Temperature analysis of a long-span suspension bridge based on field monitoring and numerical simulation

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Abstract

Structural temperature is an important form of loading for bridges, particularly for long-span steel structures. In this study, the temperature distribution of the Humber Bridge in United Kingdom is investigated based on numerical simulation and field measurements. A 2D fine finite element (FE) model of a typical section of the box girder of this long-span suspension bridge is constructed. The time-dependent thermal boundary conditions are determined based on the field meteorological measurements with external surface heat convection coefficients varying according to differing local wind speeds they experience. Pre-analysis is adopted to determine the initial thermal condition of the model, then transient heat-transfer analysis is performed and the time-dependent temperature distribution of the bridge is obtained leading to numerical temperature data at different locations in different time that are in good agreement with the measured counterparts. The vertical and transversal temperature differences of the box girder are also investigated. Both measured and numerical results show that the transversal temperature variation across the streamlined girder is significant. The effects of the box girder shape, pavement of the upper webs, and bridge orientation on the transversal temperature difference are finally investigated.

Keywords: long-span suspension bridge, temperature behavior, field monitoring, heat-transfer analysis, transversal temperature difference

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1. Introduction

Bridges are subject to daily, seasonally, and annually varying environmental thermal effects caused by solar radiation and surrounding air temperature. The changes in structural temperature and temperature distribution of a bridge result in movements and deformations, heavy demands on connections and supports and potentially excessive stresses and cracks. For example, repeated cycles of heating and cooling induced by thermal actions may result in large amplitude stress cycles and fatigue damages. In fact structural behavior of bridges is more significantly affected by environmental thermal effects than by external operational loads (Priestley, 1976, 1978; Kennedy and Soliman, 1987; Salawu, 1997; Xia et al., 2011; Bojovic and Velovic, 2014).

Analyzing the thermal effects on bridges mainly consists of two studies: structural temperature and induced structural responses. To calculate temperature-induced responses and evaluate the thermal effects on bridge behavior, the entire structural temperature distribution must be accurately known. Since the 1960s, considerable efforts have been devoted to investigating temperature distribution and thermal effects on bridges based on laboratory experiments and field investigations and Zuk (1965) was considered the first to study the thermal behavior of bridges. He identified the effects of solar radiation, air temperature, wind, humidity, and material types on temperature distribution by investigating several highway bridges. Emanual and Reynolds (1978) investigated the temperature variations of a composite-girder highway bridge and calculated the bridge temperatures as a function of time by using finite element (FE) analysis. Since the negative effects of temperature are mainly induced by uneven temperature distribution, the temperature gradient (difference) of various types of bridges became the research focus then. Priestley (1976, 1978) analyzed the vertical temperature gradients of pre-stressed and reinforced concrete bridges and compared the analytical results with those from laboratory and field experiments. Kennedy (1987) studied the temperature distribution of composite bridges and proposed the linear temperature distribution through the depth of the slab and uniform distribution through the depth of the steel beam by synthesizing several theoretical and experimental studies on prototype bridges. Churchward and Yehuda (1981) continuously recorded the temperature of a post-stressed twin box concrete bridge and presented an analytical expression of the vertical temperature profile as a function of the maximum differential temperature and environmental parameter insolation. A long-term field measurement was conducted by Dilger et al. (1981) to investigate the thermal effects on a continuous, steel-concrete composite box girder bridge during its construction and its first three years of operation.

Analytical equations and numerical methods have also been proposed to calculate the temperature distribution of simple structures, including girder bridges, since the 1970s (Emerson, 1973; Hunt and Nigel, 1975; Priestley, 1976; Kehlbeck, 1981; Elbadry and Ghali, 1983). These methods are basically one-dimensional (1D) approaches that assume temperature only varies along the depth of the cross-section and that variations along other directions are insignificant. As structural configuration becomes increasingly complicated, the 1D models can hardly capture the temperature variation and distribution of relatively complicated structures, including box girder bridges. Elbadry and Ghali (1983) proposed a two-dimensional (2D) FE method to determine the time-dependent temperature variation of a concrete box girder bridge by considering geometry, location, orientation, material, and meteorological conditions. Tong et al. (2001, 2002) conducted such a study on a steel bridge in Hong Kong while Lucas et al. (2003) statistically analyzed the average temperature and thermal gradient of a steel box girder bridge. These studies show that steel bridges have a large temperature gradient along the cross-section and significant variation over time because of the high conductivity of steel.

