Field Study of Bridge Temperatures in Composite Bridges

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An experimental and analytical investigation was conducted to obtain thermally induced movements and bridge temperatures of a newly constructed bridge in central Louisiana. The instrumentation, field monitoring, and temperature data analysis are described. On the basis of a measured distribution of temperatures through the depth of the bridge sections, a model to predict this distribution was developed. The model is accurate and easy to use and can be easily adopted and applied as thermal loading in finite element programs predicting thermal movements and stresses in bridges.

Highway bridges generally require expansion joints between sections of the deck or between the deck and the approach roadway. The current practice for the design of expansion joints for Louisiana highway bridges (*1*) is based on elementary strength of materials formulas, and these may not accurately predict actual joint movements in modern bridges. Therefore an experimental research project was conducted on an actual bridge in central Louisiana to obtain longitudinal movements and bridge temperatures. The purpose of this paper is to describe the instrumentation, field monitoring, and data analysis as related to bridge temperatures. A detailed description of instrumentation and presentation of results pertaining to longitudinal movements appears elsewhere (*2*).

RELATED STUDIES

Reynolds and Emanuel (*3*) have written a concise summary of relevant research conducted in this area between 1957 and 1970. They concluded that relating environmental conditions to bridge movements is extremely complex. Mortlock (*4*) investigated various types of instruments used to obtain bridge movements and temperatures. He considered measuring devices that could be left at a bridge site for continuous field monitoring and concluded that the following should be used: (*a*) thermocouples placed in the slab during construction to obtain the variation of temperature through the slab depth, (*b*) linear variable differential transformers (LVDTs) mounted across the expansion joint to measure the joint movements, and (*c*) a Kipp solarimeter to measure the solar radiation of the slab. Combinations of these measuring devices were placed at seven bridge sites in England and Wales. The data obtained were compiled and analyzed by Emerson (*5*). A major conclusion was that the instrumentation had functioned satisfactorily. From the gathered data, a coefficient of thermal expansion for each bridge was developed. It was finally concluded that, with certain limitations, it is possible to estimate the extreme range of movements likely to occur during the life of a bridge if the shade temperatures are known. Dillon and Kissane (*6*) summarized the movements of prestressed concrete girders located throughout New York State over a 2-year period. This information was compared with that from climatic records, and it was concluded that the actual temperature ranges were greater than the design ranges; however, the average annual end movement was not significantly different from design values. Emerson (*7*) describes a method for determining the effective temperatures in composite bridges when shade temperatures and bridge movements are known. The method was applied on two bridges in England. Thermocouple wires were used to measure the temperature in the bridge slab and the ambient temperature. The method of prediction was based on these measurements, and the results were reasonably accurate. Abdul-Ahad (*8*) developed a theoretical method of calculating thermally induced stresses and movements in continuous bridge structures. The experimental monitoring was done on a composite box girder bridge. The bridge was 2,700 ft long with 29 spans and no expansion joints except at the abutments. The experimental and analytical results were close; however, the experimental data were limited and no generalized conclusions could be drawn.

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OBJECTIVES AND SCOPE

The purpose of this paper is to describe the experimental procedures and associated instrumentation and to discuss the general behavioral characteristics of a specific bridge as related to thermal gradients. Reported here are the results of a systematic study of bridge temperatures and temperature distributions. An upcoming paper will address the thermal joint movements. The study was focused on a newly constructed bridge on US-190 over the Atchafalaya River at Krotz Springs, Louisiana. The bridge was instrumented using LVDTs, thermocouples, and optical devices. The objective was to study the thermal characteristics and temperature distribution through the depth of bridge sections.

BRIDGE DESCRIPTION

The bridge to be investigated is the east approach of US-190 over the Atchafalaya River at Krotz Springs, Louisiana. It consists of cast-in-place concrete slabs acting compositely with either Type IV AASHTO prestressed concrete girders or steel plate girders. This superstructure is supported by twelve bents, as shown in Figure 1. The abutment is labeled Bent 1 and the rest of the bents are numbered in ascending order from east to west. Five expansion joints are provided to allow for bridge movements. These joints are num-

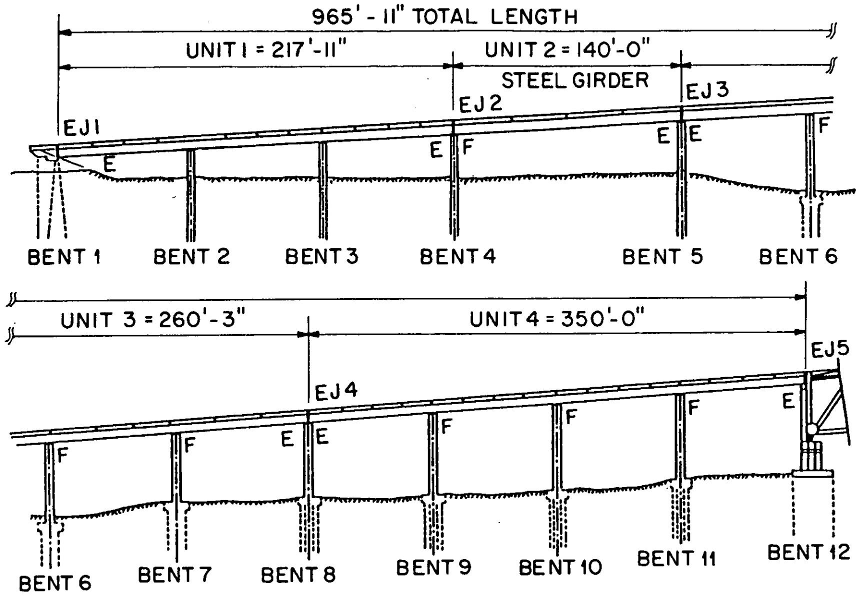

FIGURE 1 North elevation of the east approach of the U.S. 190 bridge at Krotz Springs, Louisiana.

