Temperature-Dependent Bridge Movements

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Abstract: The response of bridges exposed to thermal environment conditions is studied. Analytical methods are developed to obtain temperature distributions and the maximum bridge temperature ranges. Thermoelastic analysis is conducted to obtain the temperature induced movements and the associated stresses in bridges. Parametric studies are conducted for different bridge geometries, temperature distributions, and support conditions. A field test is conducted on a bridge to verify the analytical models.

The analysis suggests that concrete bridges sometimes are designed for smaller temperature ranges and corresponding thermal movements than may be expected in practice. The analysis suggests that steel bridges with composite concrete decks sometimes are designed for larger temperature ranges and thermal movements than may be expected at many locations in the United States. Skew and curved bridges often exhibit magnitude and direction of movement that is different than commonly expected. Additional guidelines and recommendations are provided regarding the analysis required to accurately predict thermal movements and the placement of bearings and expansion joints. This study will help engineers better understand thermal movements and stresses developed in bridges.

Introduction

Temperature variations cause bridges to expand and contract. The movements are often accommodated by the bearings and expansion joints. If any component of the movements is restrained, large forces may develop in the structure, and these forces sometimes cause damage (Moorty and Roeder 1990). Integral abutment bridges and bridges with few joints have become popular. The movement in these bridges is accommodated by deflection of the substructure, or the bridge components are designed to withstand the large forces that may result due to restraint of the movements.

Accurate prediction of thermal movements or the associated forces is important during the design of bridges. The American Association of State Highway and Transportation Officials (AASHTO) specification (*Standard* 1985) has provisions for predicting longitudinal thermal expansion in bridges, which is given by $\alpha L \Delta T$, where α is the coefficient of expansion, L is the length of the bridge, and ΔT is the range of mean bridge temperature. The temperature range specified by AASHTO is based on two climates: moderate and cold. The temperature ranges of concrete and steel bridges for each of these climates is specified. This method of predicting bridge temperature ranges and the resulting movements does not recognize the full complexity of the thermal behavior of bridges.

A theoretical method of predicting bridge temperatures and the resulting movements is presented in this paper. Heat-transfer analyses are performed, and the transient temperatures are computed for different types of bridges in different locations in the United States. These predicted temperatures are then correlated to observed behavior and the AASHTO design method,

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and they are used to predict temperature-dependent movements. Skew and curved bridges are emphasized, because the research shows that these movements may be more complex and quite different from that suggested by the AASHTO method. A series of field measurements is then described, and the results are compared with the theoretical predictions.

The paper provides an improved understanding of thermal movements in bridges. It shows that the AASHTO method provides a reasonable prediction of thermal movements in many bridges, but may be unrealistic for some others. It provides guidelines for determining which bridges are not easily handled by the AASHTO method, and it describes analytical methods that can be used with them. Design recommendations that help minimize problems with thermal movements are presented.

Heat-Transfer Analysis

The first step in analyzing thermal movements is determining the temperatures within the bridge. Structures exposed to the atmosphere are subjected to an exchange of heat energy between their surfaces and their surroundings. The three principal mechanisms of heat transfer in a bridge are radiation from the sun and reradiation from the surface, convection with the ambient air, and conduction to and from the surroundings and through the structure and the substructure. The Poisson equation models the transient heat flow process and is given by

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

where ρ = the density; c = the specific heat; k = the conductivity; T = the temperature of the material; t = the time; and x, y, and z = the coordinates along the length, depth, and width, respectively, of the bridge. The boundary conditions are those of insolation, convection, and reradiation. The boundary conditions are written as

$$\eta I_n - h_c(T_s - T_a) - \varepsilon\sigma(K_s^4 - K_a^4) + k_n \frac{\partial T}{\partial n} = 0 \qquad (2)$$

where η = coefficient of absorptivity; I_n = insolation at boundary, W/m^2; T_s = temperature at surface, °C; T_a = ambient temperature, °C; K_s = absolute temperature at surface, °K; K_a = absolute ambient temperature, °K; k_n = conductivity of material in direction normal to surface, W/(m °C); h_c = coefficient of convective heat transfer, W/(m^2 °C); σ = Stefan-Boltzman constant = 5.67×10^{-8} W/(m^{2}K^4); and ε = emissivity.

The reradiation heat transfer can be expressed as

$$\varepsilon\sigma(K_s^4 - K_a^4) = \varepsilon\sigma(K_s + K_a)(K_s^2 + K_a^2)(T_s - T_a) = h_r(T_s - T_a) \qquad (3)$$

where h_r = the radiative heat-transfer coefficient and is equal to $[\varepsilon\sigma(K_s + K_a)(K_s^2 + K_a^2)]$. The combined heat-transfer coefficient is $h = (h_c + h_r)$ and its value at the top of the deck is given by the empirical formula

$$h = 13.5 + 3.88u \qquad (4)$$

where u = the wind speed in m/s at the bridge site (Potgeiter 1983). The heat-transfer coefficient below the deck is taken to be half of that at the top, since this was recommended by past research (Potgeiter 1983; Thepchatri et al. 1977). The boundary condition then becomes

$$\eta I_n - h(T_s - T_a) + k_n \frac{\partial T}{\partial n} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

The solar radiation incident on a surface consists of direct radiation and diffusion radiation, which is analytically obtained by the method outlined in Moorty (1990). The latitude, the angle of declination, the solar zenith angle, the angle of tilt, and the wall azimuth angle of the surface are required to determine the insolation at a point on the surface of the earth. The developed algorithms also take into consideration the cloud cover. The hourly ambient temperature distribution is estimated by interpolating a sine curve between the maximum and minimum temperatures for the day (Elbadry and Ghali 1983). The daily average wind speed and the daily maximum and minimum temperatures are obtained from weather data.

