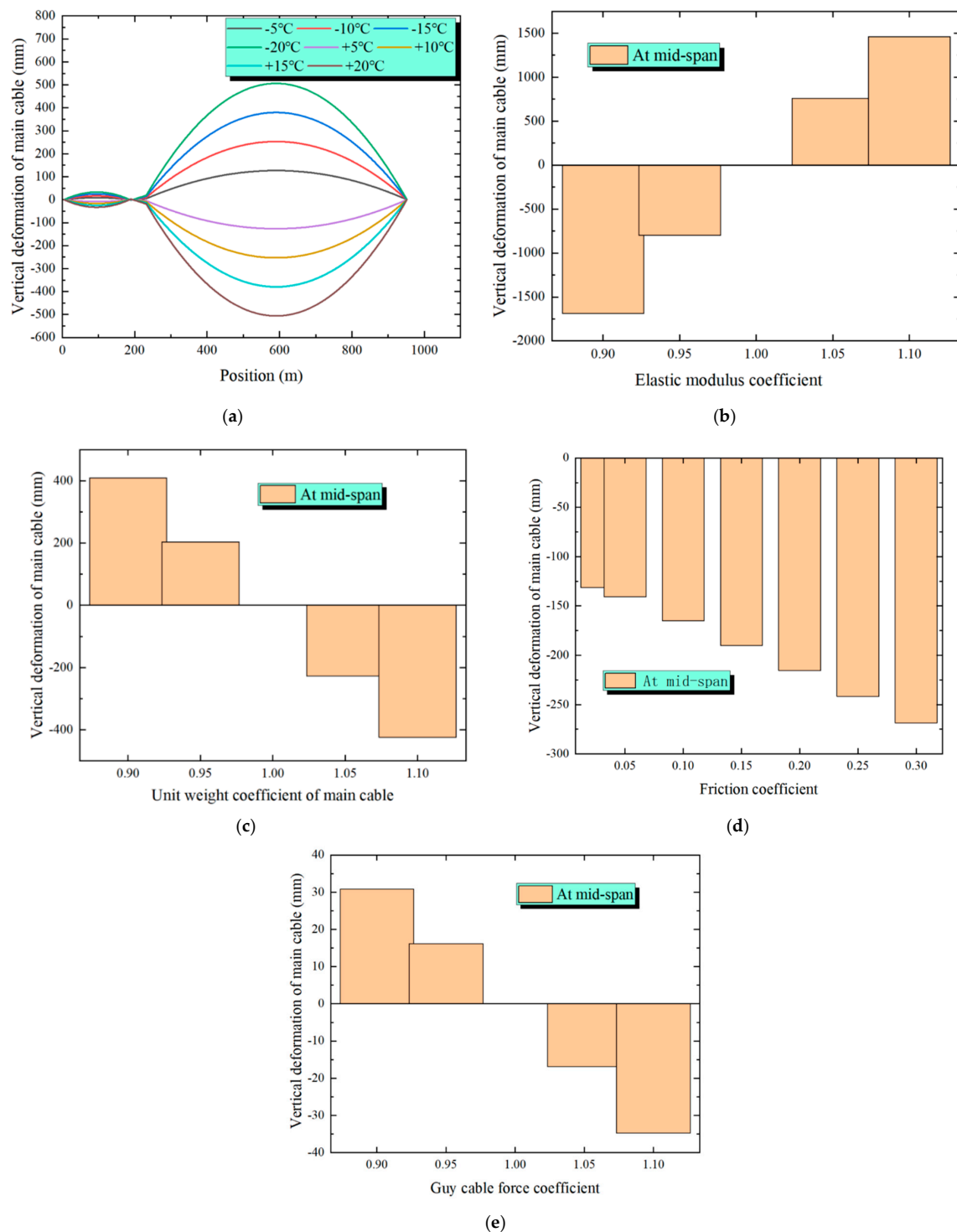


Figure 14. Influence of key parameters on vertical deformation of main cables. (a) Influence of temperature. (b) Influence of elastic modulus of main cables. (c) Influence of unit weight of main cables. (d) Influence of saddle friction resistance. (e) Influence of guy cable tension.

Figure 17 is the influence of key parameters on pylon stress.

