Effect of the Presence of Pre-Service Construction Cracks in Concrete Decks on the Thermal Profile of Composite Steel-Concrete Bridges in Cold Regions

Omar Y. El Masri¹ and Caesar Abi Shdid²

¹Ph.D. Candidate, Dept. of Civil and Environmental Engineering, Syracuse Univ., Syracuse, NY 13210. E-mail: oyelmasr@syr.edu
²Associate Professor and Chair, Dept. of Civil and Environmental Engineering, Lebanese American Univ., 211 E 46th St., New York, NY 10017. E-mail: caesar.abishdid@lau.edu.lb

Abstract

Thermally induced stresses in composite steel-concrete bridges are higher than those experienced by their concrete and steel cousins, leading to significant damage in the concrete deck and corrosion of the steel reinforcement. Bridge design engineers use thermal profiles prescribed by codes such as AASHTO to predict future service stresses. A 3D finite element model is presented that investigates the temperature distribution in a case study bridge with pre-existing construction deck cracks. The non-linear transient simulation is performed using actual environmental loads for a geographic region with severe climate (North Dakota), and the resulting profile is compared to that of AASHTO. The results show the thermal gradient proposed by AASHTO to be overly conservative in cold regions. Existing models seem to ignore the nonlinearity of the thermal gradient, which can be critical for thermal stress calculations. The pre-service deck cracks appear to have a considerable effect on both the vertical and the longitudinal temperature distributions, and it is recommended that they be given careful consideration by design codes.

1. Introduction and Justification

Bridges are subjected to continuously changing diurnal environmental conditions that lead to continuous heat gain and loss with their surroundings. The thermal gradient that develops within a bridge cross section is affected by four basic heat transfer phenomena: a- convection at the surfaces, b- irradiation, c- solar radiation, and d- conduction within the bridge. While the solar radiation intensity has the highest effect on changing the bridge temperature, the thermal gradient is largely affected by the thermal diffusivity of the constituent materials. It is the difference in this thermal diffusivity of concrete and steel that makes the thermal gradient in composite steel-concrete bridges rather high. The non-uniform temperature distribution within a bridge cross section when combined with different coefficients of thermal expansion and shear connectors that prevent slip between the concrete deck and steel girders, will lead to considerable thermal stresses. These stresses are known to be relatively high when compared to service load stresses, leading to considerable damage in the concrete deck. The major damage attributed to thermal stresses is developing deck cracks. However, and despite its importance, limited studies have been dedicated to investigating the temperature distribution in composite bridges [1,2,3].

Design codes, such as the American Association for State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications, assert the importance of accounting for thermal stresses in bridge design by providing designers with proposed thermal gradients that describe...
the vertical temperature distribution in bridges located in various geographic regions [4]. However, previous studies, on which the proposed AASHTO gradient is based, mostly consider two- or one-dimensional models with, in some cases, even steady state analysis. They have thus failed to consider the effect of construction cracks, which are pre-existing in the bridge deck, on the temperature distribution within the deck in both the transverse and longitudinal directions. These cracks are found to develop directly after the concrete deck casting and before the opening of the bridge for traffic [5]. The work presented here uses a three-dimensional computational model to examine the accuracy of presently used thermal profile models in cold regions.

2. Related Work

Analytical, numerical, and experimental investigations have led to the development of various thermal profiles that have been adopted by different codes around the world. Zuk [1] developed equations to calculate the longitudinal and transverse stresses in composite bridges under different conditions of temperature and shrinkage. These equations were developed for four (4) different and critical cases of temperature distribution; however, a uniform temperature for the steel beam is adopted in all cases due to its high thermal conductivity and its ability to adjust its temperature quickly to that of the surrounding environment [1]. In a later study, Berwanger [6] developed a numerical procedure that uses two-dimensional thermo-elastic finite element analysis (FEA) to predict the transient temperature in the cross sections of composite bridges. Results showed a slower response for the concrete slab with a very rapid increase in thermal moments. The study concluded that a linear temperature profile could be used satisfactorily to represent the temperature in the transverse cross section. The study also stresses that possible existing cracks in the concrete deck were ignored.