A one-dimensional transient heat-transfer analysis along the depth of the superstructure is conducted using a finite difference formulation of (1) with the Crank-Nicholson implicit scheme (Potgeiter 1983). A two-dimensional analysis along the bridge cross-section is conducted using the finite element method with ANSYS (*ANSYS* 1987). Isoparametric quadrilateral thermal shell elements (STIF 57) were used to model the deck, concrete T-beams, and box girders. Two-dimensional conducting bar elements (STIF 32) were used to model the steel girders. The convection boundary conditions were achieved using convection links (STIF 34). The transient analysis was started at 7:00 A.M. in the summer and 6:00 P.M. in the winter, and the initial temperature in the bridge was considered to be uniform and equal to the ambient temperature at that time. The analysis was conducted for four days in order to eliminate the effect of the assumed initial conditions. The analyses were performed for a typical bridge with concrete box girder, concrete T-beam, and steel girder with a composite deck. The one-dimensional model overestimated the temperature in the webs of concrete bridges by 3–5°F (+1.5 to +2.7°C). The temperature distributions obtained from the one-dimensional and two-dimensional models are nearly the same for composite steel bridges. The computed temperatures were compared with the one series of field observations (Thepchatri 1977) reported in the United States, and the theoretical prediction compared well with these field measurements. The calculated temperatures were also compared with past theoretical studies, and the comparison lends additional credibility to the theoretical predictions. However, most of the prior work has focused on temperature gradient and thermal stress rather than bridge movements. The temperature conditions that cause maximum movement are different from those that cause maximum thermal stress, since they typically occur at different times. Thus, direct comparison of computed data with past temperature profiles is not meaningful.

Emerson (Emerson 1976; Black and Emerson 1976) noted that the mean bridge temperature governs the longitudinal movement of the bridge. A study was conducted to obtain the maximum and minimum mean bridge temperature, and the maximum mean bridge temperature range in different types of bridges at 11 different locations in the United States. These stations were selected to provide a wide range of possible temperatures in the country. The days of maximum and minimum temperatures in the past 50 yr were used, and clear sky conditions were assumed for these days. Wind speed of 7 mi/hr (3 m/s) in the summer and 8 mi/hr (3.6 m/s) in the winter

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and an absorptivity coefficient of 0.65 were used. These calculated temperature ranges were compared to the temperature ranges specified by AASHTO.

The maximum range in the computed mean bridge temperature as a function of the maximum range in the ambient temperature is shown in Fig. 1. The maximum mean bridge temperature range is higher for composite bridges than concrete bridges. The AASHTO specification recommends a temperature range of 120°F (67°C) for metal structures and 70°F (39°C) for concrete structures in moderate climate. In cold climates, a temperature range of 150°F (83°C) for metal structures and 80°F (44°C) for concrete structures is recommended. The temperature ranges shown in Fig. 1 are exceeded sometimes for the concrete bridges, while those of the steel and composite bridges are well within the AASHTO-recommended ranges. This suggests that the temperature range used by AASHTO for concrete bridges may be smaller than required, but it must be noted that concrete bridges are also designed for additional movement due to creep and shrinkage. This additional movement requirement may compensate for any deficiency in the AASHTO design method. The AASHTO limits for some steel bridges may be too large for many locations.

Thermal Movements in Skewed Bridges

The mean bridge temperature ranges provided by AASHTO are commonly used to predict longitudinal movement, but true thermal movements are often more complex (Roeder and Moorty 1991). In particular, large transverse movements in addition to the longitudinal ones were noted for some skew bridges. Therefore, a parametric study that considered the skew angle, width and length of the bridge, and the expansion joint characteristics was performed. The thermoelastic analysis used the finite element method with ANSYS version 4.3A and 4.4. The deck was modeled using rectangular plate elements, and the girders were modeled with three-dimensional beam elements (STIF 4). The bracings were modeled by three-dimensional truss elements (STIF 8). The deck and the girders were rigidly connected by

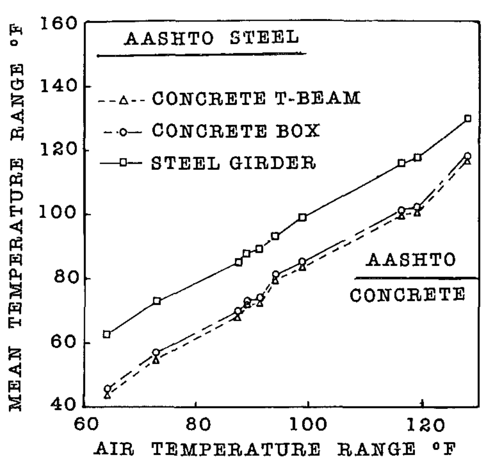

FIG. 1. Maximum Mean Bridge Temperature Range versus Maximum Ambient Temperature Range (°F = 1.8°C)

constraint equations. The movement of the bearings and expansion joints used a slip-stick mechanism as observed by Emerson (1976). That is, the bearings and expansion joints do not move until a slip force is reached, and they move freely after this force is achieved. Nonlinear spring-slider elements (STIF 40) model this behavior.

