THERMAL BEHAVIOR OF MISSOURI'S FIRST HIGH PERFORMANCE CONCRETE SUPERSTRUCTURE BRIDGE

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ABSTRACT

High performance concrete is becoming more widely utilized in highway bridge structures due to its economical savings and greater design flexibility. The first high performance superstructure concrete bridge in Missouri was instrumented to investigate the thermal behavior of the bridge using thermocouples and thermistors from construction through it service life. Mean bridge temperature and thermal gradients for design suggested by NCHRP and AASHTO were compared with the measured values. Modified methods were recommended for bridge mean temperature and thermal gradients. Computed values by modified methods correlated well with measured values. Hydration temperatures were also investigated.

Keywords: High Performance Concrete, Instrumentation of Bridges, Hydration Temperature, Mean Bridge Temperature, Thermal Gradient.

INTRODUCTION

High performance concrete (HPC) is becoming more widely utilized in highway bridge structures due to its economical savings and greater design flexibility. To implement more widespread use of HPC in Missouri, the Missouri Department of Transportation (MoDOT) co-sponsored a research study in Missouri to investigate both the early-age and later-age performance of a widely used PC bridge system in Missouri that includes the use of HPC and larger prestressing strands. In this study, the first HPC superstructure bridge in Missouri, Bridge A6130, was constructed, instrumented and monitored. Thermal behavior of the bridge is discussed and presented in this paper.

Temperature can have a significant impact on the behavior of concrete highway bridge structures. Large stresses and strains may results as a structure heats or cools, depending on the distribution of temperature and level of restraint present in the structure. Since a real structure is rarely completely free to move or completely restrained, a combination of strain and stress is usually present[1].

High hydration temperatures are often developed in members using high strength / high performance concrete (HS/HPC) since large quantities of cementitious materials are typically used. In addition, if the cooling of a member shortly after hydration is restrained, thermal cracking may result.

Variations in environmental conditions lead to two basic thermal cycles for any bridge structure: the seasonal cycle, and the diurnal cycle. Ambient temperatures are highest during the summer months and lowest during the winter months. Average bridge temperatures follow the same basic trend. Bridge structures must be designed to accommodate the axial movements associated with this seasonal cycle. Restraint of these movements result in additional stresses that must be considered in design[1]. The daily temperature cycle is primarily governed by the path of the sun in the sky at the bridge site and the changes in ambient conditions during the course of the day and night. Thermal gradients produce a combination of axial and flexural stresses and strains through the depth of a structure. Radolli found these stresses and strains, can exceed those produced from live loads in certain cases though they are temporary in nature[2].

BRIDGE DESCRIPTION

Bridge A6130 was designed as a five-span bridge on Route 412 spanning drainage ditch No.8 in Pemiscot County located near Hayti, Missouri. The span lengths of the bridge are 15.5 m (50.9 ft), 17 m (55.8 ft), 17 m (55.8 ft), 17 m (55.8 ft) and 15.5 m (50.9 ft), respectively. This is the first bridge in Missouri to fully implement HPC into the superstructure including the girders and the bridge deck.

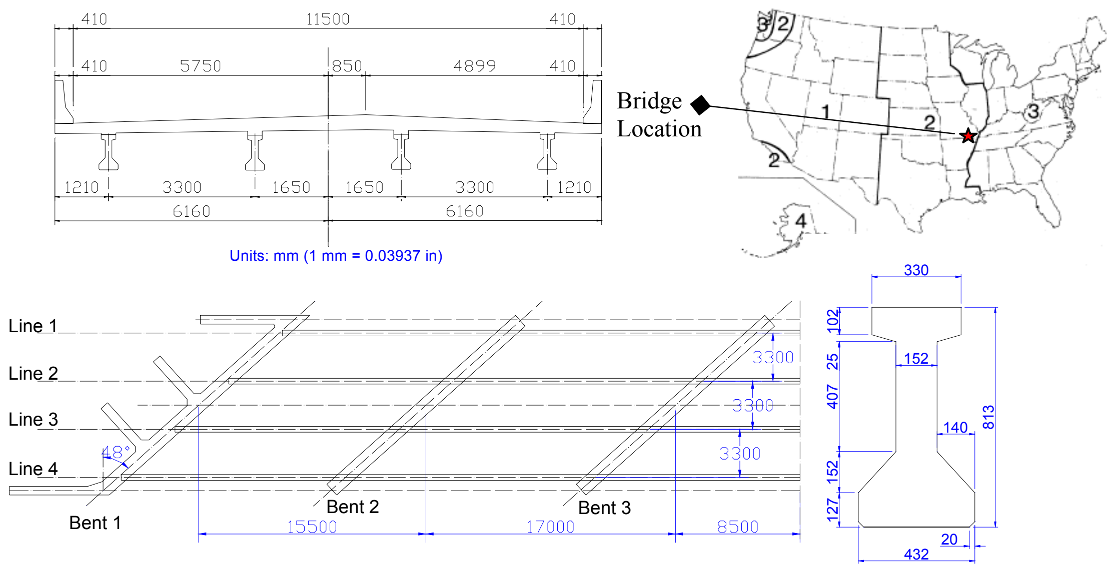


Fig. 1 Bridge A6130 Alignment (Half Bridge), Cross-Section and Location

Precast prestressed beams were designed to incorporate HSC. The required design compressive strength was 70 MPa (10,160 psi). The required release strength was 52 MPa (7550 psi). The use of 15.2 mm (0.6 in.) diameter pretensioned strands is employed to make full use of the high strength concrete. All twenty main span girders used in the bridge are MoDOT type 2 girders. Dimensions of these girders (see Fig. 1) vary slightly from standard AASHTO type II girders. HPC is also used in the cast-in-place deck with a thickness of 230 mm (9.1 in.). The abutment and bent lines are projected on a skew at an angle of 48^{o}. Half of the bridge plan and the cross section of the bridge are shown in Fig. 1.

RESEARCH SIGNIFICANCE

As the first HPC superstructure bridge in Missouri, Bridge A6130 was designed with a girder spacing up to 3.3 m (10.8 ft). The use of high strength concrete (HSC) enabled the designers to reduce the number of girders from 6 using conventional strength concrete to 4 using HSC. HPC is also used in the cast-in-place deck including the use of mineral admixtures to obtain a highly impermeability. Monitoring of this structure during the construction period and the service life can provide a beneficial understanding of its thermal behavior, including hydration temperatures, mean bridge temperatures and thermal gradients. Based on the field data acquired, a design recommendation on mean bridge temperature and thermal gradients are developed herein for bridges in Missouri and therefore offers a useful reference for designers, contractors and researchers.

INSTRUMENTATION PROGRAM

An instrumentation program was developed to monitor components of the bridge superstructure during early-age and later-ages to identify trends in the measured and observed behavior. A total of 16 internal thermocouples, 64 internal vibrating wire strain gauges (VWSG), and 14 internal bonded electrical resistance strain gauges (ERSG) were embedded in the PC girders and CIP deck. A data acquisition system (DAS) was designed and assembled for the monitoring program. In total, 6 girders and 4 locations were instrumented in the deck as illustrated in Fig. 2. Thermocouples were embedded in girder B21 and B22. A thermocouple labeled EX-BS was located adjacent to an external strand at the bottom layer of the prestressing strands. The hydration temperature for the concrete at the bottom strand level was collected by a thermocouple labeled IN-BS attached to one bottom strand in the beam, at a section approximately 3.05 m (10 ft) away from the beam end. VWSG and ERSG were embedded in girders B13, B14, B23 and B24 at mid-span section and near support section as shown in Fig. 2. Gauge locations along the height of the section include the top flange (TF), top web (TW), middle web (MW), center gravity of I-section (CGI), center gravity of strands (CGS), and bottom flange (BF).

Concrete temperatures were recorded by thermocouples and thermistors integrated in VWSG. Temperature data were used to investigate thermal gradients, extreme seasonal bridge temperatures, hydration temperatures, and corrections for strain and deflection measurements. ERSG were used as redundant gauge for strain measurement. Ambient temperature values were obtained from a National Climatic Data Center (NCDC) weather station location in a close proximity to the bridge. More specifics about the instrumentation plan for this bridge may be found in a paper by Yang, Shen, and Myers[3].

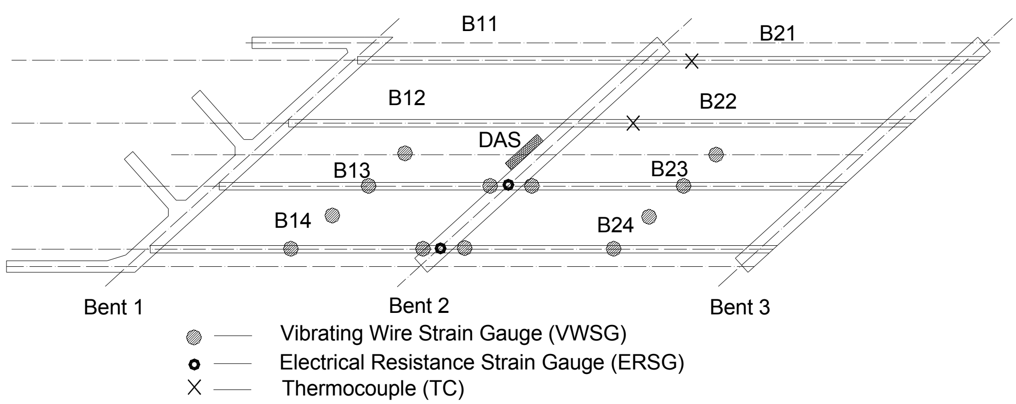


Fig. 2 Strain Gauge Connected with DAS

RESULTS

After download of field data acquired by DAS, temperature data were analyzed and studied. