

Temperature field effects and thermal-induced response of curved steel box-girder bridges using field monitoring data and numerical assessments

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Abstract: Curved steel box girder bridges have been increasingly used in practice due to their distinctive structural merits. However, compared to straight configurations, their geometry may induce significant torsion and coupled bending-torsion effects. Under large ambient temperature variations, they can also be susceptible to considerable temperature gradients and complex thermal stresses, leading to the temperature fields with intricate spatiotemporal characteristics. To this end, this paper investigates the temperature field distributions and associated mechanical response of curved steel box girder bridges through field monitoring and finite element analysis. Based on the field measurements carried on a representative case study bridge, the vertical and transverse temperature gradient models are proposed for curved steel box girder bridges. It is shown that the vertical temperature gradient can be closely characterized by an exponential function, whereas the transverse temperature gradient closely follows a Gaussian distribution. The proposed temperature gradient models are subsequently applied to derive the vertical and transverse temperature differences for the curved steel box girder bridges located in 12 major cities across various climatic zones in China. The resulting temperature field distributions and thermal-induced mechanical responses, including thermal stresses and deformations under individual as well as coupled vertical and transverse temperature gradients are assessed in detail. Particular focus is given to examining the influence of the curvature radius on the thermally induced deformations, stresses, and torsional behavior of curved steel box girder bridges. The findings provide a fundamental basis for improving the design procedures and offer systematic region-specific thermal loading parameters in support of climate-adaptive design approaches for curved steel box girder bridges.

Keywords: bridge engineering; curved steel box girder bridges; field monitoring; temperature gradients; climate-adaptive design; structural response.

1. Introduction

Curved steel box girder bridges can be adapted to complex terrain conditions and enable multi-directional traffic integration, making them an increasingly common structural type in urban transportation systems [1]. However, due to solar radiation, mutual shading among different components of curved steel box girder bridges causes a lag in internal temperature variation compared to the surface. This thermal lag leads to vertical and transverse temperature gradients, which may have a significant influence on the mechanical performance of such bridges [2, 3].

Neglecting such temperature gradient effects in the design may result in structural deterioration or even failure of box girder bridges [4–6]. Thus, extensive research efforts have been undertaken to investigate the temperature field behavior in bridges. Based on the field measurements in concrete box girder bridges, Abid et al. [7] examined the temperature distribution characteristics using one year of field monitoring data in conjunction with empirical analysis. Based on year-long temperature measurements from the Waterloo Bridge, Nepomuceno et al. [8] modified temperature gradient models specified in AASHTO [9] and New Zealand bridge design codes [10]. By utilizing field data from a 5×35 m continuous steel-concrete composite box girder bridge, Li et al. [11] proposed a vertical temperature gradient model for these bridges. Through a combination of year-long monitoring of a three-cell concrete box girder bridge and concurrent local meteorological data, Zhang et al. [12] developed predictive equations for temperature fields incorporating real-time air temperature, solar radiation, and wind speed. Based on field measurements from three long-span concrete box girder bridges located in cold regions, Li et al. [13] revealed the distinct features of vertical temperature distributions in these bridges subjected to cold climates. Based on the long-term monitoring data, Ding et al. [14] also applied hypothesis testing and extreme value analysis to develop a temperature difference model and identified the distribution patterns of both transverse and vertical temperature differences in steel box girder bridges. Wang et al. [15] systematically analyzed the vertical and transverse temperature distributions of steel box girder bridges based on long-term monitoring data. Based on the temperature data from a steel box girder bridge in a cold region, Hu et al. [16] combined clustering and Weibull distribution analysis to determine the standard vertical temperature differences in steel plates and proposed a practical vertical temperature gradient model for such bridges. Based on the monitoring data from a cable-stayed bridge, Zhu et al. [17] also examined the vertical temperature distribution characteristics of steel box girder bridges. Although notable advancements have been made in understanding the

temperature distributions in concrete and steel box girder bridges through field measurements [7–17], detailed studies on both the vertical and transverse temperature gradients for such bridge structures are comparatively lacking.

Due to the challenges in acquiring in-situ temperature field data, numerical simulations have been widely used to investigate the thermal behavior of bridges. Taysi et al. [18] conducted a three-dimensional (3D) finite element (FE) thermal analysis to characterize the temperature distribution and variation patterns induced by thermal loads in concrete box girder bridges. Fan et al. [19] developed a refined numerical model for the temperature fields in steel-concrete composite bridges, enabling thermal analysis across various cross-sectional configurations. Wen et al. [20] proposed a time-dependent FE model for composite box girder bridges and identified significant discrepancies in temperature gradients between the edge and central girders. Cai et al. [21] conducted a two-year continuous thermal simulation across seven distinct climatic regions to examine the effects of thermal loads on the structural behavior of concrete box girder bridges. Musab et al. [22] performed transient FE thermal analyses on concrete box girder bridges and assessed the effects of geometric parameters on temperature distributions. On the other hand, for steel box girder bridges, Xia et al. [23] developed a FE model of a typical cross-section in a long-span suspension bridge using field meteorological data, and examined the effects of girder geometry, deck paving on the top flange, and bridge orientation, on both the vertical and transverse temperature differentials. Lu et al. [24] integrated thermal analysis with field monitoring data to simulate the spatiotemporal temperature fields of steel box girder bridges, offering detailed insights into their temperature distributions. Tao et al. [25] employed a Hermite transform-based spectral representation method to simulate the long-term temperature field of the Su-tong Yangtze River Bridge, validating the accuracy of numerical simulations by comparing it with spectral and probabilistic gradient models. Hu et al. [26] developed a FE model to characterize the vertical temperature gradients in steel box girder bridges under cold climate conditions, and verified the effectiveness of the model against field measurements. Zhou et al. [27] conducted thermomechanical numerical simulations for the Peljesac Bridge and developed a temperature field model for steel box girder bridge under solar radiation. Hao et al. [28] constructed a FE model of a steel box girder based on the measured meteorological data and examined the distribution patterns of vertical temperature gradients.

Importantly, in-depth understanding of the distribution of temperature fields and their induced mechanical effects is critical for evaluating the structural safety of bridges. To this end, Lee et al.

[29] examined the thermal deformation of prestressed concrete beams under vertical and transverse temperature gradients based on temperature monitoring and experimental investigations. Wang et al. [30] assessed the thermal stress distribution characteristics in concrete box arch bridges through FE analysis and field monitoring data. By integrating field observations with numerical simulations, Shen et al. [31] identified the temperature fields and corresponding thermal stress distributions across the thickness of the top and bottom plates in concrete box girders. Mu et al. [32] conducted numerical analyses to investigate the thermal effects on orthotropic flat steel box girder bridges. By taking a curved steel box girder bridge as a case study, Wang et al. [33] investigated the influence of nonlinear temperature fields, self-weight, and eccentric loading, on the variation of support reactions. Based on both numerical simulations and field measurements, Zhou et al. [34] studied the static response of long-span suspension bridges subjected to temperature-induced actions during service. Luo et al. [35] employed FE modeling to evaluate the thermal effects on two steel box girder bridges with different temperature gradient patterns, and revealed their stress distributions under spatial temperature fields.

Overall, based on the above discussions, most previous studies [29–35] have mainly focused on the thermo-mechanical behavior of concrete box girder bridges and on straight steel box girder forms. However, due to the distinct bending-torsion coupling characteristics inherent to curved steel box girder bridges, their stress and deformation patterns under both individual and coupled vertical and transverse temperature gradients could significantly differ from those observed in their straight and concrete counterparts. Further studies are therefore needed on the differential mechanical behavior between the inner and outer edges of curved steel box girder bridges under spatially varying temperature fields. Given the more complex mechanical characteristics of curved steel box girder bridges under thermal actions, investigating these temperature field effects and associated thermally induced mechanical behavior is crucial for improving the load combination strategies and ensuring their service safety.

To address the above-mentioned research needs, this paper describes a detailed investigation into the temperature field characteristics and thermally induced mechanical response of curved steel box girder bridges through field measurements and numerical simulations. A typical curved steel box girder bridge is considered as a case-study. Based on the field monitoring data, the vertical and transverse temperature gradient models for curved steel box girder bridges are developed and validated using the field measurements. The effectiveness of an integrated FE modeling approach,

incorporating realistic geometric and thermal boundary conditions, is verified through comparison against the field monitoring data. By using the validated simulation approach, the proposed temperature gradient models are used to derive representative vertical and transverse temperature differences for the curved steel box girder bridges located in 12 major cities across various climatic regions in China. The resulting thermal response characteristics and structural response of the bridges under the individual and coupled vertical and transverse temperature gradients are assessed in detail. Particular attention is given to quantifying the influence of the curvature radius on the thermally induced deformations and torsional performance of curved steel box girder bridges. The implications of the findings on the climate-adaptive design of curved steel box girder bridges are highlighted within the discussions.

2. Field monitoring study

2.1 Case study bridge

A high-speed railway bridge located in Northern China, at Jinan, Shandong Province, is considered as the case-study. Fig. 1 provides schematic information for the bridge, which has a span length of 185 m. The girder components, including the web, top flange, bottom plate, diaphragms, and stiffeners are fabricated using the Q370 steel [36]. The deck slab is constructed using a 50 mm thick ultra-high-performance concrete (UHPC) [37]. The steel box girder is aligned along a circular curve with a radius of 950 m. The main girder employs a single-box single-cell cross-section with a constant height. The thicknesses of the top and bottom plates are 20 mm, and both plates are thickened outward from the center. The internal web plates have a thickness of 20 mm. Each side of the girder is equipped with 1.6-meter-wide cantilever flanges. Standard diaphragms are spaced at 5 m intervals in the longitudinal direction of the bridge, with additional annular diaphragms installed between each pair of standard diaphragms, with diaphragms having a thickness of 20 mm.

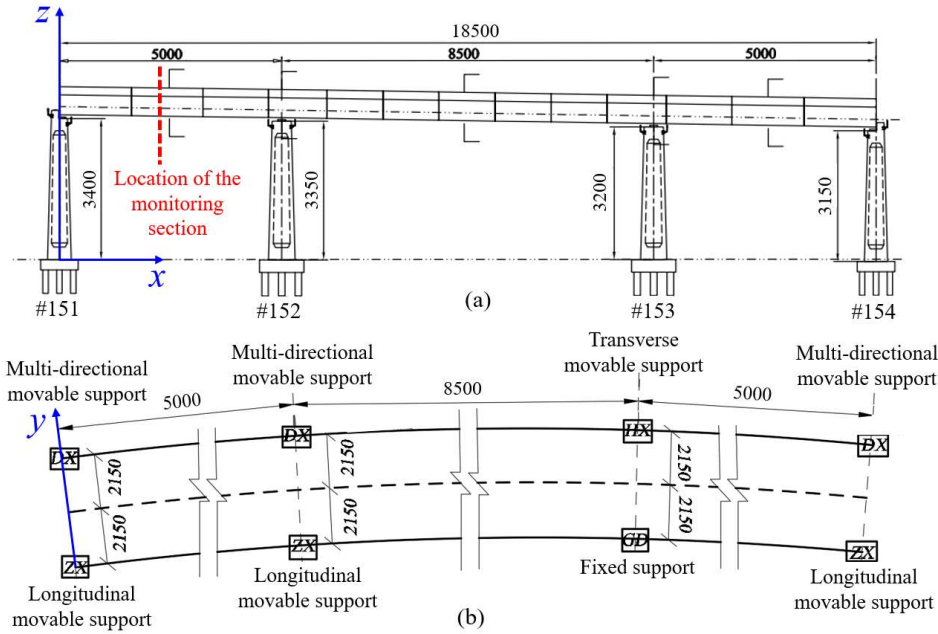


Fig. 1. Schematic information of bridge: (a) elevation and (b) plan view (units in cm).

To collect in-situ temperature data for the bridge, temperature sensors were employed to monitor the temperatures on both the internal and external surfaces of the girder. Fig. 2 shows the locations of field monitoring points. To capture the measured temperature field accurately, the measurement points were optimally arranged along the girder cross-section. A total of 17 temperature monitoring points were installed on the inner and outer surface of the girder cross-section.

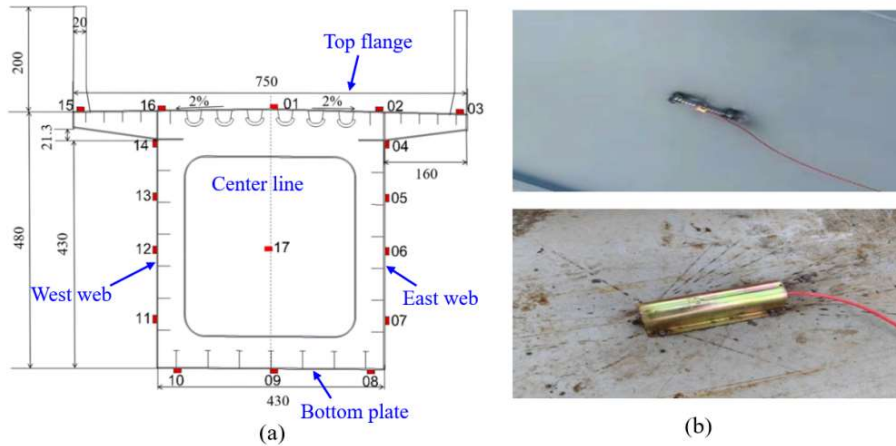


Fig. 2. Arrangements of: (a) field monitoring points and (b) temperature sensors (units in cm).

2.2 Measured temperatures

In natural environments, the temperature field of curved steel box girders varies due to the influence of solar radiation, ambient temperature, and wind speed [38]. Seasonal changes in solar radiation intensity could further lead to variations in the temperature distributions in the girders. Thus, this study focuses on detailed analysis of field-measured temperature data during the periods of solar-induced heating in summer and winter at the bridge site. Fig. 3 shows the recorded temperature profiles recorded at the bridge site on 7-9 July 2022 (summer) and 6-8 January 2023 (winter).

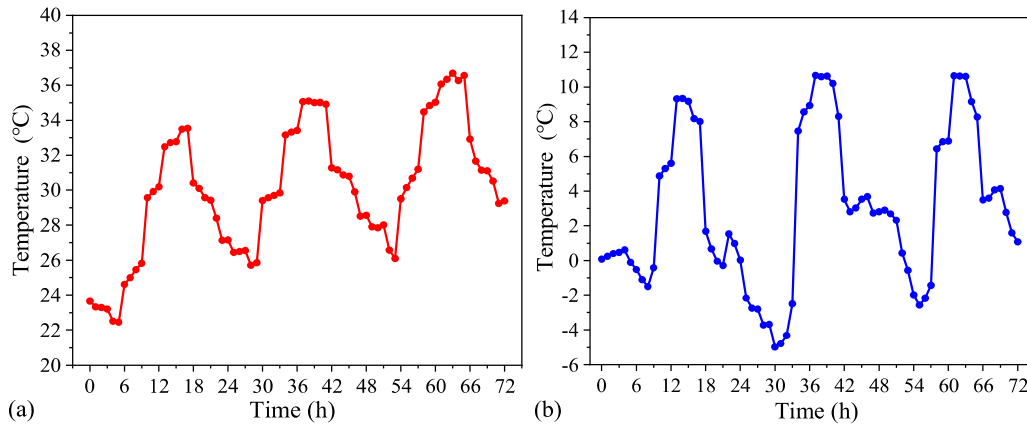
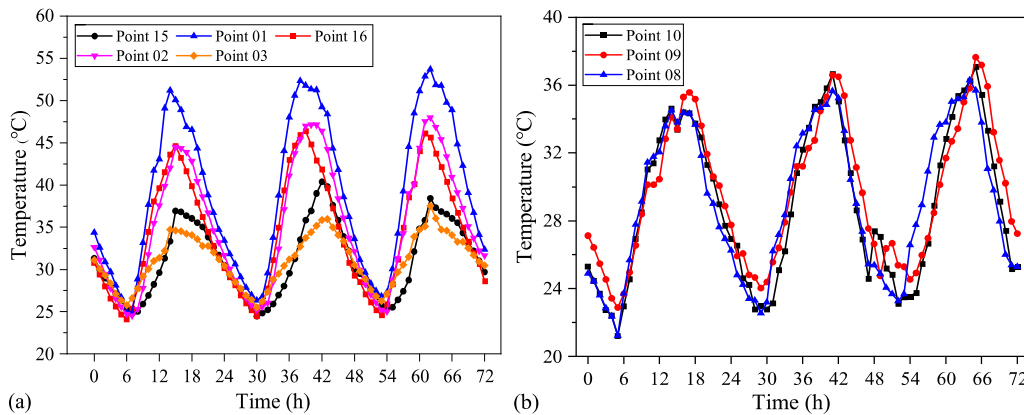


Fig. 3. Recorded temperature at bridge site: (a) 7-9 July 2022 and (b) 6-8 January 2023.

Fig. 4 and Fig. 5 show the recorded temperature variations at different measurement points on the girder from 7 to 9 July 2022 and from 6 to 8 January 2023, respectively. According to the Chinese design code [42], the negative temperature gradient can be generally specified to be -0.5 times the magnitude of the positive temperature gradient under normal service load conditions. Thus, only the summer case is examined in present study in detail, whereas that of the winter case (i.e., with similar assessment method of the summer case) is not presented in this study, due to the page limitation. As shown in Fig. 4(a), the temperature at the top flange followed a clear diurnal sinusoidal pattern, ranging from 24.1 °C to 53.7 °C. The temperatures began to rise at 6: 00 and peaked around 15: 00, closely aligning with ambient air temperature cycles, indicating strong thermal coupling with the atmospheric conditions. Under direct solar radiation, the central point (01) experienced higher temperatures compared to the edge points (15 and 03), resulting in a significant transverse temperature gradient across the top flange, with a maximum difference of 18.6 °C. This disparity is primarily due to the shading effect of parapets, which reduced solar exposure near the edges.



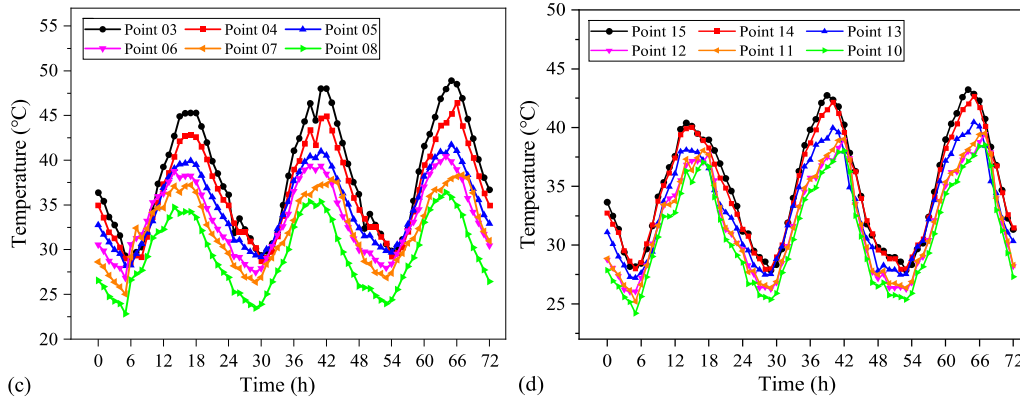
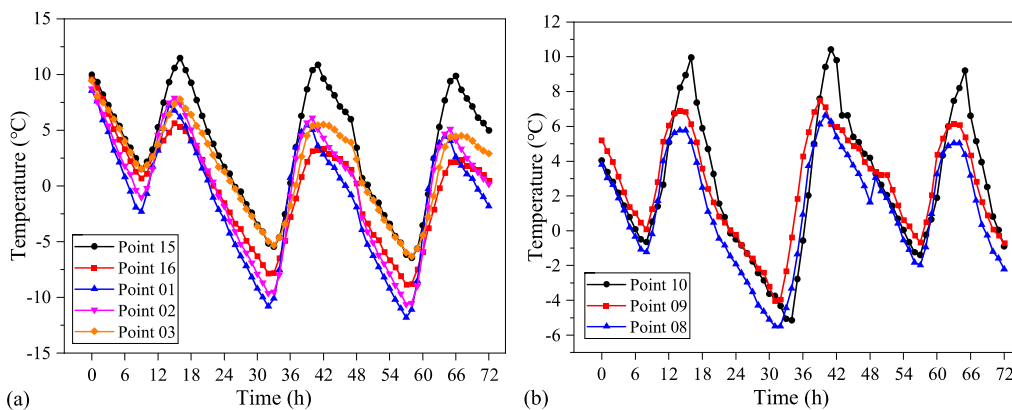


Fig. 4. Measured temperatures in girder on 7-9 July 2022: (a) top flange, (b) bottom plate, (c) east web, and (d) west web.

Fig. 4(b) shows that the bottom plate also exhibited sinusoidal thermal fluctuations, with temperatures ranging from 21.2 to 37.6 °C. Peak temperatures were observed at 17: 00 at the centre of the bottom plate, indicating a delayed thermal response compared to the top flange. At 14: 00, the top flange reached its maximum of 53.7 °C, whereas the bottom plate remained at 35.4 °C, resulting in a vertical temperature gradient of 18.3 °C. This lag is attributed to the absence of direct solar exposure on the bottom plate, which is primarily heated by the reflected ground radiation and atmospheric diffuse radiation. Besides, as illustrated in Figs. 4(c) and (d), the east and west webs also showed diurnal sinusoidal fluctuations. The former exhibited temperatures ranging from 22.8 to 48.9 °C, while that of the latter ranged from 24.2 to 43.2 °C. The former generally exhibited higher peak temperatures, occurring between 16: 00 and 17: 00, whilst the latter reached its daily peaks earlier, between 14: 00 and 16: 00. The elevated temperatures on the east web are mainly due to its greater exposure to solar radiation compared to the west web.



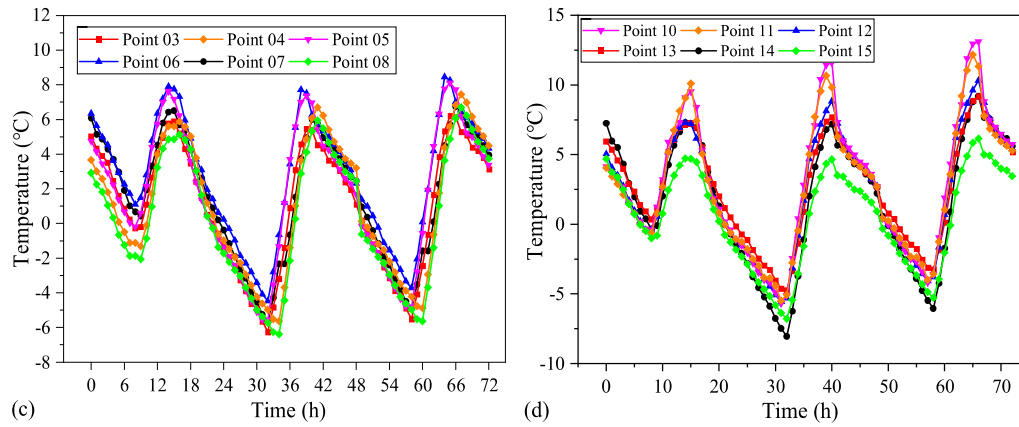


Fig. 5. Measured temperatures in girder on 6-8 January 2023: (a) top flange, (b) bottom plate, (c) east web, and (d) west web.

3. Development of temperature gradient models

3.1 Vertical temperature gradient

To characterize the vertical temperature profile, a day exhibiting a large vertical temperature difference (9 July 2022) was selected as a representative case. It was found that the temperature difference on the east web consistently exceeded that on the west web throughout the day. Hence, the temperature distributions at various measuring points on the east web were evaluated across different time intervals, as shown in Fig. 6.

As seen from Fig. 6, between 0:00 and 8: 00 on 9 July 2022, temperatures at various points on the east web varied uniformly with minimal differences, exhibiting an overall declining trend. After 10: 00, the top flange was directly exposed to solar radiation, leading to a temperature increase, whereas the bottom plate, receiving little solar radiation, heated more slowly. Hence, a temperature gradient developed between the top and bottom plates. From 13: 00 to 16: 00, significant temperature fluctuations occurred in the web, especially within 1 m of the top flange, where changes were most pronounced. After 16: 00, due to the steel plate high thermal conductivity delaying the bottom plate cooling, the temperature of the top flange began to decrease, whereas that of bottom plate continued rising. After 19: 00, the temperatures of both the top and bottom plates declined more rapidly than the web, gradually reducing the temperature difference.

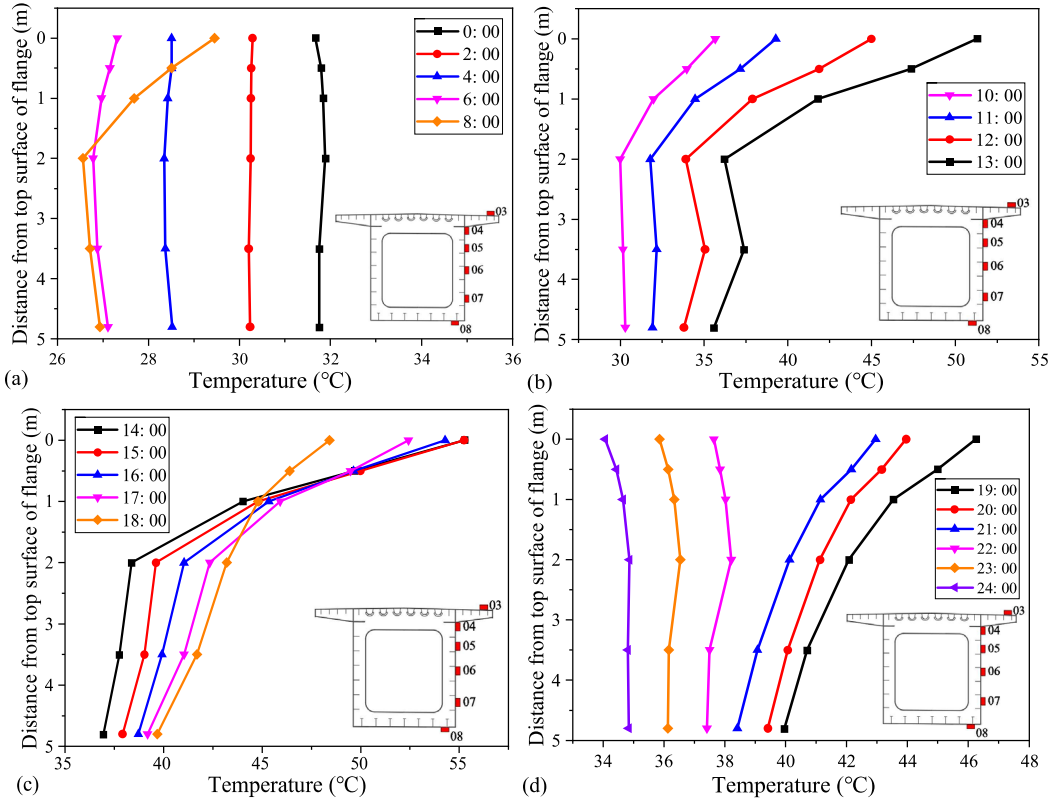


Fig. 6. Measured temperature variations at the east web on 9 July 2022: (a) 0: 00 to 8: 00, (b) 10: 00 to 13: 00, (c) 14: 00 to 18: 00, and (d) 19: 00 to 24: 00.

Based on the measured vertical temperature data, the period between 13: 00 and 16: 00 was identified as the high-temperature interval. During this period, a pronounced vertical temperature gradient develops within the girder. Considering the top flange as the starting point for height measurement and increasing downward along the vertical axis, the temperature recorded at the bottom plate was used as the reference temperature. The temperature differences from 13: 00 to 16: 00 were then calculated and plotted along the height of the girder, as illustrated in Fig. 7. As seen from Fig. 7, the temperature exhibited a nonlinear decreasing trend along the depth of the girder. The temperature differences between the top and bottom plates at 13: 00, 14: 00, and 15: 00 were 15.7, 18.2, and 17.3 °C, respectively. The distribution pattern of the vertical temperature gradient closely resembled the nonlinear exponential function, which is also used in the Chinese design code [39]. Based on the above, an exponential function is used in this study to fit the vertical temperature gradient of the girder (see Fig. 7), which can be expressed through Eq. (1) below.

$$T(y) = \Delta T_1 \cdot e^{-a \cdot y} \quad (1)$$

where $T(y)$ is the temperature difference at the calculation point, in °C; y is the vertical distance from the top flange, in m; ΔT_1 is the temperature difference between the top and bottom plates of the girder along the vertical direction, in °C; and a is a fitting parameter. The temperature data measured at 14: 00 on 9 July 2022, when the maximum temperature difference between the top and

bottom plates reached 18.2 °C, was used for the curve fitting. The fitting resulted in a value of a of 1.4, with a correlation (R^2) of 0.96, indicating the effectiveness of the proposed vertical temperature gradient model. The resulting temperature difference profile (i.e., dashed line) was then compared with those obtained at other times, as illustrated in Fig. 7.

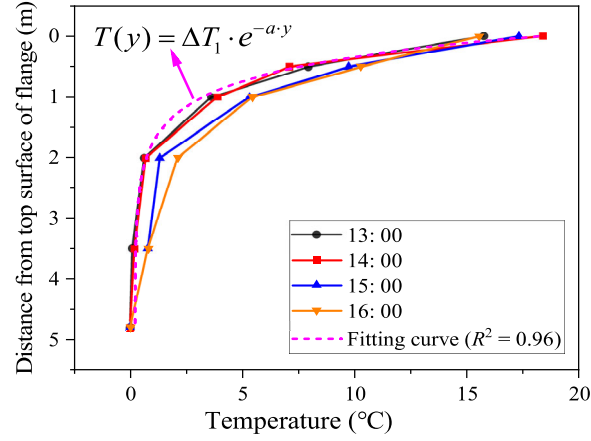


Fig. 7. Comparison of measured and theoretical vertical temperature distributions for the girder.

3.2 Transverse temperature gradient

To investigate the pattern of transverse temperature gradient, a representative day is considered with a significant lateral temperature difference (9 July 2022). The temperature distributions at various measuring points on the top flange were examined at different time intervals throughout the day, as illustrated in Fig. 8. From 0: 00 to 8: 00, the top flange experienced relatively stable temperature variations; however, between 10: 00 and 16: 00, the temperatures increased due to the rising ambient temperatures, leading to an increasingly pronounced temperature gradient. The top plate temperature peaked at 55.8 °C at 14: 00. After 15: 00, as the ambient temperature began to drop, the temperature of the top flange gradually decreased and stabilized.

As shown in Fig. 8, the period from 14: 00 to 17: 00 is identified as the high-temperature interval, during which a pronounced transverse temperature gradient developed across the top flange. Taking the temperature measured at the eastmost point of the top flange as a reference, the temperature differences at 14: 00 to 17: 00 were determined and plotted along the transverse direction of top flange, as shown in Fig. 9. The temperature difference profile exhibited a bell-shaped curve closely resembling a Gaussian function. Therefore, the transverse temperature gradient pattern of the girder is fitted using a Gaussian function, as illustrated in Fig. 9 and expressed in Eq. (2) below.

$$T(x) = \Delta T_2 \cdot e^{-\frac{(x-x_c)^2}{2w^2}} + b \quad (2)$$

where $T(x)$ is the temperature difference between a transverse point on the girder and the eastmost point of the top flange, in °C; ΔT_2 is the maximum transverse temperature difference, in °C; x is

the distance from the east side of top flange; x_c is the location of the center of the top flange; w is the shape parameter; and b is a fitting parameter. The temperature data measured at 14: 00 on 9 July 2022, when the transverse temperature difference of the top flange reached its maximum value (6.7 °C), was selected for model fitting. The fitting resulted in values of $b = -0.5$ and $w = 1.55$, with a correlation (R^2) of 0.95, indicating the effectiveness of the proposed transverse temperature gradient model. The fitting result (i.e., dashed line) is illustrated in Fig. 9.

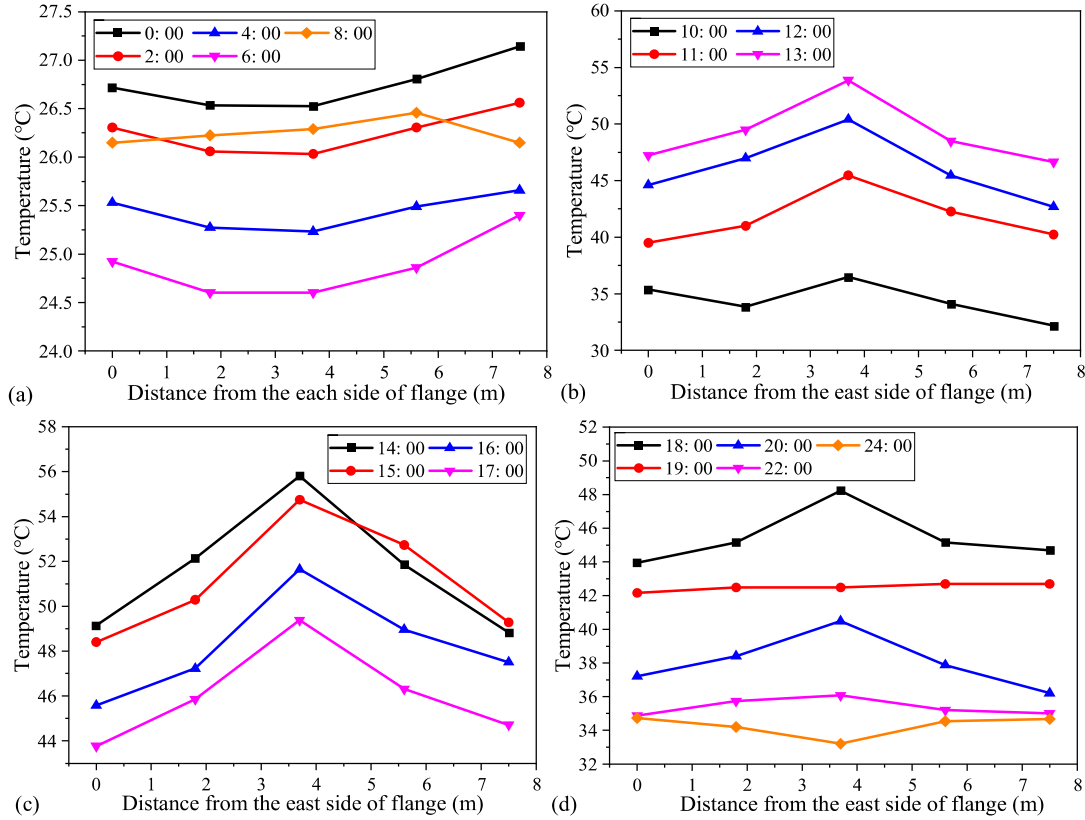


Fig. 8. Measured temperature variations at the top flange of the girder on 9 July 2022: (a) 0: 00 to 8: 00, (b) 10: 00 to 13: 00, (c) 14: 00 to 18: 00, and (d) 19: 00 to 24: 00.

Moreover, it should be noted that, compared with the linear models adopted in several existing design codes [40–42], the developed nonlinear exponential and Gaussian functions in representing the vertical and transverse temperature gradient models (i.e., Eqs. (1) and (2)) based on the field monitoring data could better reflect the underlying physical mechanisms and demonstrate significantly improved agreement with the measured data. The proposed temperature gradient models account for both vertical and transverse temperature variations, thereby capturing more comprehensive and critical structural responses, which can serve as a valuable reference for the design of the curved steel box girder bridges.

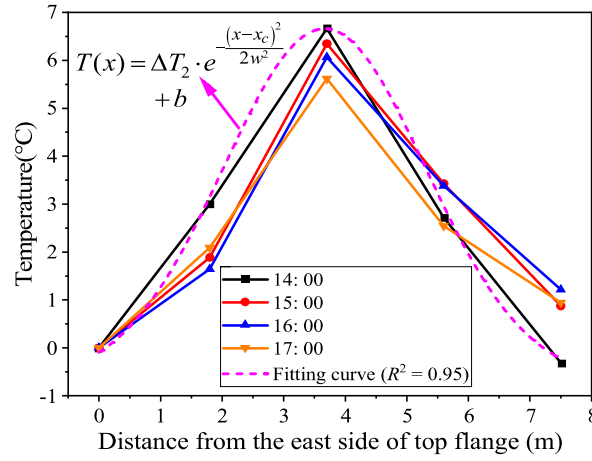


Fig. 9. Comparison of measured and proposed transverse temperature curves.

3.3 Numerical simulation of temperature profiles

A variety of numerical simulation approaches have been developed for the temperature profiles of structures, including finite difference and finite volume methods [2–5]. Among them, the conventional finite element method (FEM) remains the most widely used in the field of solid and structural mechanics, owing to its ability to accurately handle complex geometries and boundary conditions through mesh discretization and basis function construction. In order to overcome the limitations of traditional discretization schemes, Samaniego et al. [43] introduced the thermodynamic and variationally consistent deep energy method (DEM) and its variational neural operator extension (VINO) [44]. According to the works conducted by Samaniego et al. [43, 44], DEM employs the energy functional as the loss term, enabling physics-informed and efficient solutions via neural networks, whilst VINO integrates variational principles with neural operators to analytically evaluate the element-level responses, thereby eliminating the numerical quadrature errors. These methods have demonstrated superior convergence and generalization performance on refined meshes, achieving high accuracy and computational efficiency while remaining free from the constraints of mesh-based discretization. However, for the thermo-mechanical coupling problems, most studies still rely on the FEM-based analyses [45–48], as the corresponding results are more readily interpretable and applicable for the bridge engineers in practice. Therefore, the present study continues to employ the conventional finite element framework to ensure consistency with widely accepted engineering methodologies.

3.3.1 FE modeling

For comparison and validation, the temperature field of a steel box girder was simulated by using ABAQUS [49] to determine the spatial temperature distribution under solar radiation. As the longitudinal temperature distribution along the bridge is generally uniform, the thermal analysis of

the steel box girder can be simplified from a three-dimensional (3D) spatial to a 2D planar problem. However, to clearly visualize the spatial distribution of the temperature field, a 3D model of the 5 m long girder was developed in ABAQUS, with appropriate meshing applied, as illustrated in Fig. 10. Finite element (FE) modeling enables a more detailed assessment and prediction of the temperature field of the curved steel box girders under actual solar conditions.

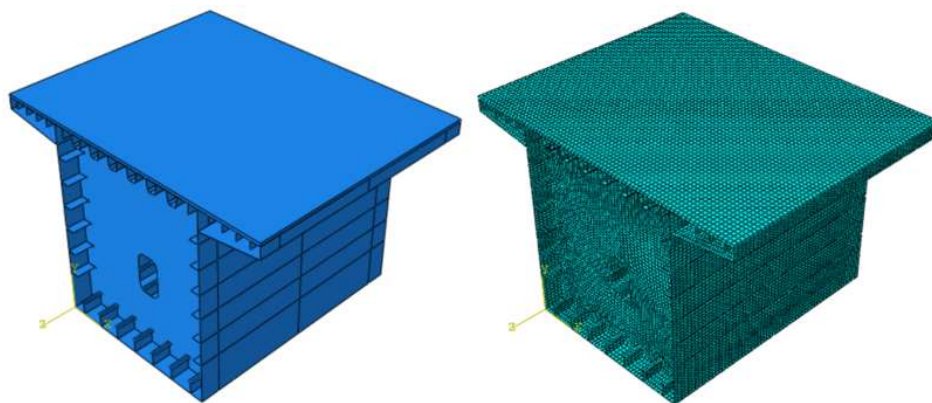


Fig. 10. FE model of curved steel box-girder developed using ABAQUS.

To ensure both the accuracy and computational efficiency of the numerical analyses, a systematic meshing strategy was adopted. The steel components, including the top flange, bottom plate, web, diaphragms, transverse stiffeners, and U-ribs of the girder were modeled using the four-node quadrilateral heat transfer shell elements (DS4), with a shell thickness of 20 mm. The UHPC deck was modeled using the 3D heat transfer solid elements (DC3D8). Tie constraints were defined between the steel box girder and UHPC deck to ensure full thermal and mechanical compatibility during analysis. In addition, the fully constrained connections were assigned between the steel components to replicate the heat conduction behavior under the welded conditions. A progressively refined mesh scheme was implemented—starting from a global reference size and applying local mesh refinement in regions exhibiting significant temperature gradient variations, particularly the upper flanges and webs. Element quality was rigorously controlled by maintaining an aspect ratio below 5 and a warping angle less than 10° , and by assigning at least two element layers through the 20 mm-thick steel plates to capture the through-thickness temperature variations. A meshing convergence study was conducted with the element sizes of 30, 25, 20, 15, and 10 mm. The results indicated that further refinement from 15 to 10 mm led to negligible differences in the thermal response (e.g., stress and deformation). Thus, a global mesh size of 15 mm was ultimately selected, in order to balance the accurate capture of temperature gradients and structural responses with computational efficiency.

During the numerical simulations, the temperature distribution of the girder under solar radiation

on 9 July 2022 was simulated. A transient heat transfer analysis step was established with a total duration of 43200 s to simulate the considered 24 h temperature variation on 9 July 2022. A fixed time increment of 1,800 s per analysis step was used, and a full Newton-Raphson iterative scheme was employed to address the nonlinear effects of radiative heat transfer. The temperature tolerance was set to 10^{-3} to ensure the numerical convergence. Moreover, based on the theoretical formulations of solar radiation, convective heat transfer, and thermal radiation presented in this study (as introduced in the following sub-sections), the surface-specific thermal boundary conditions were defined in the interaction module. A time-dependent ambient temperature profile was created using the Amplitude function available in ABAQUS and simultaneously applied to both convective and radiative boundary conditions to simulate the diurnal atmospheric temperature fluctuations. Solar heat fluxes computed from the theoretical equations were applied as surface loads in the load module. Non-symmetric matrix storage was activated to handle radiation-related nonlinearity. Field outputs included nodal temperature distributions, while history outputs recorded the temperature-time histories at key monitoring points. During the numerical simulations, the convergence parameters could be closely monitored to ensure the numerical stability.

Table 1. Material properties of the steel and UHPC.

Parameter (units)	Steel	UHPC
Young's modulus (MPa)	2.06×10^{11}	5.15×10^{11}
Poisson ratio	0.3	0.2
Density (kg/m^3)	7854	2360
Coefficient of linear thermal expansion ($^{\circ}\text{C}^{-1}$)	1.2×10^{-5}	1.0×10^{-5}
Specific heat capacity ($\text{J} \cdot (\text{kg} \cdot ^{\circ}\text{C})^{-1}$)	460	960
Emissivity	0.6	0.9

As the thermal field varied continuously with time, transient heat transfer analysis was adopted for the numerical simulations. The detailed configuration of the girder segment model is shown in Fig. 10, and the material parameters for the steel and concrete materials are summarized in Table 1. Finally, the temperature validation points corresponding to the field measurement locations were extracted from the FE model. The simulated temperature-time curves were compared against the measured data, achieving a temperature deviation within 2 °C and a correlation coefficient above 0.95 for temperature gradient profiles; and thereby confirming the accuracy and reliability of the developed models.

3.3.2 Applied thermal boundary conditions

As noted above, determining the temperature field in a steel box girder can be simplified to a 2D transient heat conduction problem. Under solar exposure, the boundary conditions (BCs) of the

girder involve heat exchange between the girder surface and the surrounding environments, primarily consisting of three mechanisms: solar radiation, air convection, and thermal radiation, as illustrated in Fig. 11. These mechanisms are introduced in the following sub-sections.

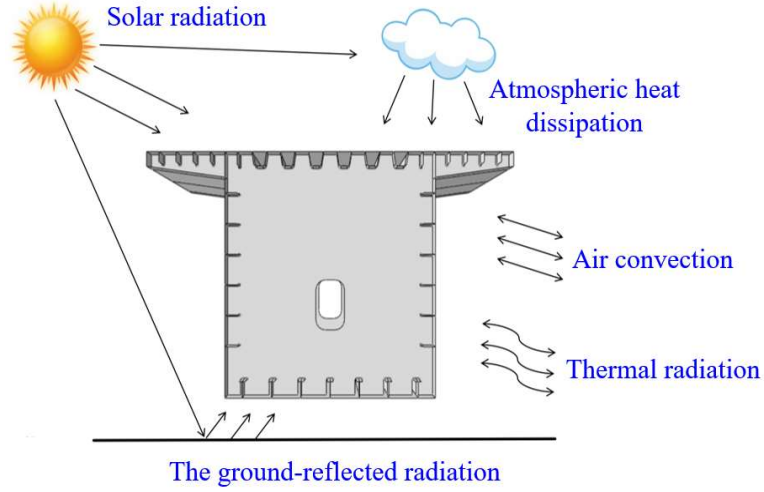


Fig. 11. Schematic diagram of heat exchange between a steel box girder and surrounding environment.

(1) Solar radiation

Due to the continuous variation in the solar incidence angle, different surfaces of a curved steel box girder receive varying amounts of solar radiation. Solar radiation plays a critical role in accurately determining the temperature distribution of steel box girders. Kim et al. [50] provided a detailed discussion on the specific influence of solar radiation on bridges, categorizing solar radiation into three components: direct solar radiation, diffuse solar radiation, and ground-reflected radiation. A simplified formula for the direct solar radiation (I_m) received by the box girder can be expressed through Eq. (3) below [50, 51].

$$I_m = I_0 \cdot \rho_l^m \quad (3)$$

$$\rho_l = 0.9^{t_u \cdot k_a} \quad (4)$$

$$t_u = A_u - B_u \cdot \cos\left(\frac{360^\circ \cdot N}{365}\right) \quad (5)$$

where I_0 denotes the solar constant; m is the optical air mass; k_a is the relative atmospheric pressure; A_u and B_u are the defined empirical parameters; and N is the sequential day number counted from January 1 in one year. Besides, for a plane (i.e., structural surface) with a solar incidence angle of θ , the intensity of direct solar radiation (I_θ) received by the plane can be calculated as:

$$I_\theta = I_m \cdot \cos \theta \quad (6)$$

In addition, the simplified formula for the diffuse solar radiation (I_d) received by the steel box girder can be represented as:

$$I_d = I_m \cdot (1 - 1.13 \cdot K_T) \quad (7)$$

where K_T denotes the daily clear-sky index. Besides, the scattering intensity on an arbitrary inclined surface can be expressed as:

$$I_\beta = \frac{I_d \cdot (1 + \cos \beta)}{2} \quad (8)$$

where β is the angle between the ground and box girder surface. Moreover, the ground-reflected radiation (I_f) can be determined from:

$$I_f = \rho^* \cdot (I_d + I_m) \cdot \frac{(1 - \cos \beta)}{2} \quad (9)$$

where ρ^* is the ground reflection coefficient, which can be set as 0.2. Thus, the total solar radiation (I) can be determined as:

$$I = I_\theta + I_\beta + I_f = I_m \cdot \cos \theta + \frac{I_d \cdot (1 + \cos \beta)}{2} + \rho^* \cdot (I_d + I_m) \cdot \frac{(1 - \cos \beta)}{2} \quad (10)$$

The key factor determining the magnitude of solar radiation absorbed by the surface of a curved steel box girder is the solar absorption coefficient of the girder surface (α). Thus, the total solar radiation intensity absorbed by the surface of a steel box girder at any inclination angle can be expressed as:

$$q_s = \alpha \cdot I = \alpha \cdot \left[I_m \cdot \cos \theta + \frac{I_d \cdot (1 + \cos \beta)}{2} + \rho^* \cdot (I_d + I_m) \cdot \frac{(1 - \cos \beta)}{2} \right] \quad (11)$$

(2) Air convection

Convection refers to the heat transfer process caused by the interaction between hot and cold fluids, which can be categorized into natural and forced convections. The natural convection primarily arises from the density differences between hot and cold fluids, which typically occurs within the steel box girder chambers under conditions of no internal air flow. Forced convection, on the other hand, generally refers to heat exchange between the surface of the girder and the surrounding air due to air movement. The curved steel box girder surface exchanges heat with the atmospheric environment. According to Newton's law of cooling, the heat flux density of air convection (q_c) can be expressed using Eq. (12) below [51].

$$q_c = h_c \cdot (T - T_a) \quad (12)$$

where T and T_a are the surface temperature of curved steel box girder and ambient air temperature, respectively; and h_c denotes the convective heat transfer coefficient, which depends on the local wind speed (v) and can be calculated by Eq. (13) below.

$$h_c = 4v + 5.8 \quad (13)$$

(3) Thermal radiation

Under environmental radiation, the radiative heat transfer occurs on the surface of a curved steel box girder, with the heat flux density (q_a) given by [51]:

$$q_a = \varepsilon \cdot C_0 \cdot \left[(273.15 + T)^4 - (273.15 + T_a)^4 \right] \quad (14)$$

where ε is the radiation heat transfer coefficient of the girder surface, which is taken as 0.6; and C_0 is the Stefan-Boltzmann constant, which is taken as $5.67 \times 10^{-8} \text{ W}/(\text{m}^2 \cdot \text{K}^4)^{-1}$. The expression for determining ε can be derived from the Stefan-Boltzmann law as follows:

$$h_r = \frac{\varepsilon \cdot C_0 \cdot \left[(273.15 + T)^4 - (273.15 + T_a)^4 \right]}{T - T_a} \quad (15)$$

In addition, the ambient air temperature (T_a) at the bridge site during the investigated periods can be determined from Eq. (16) below [51].

$$T_a = 0.5 \cdot (T_{\max} + T_{\min}) + 0.5 \cdot (T_{\max} - T_{\min}) \cdot \sin \frac{(t - t_0) \pi}{12} \quad (16)$$

where $T_{\max}$ and $T_{\min}$ are the daily maximum and minimum ambient temperatures recorded at the bridge site, respectively; where t is the local time in hours (h); and t_0 denotes the initial time at which the daily minimum temperature occurs. Based on the above, the thermal BCs at the external surface of the girder can be applied during the numerical simulations using ABAQUS. The total heat exchange at the external surface consists of three components: solar radiation (q_s), air convection (q_c), and thermal radiation (q_a), which are all defined positive heat flux as the flow from the external environment into the structure and negative as the reverse. The thermal BCs can be expressed as [52]:

$$\lambda \cdot \left(\frac{\partial T}{\partial n} \right)_{\Gamma} = q_s + q_c + q_a \quad (17)$$

where λ is the thermal conductivity, in $\text{W}/(\text{m} \cdot \text{K})$.

Based on the above-introduced theoretical formulations of radiations on the curved steel box girder structure, the surface-specific thermal boundary conditions could be defined in the interaction module available in ABAQUS. The top flange was subjected to both direct and diffuse solar radiation, while the web surfaces were treated according to their orientations and shadowed regions. The bottom plate considered only reflected radiation from the ground. All exterior surfaces were assigned convective heat transfer coefficients and radiative exchange conditions defined by

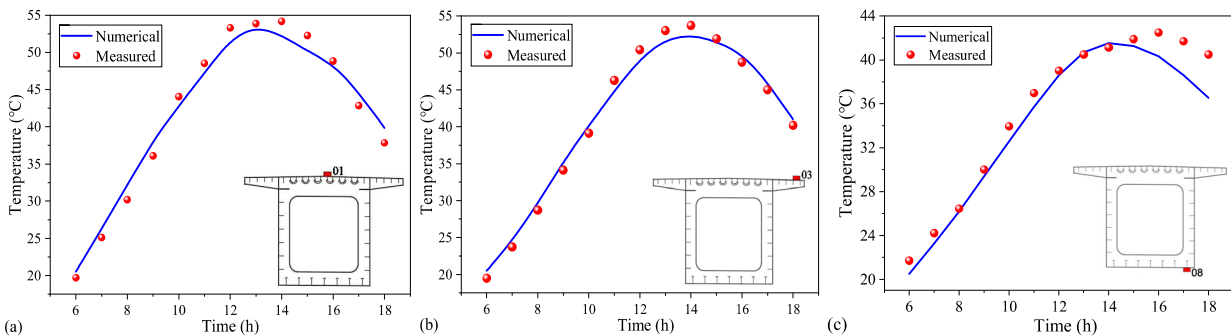
empirical correlations. In addition, information of the relevant parameters used in the theoretical formulations of radiation is summarized in Table 2.

Table 2. Information of the relevant parameters used in the theoretical formulations of radiations.

Radiation type	Description	Value or formulation	References
Solar radiation	Solar constant, I_0	1367 (W/m ²)	[50]
	Optical air mass, m	$m = k_a/\sin\beta$	[50]
	Empirical parameter, A_u	$A_u = 0.4237 - 0.00821 \cdot (6.0 - H)^2$	[51]
	Empirical parameter, B_u	$A_u = 0.5055 + 0.00595 \cdot (6.5 - H)^2$	[51]
	Ground reflection coefficient, ρ^*	0.2	[51]
Air convection	Solar absorption coefficient, α	0.6 for steel; 0.9 for UHPC	Present study
	Convective heat transfer coefficient, h_c	Eq. (13) in present study	[51]
	Radiation heat transfer coefficient, ε	0.6 for steel; 0.9 for UHPC	Present study
Thermal radiation	Ambient air temperature, T_a	Eq. (16) in present study	[51]

3.3.3 Comparison between simulations and measurements

To validate the accuracy of the numerical simulations, representative temperature monitoring points were selected on the top flange, bottom plate, and east web of the girder. Fig. 12 shows comparisons of the numerical and measured results at different field measuring points. The simulated temperatures at the two measurement points on the top flange deviated most significantly from the measured values at 14: 00, with differences of 2 and 1.5 °C, respectively. However, the overall temperature-time profiles showed good agreement in terms of both magnitude and trends. For the bottom plate, the peak temperature is predicted numerically at 15: 00, while the measured peak occurred at 17: 00, indicating a temporal discrepancy but consistent general behavior. On the east web, both the simulated and measured temperatures reached their maximum at 16: 00, with a maximum difference of less than 2 °C. These results indicate that the thermal boundary conditions employed in the FE model could effectively reflect the actual field conditions. Overall, the numerical simulations demonstrated good agreement with field monitoring data, confirming the validity and reliability of the modeling approach.



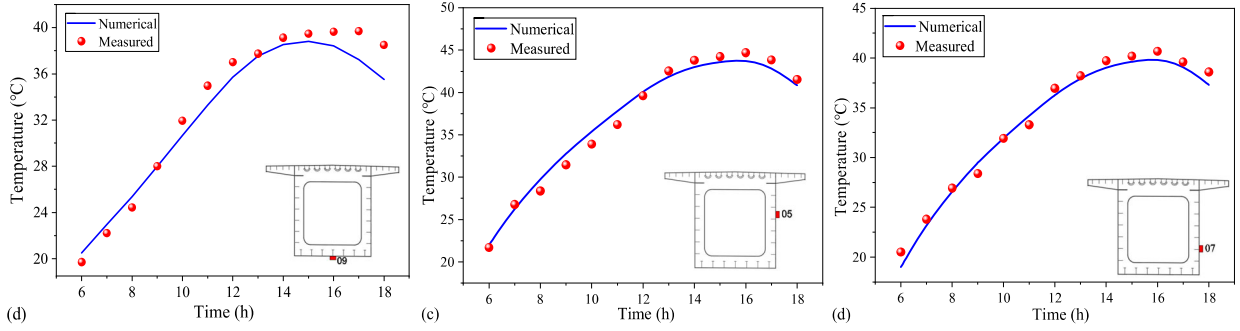


Fig. 12. Numerical versus measured temperatures: (a) Point 1, (b) Point 3, (c) Point 5, and (d) Point 7.

It was also found that the vertical temperature difference across the web and the transverse temperature difference across the top flange reached their peak values at 14: 00 and 15: 00, respectively. To evaluate the temperature gradients, both the vertical and transverse temperature distributions were determined at the monitoring points on both the top and bottom plates. These results were then compared with the measured temperature gradients, as shown in Fig. 13. Under the most critical temperature conditions, the simulated temperature differences on the top flange and the web closely matched the measured values, with a maximum deviation of less than 2°C. This confirms the effectiveness of the proposed method in capturing the temperature fields in the curved steel box girders.

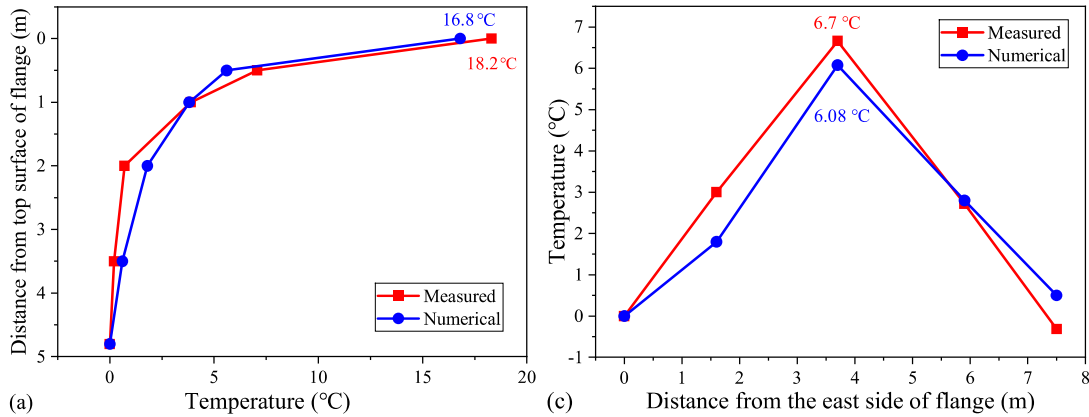


Fig. 13. Comparison of numerical and measured temperature gradients: (a) vertical and (b) transverse.

3.4 Regional differences in temperature gradients

To consider the effects of regional variability in climatic conditions on the temperature gradients, this study examines the temperature differentials in curved steel box girder bridges situated in diverse climatic zones across China. Building upon the validated numerical simulation approach, the FE models were employed to establish the correlations between the regional meteorological parameters and temperature differences in a set of curved steel box girder bridges. 12 major cities were selected as case studies. The historical meteorological data, including the daily maximum (T_{max}) and corresponding minimum (T_{min}) ambient temperatures over a 5-year period of 2019~2024

were obtained from the National Meteorological Information Center of China [53] to compute the daily maximum temperature variations (T_v) in each city. The proposed and validated temperature gradient models (Eqs. (1) and (2)) were then used to derive the vertical and transverse temperature differences, as summarized in Table 3. Thus, both the transverse and vertical temperature gradient models of the curved steel box girder bridges situated in the considered 12 major cities could be developed and their effectiveness could be verified by using the suggested numerical simulation approach, if the field monitoring data is available. Moreover, during the determination of the temperature differences in several representative cities in diverse climatic zones across China, Table A1 given in the Appendix summarizes the considered thermal boundary condition parameters for the computational process of the curved steel box girder bridges in these considered cities.

Table 3. Recorded meteorological data and calculated temperature differences in representative cities.

Region	City	Meteorological data			Date	Calculated vertical temperature difference (°C)	Calculated transverse temperature difference (°C)	Parameter α
		T_{max}	T_{min}	T_v				
Northeastern China	Harbin	26.3	7.8	18.5	2022.9.5	22.5	3	2.44
	Shenyang	32.75	13.27	19.48	2023.9.7	21.6	4.5	3.2
Northwestern China	Xi'an	31.1	12.2	18.9	2023.5.20	23.0	10.6	2.9
	Lanzhou	31.71	14.08	17.63	2023.9.6	14.33	3	1.6
Eastern China	Jinan	36.52	18.03	18.49	2023.7.9	18.2	6.7	1.4
	Hangzhou	28.96	11.12	17.84	2022.8.19	23.6	11.1	2.3
Central China	Wuhan	34.89	21.67	13.22	2023.9.19	14.9	6.4	1.3
	Changsha	36.08	20.94	15.14	2022.8.23	12.8	5.9	1.5
Southwestern China	Chengdu	38.3	23.62	14.68	2022.8.23	14.3	5.8	2.2
	Guiyang	30.21	16.03	14.18	2022.9.6	14.9	6.5	1.4
Southern China	Guangzhou	34.64	24.06	10.58	2023.5.30	21.2	6.3	3.2
	Nanning	37.66	25.54	12.12	2023.5.6	21.8	5.4	3.5

As shown in Table 3, both the transverse and vertical temperature gradient models exhibited significant regional variability across different cities. For example, in Northeastern and Southern China, large diurnal temperature ranges and intense solar radiation led to substantial vertical temperature differences between the top and bottom plates. In Eastern China, Hangzhou experienced greater vertical gradients than Jinan due to its higher humidity levels. In contrast, Central and Southwestern China exhibited the smallest vertical gradients, due to the combined effects of elevated humidity and strong solar scattering. For the transverse gradients, Xi'an and Hangzhou showed a "hot center-cool edges" pattern on the top flange during summer. In Lanzhou and Shenyang, however, the top flange temperatures were more uniform, which is attributed to strong convective heat transfer driven by frequent winds and relatively stable solar radiation. In most other cities, the transverse temperature differences ranged from 5.4 to 6.7 °C and showed a

positive correlation with local solar radiation intensity. These regional differences suggest that, in design practice, enhanced vertical flexural capacity should be considered in areas with high vertical temperature gradients, while optimization of lateral thermal stress in the top flange is necessary in those regions with pronounced transverse gradients. These findings provide region-specific temperature loading parameters for the design of curved steel box girder bridges across various climatic regions.

4. Thermal-induced structural response

The vertical and transverse temperature gradients derived under the most critical conditions identified in Section 2 were applied in FE models. A dataset of thermal response of the case-study bridge was generated. The resulting stress and deformation behavior under the individual and coupled vertical and transverse temperature gradients were assessed. The findings provide a theoretical basis for the design, construction, and operation of curved steel box girder bridges.

4.1 FE modeling of the bridge

A FE model, as depicted in Fig. 14, was developed using ABAQUS [46] to investigate the temperature response characteristics of the bridges. The case-study bridge lies along a circular arc with a radius of 950 m, and the spacing of diaphragms is set to 2.5 m. The model was configured using the Static-General analysis step, with realistic geometric and thermal boundary conditions. Temperature gradient loads were introduced through the predefined fields. The mesh size was kept consistent with that used in the preceding thermal analysis of the box girder (i.e., 15 mm), as introduced in Section 3.3. The four-node quadrilateral heat transfer shell elements (DS4) (with a shell thickness of 20 mm) were adopted to model the top and bottom plates, webs, U-ribs, and diaphragms, while the UHPC deck was modeled using 3D heat transfer solid elements (DC3D4). The detailed FE modeling procedure of the bridge was similar to that of the box girder (as given in Section 3.3), which is not introduced in detail herein due to space limitation. In addition, the detailed configuration of the bridge model is shown in Fig. 14. The material parameters for the steel and concrete materials are as provided in Table 1.

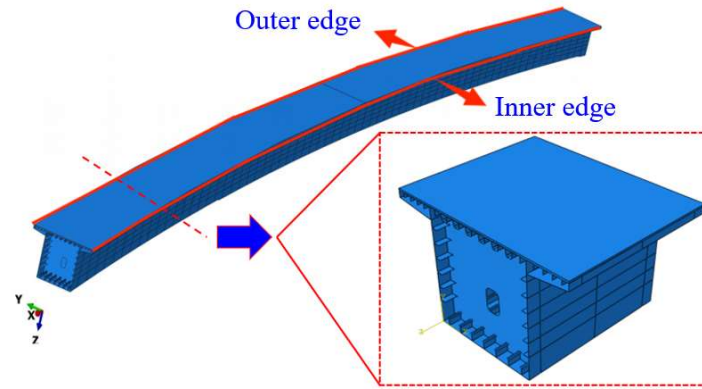


Fig. 14. Developed FE model of curved steel box girder bridge using ABAQUS.

4.2 Behavioural characteristics

4.2.1 Response under vertical temperature gradient

A curvilinear coordinate system is established for the girder, where the x -, y -, and z -axes correspond to the longitudinal, transverse, and vertical directions of the bridge, respectively. A vertical temperature gradient is applied along the z -axis using a predefined field function in ABAQUS. Unlike their straight counterparts, curved steel box girder bridges exhibit bending-torsion coupling effects under thermal loads, which can result in more pronounced transverse and longitudinal response. To investigate the thermally induced response of the girder under vertical temperature gradients, both the stresses and deformations were assessed.

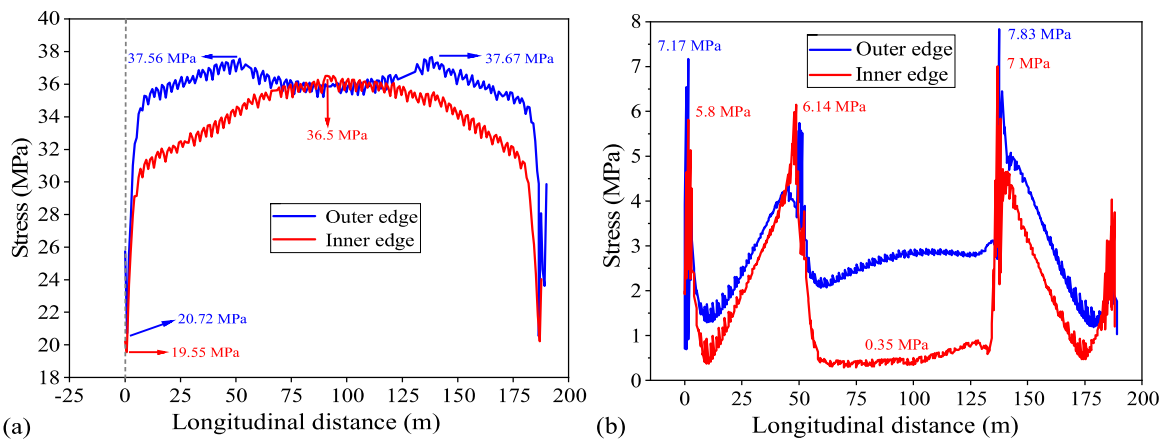


Fig. 15. Stress distributions under a vertical temperature gradient: (a) top flange and (b) bottom plate.

Fig. 15 illustrates the effective stresses induced along the outer and inner edges of the top flange and bottom plate under a vertical temperature gradient. As shown in Fig. 15(a), a symmetrical stress distribution was observed along the longitudinal direction between the inner and outer edges of the top flange. However, distinct differences exist in the location and variation patterns of the stress extrema. The minimum stresses at both the outer and inner edges of the top flange occurred at the side spans, with values of 20.72 and 19.55 MPa, respectively. The maximum stress on the outer edge appeared near the mid-span support (37.67 MPa), while that on the inner edge occurred at the

mid-span center (36.5 MPa). This stress disparity is attributed to the differential solar exposure: the inner edge, being partially shaded, underwent a lower temperature rise than the fully exposed outer edge, leading to reduced thermal expansion and thus lower stress levels. Besides, as observed from Fig. 15(b), a distinct stress discontinuity is observed near the bearing locations. The maximum tensile stress in the bottom plate occurred at the outer edge adjacent to the fixed bearing, reaching 7.83 MPa. This elevated stress is attributed to the significant restraint imposed by the fixed bearing, which amplifies the stress concentration in this region. Under the vertical temperature gradient, the bottom plate reached a peak stress of 7.83 MPa, while the top flange reached a peak stress of 37.67 MPa. This pronounced difference arises from the non-uniform thermal distribution, with the vertical temperature gradient being greatest at the top surface and decreasing downward, thus inducing larger thermal stresses in the upper regions of the girder.

In addition, as shown in Fig. 15, notable stress variations were observed at the bearing locations, compared to other regions. To further investigate this phenomenon, Fig. 16 presents the stress contours of the transverse diaphragms at Supports #151 and #152, as indicated in Fig. 1. Under a vertical temperature gradient, the diaphragms exhibited peak stresses of 43.8 and 33.4 MPa at the mid-span and side-span supports, respectively. These maximum stresses were concentrated at the cantilevered ends. This can be attributed to the fact that the upper portion of the girder had greater temperature differentials under a vertical gradient, resulting in higher thermal stresses.

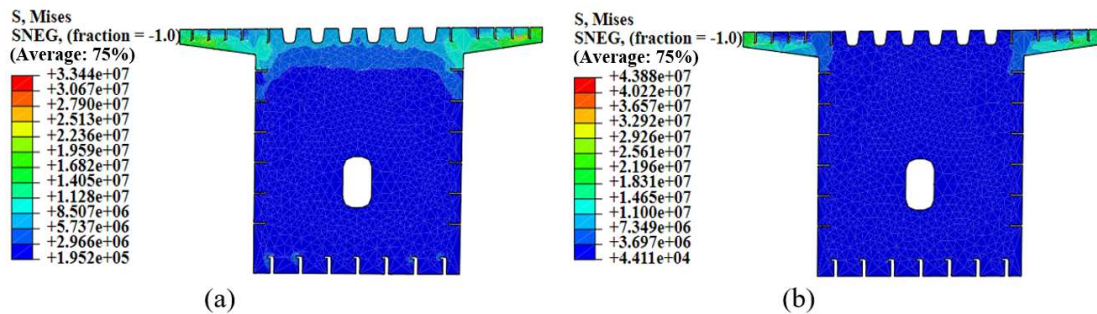


Fig. 16. Stress contours of cross-section of girder under a vertical temperature gradient: (a) Support #151 (side span) and (b) Support #152 (mid-span).

Fig. 17 shows the longitudinal displacement profiles at the inner and outer edges of the bridge under a vertical temperature gradient. The longitudinal displacements exhibited an antisymmetric distribution. Thermal expansion led the girder to elongate from the midspan toward both ends. The maximum longitudinal displacements of the top and bottom plates occurred in the side spans, reaching 1.38 and 0.76 mm, respectively. The displacement of the top flange ranged from -1.38 to 1.3 mm, and that of the bottom plate varied from -0.76 to 0.68 mm. Due to the higher solar radiation exposure, the top flange exhibited greater thermal deformation compared to the bottom plate. The

longitudinal deformation at the outer edge also exceeded that at the inner edge, mainly due to the relatively high thermal expansion coefficient of steel and the differential solar heating. As the outer surface of the girder is more directly exposed to sunlight, it exhibits a more rapid temperature rise compared to the inner surface, leading to a greater thermal expansion along the outer edge.

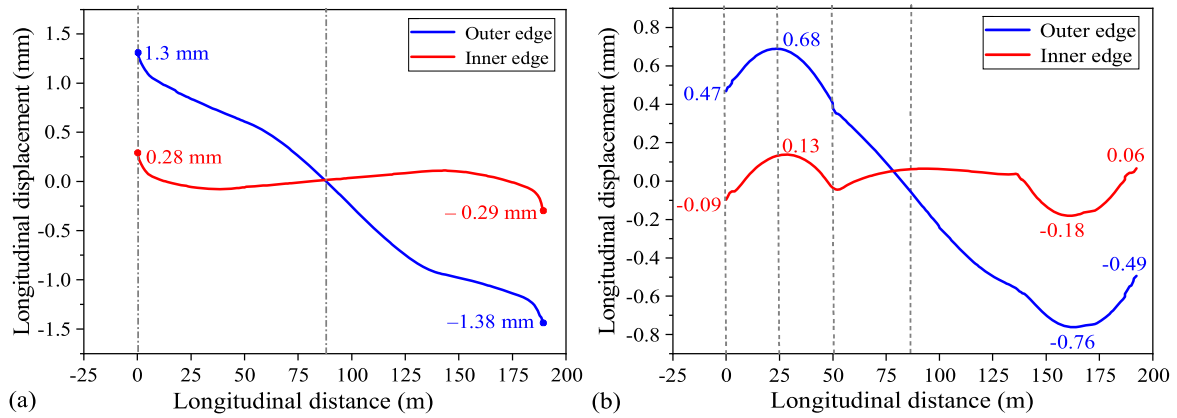


Fig. 17. Longitudinal displacement distributions of the bridge under a vertical temperature gradient: (a) top flange and (b) bottom plate.

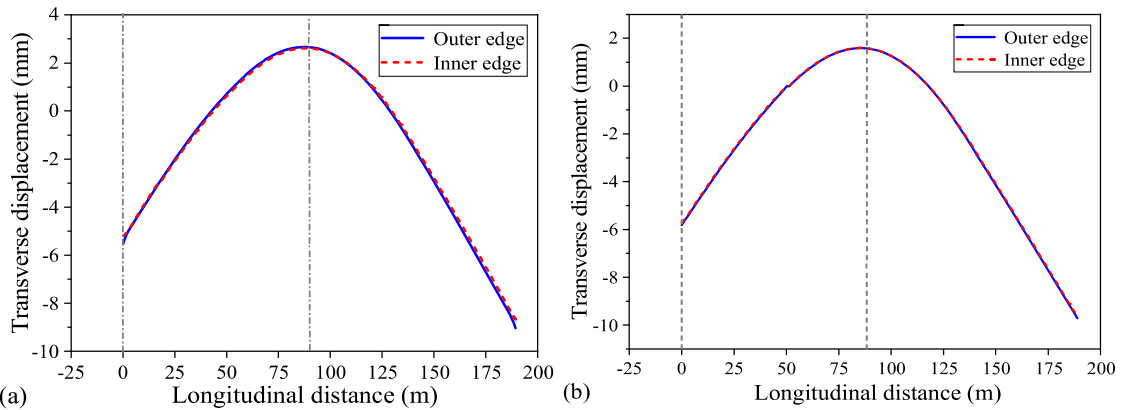


Fig. 18. Transverse displacement distributions of the bridge under a vertical temperature gradient: (a) top flange and (b) bottom plate.

Fig. 18 illustrates the transverse displacement variations of the bridge under a vertical temperature gradient, indicating that: (i) the transverse displacements at the inner and outer edges were nearly identical, with their curves almost completely overlapping. Both edges exhibited a parabolic distribution, with larger displacements at the girder ends and smaller values at midspan; and (ii) when subjected to the maximum vertical temperature differential, the girder experienced a lateral offset. Inward displacements were observed at the girder ends, reaching a maximum of 9.6 mm, while the outward displacements occurred near the midspan of the main span, with a peak value of 2.66 mm. The maximum transverse displacement in the side spans was about 3.6 times greater than that in the main span. This discrepancy is primarily attributed to the lateral restraint provided by Support #153, which limits transverse movement at the midspan. In contrast, supports #151 and #154 are equipped with sliding bearings, allowing thermal expansion and resulting in the

larger transverse displacements in side spans.

Fig. 19 illustrates the vertical displacement variations along the girder under a vertical temperature gradient, indicating that: (i) the bridge exhibited a wavelike displacement profile along the longitudinal direction at both the inner and outer edges of the top and bottom plates; both had downward displacements at mid-span, with a maximum value of 0.62 mm, while upward displacements occurred near the side spans, reaching up to 1.09 mm. Vertical deformations near the side spans were relatively small due to the presence of supports that constrain vertical movements. (ii) The outer edge experienced larger vertical displacements than the inner edge, leading to a displacement differential across the cross-section. Such an uneven deformation induces torsional effects, leading to an overall twisting of the girder toward the outer curve.

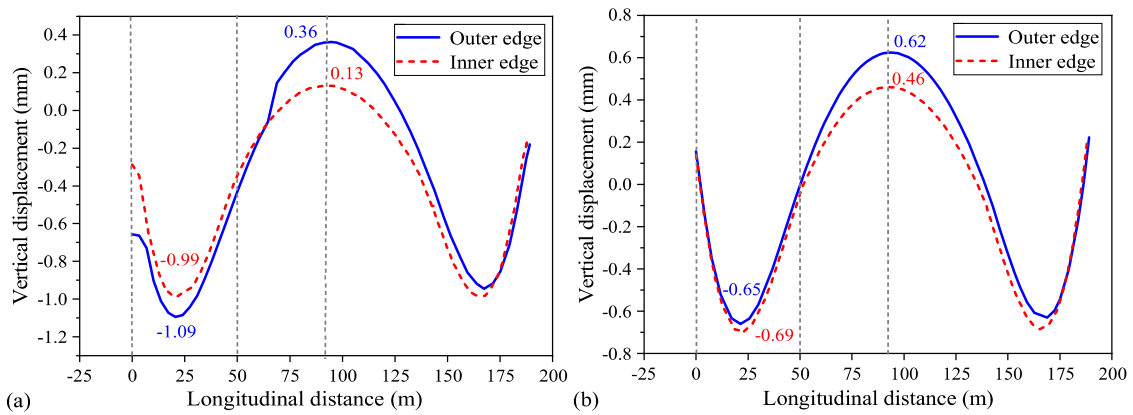


Fig. 19. Vertical displacement distributions of the bridge under a vertical temperature gradient: (a) top flange and (b) bottom plate.

4.2.2 Response under transverse temperature gradient

Fig. 20 compares the induced effective stress response under a transverse temperature gradient. As shown in Fig. 20(a), the stress along the midline of the top flange ranged from 6.03 to 14.9 MPa, whereas the stress at the outer and inner edges varied from 1.04 to 3.09 MPa and 0.28 to 3.15 MPa, respectively. The midline stress is about 4.82 and 4.73 times greater than that at the inner and outer edges, respectively, indicating significant stress concentration at the center of the top flange. This can be attributed to the higher temperature at the midline compared to the edges under the maximum transverse temperature difference, where thermal expansion and contraction effects are more pronounced, resulting in higher stress concentration in the central region of the top flange.

As shown in Fig. 20(b), the stress distribution under a transverse gradient is similar to that under a vertical gradient, both exhibiting abrupt stress changes near the supports. The maximum stresses at the inner and outer edges of the bottom plate occurred near the fixed support, reaching 7.47 and 4.83 MPa, respectively. In contrast, the maximum stresses at the same locations were 7.0 and 7.83

MPa, indicating that the bottom plate experienced slightly higher stress under a vertical gradient compared to the transverse case. Fig. 21 depicts the stress contours of the transverse diaphragms located at the #151 and #152 supports under a transverse temperature gradient. As shown in Fig. 21, the maximum stresses at the side- and center-span diaphragms reached 22.9 and 22.5 MPa, respectively. These peak stresses were concentrated at the U-shaped notch, primarily due to the intensified thermal effects in the upper portion of the bridge induced by transverse thermal gradients.

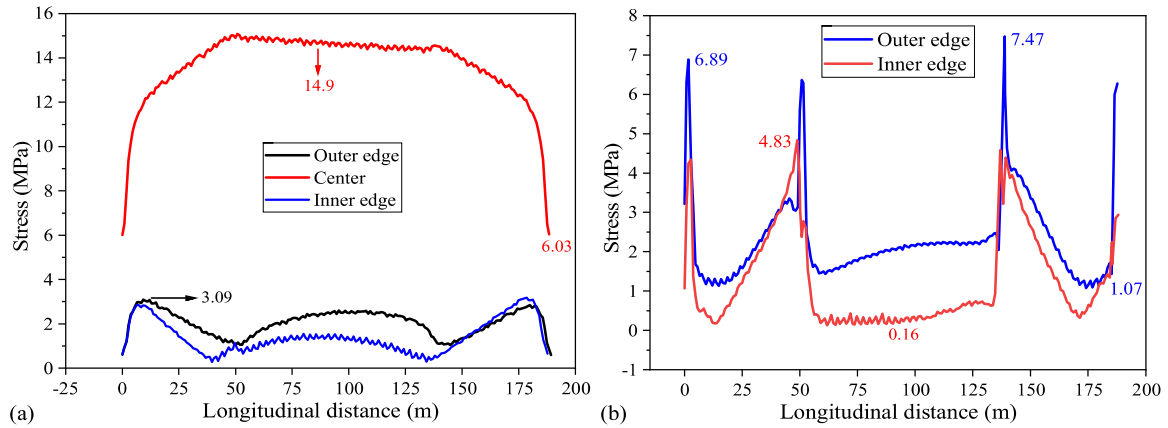


Fig. 20. Stress distributions under transverse temperature gradient: (a) top flange and (b) bottom plate.

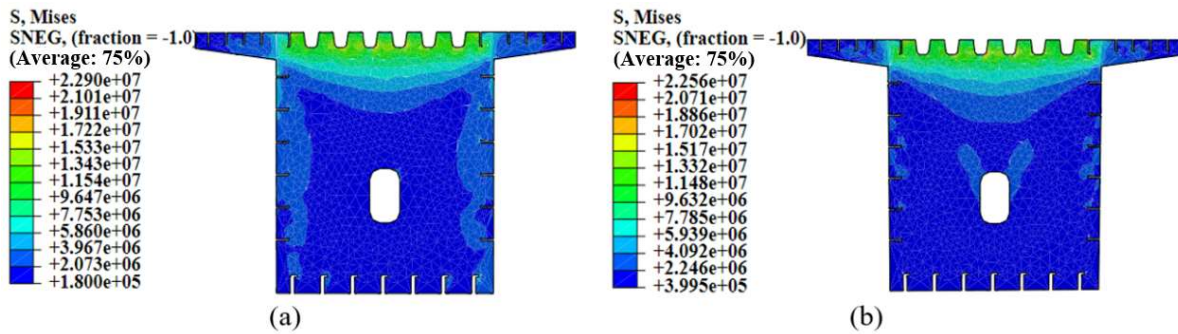


Fig. 21. Stress contours of the cross-sections of the bridge under the transverse temperature gradient: (a) at Support #151 (side span) and (b) at Support #152 (center span).

Fig. 22 depicts the longitudinal displacement profiles of the bridge under a transverse temperature gradient. It is observed that the longitudinal displacements of the top and bottom plates exhibited an antisymmetric distribution. The maximum longitudinal displacements occurred in side spans, which was 1.0 mm at the top flange and 0.57 mm at the bottom plate. The peak displacement under a vertical temperature gradient is 1.38 times greater than that under transverse gradient. Such discrepancy arises from the larger vertical temperature differential, which exerts a more pronounced effect on longitudinal expansion. The relatively smaller influence of a transverse temperature gradient is due to its lower magnitude, even under the peak solar radiation. However, the transverse gradient induced differential thermal expansion across the width of the top and

bottom plates: longitudinal elongation increased from the inner edge toward the centerline and reached a maximum at the outer edge.

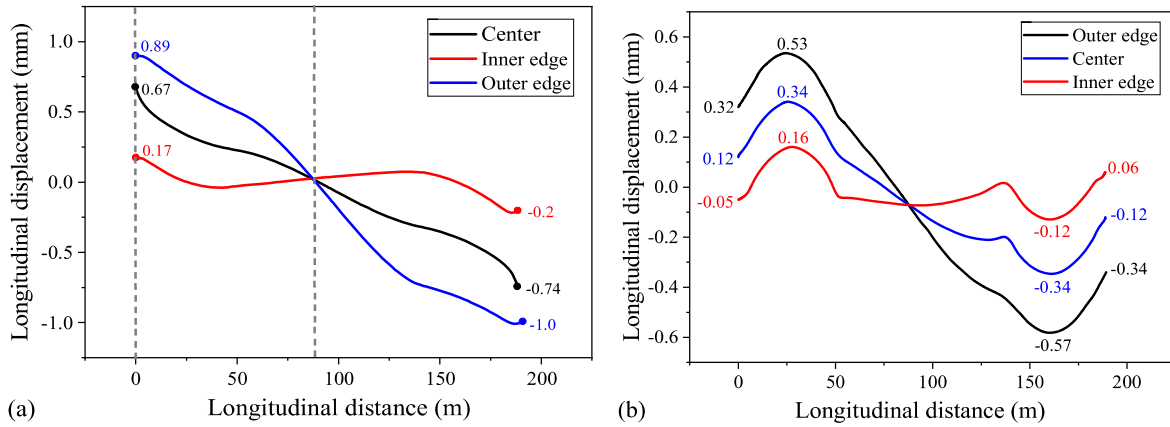


Fig. 22. Longitudinal displacement distributions of the bridge under the transverse temperature gradient: (a) top flange and (b) bottom plate.

Fig. 23 shows the transverse displacement profiles of the girder under the transverse temperature gradient. It is observed that the transverse displacements at the inner and outer edges of the bottom plate are nearly identical, indicating negligible differential movement. In contrast, the top flange showed a maximum displacement difference of 0.34 mm between its inner and outer edges. Under the peak transverse temperature difference, the girder experienced inward lateral displacements of up to 5.7 mm in the side span and outward displacements of up to 3.14 mm at the mid-span. Besides, the transverse displacement induced by vertical temperature gradient was approximately 1.68 times that caused by the transverse gradient.

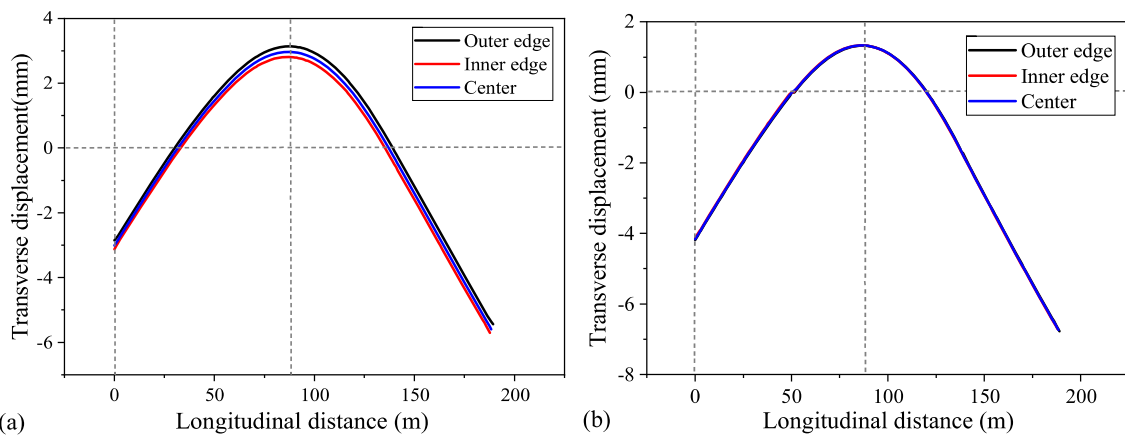


Fig. 23. Transverse displacement distributions of the bridge under the vertical temperature gradient: (a) top flange and (b) bottom plate.