bered 1 through 5 in consecutive order from east to west as well. Joints 1 through 4 are membrane seals, whereas Joint 5 is a toothed type. The bridge continues over the river as a steel through truss.

Unit 2 is the longest single span of the approach at 140 ft. It consists of a cast-in-place slab 8½ in. thick acting compositely with four steel plate girders 72 in. deep. The other three sections of the approach (Units 1, 3, and 4) consist of a slab 7½ in. thick acting compositely with five Type IV AASHTO prestressed concrete girders.

The supporting Bents 2 through 5 consist of concrete caps poured at the top of precast concrete piles 30 in.^{2}. Bents 2 and 3 each have four precast concrete piles supporting a level cap. Bents 4 and 5 each have five piles supporting the cap. The cap is stepped to allow the top of the steel girders to match flush at the same level as the top of the concrete girders. The supporting Bents 6 through 11 consist of level concrete caps poured at the top of two cast-in-place concrete columns 54 in. in diameter. Bent 12 consists of two cast-in-place concrete columns 30 in. in diameter anchored to a bridge pier. This pier also supports the end rocker bearings of the river crossing truss. The ends of the girders at the expansion joints and at continuous joints over the bents were placed on neoprene bearing pads of the standard type used in Louisiana.

At continuous joints, the girders were connected to the bent cap by imbedding a dowel into the cap extending into the continuous joint. At some expansion joint locations, the girders were pinned to the bent cap. The connection consists of a steel angle-shaped bracket bolted to both the girder and the cap. The bolt holes do not allow for any longitudinal movement between the cap and the girders. At some expansion joint locations, the girders were allowed to slide on the cap. This roller type of connection consists of a steel angle-shaped bracket with slotted holes, which allows for movement. The location of each type of joint connection is shown in Figure 1. Pinned joint connections are denoted by the letter F, whereas joints allowed to move are denoted by the letter E. Additional information and bridge design details appear elsewhere (2). The bridge was already under construction at the beginning of this research project (October 1986). The supporting bents had been erected and the girders were already in place. It was during that period of construction that the first instrumentation was installed. At that time the decks were also constructed. On October 27, 1988, construction was completed and the bridge was opened to traffic.

INSTRUMENTATION

LVDTs were chosen to obtain the joint movements. A theodolite was chosen to obtain the bent sway, and thermocouples were used to obtain all temperature measurements. The LVDTs and thermocouples were wired to the monitoring station where they would be connected to a Hewlett Packard microcomputer and data acquisition system. The computer would store the readings for later processing. Electrical power for the system was supplied through a portable generator.

Thermocouple wires type PP20TX were used to measure the temperatures of the Krotz Springs bridge; three advantages made them the choice for this investigation. The thermocouples were placed along the depth of the section to detect the temperature variation. Each array consisted of six thermocouples located on both slab and girder. The location of these arrays is shown in Figure 2. The slab thermocouples were placed near the top, center, and bottom of the slab at the time of pouring. The girder thermocouples were placed at a later time. These were bonded on the outer surface of the concrete girders using epoxy and a layer of hydraulic cement to ensure a more consistent thermal conductivity. Two additional thermocouples were placed hanging under the slab to record the ambient temperature. All thermocouples were run under the bridge to the data acquisition system of the microcomputer at the monitoring station.

LONG-TERM THERMAL BEHAVIOR

The mechanism of bridge joint movements is very complex. The strains that influence joint movements are caused by a variety of factors, including thermal changes, time-dependent creep and