The geometry of the basic bridge model for the parametric studies is shown in Fig. 2. The bridge is a three-span, composite, steel-girder bridge, oriented east to west. The skew angle, length, and width are varied. The eight girders running along the length of the bridge rest on fixed pin bearings at the west abutment and on rocker bearings, which allow movement only in the longitudinal direction at the two interior piers and the east abutment. There are expansion joints at the two ends of the bridge deck in the basic model, but other analyses that constrained the deck displacement at the fixed end (west end) were also performed. The temperature distribution shown in Fig. 3 is typical of that obtained in this study in the thermal analysis of composite bridges and was used for the movement calculations. The thin

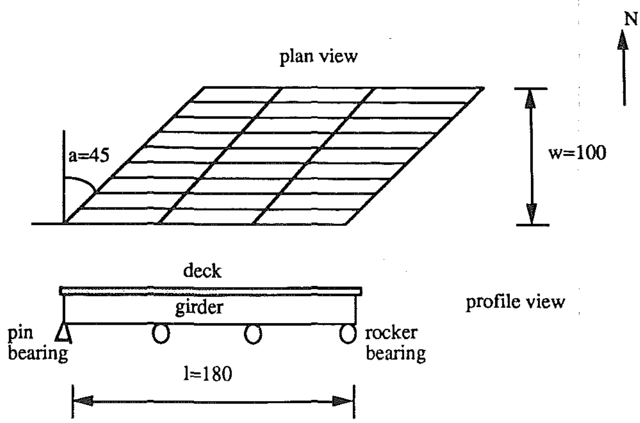

FIG. 2. Model of Bridge (All Dimensions in ft; 1 m = 3.2787 ft)

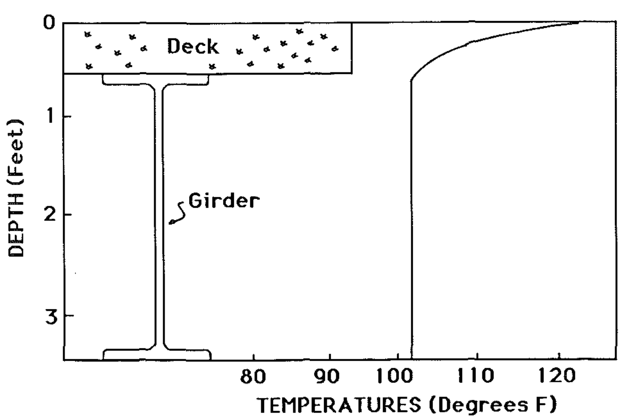

FIG. 3. Temperature Distribution along Depth (°F = 1.8°C + 32)

steel girders have high conductivity and their temperature is mainly determined by convection. The temperature in the girders is nearly uniform and equal to the ambient temperature. This temperature distribution is similar to that suggested in other work (Kennedy and Soliman 1987), but it is also a bit different because it occurs at a different time of the day. Maximum bridge movements tend to be associated with mid-to-late afternoon summer temperatures and early morning winter temperatures. Maximum temperature gradients and thermal stress effects occur at different times.

The computed longitudinal displacements of the deck for a right bridge and a 45° skew bridge are shown in Fig. 4. The longitudinal displacements of a right bridge are uniform along the cross section. However, as the skew angle increases, the displacements are no longer uniform, and maximum longitudinal displacement is along the long diagonal at the northeast corner of the bridge. The transverse displacements of the deck for a right bridge and a 45° skew bridge are shown in Fig. 5. The transverse displacements of a right bridge are symmetric about the longitudinal axis. However, as the skew angle increases, the maximum transverse displacements are along the long diagonal. The maximum transverse displacement is at the fixed end of the bridge, at the southwest corner. These figures clearly illustrate the large transverse movements that are noted in practice (Roeder and Moorty 1991) but not predicted by simple, one-dimensional design methods.

The longitudinal and transverse displacements increase with an increase in the skew angle of the bridge as shown in Fig. 6. The displacements are normalized with respect to the length of the bridge. Since there is a rigid connection between the deck and the girders, the increase in the transverse displacements of the deck is accommodated by an increase in the rotations of the girders at that location. The bottoms of the girders are restrained against transverse movements. Hence, the increase in transverse movements with an increase in the skew angle results in an increase in the temperature-dependent forces in the girders and the bearings. The transverse movements are smaller than the longitudinal movements, but they are particularly damaging to the bridge because they are not accommodated by the bearings.

An increase in temperature results in uniform free expansion in all directions. Therefore, the expansion in the transverse direction can be relatively large in short, wide bridges. Many types of bridge bearings allow movement only in the longitudinal direction, and so a portion of the transverse movement is restrained, and large forces develop in the bearings and the girders. Short spans are particularly susceptible to large forces because of their greater stiffness. The transverse movements of the deck are accommodated by the rotations about the longitudinal axis of the girders. The transverse movements and, as a result, the girder rotations increase with an increase in the width of the bridge.

In prior calculations, the bearings were assumed to be in good condition and offered small resistance to longitudinal movement (about 10% of the dead load on the bearings). However, the bearings and expansion joints may offer large resistance to movement due to deterioration. The effect of the resistance to movement offered by the bearings and the expansion joints was examined by varying the slip force between 0% and 80% of the dead load on the bearing. Increased bearing stiffness also has a parallel in that it approaches the condition of integral construction that is sometimes used in the United States. As the bearing resistance is increased, there is an increase in the transverse movement along the long diagonal and the forces in the girders and the bearings increase considerably. The increase in the

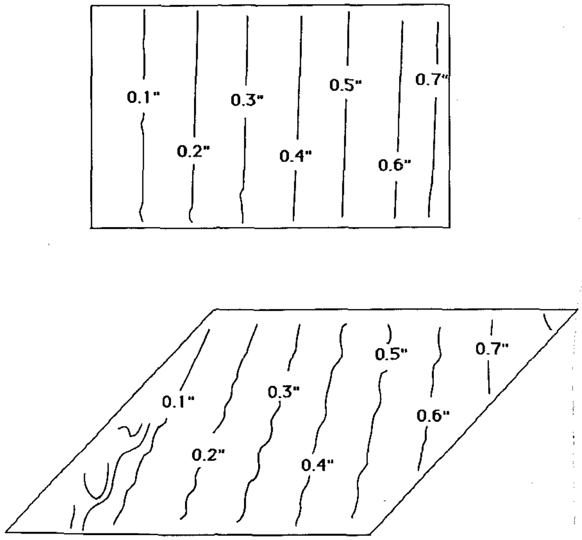

FIG. 4. Longitudinal Displacements (in in.) of Deck for Normal and 45° Skew Bridge (1 in. = 2.54 cm)

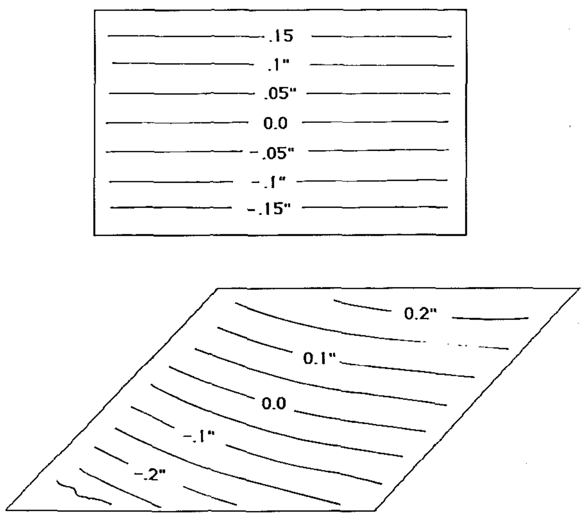

FIG. 5. Transverse Displacements (in in.) of Deck for Normal and 45° Skew Bridge (1 in. = 2.45 cm)

resistance to movement of the expansion joints also results in an increase in the transverse movements, a reduction in the longitudinal movements, and an increase in the stresses in the deck and the bearings.

Some bridge bearings, such as elastomeric bearings or unguided sliding surfaces, do not provide resistance to transverse movement. An analysis was conducted that used longitudinal and transverse bearing stiffness in the

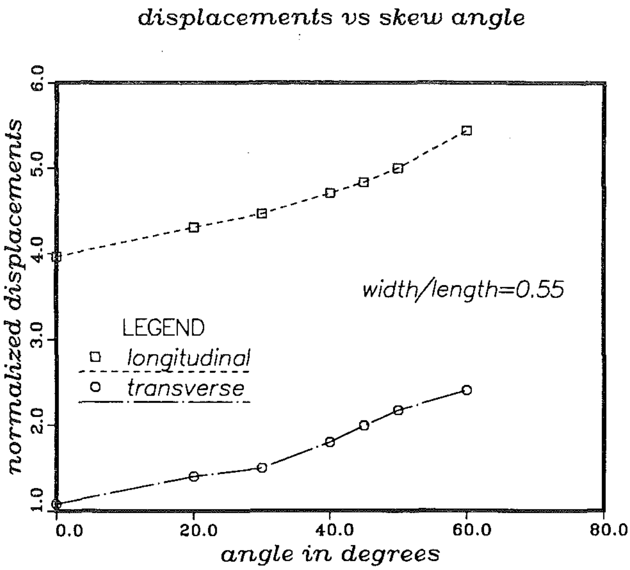

FIG. 6. Maximum Normalized Longitudinal and Transverse Displacements versus Skew Angle.

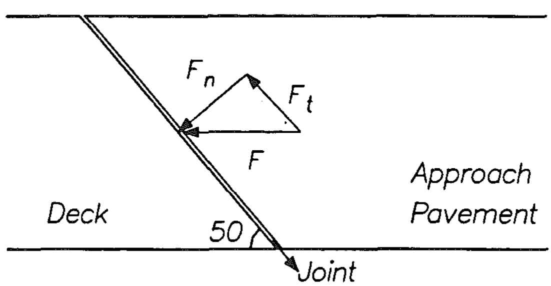

FIG. 7. Forces on Deck due to Resistance Encountered in Expansion Joints

order of that achieved with elastomeric bearings. The transverse displacements are larger and the longitudinal displacements are smaller than the case when rocker bearings or other guided bearings were used. Since the potential transverse movement is permitted, the forces in the girders and the bearings are smaller than when guided bearings were used. Hence, elastomeric bearings or those with unguided sliding surface may be a better choice for sharply skewed, short, wide bridges.

Resistance encountered in expansion joints or support locations can dramatically increase the transverse movement of skew bridges (Moorty and Roeder 1990). This occurs because points of resistance develop compressive force, F, across the skew angle as depicted in Fig. 7. Compressive force, F_n, is developed in the normal direction because pavements and granular material have limited shear strength, and the transverse component of this force tends to drive the bridge toward larger transverse movement.

Several analyses were performed including the stiffness of the piers. Pier stiffness did not have a great impact on the movements noted for relatively short skew bridges that experience large transverse movements. Pier stiffness has a greater impact on the movements noted for longer bridges, and it may influence the forces and distributions of force noted within the structure.

Thermal Movements in Curved Bridges

The thermal movements in curved bridges are complicated by virtue of their geometry. In addition, engineers do not have a uniform way of placing the bearings. Some engineers orient the bearings along the tangent to the curve of the bridge, while others orient them on the chord from the fixed point of the bridge. A study was conducted to better understand the thermal movements in curved bridges. Thermal-structural analyses were done to determine the effect of various geometric parameters, orientation of the bearings, and the stiffness and resistance of the substructure on the thermal response of curved bridges.

A three-span, curved, steel-girder bridge, oriented north to south, was used for the studies. The basic structure had four steel girders supporting the deck that rest on fixed pin bearings at the south abutment and on guided bearings at the two interior piers and the north abutment. There are expansion joints at the north and south ends of the deck. A temperature distribution similar to that shown in Fig. 3 was used as the thermal load. The fixed location of the curved girders is often at a pier that is not rigid. Several analyses were conducted to investigate the effect of the location of the fixed point. The effect of bearing orientation was also studied in various analyses. The angle of curvature of the bridge is 40° and the length of the bridge is 600 ft (183 m) for this study.

The first analysis was conducted with guided bearings that allow movement only in the tangential direction. The displaced shape of the bridge and the radial and tangential displacements of the deck when the bearings are oriented along the tangent are shown in Fig. 8. There is significant radial movement at the center of the bridge, which is accommodated by the radial displacement of the piers. There is some tangential movement of the piers as well.

The displaced shape and the radial displacements of the deck for guided bearings along the chord are shown in Fig. 9. The tangential displacements are nearly the same for both the bearing orientations. The displacements at the two piers and the north abutment are nearly along the chord from the fixed point when the bearings are oriented along the chord. Hence, the bearing orientation along the chord from the fixed point is a better choice when the fixed point is at a rigid support such as an abutment. However, when the fixed point is at a pier, the deflection of the pier complicates the bridge movement, and the movement is not along the chord from the fixed point at some locations. In such a situation, elastomeric bearings or bearings with unguided sliding surfaces may be a better choice.

The effect of the relative stiffness of the bearings and the stiffness of the piers was also examined. When the stiffness of the bearings is large compared to the stiffness of the pier, all the tangential and radial movement of the superstructure is accommodated by the displacement of the piers. With flexible bearings, the tangential movement is accommodated by the rolling of the bearings. The tangential displacement is nearly the same for different bearing and pier stiffness, but the radial displacements increase with an

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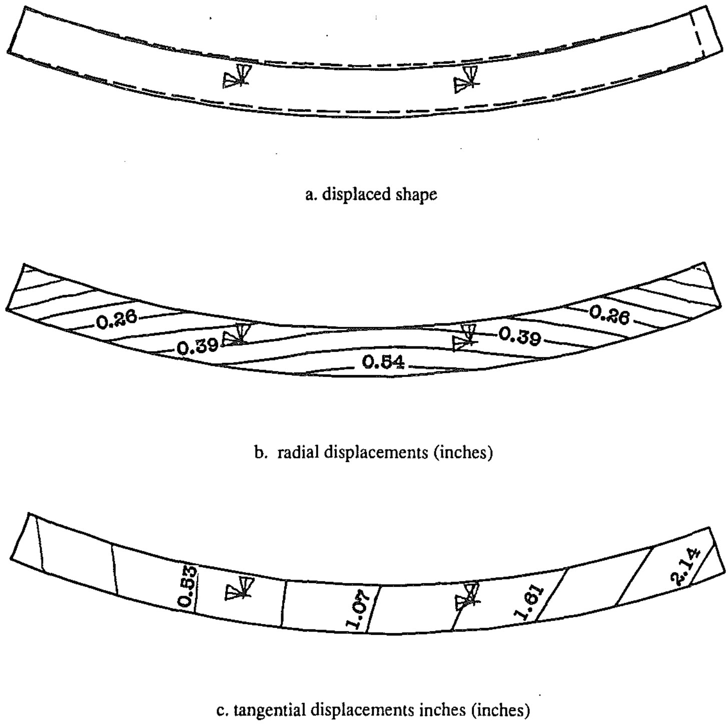

FIG. 8. Displaced Shape and Radial and Tangential Displacements of Deck when Guided Bearings are Oriented along Tangent (1 in. = 2.54 cm) (*a*) Displaced Shape; (*b*) Radial Displacements (in.); and (c) Tangential Displacements (in.)

increase in the relative stiffness and resistance of the bearings. The forces in the bearings, girders, and the piers also increase with an increase in the stiffness and the resistance of the bearings.