Naturally the top surface of a box-girder bridge receives more solar radiation than the web and soffit in general, resulting in considerable vertical temperature difference, an effect that has been widely investigated, with detailed specifications provided in bridge design codes, for example Eurocode 1 (European Committee for Standardization, 2003). The transversal temperature difference (TTD) is usually smaller than the vertical difference for most types of bridge, especially for concrete bridges (Mondal and DeWolf, 2007), hence present codes do not provide much information on this. However, particular types of bridge may also experience significant TTD and the induced structural responses, such as transverse movements, can pose a significant threat to structural performance (Moorty and Roeder, 1992). For example, Kromanis et al. (2014) investigated the quasi-static temperature effects on the Cleddau Bridge based on continuous monitoring measurements, showing the TTD up to 15 degrees. It resulted in plan bending of the main box girder, generating plan rotations at the roller bearings. These movements, which were not considered at the design stage, imposed large forces on the bearings and led to their degradation.

The number of constructed long-span bridges has dramatically increased over the past decades. These bridges have a complicated temperature distribution because their main structural elements, including decks, towers, and main cables, have different thermal characteristics. The temperature action of these bridges is a major concern, and long-term monitoring has become a standard procedure through rapid development of structural health monitoring (SHM). While the prime focus of these systems is deformations and their temporal and spatial derivatives, a number of exercises have used temperature data to study thermal effects. For example Xu et al. (2010) analyzed the temperature characteristics of Tsing Ma Bridge using several years of monitoring data, while Xia et al. (2013) performed extensive thermal and structural analyses of the temperature effects on the bridge. Ding et al. (2012, 2013) used long-term monitoring data to estimate the extreme temperature differences of a steel box girder suspension bridge, while Westgate (2012) and de Battista et al. (2014) investigated the effects of traffic and thermal actions on the static and dynamic responses of Tamar Suspension Bridge.

To resist wind loading these long-span bridges normally have a wide cross-section, resulting in relatively large transversal temperature differences, an effect which has not been sufficiently investigated in previous studies. The Humber Bridge, on which a long-term SHM system has been installed, provides an opportunity for an in-depth study of this issue. This paper is organized as follows:

1. The Humber Bridge and the installed SHM system are briefly introduced.
2. The thermal boundary conditions of the bridge are discussed.
3. The FE model of the box girder is developed, and thermal analyses are performed for each model.
4. The analytical results are compared with the measurements to validate the method.
5. Time-dependent structural temperature and distribution are obtained.
6. The temperature differences, particularly TTD, of the box girder are investigated.
7. Conclusions and suggestions are drawn for analyzing temperature actions of long-span

suspension bridges.

2. Humber Bridge and the monitoring system

2.1 Humber Bridge

The Humber Bridge, completed in June 1981 has a total length of 2220 m with an asymmetric layout comprising the 280 m Hessle side span, 1410 m main span, and 530 m Barton (south) side span, as shown in Fig. 1. The main span was the longest in the world for 17 years from its inauguration and is at the time of writing the seventh longest of its type in the world.

The bridge girder is not continuous through the towers, having a complex arrangement of bearings and expansion joints to accommodate movement due to wind, traffic and thermal loading. It was assembled from 124 box segments, each typically 18.1 m long, 22 m wide, and 4.5 m high, with four stiffened bulkheads and has cantilevered footpaths and cycle tracks bringing the total width to 28 m. The upper roadway surface of the box is an 18.2 m wide orthotropic steel deck that was originally covered with 41 mm thick rubberized bitumen asphalt, but which has been replaced at least once while cantilevers retain a thin asphalt surfacing (Brownjohn et al., 1987).