Thermal gradients used in composite bridges differ from one code to another. Imbsen et al. [2] evaluated the thermal effects on bridge superstructures based on different codes. Many of the findings and recommendations of this study were included in the following revision of the AASHTO code: Thermal Effects in Concrete Bridge Superstructures [7]. Kennedy and Soliman [8] synthesized the various theoretical and experimental studies that had been conducted on composite concrete slab on steel beam bridges, and proposed a simple one dimensional vertical temperature distribution within the section. The distribution they proposed is uniform through the depth of the steel beam and is linear through the concrete deck. A study by Fu et al. [3] concluded that a steady-state thermal condition never exists within a bridge structure, and that the time dependency of the ambient air temperature and solar radiation would dictate a transient analysis. A more recent thermal profile was proposed by Chen [9] based on numerical analysis using two-dimensional finite element (FE) analysis. Emanuel and Taylor [10] conducted a computer-based study on composite bridges to investigate the relationship between uniform, linear, and non-linear components of thermally-induced stresses on the one hand and varying span lengths, number of spans, and support conditions on the other hand. The study concluded that the three components of thermally induced stresses are independent of the span length.

Bridge decks with overhangs present a problem for predicting the daily temperature in a cross section due to the shading effect that they will have on the steel girders—an effect that will vary between geographical locations and throughout the time of the day. An analytical parametric study was conducted by Fu et al. [3] on composite bridges to determine the effects of shading.
The study concluded the shading on the girders from the slab overhang to be the most influential factor on the vertical thermal distribution [3]. Another study confirmed the previous theoretical findings through temperature measurements on experimental bridge scaled models placed on the roof of a building as well as on an existing steel bridge in Hong Kong. [11].

Cracks occur in concrete bridge decks in different forms: transverse cracks, longitudinal cracks, and map cracks. Map cracks (also known as pattern cracks) are a very common form of cracking in all types of concrete bridge decks. Map cracks initiate at the bottom of the concrete deck and propagate their way up through the deck until they immerse at the surface where they appear like a map with squiggly lines. Such cracks are often the product of improper curing where the concrete surface moisture is allowed to evaporate too quickly, and the movement or shrinkage of the concrete deck is restrained. Studies have found transverse cracks (cracks that run perpendicular to the girders) to be the predominant form of cracking in the reinforced concrete bridge decks [5,12]. Transverse cracks have been found to be of full depth of the deck and occur at regular intervals of 0.9 to 3.1 meters apart along the bridge length [13, 14, 15], in both the positive and negative moment regions of the bridge [16]. The widths of the cracks have been reported in the range of 0.1 to 0.5 mm. These cracks have been observed along the entire length of bridges, in both simple and continuous span construction. Ramey et al. [12] noted that transverse cracks occur early during the construction process typically after the casting of the concrete, and before the bridge has been placed in service.

3. Composite Bridge Model

The City of Fargo in North Dakota (Latitude: 46°, 52’, 38”; Longitude: 96°, 47’, 22”; Elevation: 275 m.), with its extremely cold winters and very warm summers, was selected as an appropriate location for to represent cold regions. This choice is made because such extreme climate highlights the vast thermal differentials that can develop in composite bridges, and consequently produce high thermal stresses.

3.1 Time Domain

For any thermal stress calculations for bridges, two cases of temperature conditions must be considered by an engineer in order to estimate the critical stresses for design. The first case is in the summer when the deck is hotter than the steel beams, and the second case is in the winter. The time domain chosen for this transient analysis consists of two 24-hour time spans occurring over two separate days: December 23 and June 4. The selection was based on the lowest and highest radiation intensity days of the year, respectively, for Fargo, ND. The selection was based as such since it has been shown that the solar radiation has the highest influence on the thermal gradients in bridges [11]. This presents an improvement over existing models that have always assumed that the two days for the simulation that represent the critical thermal cases of the year are those with the highest and lowest ambient temperatures, where these temperatures were calculated based on existing sinusoidal empirical expressions [10]. A transient time step of one hour, divided into four (4) equal increments, was used for the thermal simulation. This choice was based on two factors: 1- the fact that weather data used is only available in increments of one (1) hour; and 2- very little temperature, radiation, and wind speed changes occurred over a time span of one hour.
3.2 Bridge Model Properties