The effect of the angle of curvature on the thermal response of curved bridges was also examined. If L is the arc length of the bridge, and R is the radius, then the angle of curvature is given by L/R. The radial displacements increase with an increase in the angle of curvature as shown in Fig. 10. The displacements are normalized with respect to the radius of the bridge. Set I has a larger bearing stiffness and bearing resistance than set II. The forces in the girders and the bearings also increase with an increase in the angle of curvature.

Field Test

The analytical studies indicate that the thermal movements of bridges depend on the bridge geometry and the support conditions. A field test was conducted on the Sutton Creek Bridge, in the Kootenai Forest in Montana, and the results were compared with those of the analytical models. The field test consisted of measuring the bridge temperature and the ambient temperature, estimating the wind speed and cloud cover, and measuring the bridge movements over a three-day period. The measurement program was

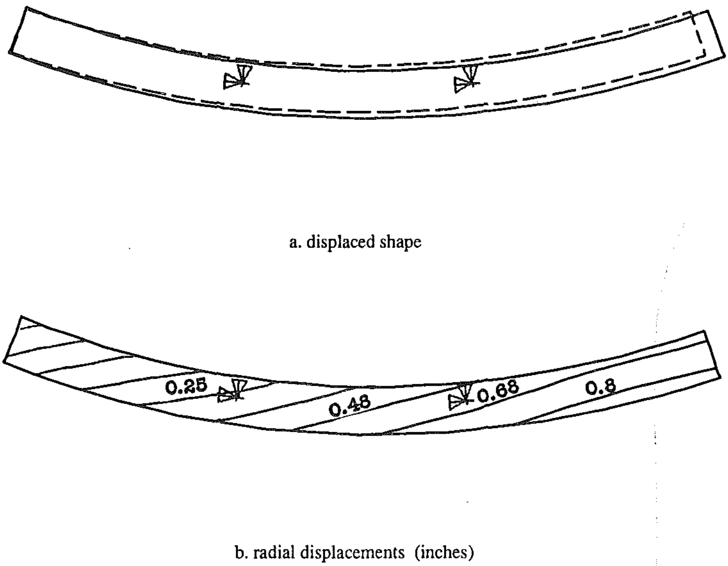

FIG. 9. Displaced Shape and Radial Displacements of Deck when Bearings Are Oriented along Chord (1 in. = 2.54 cm) (*a*) Displaced Shape; and (*b*) Radial Displacements (in.)

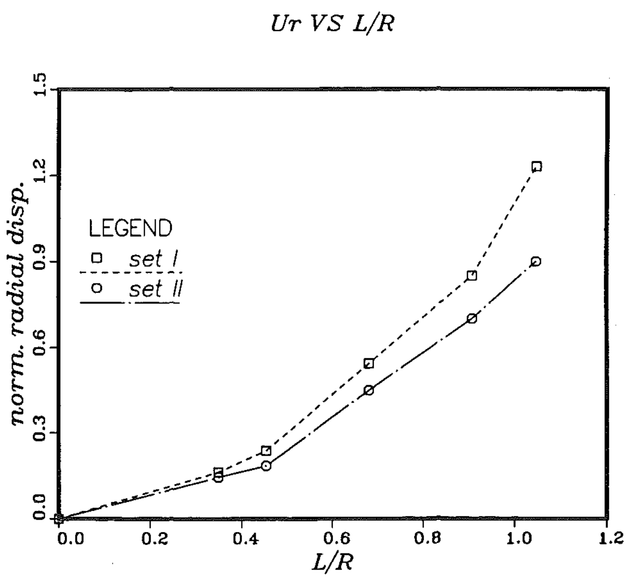

FIG. 10. Maximum Normalized Radial Displacements versus Angle of Curvature

a very economical experimental program, and so the data were taken only at critical times and locations.

The Sutton Creek Bridge is a continuous three-span curved steel-girder bridge. It spans a total arc length of 657.5 ft (200.4 m) north to south and

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has a radius of curvature of 881.5 ft (269 m). The first three spans from the south of the bridge are continuous, while the fourth span at the north is simply supported. The first pier from the south (pier 1) is 85 ft (26 m) tall, and the third pier (pier 3) is 75 ft (23 m) tall. The center pier (pier 2) is quite slender and is 200 ft (70 m) tall. The girders rest on fixed pin bearings at the north abutment and at pier 1, and on rocker bearings on the south abutment, pier 2, and pier 3. All the bearings are oriented tangential to the bridge curve, allowing movement only in the tangential direction. There are preformed elastomeric expansion joints at the north and south abutment and also between the continuous and simply supported span at pier 3.