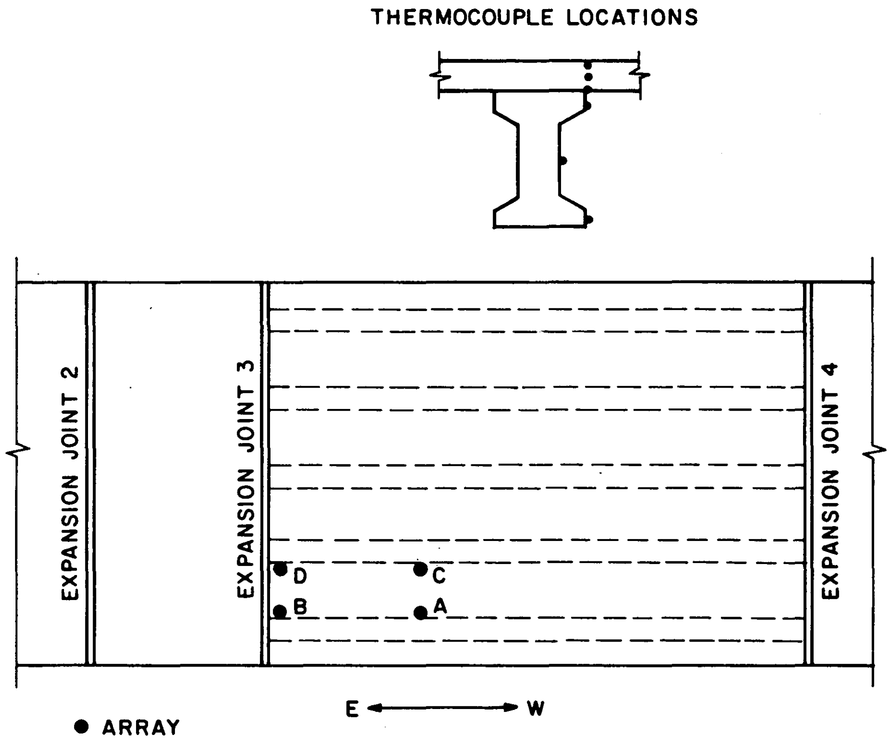

FIGURE 2 Plan view of bridge showing location of thermocouple arrays.

shrinkage, loss of prestress, and applied live loads. Furthermore the movements caused by thermal changes are greatly affected by the profile of temperature distribution through the depth of the cross section. A change in temperature, which varies linearly over the cross section of a simply supported bridge, produces no stresses. However, when the temperature variation is nonlinear, the same bridge will be subjected to stresses, because any fiber, being attached to other fibers, cannot exhibit free temperature expansion. These thermal stresses in the cross section are referred to as the self-equilibrating stresses. When the temperature variation is nonlinear, the strain distribution over the cross section hypothetically would be nonlinear, but because plane cross sections tend to remain plane, the actual strain distribution is linear. *(9,10)* The difference between the hypothetical and actual strain curves represents expansion or contraction, which is restrained by the self-equilibrating stresses.

The bridge was monitored over an approximate 2-year period. It was not practical to provide continuous monitoring over such an extended time. Instead, the bridge was monitored once per month continuously for a 24- or 12-hr period. The monitoring time was alternated between 12 and 24 hr on a month-to-month basis. The results of these intermitent cycles of monitoring are summarized in Figures 3 and 4. Actual temperature measurements were taken at the top, center, and bottom of both the slab and concrete girder, respectively. The results were a relatively narrow band of temperature variations for slab and girder, respectively, with the band of girder temperatures generally a step lower than the slab temperatures during the heat of the day but similar during the nighttime hours. The plots in Figures 3 and 4 are limited to the band width of slab and girder temperatures. Also shown is the variation of ambient temperature during the same periods. Figure 3*a* and *b* shows the temperatures recorded at Locations A and B, whereas Figure 4*a* and *b* gives the temperatures recorded at Locations C and D. The bottom slab thermocouple at Location C did not function properly, however, and readings recorded by it were discarded.

It can be seen from Figures 3*a* and *b* and 4*a* and *b* that there is a small variation between the temperatures recorded at Locations A, B, C, and D. For example, on the coldest day, December 16, 1987, the highest slab temperatures recorded at Locations A, B, C, and D were 57°F, 53°F, 57°F, and 55°F, respectively. Similarly for the hottest day, May 16, 1988, the highest slab temperatures recorded at Locations A, B, C, and D were 113°F, 108°F, 113°F, and 110°F, respectively. It can also be seen from the figures that, with the exception of January 12, 1988, and January 5, 1989, the slab temperatures rose higher than the girder temperatures during the heat of the day, with the ambient temperature falling somewhere in between. Again this is because the slab was exposed to the sun and solar radiation while the girders were in the shade. The largest differential between slab and girder temperatures was about 20°F and occurred during the hottest monitoring days of April 15, May 16, June 10, and August 25, 1988. A large temperature differential through the depth of the bridge section as well as a large maximum and minimum temperature differential was of particular significance for future studies of expansion joint movements in bridges of this type.

To better illustrate the long-term temperature trends of the composite system, the maximum and minimum values of the average slab and girder temperatures as well as ambient temperatures are plotted in Figure 5. Only the values from the 24-hr continuous monitoring periods are shown with lines connecting points for clarity in reading the trends. Because this bridge is located in a temperate climate (only 1 day with below-freezing temperatures), the trends reflected here do not necessarily apply to colder climate conditions.

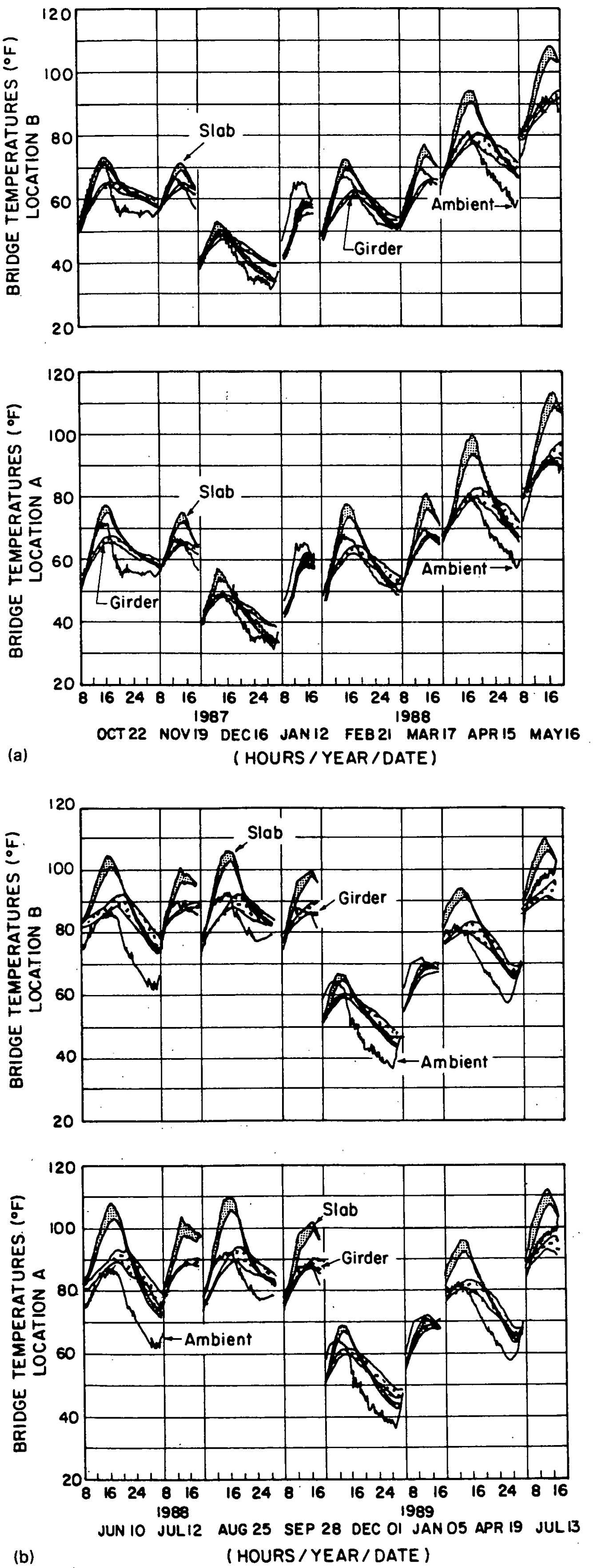

FIGURE 3 Bridge temperatures obtained from thermocouples at locations A and B.

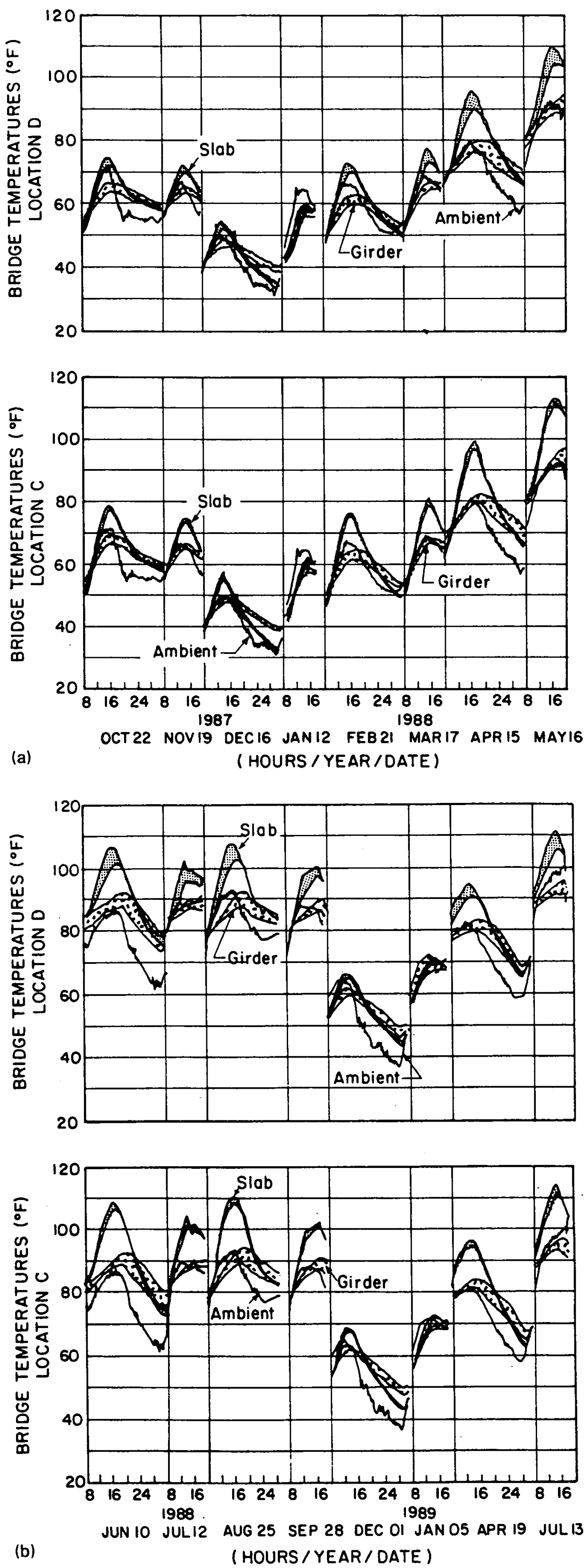

FIGURE 4 Bridge temperatures obtained from thermocouples at locations C and D.

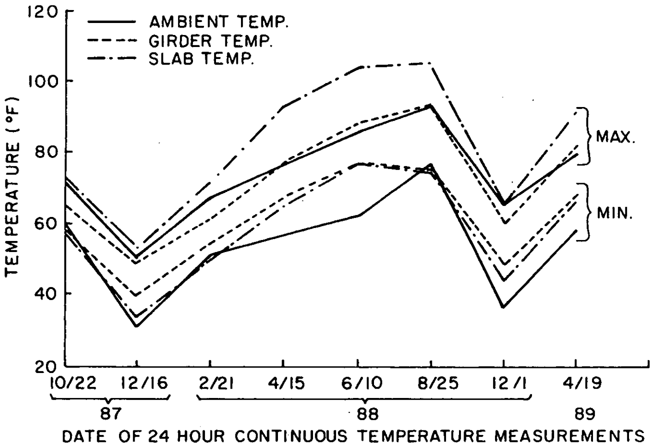

FIGURE 5 Seasonal variation in extreme temperatures for slab, girder, and ambient (slab and girder temperatures are average values).

The maximum average temperature in the girders closely followed the maximum ambient temperature (although with a phase shift as seen in Figures 3 and 4). However, usually the minimum average temperature of the girders was somewhat higher than the minimum ambient temperature. Thus the girders often did not reach temperature equilibrium with the ambient before reheating with the next day's temperature rise.

For the slab the same trends occurred except when the maximum ambient temperature began to exceed 70°F. At these higher temperatures, the solar radiation effect serves to magnify the slab temperatures in a somewhat linear manner. For example, a linear least-squares curve fit (slightly rounded to whole numbers) relating maximum average temperature in the slab, T_s, to maximum ambient temperature, T_a, is

$$T_s = T_a \qquad \text{For } T_a \leq 70$$
$$T_s = 2T_a - 70 \qquad \text{For } T_a > 70 \tag{1}$$

TEMPERATURE VARIATIONS THROUGH DEPTH

The data obtained during the 24-h monitoring days were used to further study the temperature distribution. For these days, the temperature distribution through the depth of the bridge section is plotted at 4-hr intervals starting at 8:00 a.m., as indicated in Figures 6 through 9. The dashed line indicated in the upper left plot of each figure represents the temperature distribution at the end of the 24-hr cycle or 8:00 a.m. the next day. The ambient temperature is also given in each plot for relative comparison. Figures 6 through 9 indicate that the thermal profiles follow a certain path over time. In particular, the slab temperatures generally are lower than, or close to, the girder temperatures during the morning hours, then rise higher than the girder temperatures, reaching their peak values around 4:00 p.m. Finally, during the evening hours the slab and girder temperatures converge again while falling to their lowest values over night.

Thermal stresses are known to cause considerable damage in bridges. Although current bridge specifications such as those of

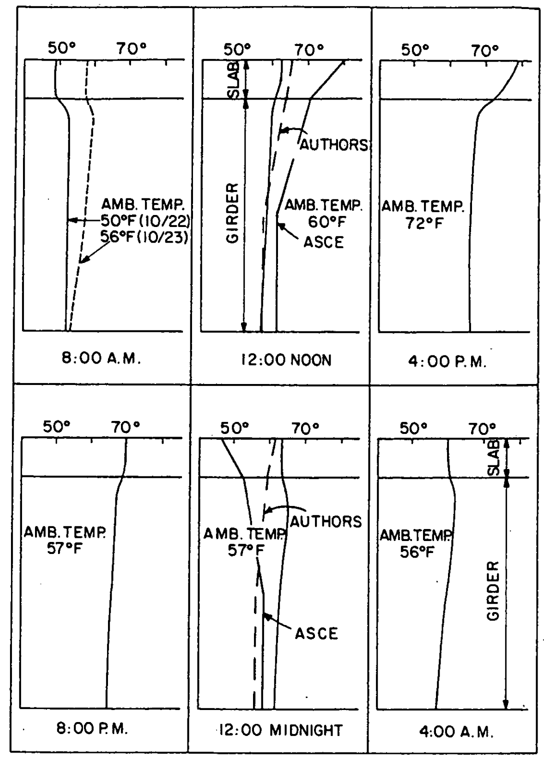

FIGURE 6 Temperature distribution through the depth of the section for October 22, 1987.

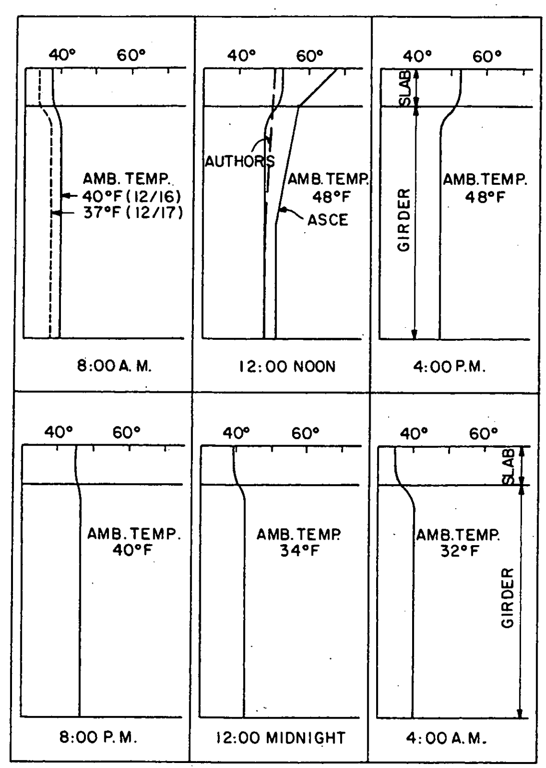

FIGURE 7 Temperature distribution through the depth of the section for December 16, 1987.

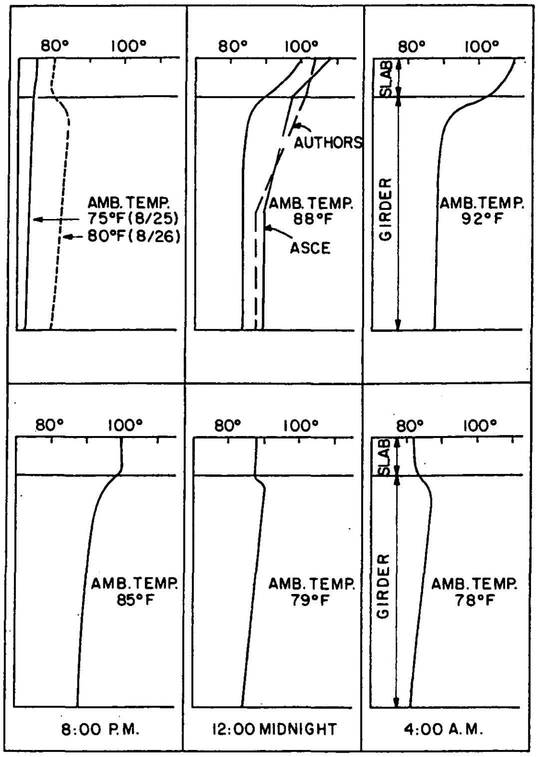

FIGURE 8 Temperature distribution through the depth of the section for August 25, 1988.

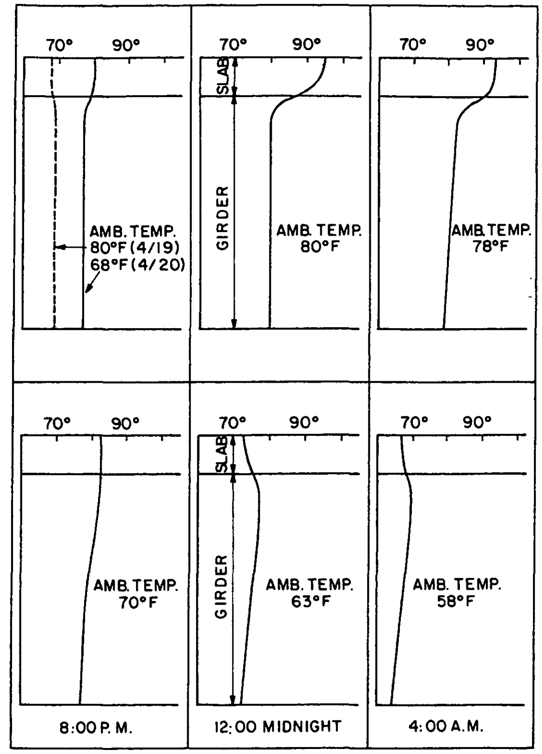

FIGURE 9 Temperature distribution through the depth of the section for April 19, 1989.

AASHTO (*11*) recognize the existence of thermal expansion and thermal forces, they are vague about values. In particular AASHTO recommends a range of temperature variation in bridges to account for the expansion movements; however, it does not provide guidelines about the vertical temperature distribution through the depth of the section. Several analytical and experimental studies have been conducted in relation to the vertical temperature distribution through the section depth of concrete and composite concrete slab-on-steel beam bridges. (*12–15*) Although many complex factors, such as solar radiation, ambient temperature, wind velocity, conductivity, and evaporation come into play, many researchers tried to obtain a simple but reasonable method of predicting the temperature distribution of bridge sections. The work of Imbsen et al. (*13*) has been incorporated into the AASHTO Guide Specifications. However, the approach was to develop maximum temperature differentials to be expected for a bridge at a given location. No relationship was developed that relates the slab/girder temperatures to the ambient temperature. Therefore a direct comparison cannot be made between the model developed here and Imbsen's work. However, a comparison can be made with another widely recognized model. The Committee on Loads and Forces on Bridges, ASCE, (*16*) recommends as thermal loading a temperature distribution through the depth of the section on the basis of the ambient temperature variation. The temperature distribution is recommended as a positive thermal loading for concrete bridges. The temperature at the top and bottom of the deck is found by adding 20°F and 10°F, respectively, to the ambient temperature and is assumed to vary linearly in between. The bridge temperature is assumed to vary linearly from the bottom of the deck to the middle of the girder where the bridge temperature is taken equal to the ambient temperature. The temperature from midgirder to the bottom of the girder is also taken equal to the ambient temperature. Similarly, ASCE provides a negative thermal loading for dropping temperatures. In this profile, the temperature at the top and bottom of the slab is found by subtracting 10°F and 5°F, respectively, from the ambient temperature.

On the basis of the experimental data a modified profile is developed by the author that is both accurate and easy to use. Polynomials of various degrees were used to curve fit the temperature data obtained from the thermocouples. On the basis of the calculated sum of the squares of the residual, a second-order polynomial is deemed to be both accurate and simple. A different curve fit is used to predict the bridge temperatures at the top of Deck T_1, the bottom of Deck T_2, and the middle and bottom girder Temperature T_3. These curve fits are indicated in Figure 10. The horizontal axis of the figure represents the ambient temperature T_a, whereas the vertical axis represents the bridge temperatures T_1, T_2, and T_3. The approach used to analyze these data was to first select a series of ambient temperatures over the range of 30°F to 90°F degrees (14 were used). Then the eight 24-hr continuous monitoring periods (Figures 3 and 4) were used to select all slab temperatures associated with each of the selected ambient temperatures. The number of data points associated with each slab temperature typically ranged between 5 and 10 per selected ambient temperature location. These data points were grouped and averaged for illustrative purposes. Each data point shown in Figure 10 represents this averaging process. The least-squares curve fit was thus based on all points and corresponds to approximately 75 data points per curve. Once T_1, T_2,

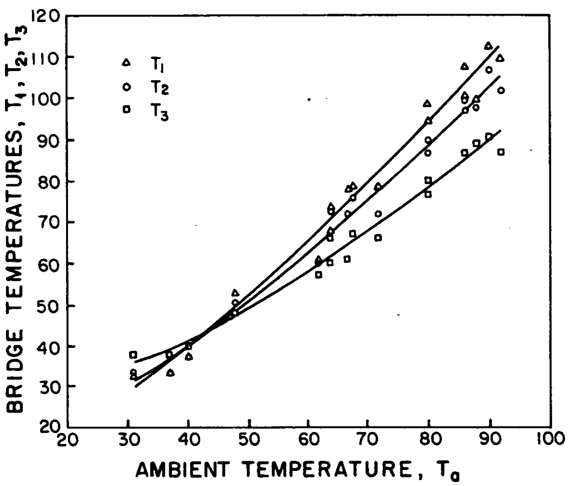

FIGURE 10 Bridge temperatures as a function of ambient temperature.

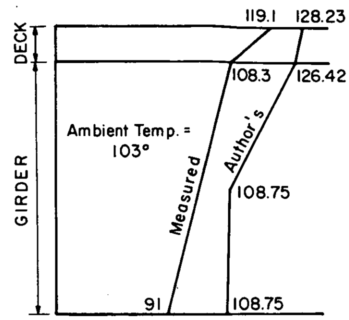

FIGURE 11 Comparison of author's temperature distribution model with Boone River Bridge measurements.

and T_3 are found the thermal profile is obtained by assuming a linear temperature variation between T_1, T_2, and T_3. The values of T_1, T_2, and T_3 can also be calculated from the following equations:

$$T_1 = 0.095 + 0.832\,T_a + 0.004\,T_a^2 \quad (2)$$

$$T_2 = 6.63 + 0.648\,T_a + 0.005\,T_a^2 \quad (3)$$

$$T_3 = 23.88 + 0.206\,T_a + 0.006\,T_a^2 \quad (4)$$

The model was developed on a bridge constructed of concrete slab on Type IV AASHTO girders, which is a common type of construction in Louisiana and elsewhere. It should be applicable also to concrete bridges of similar construction using Types II and III AASHTO girders. The developed thermal profile is compared with the experimental measurements and the ASCE profile on 3 typical days in various seasons, as shown on Figures 6 through 8. Figures 6 and 7 (October 22, 1987, and December 16, 1987) indicate that when the ambient temperatures are low the author's experimental and analytical results agree very well, whereas ASCE'S profile differs in the range of 30 to 40 percent in the slab. On August 25, 1988, however, when the ambient temperatures were high, both the authors' and ASCE'S profiles overestimated bridge temperatures by approximately the same amount of 5 to 10 percent.

The temperature distribution predicted by the model is also compared to the experimental measurements obtained at the Boone River Bridge (*17*). Figure 11 shows the temperature distribution through the depth at the time of the highest temperature as well as the author's distribution corresponding to the recorded ambient temperature of 103°F. The figure shows that the author's model overestimated bridge temperatures by approximately 10 percent at the top of the slab and 20 percent at the bottom of the girders. The large difference at the girder bottom is because at the Boone bridge the girder thermocouples were placed at the center of the girder and during a hot day when the temperature was rising quickly there is a time lag between girder center and surface temperatures. In addition, the model's accuracy decreases at high ambient temperatures because it was developed using data corresponding to ambient temperatures of up to 92°F.

CONCLUSIONS

A composite concrete deck-girder bridge (typical of the type constructed in the southeast and much of the United States) was experimentally studied to determine thermal distributions in both deck and girders. Periodic observations over a 2-year period were used to evaluate long-term trends. Each observation was conducted continuously for 12 or 24 hr to also evaluate short-term behavior. The following conclusions and observations are of some significance.

1. For the 8½-in. concrete deck, the temperature variations through the thickness were relatively small, rarely exceeding 8°F and usually less than 5°F.
2. Temperature variations through the depth of the 72-in. prestressed concrete girders were also relatively small and seldom exceeded 10°F.
3. On the basis of the average temperature values in both the slab and girder, long-term trends indicated that (*a*) girder temperatures closely follow the maximum ambient temperature over the range of 50°F of 95°F (with a phase shift), (*b*) for the same temperature the minimum girder temperatures often remain 5°F to 10°F higher than the lowest ambient temperatures, and (*c*) slab temperatures follow the same trend as girders except when the maximum ambient temperature exceeds 70°F, in which case solar radiation magnifies the slab temperature in a somewhat linear manner.
4. The distribution of temperatures through the depth of the slab and girders varied significantly. On the basis of the measured distribution of temperatures through the depth of the bridge sections, a model to predict this distribution was developed. The model relates the temperatures at the top and bottom of the slab as well as the girder temperatures to ambient temperatures.
5. The developed model provides a good description of thermal profiles through the depth of the slab and girder. For the low ambient temperature range, the values obtained from the model are almost identical to the actual temperature measurements and are more accurate than those of the ASCE profile. For the high temperature range, the model varies only 5 to 10 percent and is similar to that of the ASCE profile. The use of such a model in a finite element program that predicts thermal movements and stresses in bridges should give more realistic results and significantly aid the design of bridge expansion devices.
6. The developed model is applicable to both positive and negative temperature distributions (rising or falling temperatures) and

can be easily adopted and used as thermal loading in finite element programs.

REFERENCES

1. *Bridge Design Manual*, 3rd ed. Louisiana Department of Transportation and Development, Baton Rouge, 1987.
2. Pentas, H. A. *Experimental Investigation of the Longitudinal Movements Associated with Highway Bridge Joints*. Ph.D. dissertation. Louisiana State University, Baton Rouge, 1990.
3. Reynolds, J. C., and J. H. Emanuel. *Thermal Stresses and Movements in Bridges*. ASCE *Journal of Structural Division*, Vol. 100, No. 1, 1974, pp. 63–78.
4. Mortlock, J. D. *The Instrumentation of Bridges for the Measurement of Temperature and Movement*. TRRL Laboratory Report 641. U.K. Transport and Road Reasearch Laboratory, Crowthorne, Berkshire, England, 1974.
5. Emerson, M. *Thermal Movements of Concrete Bridges: Field Measurements and Methods of Prediction*. Joint Sealing and Bearing Systems for Concrete Structures. ACI Special Publication SP-70, Vol. 1, Detroit, 1971, pp. 77–102.
6. Dillon, E. W., and R. J. Kissane. *Annual End Movements of Prestressed Concrete Bridges*. Final Report. U.S. Department of Transportation, FHWA, New York, 1978.
7. Emerson, M. Bridge Temperatures for Setting Bearing and Expansion Joints. TRRL Supplementary Report 479. U.K. Transport and Research Laboratory, Crowthorne, Berkshire, England, 1979.
8. Abdul-Ahad, R. B. *Effects of Restrained Thermal Movement in a Continuous, Prestressd Concrete Bridge Without Interior Expansion Joints*. Ph.D dissertation. University of Tenessee, Nashville, 1981.
9. Ghali, A., and R. Favre. *Concrete Structures: Stresses and Deformations*. Chapman and Hall, New York, 1986.
10. Will, K. M., C. P. Johnson, and H. Matlock. *Analytical and Experimental Investigation of the Thermal Response of Highway Bridges*. Report 23-2. Center for Highway Research, The University of Texas at Austin, 1977.
11. *Standard Specifications for Highway Bridges*, 13th ed. AASHTO, Washington, D.C., 1983 (with interim specifications, 1984, 1985, 1987–1988).
12. Kennedy, J. B., and M. H. Soliman. Temperature Distribution in Composite Bridges. ASCE *Journal of Structural Engineering*, Vol. 113, No. 3, 1987, pp. 475–482.
13. Imbsen, R. A., D. E. Vandershaf, R. A. Schamber, and R. V. Nutt, *NCHRP Report 276: Thermal Effects in Concrete Bridge Superstructures*. TRB, National Research Council, Washington, D.C., 1985.
14. Churchward, A., and Y. J. Sokal, Prediction of Temperatures in Concrete Bridges. ASCE *Journal of Structural Engineering*, Vol. 107, No. 11, 1981, pp. 2163–2176.
15. Zuk, W. Thermal Behavior of Composite Bridges—Insulated and Uninsulated. In *Highway Research Record 76*, HRB, National Research Council, 1965, pp. 231–253.
16. Committee on Loads and Forces on Bridges. Recommended Design Loads for Bridges. ASCE *Journal of Structural Engineering*, Vol. 107, No. 7, 1981, pp. 1161–1213.
17. Girton, D. D., T. R. Hawkinson, and L. F. Greimann. Validation of Design Recommendations for Integral-Abutment piles. ASCE *Journal of Structural Engineering*, Vol. 117, No. 7, 1991, pp. 2117–2134.

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