Colquitz River Bridge was selected as a case study for its typical nature of many composite steel-concrete bridges, and due to its perfectly symmetrical cross section. The bridge is located near Victoria in British Columbia, Canada. The bridge length is 83 meters divided into five spans of varying lengths, and it has a width of 11.9 m. The concrete deck is 18 cm thick resting on six wide-flange (W33x141) steel girders equally spaced at 198 cm on center with 99 cm overhangs on each side. The material properties that are used in the model are summarized in Table 1.

<table>
<thead>
<tr>
<th>Bridge Part</th>
<th>Unit Weight (Kg/m³)</th>
<th>Modulus of Elasticity (MPa)</th>
<th>Poisson’s Ratio</th>
<th>Coefficient of Thermal Expansion (cm/cm/°C)</th>
<th>Thermal Conductivity (W/m K)</th>
<th>Grade</th>
<th>Compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>2,400</td>
<td>26,435</td>
<td>0.2</td>
<td>10.8 E-6</td>
<td>0.7</td>
<td>NA</td>
<td>28</td>
</tr>
<tr>
<td>Steel</td>
<td>7,850</td>
<td>200,000</td>
<td>0.3</td>
<td>11.7 E-6</td>
<td>42.5</td>
<td>ASTM A992</td>
<td>NA</td>
</tr>
</tbody>
</table>

Given the bridge cross-sectional symmetry with respect to the longitudinal axis, only a portion of the bridge cross section 4 m wide that includes an exterior girder and one adjacent interior girder is needed to develop the thermal profile for the full cross section. Since the thermal stresses are not sought after in this study, the full span and the corresponding support conditions are not necessary to develop the thermal profile, and a 2.75 m longitudinal segment of the bridge was modeled. The post-construction and pre-service transverse cracks in concrete decks were incorporated in the model at a spacing of 0.9 m, a width of 0.51 mm, and extending the full depth of the deck, as reported in the literature. The study assumes that the bridge is neither equipped with a waterproofing system nor with a top asphalt pavement.

3.3 Environmental Loads and Boundary Conditions

Bridges are exposed to various environmental variables that lead to heat energy exchange between their surfaces and the surrounding. This exchange is the reason behind the unsteady thermal state within the bridge cross section. The different heat transfer components acting on the boundaries of a bridge are visually depicted in Figure 1 and may be represented in terms of heat flux as

\[ q = q_c + q_r + q_s \]

where \( q \) is rate of energy transfer, J/s -1 m\(^2\) or W/m\(^2\); \( q_c \) is the rate of energy convection; \( q_r \) is the rate of thermal irradiation, and \( q_s \) is the rate of solar radiation energy.

The convection component of the heat flux equation, \( q_c \), is calculated using \( q_c = h_c (T - T_a) \), where \( h_c \) is the convection heat transfer coefficient, W/m\(^2\) °C; \( T \) is the temperature of the surface, °C; and \( T_a \) is the ambient temperature, °C. The convective heat transfer coefficient is calculated using the empirical formula suggested by Ibrahim [20] as \( h_c = 4.67 + 3.83u \) for top surfaces, \( h_c = 2.17 + 3.83u \) for soffit surfaces and \( h_c = 3.67 + 3.83u \) for steel webs and slab outer surfaces, where \( u \) is the wind speed in m/s and \( h_c \) is in W/m\(^2\). The air temperature and the wind speed data for the two chosen days were obtained for the Hector International Airport in Fargo, ND from the National Climatic Data Center [18].

The irradiation component in the heat flux equation, \( q_r \), is calculated using \( q_r = eF(T^4 - T_a^4) \), where \( F \) is the Stefan-Boltzmann constant equals to 5.671 x 10\(^{-8}\) kg s\(^{-3}\) °K\(^{-4}\); \( T \) is the temperature of the surface, °C; \( T_a \) is the ambient temperature, °C; and \( e \) is the emissivity of the surface. The
emissivity value for concrete and rusty steel and iron is reported by ASHRAE [19] to be between 0.85 and 0.95. Thus, a value of 0.9 has been assigned in this study for both materials.

\[ I_t = I_b \frac{\cos \theta}{\cos \theta_z} + I_d \left(1 + \cos \beta\right) + \rho \left(\frac{1 - \cos \beta}{2}\right), \]

where \( I_t \) is the total solar radiation; \( I_b \) is the beam solar radiation on a horizontal surface; \( I_d \) is the diffuse radiation on a horizontal surface; \( I \) is the total radiation on a horizontal surface (\( I_b + I_d \)); \( \theta \) is the angle of incidence; \( \theta_z \) is the zenith angle; \( \beta \) is the slope of the surface; \( \rho \) is the diffused ground reflection and is equal to 0.2. The expression to determine \( \cos \theta \) is

\[ \cos \theta = \sin \delta \sin \varphi \cos \beta - \sin \delta \cos \varphi \sin \beta \cos \gamma + \cos \delta \cos \varphi \cos \beta \cos \omega + \cos \delta \sin \beta \sin \gamma \sin \omega, \]

where \( \varphi \) is the surface latitude; \( \delta \) is the declination; \( \gamma \) the surface azimuth angle; \( \omega \) is hour angle. The hourly solar radiation data at the selected location for the two chosen days, including the beam and diffused components \( I_b \) and \( I_d \), were obtained from the National Renewable Energy Laboratory [21].

The hourly height of the shade created by the deck overhang on the web of the exterior girder is given by

\[ L_s = L_c \frac{\tan \theta_a}{\sin(90+\gamma - \gamma_s) \sin \delta \cos \beta \tan \theta_a}, \]

where \( L_c \) is the length of the overhang slab in cm; \( \theta_a \) is the solar altitude angle in degrees; \( \gamma \) is the surface azimuth angle in degrees; \( \gamma_s \) is the sun azimuth angle in degrees. The hourly solar altitude angle and sun azimuth angle were obtained from the National Renewable Energy Laboratory [21] for the chosen bridge location and on the two selected days in June and December. Such actual measured data considers the actual altitude of the bridge above the sea level, and how that affects the solar angles.
3.4 Finite Element Modeling

Studies on the thermal behavior of composite bridges found in the literature have conducted one- and two-dimensional FEA in which the temperature is assumed to remain constant along the length, and sometimes the transverse width, of the bridge [3,23]. The 2D model is widely accepted as an accurate approach for conducting transient heat transfer simulation in composite bridges. However, such 1D and 2D models fall short of being able to model the thermal effect of transverse cracks that have been repeatedly reported to exist prior to the bridge being put into service. A 3D model is used in this study to better reflect the effect of such transverse cracks on the temperature distribution within the bridge cross section.

The FE model of the bridge section is constructed using homogenous solid elements. Both concrete deck and steel girders were modeled using 3D 8-node linear hexahedral heat transfer mesh elements with temperature as a single degree of freedom at each node, and second-order (quadratic) interpolation in three dimensions. The refined mesh elements measured 2.25 cm, thus resulting in eight (8) elements along the depth of the concrete deck. The overall model measured 4 m in the transverse direction and 2.75 m in the longitudinal direction, with 0.51 mm wide cracks spaced at 0.9 m in the longitudinal direction and stretching the entire width of the deck. Uncoupled heat transfer analysis was conducted in which the temperature field was calculated without consideration of any stresses or deformations. The nonlinearity in the analysis is the result of the boundary conditions being nonlinear, and the latent heat properties of the elements. The cracks were modeled as an open cavity element with constant size throughout the depth of the deck equal to the width of the crack. The air inside the crack was given the same initial conditions as the air outside the crack. Natural convection and surface irradiation interaction properties are assigned to the two surfaces of each crack. Natural convection at the crack surfaces is generated by density differences in the air occurring due to the temperature gradient across the crack surface. Solar and diffused radiation were not accounted for at the crack surfaces due to its small width and the inability of the radiations to reach such surfaces as shading is provided by the adjacent surface. The contact direction with adjacent elements was modeled to be normal to the vertical surfaces of the crack, and the environmental boundary conditions (arranged in text files) are assigned to the surface elements that recall data from these files at every transient time step.

Previous studies have indicated that initial temperature appears to have very little effect on the temperature difference within a bridge deck [3,10]. Given this fact, the bridge temperature was initially set to $T_0=0^\circ C$, and the environmental initial boundary conditions were set as obtained from the National Climatic Data Center [18]. To ensure a full composite action between the concrete deck and the steel girders with no slip, a “Tie” interaction was used at the contact surface between the two components. This approach ensures a hard pressure over-closure between the two surfaces and hence permits the heat to transfer through conduction. Surface irradiation to the surrounding medium was modeled as a time dependent surface property covering the whole model with uniform emissivity distribution and hourly ambient temperature data. Three different sets of convection heat fluxes (for the top surfaces, soffit surfaces, and side surfaces) were calculated and implemented in the FE model as boundary conditions.
4. Discussion of Results

The vertical temperature distribution was obtained at four distinct critical positions within the bridge. These positions are: Position I: midway between the cracks, for the exterior girder. Position II: near the surface of the crack, for the exterior girder. Position III: midway between the cracks, for the interior girder. Position IV: near the surface of the crack, for the interior girder.

4.1 Exterior versus Interior Girders

An initial comparison of the vertical temperature distributions between the bridge sections at the exterior and interior girders revealed similar results between Positions I and III on the one hand, and Positions II and IV on the other hand. This similarity in the temperature profile was valid for both simulated days. A maximum temperature differential of 0.45 °C was reached between the two girders at time 8:00 on June 4, 2010. This small difference is due to the sun beams radiations reaching the exterior steel web soon after sunrise. Yet, the shading effect on the exterior girder for the remainder of the day results in the same temperature distribution for the two girders.

4.2 Absolute Maximum Temperature in the Section

The absolute maximum temperature in the concrete deck is critical for calculating the bridge thermal stresses resulting from the various components of thermal strains (uniform, linear, and non-linear). On June 4, the temperature in the concrete deck reached a maximum value of 38.6 °C at time 15:00 at position I. This temperature was recorded at the top surface of the concrete deck as can be seen from the color contours in Figure 8. At this same time, the temperature at position II reached a maximum value of 33.7 °C at the top end of the deck. The reason for this difference of 4.8 °C in the maximum temperature between Positions I and II is the presence of cracks, which appear to help the concrete deck surface at mid-depth to quickly adjust its temperature to the transient conditions of the surrounding. This effect is more noticeable in the summer than it is in the winter. Yet, a very high wind speed during the winter could lead to a negative gradient at the crack position (Position II) due to forced air flow through the cracks. Figure 2 provides the vertical temperature distribution at these two locations. It is worth noting that due to solar radiations reflected from the ground, the temperature at the bottom surface of the concrete deck tends to be slightly cooler right over the two steel girders than the regions between the two girders and under the overhang.

![Figure 2. Vertical temperature distribution at Positions I and II (15:00, June 4)](image-url)
The absolute maximum temperature in the steel girder will also significantly affect the thermal stresses in the bridge. On June 4, the temperature in the steel girder reached a maximum value of 28.6 °C at time 14:00 in the middle of the steel web. The maximum temperature on December 23 peaked at only -4.4 °C at 15:00 in the middle of the steel web.

### 4.3 Vertical Temperature Distribution

Results of the three-dimensional FE computations show that the vertical temperature distribution in the steel web is almost uniform for the two selected days in June and December. This temperature gradient is expected given the high thermal conductivity of steel and its ability to adjust its temperature quickly to that of the surrounding environment. The computations show that on June 4, the maximum vertical temperature differential between the steel girder and the concrete deck occurred at time 18:00 and reached a maximum value of 11 °C (positive gradient), as shown in Figure 3(a), at Position I. This value is less than half that suggested by Kennedy and Soliman [8], which is 22.2 °C. The corresponding vertical temperature difference at Position II was only 6 °C. It is interesting to note that the absolute maximum temperature differential was reached at time 11:00 inside the concrete deck, and this is due to the low thermal diffusivity of concrete. This difference between the top surface and the mid-depth of the concrete deck reached a value of 11.2 °C. In fact, the top and bottom surfaces of the concrete deck are exposed to direct and ground reflected solar radiations at time 11:00 during the heating process. These radiations, in conjunction with the low thermal diffusivity of concrete, lead to a lower temperature at mid-depth of the deck as shown in Figure 3(a). After sunset, the thermal profile in the concrete deck, provided in Figure 3(a) at time 22:00, shows that the concrete is the warmest at mid-depth of the deck during the cooling process. This happens due to both top and bottom surfaces of the deck losing heat to the surrounding by convection faster than the concrete at mid-depth. On December 23, the vertical temperature distribution is almost uniform for the entire day due to low ambient temperature and solar radiation. The maximum negative differential temperature between the steel girder and the concrete deck reached a value of 2 °C at time 7:00 at Position I, as shown in Figure 3(b). This negative thermal gradient is due to the start of the heating process after sunrise. After sunset (after 17:00), the temperature distribution in the bridge reached almost a steady state with a maximum vertical temperature difference of 0.4 °C. Therefore, the cooling process had negligible effects on the thermal gradient in December under normal environmental conditions.

![Figure 3. Vertical temperature distribution at Position I (a) (June 4), (b) (7:00, December 23)](image-url)
4.4 Comparison with Existing Models

The AASTO LRFD Bridge Design Specification [4] provides provisions for the vertical temperature gradient in composite bridges. AASHTO LRFD [4] recommends that the vertical temperature distribution be uniform in the steel girders and linear in the superimposed concrete deck. The temperature differential in the concrete deck is based on the map of solar radiation zones in the United States, in which the city of Fargo, ND is located in zone 2. For zone 2, the AASHTO LRFD Bridge Design Specification [4] recommends a maximum positive vertical temperature differential of 18.9 °C between the top and bottom surfaces of the concrete deck, with a linear profile in between. The AASHTO temperature gradient and that proposed by Kennedy and Soliman [8] are compared to that computed using the thermo-elastic analysis of the FE model in June, as shown in Figure 4.

![Figure 4. Vertical temperature distribution at position I on June 4 at: (a) (18:00), (b) (11:00)](image)

Figure 4(a) shows that the shape of the obtained vertical temperature distribution is very similar to the one proposed by AASHTO and by Kennedy and Soliman [8]; however, the AASHTO specification provides a maximum temperature differential of at least 7.9 °C higher than the one obtained in this study at Position I (almost 13 °C at Position II). This difference is even more for the model proposed by Kennedy and Soliman [8], which reaches 11 °C at Position I and about 16 °C at Position II. This very conservative approximation adopted by AASHTO leads to considerable error in assessing the thermal stresses in composite steel-concrete bridges in locations as the one considered here.

Figure 4(b) shows a comparison of the vertical temperature distribution proposed by AASHTO and by Kennedy and Soliman [8] to that obtained in this study at time 11:00. The relative error between the two maximum temperature differentials in both profiles (AASHTO and proposed) is 7.7 °C. Another issue to consider is the nonlinearity of the thermal profile when compared with the linear distribution proposed by AASHTO. In fact, the vertical temperature distribution in the concrete deck was close to being linear only in the afternoon between time 14:00 and 19:00, and nonlinear for the remainder of the day. This nonlinearity in the temperature distribution will produce a nonlinear thermal strain component and its effects on the design of the bridge require further investigation.
It is therefore evident that the vertical thermal gradient proposed by AASHTO for the considered climate is conservative in the concrete deck due to several reasons: a) AASHTO completely ignores the effect of the depth of the concrete deck and its effect on the maximum temperature differential. b) The thermal profile prescribed by AASHTO is based on a one dimensional conduction analysis, and thus the solar radiation reflected from the ground is ignored. This reflected radiation decrease the positive thermal gradient through heating the bottom surface of the concrete deck. c) The AASHTO model does not take the overhang-to-depth ratio into consideration. This ratio has an effect on the maximum temperature differential of the critical exterior beam through the shading effect. d) A linear vertical temperature distribution is assumed by AASHTO in the concrete deck at all times. e) AASHTO does not account for the effect of pre-existing transverse construction cracks in the concrete deck, whose effect tends to lessen the average temperature differential between the concrete deck and the steel beam.