In Figure 17a, the pylon stress is compressive stress when the temperature decreases, and the stress at the pylon bottom is tensile stress when the temperature increases. When the temperature changes by 20 °C, the maximum variation of the pylon stress reaches 12.1 MPa. In Figure 17b, the pylon stress is tensile stress when the elastic modulus of the main cables decreases, and it is compressive stress when the elastic modulus of the main cables increases. When the elastic modulus of the main cables changes by 10%, the

maximum variation of the pylon stress reaches 6 MPa. In Figure 17c, when the unit weight of the main cables decreases, the stress of the steel tubes on the front side of the pylons is tensile stress, and the stress of the steel tubes on the rear side of the pylons is compressive stress. When the unit weight of the main cables changes by 10%, the maximum variation of the pylon stress reaches 13 MPa. In Figure 17d, when the saddle friction resistance increases, the stress of the steel tubes on the front side of the pylons is compressive stress, and the stress of the steel tubes on the rear side of the pylons is tensile stress. When the friction coefficient is 0.3, the maximum variation of the pylon stress reaches 59 MPa. At the maximum design hoisting load, the front steel tube at the tower bottom exhibits a stress of -168 MPa, and the rear one shows -159 MPa. At a friction coefficient of 0.3, the stress caused by friction in the steel tube at the tower bottom constitutes 35.1% of the total stress under the maximum design hoisting load. In Figure 17e, when the tension of the guy cables decreases, the stress of the steel tubes on the front side of the pylons is compressive stress, and the stress of the steel tubes on the rear side of the pylons is tensile stress. When the tension of the guy cables changes by 10%, the maximum variation of the pylon stress reaches 43 MPa. Overall, the saddle friction resistance exerts the most significant influence on pylon stress, followed by the tension of the guy cables, while the influence of other parameters is relatively minor.

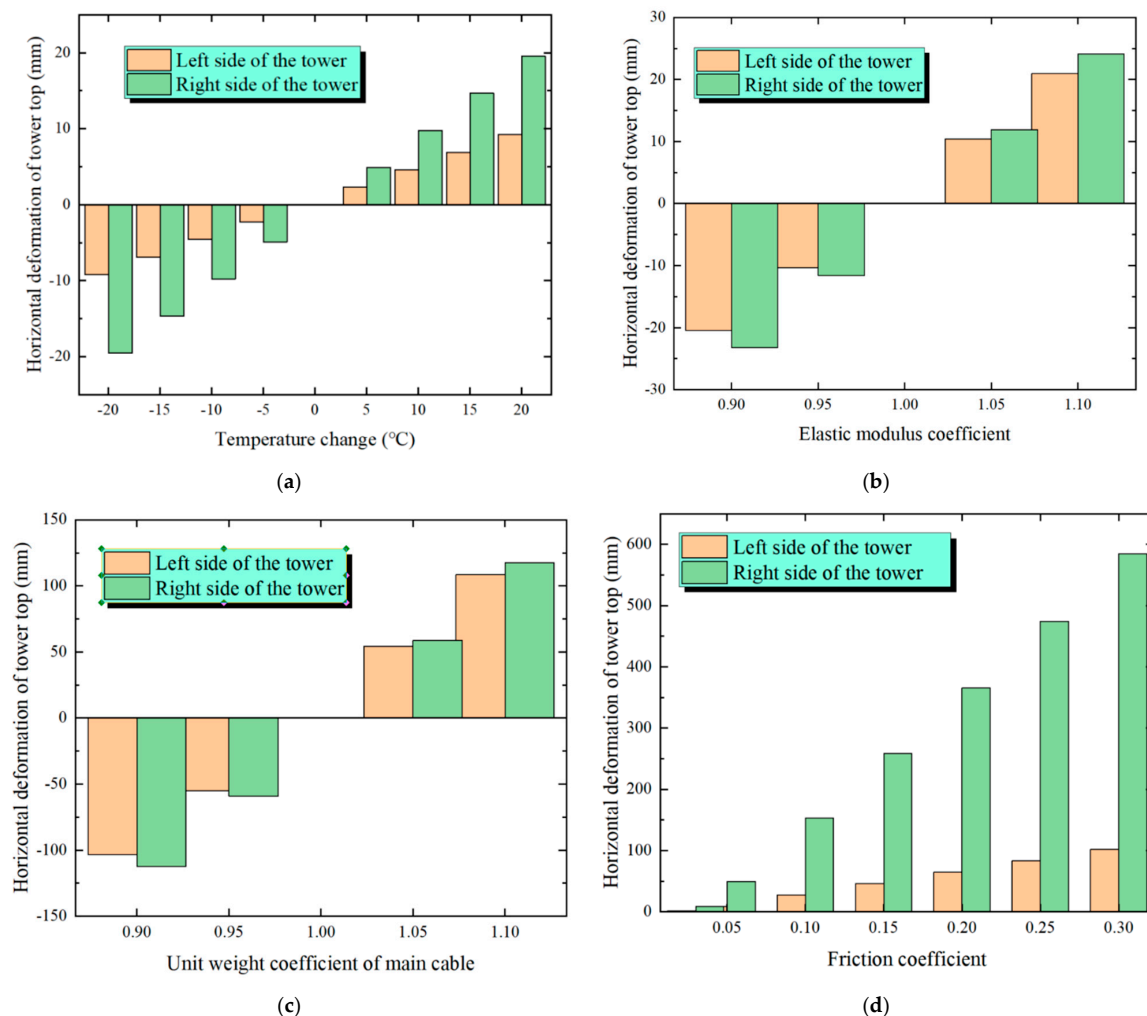


Figure 15. Cont.

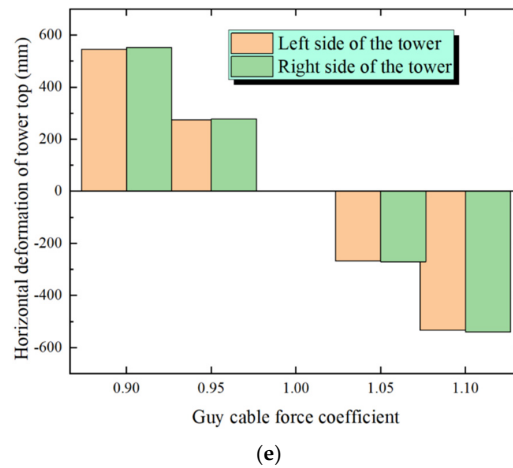


Figure 15. Influence of key parameters on horizontal deformation of pylons. (a) Influence of temperature. (b) Influence of elastic modulus of main cables. (c) Influence of unit weight of main cables. (d) Influence of saddle friction resistance. (e) Influence of guy cable tension.