2.2 SHM system of the Humber Bridge

The bridge has several systems for monitoring traffic, weather and main cable condition (Lynch, 2012). However, the most relevant monitoring system for this study was installed on the bridge in 2011. The SHM system consists of three modules: the sensors, the transmission network, and the data management system (Koo et al., 2011, Brownjohn et al., 2014). The sensors are divided into four types according to the sensing parameter.

1) Sensors for meteorological parameters. A new weather station was installed at the mid-span of the bridge which supplements data from anemometers and air temperature sensors previously

by the operator. Meteorological data from nearby Humberside Airport are also available and are integrated into the database of the SHM system.

2) Sensors for dynamics responses. Three servo-accelerometers feed acceleration signals into an automated process for real-time estimation of modal frequencies and damping ratios.

3) Sensors for static responses. Real-time kinematic position data are available for GPS antennae fixed to each main cable at mid-span, with correction data are provided by a reference station at the Hessle anchorage. This system provides the primary position data for the bridge. Four laser extensometers, one on each columns of the Hessle and Barton towers provide bearing movement data.

4) Sensors for structural temperature. Four thermocouples were installed to record the temperature of the box girder at the mid-span located on the top (T_t), bottom (T_b), east (T_e), and west (T_w) surfaces of the box girder. Fig. 2 depicts the two temperature sensors installed during construction at the middle of the left lanes of the northbound carriageway to collect the surface temperature of the asphalt (T_s) and the ground temperature (T_g) in the interface between the steel surface and paved asphalt.

3. Thermal analysis of bridge structures

3.1 Thermal environment of a box girder bridge

The temperature differences along the longitudinal direction of a bridge are generally neglected. Therefore, a single box girder section can be used to analyze the temperature distribution of the bridge, the thermal environment of which is shown in Fig. 3. The heat transfer process of a box girder bridge exposed to the open environment consists of heat conduction, heat convection, and thermal radiation (Kehlbeck, 1981). Heat conduction exists in the interior of the box girder and is governed by the Fourier heat-transfer equation. Heat convection is a kind of energy exchange between the solid surface and the surrounding fluid that results from the diffusion and bulk motion of the fluid. Thermal radiation is a kind of energy transfer caused by the structural surface emitting and absorbing radiation.

Several forms of radiation, including solar, atmospheric, diffuse, reflected, environmental, and structural irradiation, are emitted or absorbed by a bridge surface. The direct solar radiation from the sun striking is the main radiation factor affecting bridges. Atmospheric radiation is the gas in the atmosphere emitting radiation, and governed by the Stefan–Boltzmann law. Diffuse radiation is the solar radiation scattered by molecules and particles in the atmosphere. The environmental radiation is the sum of the radiation emitted by surrounding matter of the ground surface, including structures, trees, rocks, and roads. This radiation also follows the Stefan–Boltzmann law. Reflected radiation describes the non-atmospheric effects such as the ground reflecting the direct solar and diffuse radiations. The structural irradiation is the radiation emitted from the bridge surface, also governed by the Stefan–Boltzmann law.

3.2 Heat transfer analysis

(1) Heat transfer theory

The temperature of a point in a structure can be expressed as $T = T(x, y, z, t)$, where x, y, and z are the Cartesian coordinates of the point and t is the time. Heat transfer theory is governed by the typical Fourier heat-transfer equation:

$$\rho c \frac{\partial T}{\partial t} = k\left(\frac{\partial^2 T}{\partial x^2} + \frac{\partial^2 T}{\partial y^2} + \frac{\partial^2 T}{\partial z^2}\right) \tag{1}$$

where k is the isotropic thermal conductivity coefficient, ρ is the density, and c is the specific heat of the material. The temperature field of a structure at a specific time can be obtained by solving the above Fourier partial differential equation under initial and boundary conditions, which will be briefed in following sections.

(2) Thermal boundary conditions

The boundary conditions for structural thermal analysis can be generally classified into three types (Lienhard and Lienhard, 2003). Type 1 denotes that the temperature of the structural boundary is exactly known, Type 2 the heat flux on the structural boundary is determinate, and Type 3 the heat flux on the boundary is proportional to the difference between the air temperature and the bridge surface temperature.

The boundary conditions associated with Eq. 1 for the thermal analysis of a bridge can be written as a combination of Types 2 and 3 (Elbadry and Ghali, 1983).

$$k\frac{\partial T}{\partial n} = h(T_a - T_v) + q \tag{2}$$

where n is normal to the surface, $h = h_c + h_r$ (W/m^{2}K) is the heat transfer coefficient combining the heat transfer coefficients of convection (h_c) and thermal irradiation (h_r), T_a is the air temperature, T_v is the structural surface temperature, and q is the boundary heat exchange per unit area (heat fluxes, positive for inflow).

The heat transfer coefficient of convection (h_c) is related to wind speed. Kehlbeck (1981) proposed an empirical equation to calculate the convection coefficient when wind speed $w \leq 5$ m/s:

$$h_c = 2.6 \times (\sqrt[4]{|T_a - T_v|} + 1.54 \times w) \tag{3a}$$

The following empirical equation is used for wind speed $w > 5$ m/s (Elbadry and Ghali, 1983; Dilger et al., 1983).

$$h_c = 6.31 \times w^{0.656} + 3.25 \times \exp(-1.91 \times w) \tag{3b}$$

The heat transfer coefficient of thermal radiation (h_r) depends on the structural material, surface temperature, and air temperature (Kreith, 1973; Elbadry and Ghali, 1983).

The heat fluxes q absorbed by structural surface that are caused by all external radiation contributions can be expressed as follows:

$$q = \alpha I \tag{4}$$

where α ($0<\alpha<1$) is the absorptivity coefficient of the surface material, and I is the sum of the external radiation received by a surface. For the case of a structure exposed to an open environment, the emitting radiation (structural irradiation G_v) is considered as the thermal irradiation by using the heat transfer coefficients (h_r) in Eq. 3. The absorbed radiation is calculated as heat fluxes q in Eq. 4, which consists of the direct solar radiation (I_s), diffuse radiation (H), atmospheric radiation (G_a) from the sky, ground surface radiation (G_g), and the reflected radiation (I_r) from the ground surface. The absorbed radiation is depicted as follows:

$$I = I_s + H + G_a + G_g + I_r \tag{5}$$

(3) Initial temperature condition for thermal analysis

On-site temperature sensors are not sufficient to provide the complete initial temperature of the bridge. Hence a pre-analysis for one or several consecutive days is performed. The initial temperatures of the bridge in the pre-analysis are assumed uniform and the thermal boundary conditions are applied. After the pre-analysis, the temperature distribution of the bridge is non-uniform, thus providing the initial condition of the subsequent thermal analysis.

4. Temperature distribution simulation of Humber Bridge

4.1 Thermal analysis of the box girder section

(1) FE model of box girder

The temperature along the longitudinal direction of the bridge is assumed to be constant. Therefore, the FE model of a typical box girder section is constructed using ANSYS (2005) to investigate the temperature distribution of the box girder. Fig. 4 shows the details of the box girder section and the FE model which consists of 38,620 elements. The model uses PLANE55 elements for several materials: steel for the stiffened plate, asphalt for roadway, and air filling the inside hollow of the box section. PLANE55 is a type of 2D element with four nodes, each having a single degree of freedom of temperature and is endowed with thermal conduction capability making it suitable for the 2D, steady-state or transient thermal analysis.

The thermal boundary conditions of the exterior and interior structural surfaces are separately applied in the conventional thermal analysis of box girders. Also the surrounding air of both the exterior and the interior of the box girder are considered totally independent. Consequently, the thermal equilibrium of the entire system can hardly be maintained. In the present thermal analysis, the air filling inside the box is modeled by using the elements of PLANE55, thus, the thermal boundary conditions of convection on the interior surfaces of the box are not necessary. The interaction of the thermal radiation of the interior surfaces is calculated by using the AUX12 radiation matrix and the results applied to the inside surface by using the super-element MATRIX50 in ANSYS (2005).

(2) Thermal boundary condition

The thermal analysis in ANSYS cannot deal with the two thermal boundary conditions (Types 2 and 3) simultaneously on the same surface. Eq. (2) can be converted as follows:

$$k\frac{\partial T}{\partial n} = h\left(T_a + \frac{q}{h} - T_v\right) = h\left(T_{eq} - T_v\right) \tag{6}$$

where T_{eq} includes both the air temperature and radiation and is referred to as "equivalent air temperature."

The wind blowing across the bridge surface significantly affects the heat transfer convection coefficient (see Eq. 3) and consequently influences the accuracy of the thermal analysis results. Previous studies on bridge thermal analysis used a constant value of wind speed for all structural surfaces. However, the box girder consists of several surfaces with different azimuth angles so the variation of local wind speeds on different bridge surfaces must be considered.

As shown in Fig. 5, the cross-section of the box girder is divided into three zones according to the wind incidence angle θ_w: windward side ($\theta_w \leq 45^{\circ}$), crosswind side ($45^{\circ} < \theta_w \leq 90^{\circ}$), and leeward side. The wind speeds on these zones take 80%, 70%, and 60% of the incident wind speed w, respectively.

(3) Temperature variation of the box girder section

The extreme environmental conditions, including strong solar radiation, high air temperature, and low wind speed, normally generate high temperature differences throughout the box girder. Thus, a typical sunny day on 24 July 2012 with relatively high solar radiation and air temperature was selected. Fig. 6 shows the measured wind speed and wind direction and Fig. 7 shows the air temperature and cloud cover. The wind speed, wind direction, and air temperature were recorded by the weather station at the mid-span of Humber Bridge. The cloud cover condition was obtained from the meteorological measurements at the Humberside Airport from the National Oceanic and Atmospheric Administration website (NOAA). The maximum cloud cover in daytime is only 25%, which can be considered a clear day.

Based on the measured meteorological data, the thermal boundary conditions are calculated and applied to the FE model for transient heat transfer analysis. The initial temperature of the box girder is obtained from the final results of a pre-analysis of the previous day. The main material parameters are summarized in Table 1 (Kehlbeck, 1981; Tong, et al., 2001; Xia et al., 2013).

The temperature variation of the box girder on 24 July 2012 is calculated and compared with the corresponding measurements in Fig. 8. The structural temperature slightly decreased and reached the minimum in the early morning. The temperature then increased to the maximum in the early afternoon and decreased in the evening and midnight.

The temperature of the entire bridge reached a minimum of approximately 10 ºC at around 05:00. This finding indicates that the entire bridge had an approximately uniform temperature distribution at this moment. However, different components reached their maximum temperature at different time instants. The asphalt cover had a maximum temperature at approximately 15:00, the top of inside surface of box girder reached the maximum temperature at around 16:00, the bottom surface a little later. The structural temperature of the east side reached the maximum at around 11:00, five hours earlier than the west side, at approximately 16:00. For this type of box-girder bridge, the exterior reaches the maximum temperature earlier than the interior, the top earlier than the bottom, and the east earlier than the west. The maximum temperature values were approximately 35, 34.5, 34, and 26 °C for the asphalt, interface, girder top, and girder bottom, respectively.

The simulated temperatures at the observed points correlate well concur with the field measurements. Therefore, the effectiveness of the heat transfer analysis is verified. Some measured temperatures exhibit abrupt changes, which have not been well predicted in the numerical simulation. This condition is most likely attributed to the transient local cloud cover affecting solar radiation.

5. Temperature gradient of the box girder

The temperature gradients of the box girder of the Humber Bridge in both vertical and transversal direction are investigated in this section.

5.1 Vertical temperature gradient of the box girder

To study the typical seasonal temperature behavior of the bridge, four sunny days in different seasons are selected for thermal analysis. The selected days are 11 February, 16 May, 24 July, and 6 October 2012. According to the measured air temperature, 11 February was the coldest in winter, whereas 24 July was the hottest in summer. The days of 16 May and 6 October can represent the weather conditions of spring and autumn, respectively.

The vertical temperature differences of the box girder in different seasons are obtained from the numerical analysis and plotted in Fig. 9. Fig. 9(a) depicts the variation of the temperature difference of the cross-section over time. The temperature difference in this simulation refers to the difference between the maximum and minimum temperatures of the section along the web on one side, which may occur at different points at different time instances. The temperature difference of the section was considerable, and the difference in daytime was significantly larger than the difference at night. The vertical temperature difference was small before sunrise, increasing to a maximum at noon, then decreasing until after sunset. The east side had the largest temperature difference at about 9:00 and the west side had the maximum at approximately 15:00. Moreover, the maximum vertical temperature difference of the west side was larger than the east side. The vertical temperature difference was largest in summer and smallest in winter

The vertical temperature gradient profile is shown in Fig. 9(b). The upper web had higher temperature than other components because it received more direct solar radiation. The lower web and bottom deck had the lowest temperature because they were blocked from direct solar radiation. The east and west sides of the box girder reached peak temperatures different times, as shown in Fig. 9(a). This finding indicates that TTD existed in the east and west upper webs.

5.2 TTD of box girder

Eurocode 1 for thermal actions (European Committee for Standardization, 2003) states that in usual cases only the vertical temperature difference must be considered. However, the TTD must also be considered in particular cases such as that in which one side is more exposed to sunlight than the other. The box girder of the Humber Bridge is streamlined with inclined webs on the east and west sides that may cause significant TTD due to receiving different solar radiation.

Here the measured TTD between the sensors installed on the east and west deck of the bridge (T_e and T_w, see Fig. 2) is investigated first. Fig. 10(a) shows the absolute value of TTD in two clear days, representing the summer and spring, respectively. In the morning the east side received much more solar radiation than the west and the temperature difference reached the maximum of 12 °C at 8:00 in summer and 5°C at 11:00 in winter. On the contrary, in the afternoon the west side received much more solar radiation than the east and the TTD reached the maximum of 13°C at 17:00 in summer and 5°C at 15:00 in winter. Therefore, summer has more significant TTD. Fig. 10(b) shows the daily maximum TTD in 2012, and the maximal TTD could be as high as 18°C. In general, significant TTD occurred from March to September.

Eurocode 1 (European Committee for Standardization, 2003) recommends 5°C as the linear TTD between outer edges of a bridge if no other information is available and no indications of higher values exist. However, the present results show that the TTD of the Humber Bridge reaches as high as 18 °C and that value over a longer period of monitoring is likely higher. As described previously, the maximal TTD observed at the Cleddau Bridge was as 15°C. Cleddau is also a steel box girder bridge, albeit one with a very different section to Humber, but the need to consider TTD for such bridges seems to be clear.

To investigate the TTD of the entire box girder, the numerical results are then examined. Fig. 11(a) shows the maximal TTD during four chosen days. The peaks occurred around 9:00 and 15:00. Similar to the measurement data, summer and spring have larger TTD than winter and autumn.

The distribution of the TTD along the deck on 24 July 2012 is illustrated in Fig. 11(b). Again the east upper web had much higher temperature in the morning, the top deck had slightly higher temperature than both upper webs at noon, and the west web took the maximum temperature in the afternoon. An abrupt change in temperature occurred at the connection between the deck and webs. Therefore, the inclined upper webs play the critical role in the TTD.

5.3 Effect of box girder profile on TTD

To study the effect of inclined upper webs on TTD, three different box girder shapes are investigated. Case 1 is the current section of the Humber Bridge; in Case 2 the footpath is at the mid-height of the section; and Case 3 the upper webs are flush with the top deck. The TTD variation on 24 July 2012 for three cases is shown in Fig. 12. The three cases have similar distribution of TTD. However, Case 2 with deepest upper webs shows the largest TTD. In Case 3, the upper webs are located at almost the same level as the upper deck but have higher temperature than the latter. This is because the former are made of steel and are directly exposed to open environment whereas the latter is covered by a 40 mm thick asphalt pavement, which has a low thermal conductivity. Next the effect of asphalt cover on TTD will be investigated.