The temperature measurements of the bridge were made using a type-K surface probe and type-K thermocouples. The temperature readings were taken with a hand-held thermometer that has an accuracy of 1°F (.5°C). The wind speed and cloud cover were estimated from local newspapers and radio stations. The movement measurements were made using a theodolite and and electronic distance measurement device. The measurements were made from two stations, *S*1, on the east side of the north abutment, and *S*2, on the west side of the north abutment. Targets were on the pier, the bottom flange of the girder, and the top of the jersey barrier at the location of piers 2 and 3. A drawing of the placement of the targets at pier 3 is shown in Fig. 11(*a*). The targets on the east side of the bridge (P-series) were used by station *S*1, while the targets on the west side (T-series), were used by station *S*2. The placement of the targets on the east side of the bridge are shown in Fig. 11(*b*). Control points were at the north abutment and at the bottom of pier 1. These control points are essential because they allow an evaluation of the accuracy of the measurements.

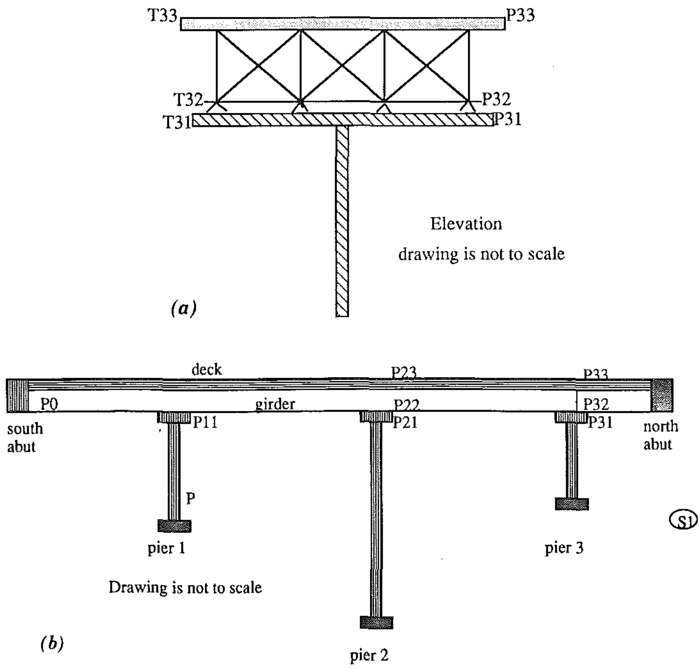

FIG. 11. (*a*) Targets at Pier 3; (*b*) Location of Targets along East Edge

The field test was conducted July 26–28, 1990. The 26th was a cloudy day with occasional rain. The sky cleared up by the afternoon on the 27th. The clear sky conditions continued on the 28th except for short periods of clouds and rain in the afternoon. The ambient temperature on the 26th varied between 54°F and 58°F (12°C–14.4°C). The maximum ambient temperature was at 3:00 P.M. on the 28th and was 93°F (34°F). The traffic on the bridge was very light, and dynamic vibration due to traffic loading was not being measured during these field observations.

Heat-transfer analysis was conducted using these environmental variables. Since the bridge is oriented north-to-south, the girder along the west edge experiences direct radiation in the afternoon and, hence, has a higher temperature than the other girders in the afternoon. The direct radiation on the girder and the amount of exposure was computed based on the solar angles, the azimuth angle of the surface, the depth of the girder, and the length of the cantilever slab. The insolation on the girder is obtained in a similar manner as described in Elbadry and Ghali (1983) and Potgeiter (1983). The calculated and measured temperatures at the top and bottom of the deck are shown in Fig. 12. The difference in the calculated and measured values is small, but there is a time phase difference in the measured and computed values. This difference can be attributed to the presence of rain water on the deck and the local shading due to the mountainous terrain, which was not included in the analytical model.

The measured and analytically obtained tangential and radial displacements at pier 1 and pier 2 are shown in Figs. 13 and 14. The experimental results show that the movements are accommodated largely by pier deflection rather than the bearings. They also show that large radial movements occur as predicted in the analyses. Significant deflection of the nominally fixed pier is also noted. The error in the movement measurements increases with the distance, since a small error in the angle measurements results in a large error in the displacements at larger distances. The maximum possible error in the measurement is around 0.12 in. (3.1 mm) and occurs at the south abutment. There is good agreement between the measured and analytical values, which gives credibility to the analytical models. The movements observed in this study are the movements caused by a daily (diurnal) temperature cycle. Much larger movements could be expected if the annual

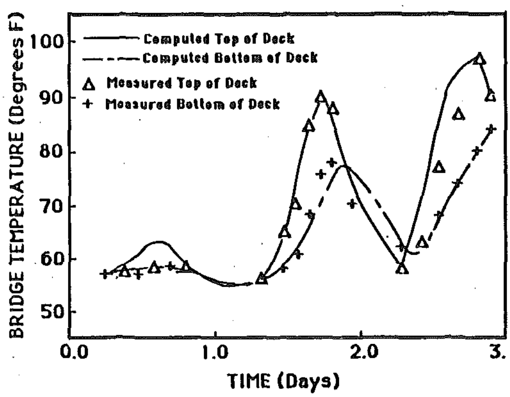

FIG. 12. Meaured and Analytical Temperatures at Top and Bottom of Deck (°F = 1.8°C + 32)

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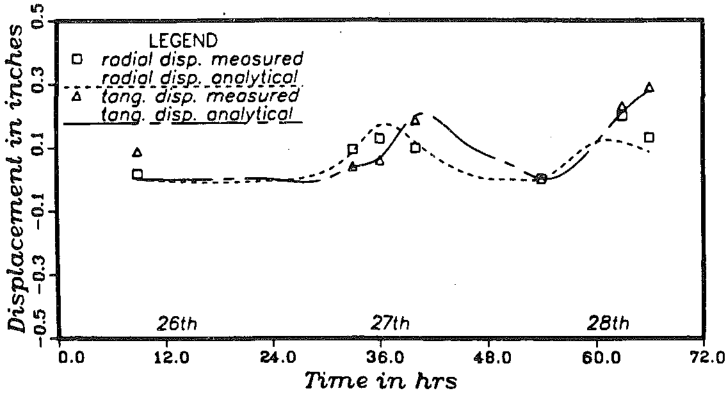

FIG. 13. Radial and Tangential Displacements of Pier 1 (1 in. = 2.54 cm)

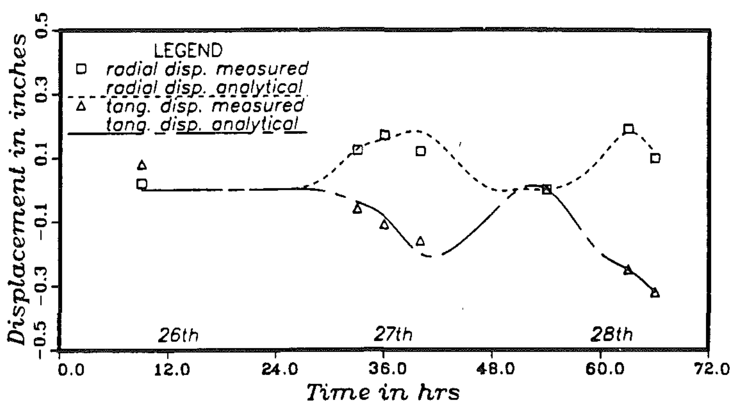

FIG. 14. Radial and Tangential Displacements of Pier 2 (1 in. = 2.54 cm)

temperature cycle was considered. Additional experiments to measure annual temperature effects are needed.

Practical Implications, Conclusions, and Recommendations

Predicting the thermal movements in bridges can be quite important to bridge design. There are numerous examples of bridges that were severely damaged when the movements were not properly accommodated by bearings, expansion joints, or appropriate integral construction techniques. A number of conclusions can be drawn from the paper, and these conclusions may have considerable practical implications if used in design. These include:

1. A one-dimensional analysis is adequate for obtaining temperature distributions in composite bridges, but it predicts higher temperatures of 3°–5°F (1.5°C to 2.7°C) in the webs of concrete bridges. When side heating of the webs due to direct insolation occurs, two-dimensional analysis is required.
2. The bridge temperature ranges specified by AASHTO are exceeded in

certain regions of the United States for concrete bridges, while they may be too large to steel and composite bridges for some locations.

3. The simple method of predicting thermal movements recommended by AASHTO is quite reasonable for straight orthogonal bridges, but more refined analysis may be required for skew and curved bridges.

4. The transverse movements and stresses in bridges increase with an increase in the skew angle and the width of the bridge. Hence, sharply skewed, short-span, wide bridges are more susceptible to damage.

5. Bearings with unguided sliding surfaces that allow movement in the longitudinal and the transverse direction are often a better choice for both curved bridges and sharply skewed bridges that have a potential for transverse movements.

6. Increase in the curvature of the bridge results in an increase in the radial movements and the stresses in the bridge. Tangential and radial movements in curved bridges are sensitive to the relative stiffness of the bridge, the bearings, and the substructure. Movements are sometimes accommodated by deflection of the piers.

7. Bearings oriented along the chord from the fixed point of a curved bridge are a better choice when the fixed point is at a rigid support. The orientation is less clear when the fixed point occurs at a pier or other flexible supports.

8. Increase in the resistance of the bearings and expansion joints results in an increase in the transverse movements and the stresses in skew bridges and an increase in the radial movements and stresses in curved bridges. Hence, skew bridges and curved bridges may require frequent maintenance to ensure satisfactory performance of the bearings and expansion joints.

9. Integral construction is clearly a viable method for accomodating thermal movements in some bridges, but it is not a solution to all thermal movement problems. Integral construction restrains the thermal movement by the strength and stiffness of the substructure, foundation, and backfill. This clearly increases the forces within the bridge. The internal forces must be accommodated in the design or they will lead to unexpected damage. These internal forces may be underestimated with simple design methods. Integral construction may increase the transverse movements in skew bridges, and it may increase the radial movements in curved bridges. Therefore, integral construction requires greater caution in complex bridge geometries.

10. The experiments show that bridge temperatures and the resulting thermal movements can be economically measured for some bridges.

11. The measured bridge temperatures are similar to those obtained analytically. The movements obtained from the analytical models compare well with the measured values. However, there are very few data of bridge temperatures and thermal movements for bridges in the United States. More experimental field data are needed to fully understand thermal movement behavior and to verify the theoretical results. Field observations are needed, particularly for curved or skew bridges.

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