Fig. 24 illustrates the vertical displacement variations of the bridge under the transverse temperature gradient. The vertical displacement patterns of the inner and outer edges of both the top and bottom plates under transverse temperature gradient are consistent with those observed under the vertical gradient. Both exhibited a wave-like distribution and, due to the constraints

imposed by the supports, the vertical displacements at the side spans were relatively small. A downward vertical deformation occurred near the mid-span with a peak value of 0.9 mm, while an upward deformation occurred near the side span with a peak value of 0.68 mm. The magnitude of downward displacement exceeds that of the upward displacement. The maximum vertical displacement under vertical temperature gradients was 1.14 mm, which is approximately 1.26 times that induced by the transverse temperature gradient. Under the transverse temperature gradient, the outer edge of the girder experiences greater thermal exposure, leading to a more pronounced thermal expansion compared to the inner edge, and thus showing a larger vertical displacement.

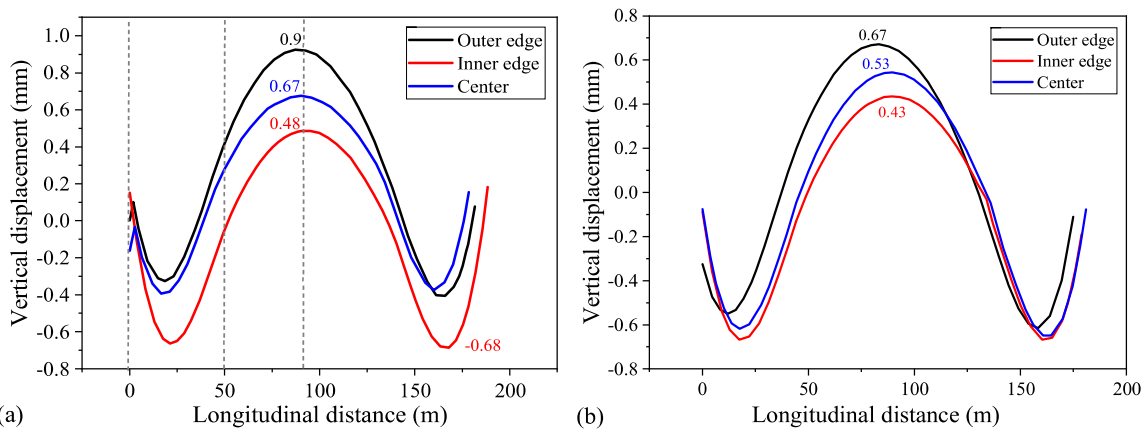


Fig. 24. Vertical displacement distributions of the bridge under the transverse temperature gradient: (a) top flange and (b) bottom plate.

4.2.3 Response under coupled vertical and transverse gradients

Fig. 25 compares the effective stress response of the bridge under coupled vertical and transverse temperature gradients. As shown in Fig. 25(a), the maximum longitudinal stresses along the centerline, outer edge, and inner edge of top flange increased in the following order: centerline < outer edge < inner edge. The peak stress (38.4 MPa) occurred at the inner edge. The maximum stress induced by the coupled vertical and transverse temperature gradients is 1.02 times that caused by the vertical temperature gradient alone and 2.57 times that of the transverse temperature gradient alone. Also, as indicated in Fig. 25(b), the stress distribution pattern in the bottom plate under coupled vertical and transverse temperature gradients is consistent with those under the individual vertical and transverse gradients, with abrupt stress variations near the support regions. The maximum stress (17.99 MPa) occurred at the fixed support (#153), which is 2.29 times higher than the stress from the vertical gradient alone and 2.4 times higher than that from the transverse gradient alone. These findings indicate that the coupled vertical and transverse temperature gradients could induce significantly larger stress than under either of the single gradient components.

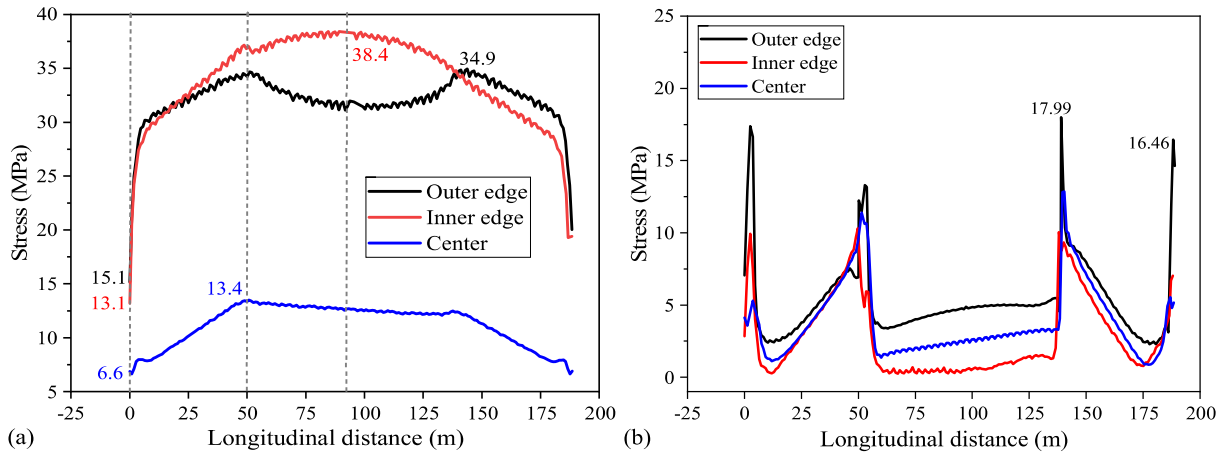


Fig. 25. Stress distributions of the bridge under coupled vertical and transverse temperature gradients: (a) top flange and (b) bottom plate.

Fig. 26 presents the stress contours of the transverse diaphragms at Supports #151 and #152 of the bridge under coupled vertical and transverse temperature gradients. As seen from Fig. 26, significant stress concentrations appeared near the flange plates and U-ribs at both the mid-span and side-span supports, with the peak stresses reaching 44.1 and 39.6 MPa, respectively. The maximum stress under coupled temperature gradients exceeds that under the vertical gradient alone by 0.6% and the transverse gradient alone by 11.3%. This can be attributed to the superimposed effects of temperature-induced stress fields, where vertical and transverse temperature gradients interact within the box girder, leading to more complex internal force distributions. To mitigate the associated risks, targeted reinforcement strategies should be incorporated into the structural design of curved steel box girder bridges under coupled vertical and transverse temperature gradients.

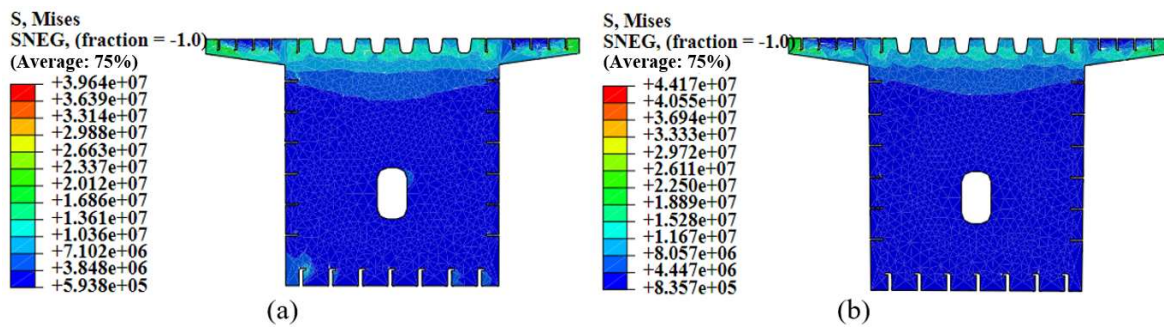


Fig. 26. Stress contours of the cross-sections of the bridge under coupled vertical and transverse temperature gradients: (a) Support #151 (side span) and (b) Support #152 (center span).

Fig. 27 shows the longitudinal displacement variations of the bridge under coupled vertical and transverse temperature gradients. The longitudinal displacement pattern of the girder is consistent with those induced by each individual gradient. The maximum longitudinal displacements occurred at the side span, with the top and bottom plates exhibiting peak values of 2.44 and 1.34 mm, respectively. The maximum longitudinal displacement under coupled vertical and transverse temperature gradients is 1.76 times that under vertical gradient alone and 2.44 times that under

transverse gradient alone. This indicates that the simultaneous action of vertical and transverse thermal gradients induces multidirectional thermal expansion, leading to significantly larger longitudinal displacements than those caused by a single thermal gradient.

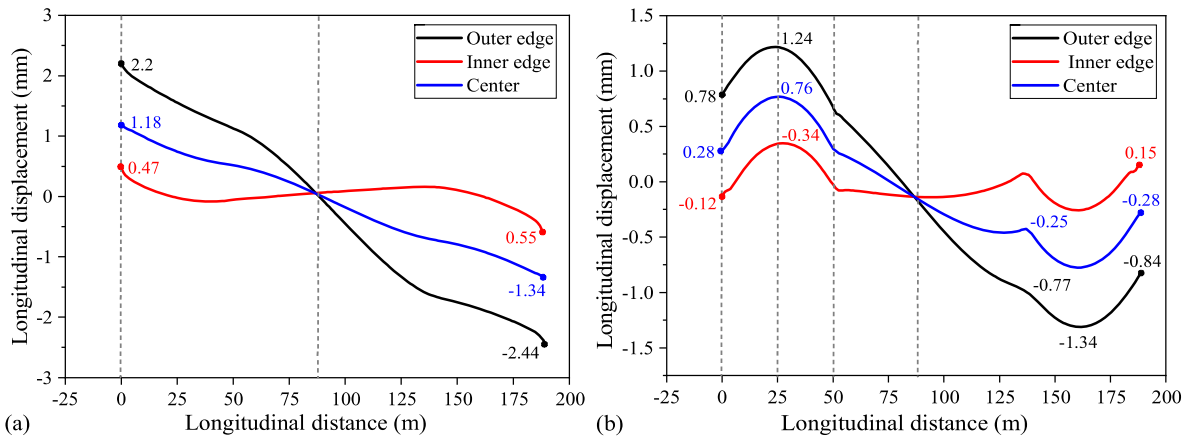


Fig. 27. Longitudinal displacement distributions of the bridge under coupled vertical and transverse temperature gradients: (a) top flange and (b) bottom plate.

The vertical displacement distributions of the bridge under coupled vertical and transverse temperature gradients are shown in Fig. 28. Under coupled temperature gradients, the vertical deformation pattern along the inner and outer edges of top and bottom plates is similar to those observed under individual temperature gradients. A downward vertical displacement occurred at the mid-span, with a maximum value of 1.47 mm, while an upward displacement of 1.52 mm appeared at the side span. The maximum displacement under coupled gradients is 1.33 times that induced by the vertical temperature gradient alone and 1.68 times that caused by the transverse temperature gradient alone. Thus, in practical design, to ensure structural safety and thermal adaptability, it is essential to consider the effects of temperature gradients and implement mitigation strategies, such as introducing appropriate camber or incorporating transverse diaphragms, to control the thermally induced vertical deformations of curved steel box girder bridges.

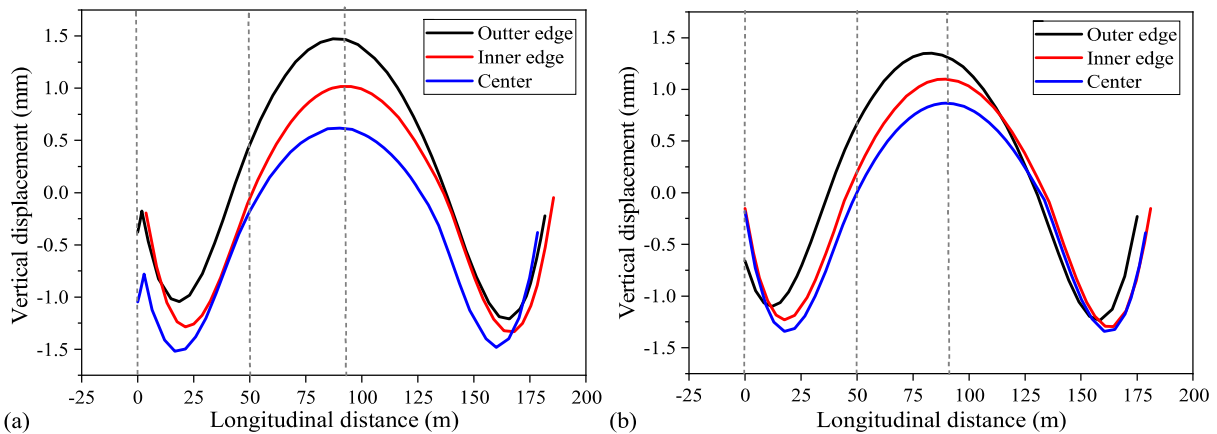


Fig. 28. Vertical displacement distributions of the bridge under coupled vertical and transverse temperature gradients: (a) top flange and (b) bottom plate.

4.3 Influence of curvature radius on the deformation, stress, and torsion behavior of the bridge

Due to the eccentricity of the centroid in curved steel box girder bridges, additional torsional moments may occur compared to those in straight counterparts. Under the solar-induced thermal loading, the bending-torsion coupling effect becomes increasingly significant with the decreasing curvature radius. To quantify the effects of the curvature radius on the thermal-induced deformation and torsional behavior of curved steel box girder bridges, the case-study bridge is analyzed with five curvature radii, namely: 300 m, 500 m, 700 m, 900 m (i.e., corresponding to the cases of $R300$, $R500$, $R700$, and $R900$ in Figs. 29 to 31), and an equivalent straight bridge. The deformation, stress, and torsion behavior of the bridges under different curvature conditions were examined. Fig. 29 shows the evolution of deformation with varying curvature radii. To ensure consistency, all five models had the same span length.

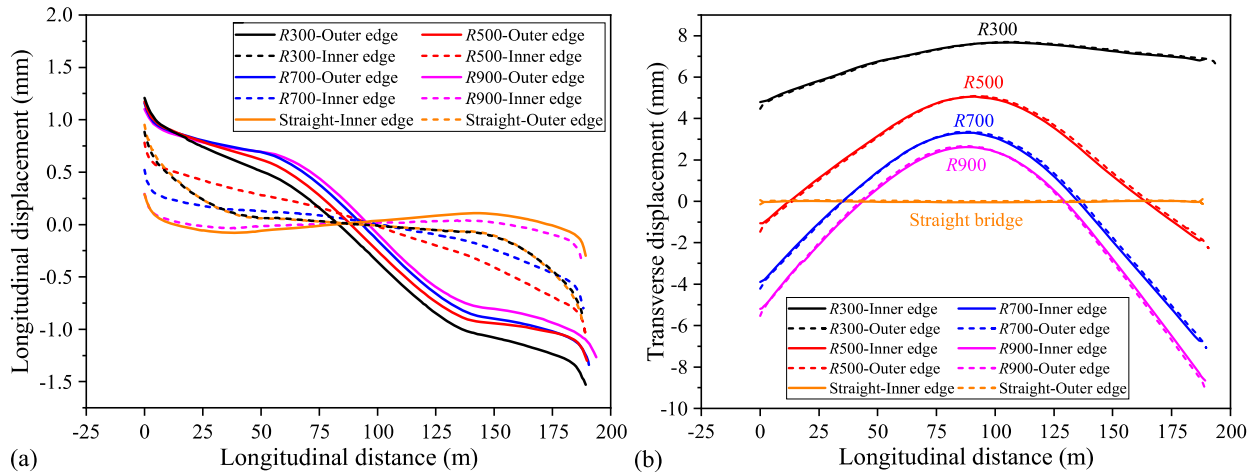


Fig. 29. Effects of curvature radius on: (a) longitudinal and (b) transverse displacements of the bridges.

As illustrated in Fig. 29, the temperature gradient induced similar deformation patterns in curved steel box girder bridges with radii of 300, 500, 700, and 900 m. The longitudinal displacements on both the inner and outer edges increased from midspan to the girder ends, reaching their maximum at the supports. The transverse displacements also shifted outward and increased progressively from the girder center toward the ends. Both the longitudinal and transverse deformations followed the order of: $R300 > R500 > R700 > R900 >$ straight bridge. In other words, curved girder bridges with the smaller radii experienced more pronounced longitudinal and transverse displacements under the same thermal gradients. As the curvature radius increased and approached that of a straight bridge, both the longitudinal and transverse displacement components diminished. This trend can be attributed to the stronger bending effect in curved steel box girder bridges with smaller radii, which leads to more significant axial elongation and transverse deformation. In contrast,

larger radius produces milder curvature and thus smaller deformations. Besides, curved steel box girder bridges with smaller radii are more sensitive to the support conditions, exhibiting larger variations in displacement under identical thermal loading.

Compared to the straight forms, the curved steel box girder bridges also exhibit more complex thermo-mechanical behavior due to their inherent bending-torsion coupling effects. As shown in Fig. 29, the transverse displacement of straight bridges under thermal loading was largely negligible. In contrast, curved steel box girder bridges exhibited maximum transverse displacements of 7.69, 5.07, 7.07, and 9.03 mm for curvature radii of 300, 500, 700, and 900 m, respectively. This indicates that curved steel box girder bridges exhibit larger lateral deformations than their straight counterparts. The discrepancy arises from the unique bending-torsion interaction in curved steel box girder bridges: the smaller the curvature radius, the greater the bending deformation, which increases the lateral displacement. In contrast, straight bridges have an effectively infinite radius of curvature, resulting in minimal bending deformation and consequently smaller transverse displacements.

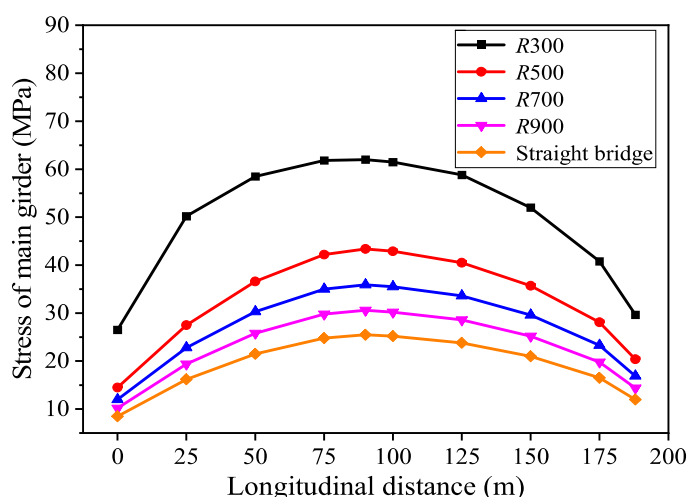


Fig. 30. Influence of curvature radius on the stress response of the bridges.

In addition, Fig. 30 illustrates the evolution of stresses of the bridges with varying curvature radii. As observed from this figure, analysis of the stress distribution data revealed a pronounced curvature effect in the curved steel box girder bridges. As the curvature radius decreased, the overall stress level increased markedly, accompanied by a systematic variation in the stress distribution pattern. The left-side of the curves exhibited a steep rising gradient, whereas the right-side ones showed a more gradual decline, resulting in an asymmetric stress profile. Notably, the stress difference between the cases with $R300$ and $R500$ is most significant, with a peak disparity of approximately 18.6 MPa, while the difference between $R500$ and the straight bridge gradually

diminished. This trend provides critical insight for the structural design of the curved steel box girder bridges, highlighting that small-radius curves are more susceptible to stress concentration in peak regions and therefore require targeted reinforcement measures.

Furthermore, Fig. 31 depicts the associated torsional angle development of the bridges with the varying curvature radii. As shown in Fig. 31, temperature gradients induce significant torsional deformation in curved steel box girder bridges. The torsional angle is positive on the outer curve and negative on the inner curve, indicating outward and inward rotations, respectively. The outward torsion predominantly occurred near the mid-span, whilst the inward torsion was concentrated near the side spans. As the curvature radius decreased, the outward torsional angle at the mid-span increased, and so did the inward torsional angle at the side spans. This trend demonstrates that the bending-torsion coupling effect of curved steel box girder bridges increased with a decreasing curvature radius, leading to a more pronounced outward torsional tendency of the girder.

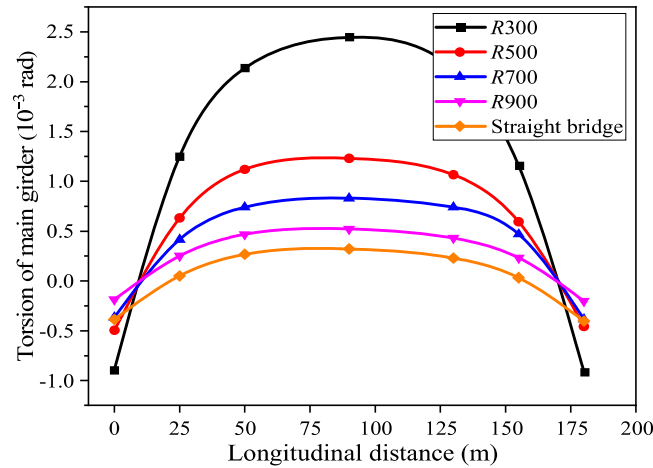


Fig. 31. Influence of curvature radius on the torsion angle of the bridges.

In summary, both the longitudinal and transverse displacements of curved steel box girder bridges increase with a decreasing curvature radius, with a marked rise observed when the radius is reduced to 300 m. A small curvature radius could induce greater torsional effects and stresses in curved girders, leading to increased deformations and stresses, as well as potentially compromising the strength and stability of the bridge. Thus, in the design and planning of curved steel box girder bridges, an appropriate curvature radius should be selected based on actual site conditions, and excessively small radii should be avoided to ensure structural safety, ride comfort, and cost-effectiveness.

5. Conclusions

This paper investigated the temperature field characteristics and associated mechanical behavior of

curved steel box girder bridges through the field measurements and numerical simulations. A representative curved steel box girder bridge was selected as a case study, and the vertical and transverse temperature gradient models were proposed based on the field monitoring data. Detailed numerical assessments were carried out to assess the validity and applicability of the proposed models. In addition, parametric evaluations were carried out to provide detailed insights into the various temperature field distributions and thermal-induced structural behavior of curved steel box girder bridges under the vertical, transverse, and coupled temperature gradients. Based on the results, the main conclusions are summarized below.

1. Based on the field monitoring data for the case-study bridge, it was shown that positive temperature gradients were predominated in summer, whereas negative temperature gradients were more prevalent in winter.

2. By using the field temperature measurements, the vertical and transverse temperature gradient models were established for curved steel box girder bridges. The vertical temperature gradients were found to be well represented by an exponential function, while the transverse temperature gradients followed a Gaussian distribution. The effectiveness and accuracy of the proposed temperature gradient models provide close prediction of the field-measured temperature data, and offer a basis for improving the design specifications of curved steel box girder bridges.

3. To extend the applicability of the proposed vertical and transverse temperature gradient models, a detailed finite element model was validated against the field measurements and employed to derive representative vertical and transverse temperature differences for the curved steel box girder bridges located in 12 major cities across diverse climatic regions in China. The results provide a reliable modeling reference and offer region-specific thermal loading parameters to support the climate-adaptive design of curved steel box girder bridges.

4. The maximum thermal stress of the case-study curved bridge under coupled vertical and transverse temperature gradients was shown to be 0.6% higher than that induced by the vertical gradient alone and 11.3% higher than that from the transverse gradient alone. These thermal stresses could be further amplified by bending stresses due to girder curvature, resulting in elevated tensile stresses along the outer edge of curved steel box girder. Thus, the combined effects of temperature gradients and structural curvature should be carefully considered in the structural analysis and design of curved steel box girder bridges.

5. Curved steel box girder bridges were found to exhibit consistent deformation patterns under

the vertical, transverse, and coupled temperature gradients. For the case-study bridge, the maximum longitudinal displacement under the coupled temperature gradients was 1.76 and 2.44 times greater than those induced by the vertical and transverse gradients alone, respectively. Similarly, the maximum transverse displacements under the combined temperature gradients were 2.08 and 2.22 times that under the individual vertical and transverse gradients, respectively. In addition, the maximum vertical displacement under the combined gradients was 2.08 and 2.22 times larger than that induced by the vertical and transverse gradients alone, respectively.

6. The results showed that the curvature radius has significant effects on the thermal-induced deformation, stress, and torsional behavior of curved steel box girder bridges. As the curvature radius decreased, both the transverse and longitudinal deformations of the bridges increased, along with a more pronounced torsional rotation toward the outer side of the curve. This indicates that a comparatively small curvature radius may adversely affect the mechanical behavior and stability of curved steel box girder bridges. It was shown that selecting an appropriate curvature radius based on site-specific conditions and project-specific requirements is crucial. To ensure the structural safety, rider comfort, and cost-effectiveness, relatively small radii should be avoided where possible, and the thermal-induced deformation, stress, and torsional behavior should be carefully and consistently considered in the design of curved steel box girder bridges.

Although this study advances the understanding of the thermo-mechanical behavior of curved steel box girder bridges, several limitations that warrant further investigations are summarized below.

(i) The current FE models neglected the effects of geometric imperfections and residual stresses. While this simplification is acceptable under normal service conditions, it may compromise accuracy when assessing structural responses near stability or ultimate limit states. Future work should incorporate the measured imperfections and residual stress distributions into advanced nonlinear FE models to more accurately evaluate their effects on global stability and ultimate capacity.

(ii) The proposed temperature gradient models were derived from one-year monitoring data of a single bridge, limiting their generality and ability to represent the stochastic variability of thermal loads across different climates, geometries, and environmental contexts. Expanding the dataset to include multi-year observations from multiple bridges or bridge networks would improve the robustness and applicability of the models. In addition, the present study did not explicitly quantify

uncertainties arising from environmental factors (i.e., ambient temperature, solar radiation, wind speed, humidity) and material properties (e.g., thermal conductivity, specific heat, elastic modulus). Future research should employ probabilistic or stochastic modeling frameworks integrating uncertainty quantification and sensitivity analysis to evaluate their impacts on structural performance.

(iii) Enhancing the predictive capability of thermal field simulations may benefit from integrating physics-informed machine learning techniques, such as the Deep Energy Method (DEM) and the Variational Implicit Neural Operator (VINO), to establish mesh-independent frameworks that capture both environmental and material uncertainties. Besides, the limited monitoring duration precluded characterization of negative temperature gradients under severe winter conditions, which are critical for bridges in cold regions. Long-term winter monitoring is therefore recommended to develop representative negative gradient models and improve thermal load prediction accuracy.

(iv) As the proposed model was developed for a specific bridge type, its applicability to other structural configurations remains uncertain. Further investigations should examine temperature gradient variability across bridges with different spans and cross-sectional geometries to establish more generalized and transferable modeling frameworks.

Acknowledgments

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Appendix

During the determination of temperature differences in several representative cities in diverse climatic zones across China, the considered thermal boundary condition parameters for different cities are summarized in Table A1 below.

Table A1. Thermal boundary condition parameters for the computational process of the curved steel box girder bridges in representative cities of China.

Region	City	I_m (W/m ²)	I_d (W/m ²)	I_f (W/m ²)	I (W/m ²)	v (m/s)	h_c (W/m ² ·°C)	T_s (W/m ² ·°C)	Q_r (W/m ²)
Northeastern	Harbin	892	241	553	1686	3.5	14.2	37.55	−142
China	Shenyang	926	244	567	1737	3.2	13.8	43.55	−151
Northwestern	Xi'an	915	253	621	1789	2.8	13.4	42.6	−148
China	Lanzhou	878	253	621	1752	4	14.7	38.87	−139
Eastern	Jinan	958	282	769	2009	3	13.7	45.62	−156
China	Hangzhou	902	254	622	1778	2.5	13.1	40.46	−143
Central	Wuhan	886	242	568	1696	2.2	12.8	42.39	−147
China	Changsha	905	241	566	1712	2.3	12.9	42.48	−148
Southwestern	Chengdu	865	240	564	1669	1.8	12.3	45.45	−153
China	Guiyang	852	242	568	1662	2	12.5	37.66	−138
Southern	Guangzhou	932	247	574	1753	2.6	13.2	45.24	−152
China	Nanning	945	248	576	1769	2.4	13	48.56	−157