The AASHTO LRFD [4] seems to provide a simple and general thermal gradient for composite bridges for various geographic regions. However, it is shown here to be overly conservative and simplistic. By incorporating the aforementioned points, the AASHTO model can be revised to obtain a more accurate thermal gradient and the corresponding maximum temperature differentials. This accuracy will help designers achieve a more optimal design by better estimating and accounting for the effects of the thermal stresses in composite bridges.

4.5 Effects of Transverse Deck Cracks

The computations performed show that the deck transverse cracks have an apparent and significant effect on the thermal profile of the composite bridge. This effect is more noticeable in the summer than it is in the winter. Yet, a very high wind speed during the winter could lead to a negative gradient at the crack position due to air flow through the cracks. This effect manifests itself in a reduction of the positive thermal gradient between the top and bottom of the concrete deck, and can be demonstrated by plotting the thermal profiles at Positions I and II or by examining the temperature contours for the positive and negative thermal gradient cases in Figure 5. The variation in the temperature differentials between Positions I and II ranges between 3.1 and 5.6 °C in June. The highest difference of 5.6 °C is reached at time 21:00 during the cooling process of the bridge. In fact, the bridge will experience faster cooling near the cracks due to the faster heat loss by convection at the crack surface.

**Figure 5.** Temperature distribution at (a) 20:00, June 4, (b) 10:00, December 23
The vertical temperature distribution in the concrete deck at Position I was nonlinear for most of the day, while that at Position II has a much lower degree of nonlinearity due to the crack’s effect in helping the concrete to quickly adjust its internal temperature to the ambient conditions.

Various codes and researchers ignore the temperature differentials in the longitudinal direction of the bridge and assume them to be negligible when compared with the vertical temperature distribution. The difference in the longitudinal temperature distribution is said not to exceed 15% of the vertical temperature differential suggested by AASHTO [4]. The computations presented here however indicate a maximum temperature differential of 6.3 °C in the longitudinal direction of the bridge when compared to the vertical temperature differential of 9.4 °C. The longitudinal temperature differential thus constitutes more than 67% of the vertical temperature differential, and the effect of this nonlinear longitudinal thermal gradient on the development of cracks and nonlinear strains requires further assessment and cannot be treated in a trivial manner.

5. Conclusions and Recommendations

The effect of pre-existing transverse construction cracks in the concrete deck on the thermal gradient in composite steel-concrete bridges was investigated using a non-linear transient thermal analysis in 3D FE formulation. The following main conclusions are reached: 1) The maximum positive vertical temperature differential between the concrete deck and steel girder reached a maximum value of 11 °C in the summer, while vertical thermal gradient is almost uniform for the entire day in winter, under normal environmental conditions. 2) For composite bridges in geographic regions having climates similar to the one considered in this case study, the AASHTO LRFD Bridge Design Specification is overly conservatively and overestimates the vertical thermal gradient, which leads to significant error in assessing the thermal stresses in composite steel-concrete bridges. The current AASHTO model is based on the 1989 guide specifications, and needs to be updated with new findings taken into considerations. 3) The FE model analysis show a nonlinear vertical temperature distribution in the concrete deck when compared to the linear distribution proposed by AASHTO and other previously-suggested models. This nonlinearity will create a nonlinear strain component that will create stresses currently not considered by designers. 4) The longitudinal thermal gradient is more than 67% of the vertical one. Its impact on the self-equilibrating transverse stresses in the bridge due to sectional continuity, Poisson’s effect, and displacement compatibility at the interface of consecutive sections needs further assessment. While the conclusions arrived at in this study can be generally applicable to similar bridges in somewhat similar climates, they remain specific to the bridge case under investigation. More numerical and experimental studies should be conducted on bridges under different weather conditions to draw a better assessment of the accuracy of currently used thermal gradient models, such as that of AASHTO in more moderate climates.

References