The upper webs are assumed to be covered with 40 mm of asphalt, same as the upper deck. The corresponding TTDs for the above three shapes are illustrated in Fig. 13. The TTDs decrease significantly as compared with those without asphalt cover. In particular, the maximum TTD decreases from 17°C to 11°C in Case 2 and from 10°C to 3°C in Case 3.

The effects of bridge orientation with respect to the sun (the bridge azimuth) on TTD are also investigated based on the current box girder section of the Humber Bridge. Four typical orientations, 0°, 45°, 90° and -45° as defined in Fig. 14 are considered. It is noted the actual Humber Bridge is almost in the south-north direction, corresponding to 0°. Here the variation of wind speed on different surfaces is not considered for simplicity and thus the convection coefficient is uniform for all surfaces. The TTD on the day is illustrated in Fig. 14. Although the time variation of TTD is different for different orientations, the maximal TTD is almost similar. Detailed investigations show that regardless the bridge orientation, when one upper web receives direct solar radiation in the morning, the opposite upper web is blocked from solar radiation by the upper deck. Therefore, the bridge orientation has a slight effect on the maximum value of TTD in one day.

6. Conclusions and Discussions

Accurately analyzing the temperature behavior of long-span bridges is a challenge because of complex configuration and high uncertain and varying meteorological environment. This study investigates the temperature behavior of the Humber Bridge, a long-span steel suspension bridge for which a high resolution FE model is constructed. Thermal boundary conditions are calculated accounting for various environmental conditions on different surfaces and transient heat transfer analyses in different seasons performed by using the ANSYS FE software package. The numerical results are verified through a comparison with the measurements.

The boundary and initial conditions and the thermal coefficients significantly affect the results of the thermal analysis. The new approaches employed in the present numerical analysis are as follows:

(1) The air inside the box girder is modeled by using air elements and the thermal radiation of the interior surface is analyzed by using the AUX12 Radiation Matrix. This method provides a more reasonable way to ensure the thermal equilibrium condition in thermal analysis of box girder bridges. The method also improves the computational efficiency by eliminating the thermal boundary conditions of the interior surfaces.

(2) Different wind speeds on different exterior surfaces are adopted in calculating the heat convection coefficients of the surface.

(3) A pre-analysis approach is adopted to obtain the initial thermal condition of the bridge. After the 24 hours pre-analysis, the thermal distribution of the entire bridge can be used as the initial thermal condition.

Employing the above approaches, the numerical results in different seasons concur with the field measurements.

The streamlined steel box girder (with inclined upper webs) is designed to perform well for wind loading. However, this study has shown that this type of girder may exhibit significant vertical and transversal temperature difference because the upper webs on each side receive very different solar radiation in daytime. In particular, the TTD has not been considered sufficiently in previous studies and current standards. Such a large TTD may cause in-plane bending of the box girder (or deck) and thus generate rotations at the bearings. To avoid resulting excessive movement or forces of bearings special attention must be given during design and analysis. Parametric studies with the established numerical model show that deep upper webs play a critical role in TTD and asphalt cover can reduce TTD considerably. The bridge orientation has limited effect on the maximum TTD.

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Table 1. Material parameters for thermal analysis

Parameters	Notation	Steel	Asphalt	Air	Concrete
Density	ρ (kg/m^3)	7850	1530	1.2×10^{-3}	2400
Heat capacity	c (J/kg/°C)	460	1075	1.007	925
Thermal conductivity	k (W/m/°C)	60	1.80	0.026	2.71
Emissivity coefficient	ε_v	0.80	0.92	0	0.88
Absorptivity coefficient	α	0.75	0.90	0	0.65