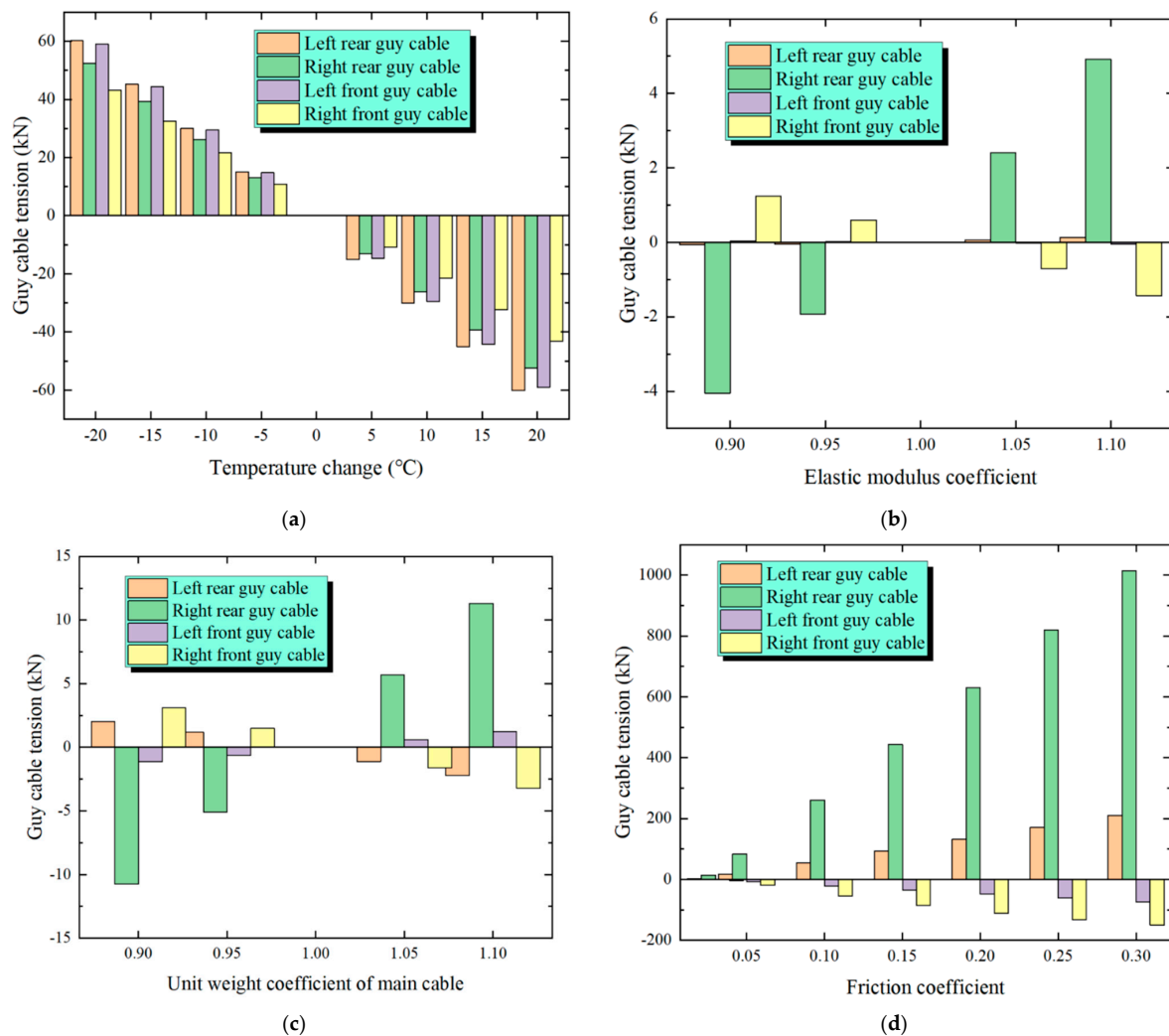


Figure 16. Influence of key parameters on guy cable tension. (a) Influence of temperature. (b) Influence of elastic modulus of main cables. (c) Influence of unit weight of main cables. (d) Influence of saddle friction resistance.

Based on the comprehensive analysis of Figures 15–17, the saddle friction resistance exerts a considerable influence on the structural stress. During construction, the frictional

resistance of the rollers can be reduced by applying lubricating oil. Alternatively, increasing the curvature radius of the cable saddle can reduce the contact pressure on the main cable and thereby lower the frictional resistance.

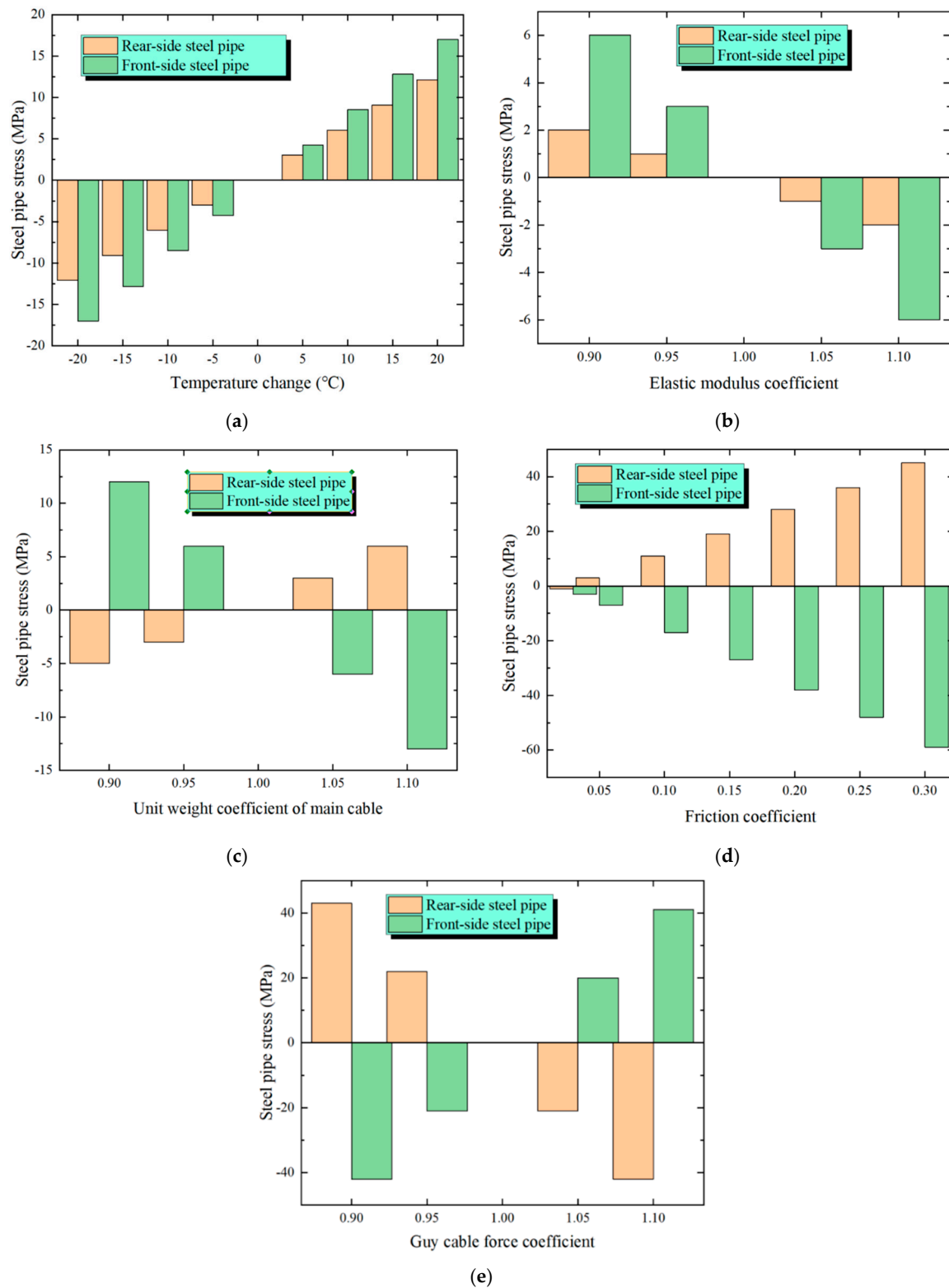


Figure 17. Influence of key parameters on pylon stress. (a) Influence of temperature. (b) Influence of elastic modulus of main cables. (c) Influence of unit weight of main cables. (d) Influence of saddle friction resistance. (e) Influence of guy cable tension.

5. Conclusions

- (1) Drawing on the temperature trial data of the steel tube truss of the pylon, the temperature at the measuring points of the steel tube varies periodically, with a maximum temperature of 47.3 °C and a minimum temperature of 17.4 °C. The maximum radial temperature gradient of the steel tube section occurs at 17:00, with a value of 15 °C, which is much higher than the temperature gradient specified in the code. The thermal gradient model advanced in this research is linear, with a maximum radial temperature difference of 15 °C across the section. The temperature characteristic points are $T_1 = 15\text{ °C}$ and $T_2 = 0\text{ °C}$.
- (2) Under the thermal gradient pattern advanced in this research, the stresses of the steel tubes on the front and rear sides of the pylon are -19.7 MPa and 19.8 MPa , respectively. However, under the temperature gradient model specified in the code, the stresses of the steel tubes on the front and rear sides of the pylon are -2.6 MPa and 2.6 MPa , respectively. The steel tube stress under the thermal gradient pattern advanced in this research is much higher than that under the temperature gradient model specified in the code.
- (3) Temperature and the elastic modulus of the main cable have a significant impact on the deformation at the mid-span of the main cable. During the fabrication of the main cable, the machining accuracy shall be strictly controlled to avoid deviations in the elastic modulus. During the erection of the main cable, its sag shall be adjusted in accordance with the law governing the temperature effect on the mid-span of the main cable to avoid elevation deviations of the main cable caused by temperature. The saddle friction resistance has a significant effect on pylon deformation, guy cable tension, and pylon stress. At a friction coefficient of 0.3, the stress caused by friction in the steel tube at the tower bottom constitutes 35.1% of the total stress under the maximum design hoisting load. During construction, the frictional resistance of the rollers can be reduced by applying lubricating oil. Alternatively, increasing the curvature radius of the cable saddle can reduce the contact pressure on the main cable and thereby lower the frictional resistance.

The temperature gradient mode for the steel tube of the pylon proposed in this paper can lay a theoretical foundation for determining temperature gradients in the design of similar pylons in the future. In addition, the influences of temperature on the geometry of the main cable and of cable saddle friction on the mechanical behavior of asymmetric cable hoisting structures obtained in this paper can provide a reference for the construction and construction control of similar structures.

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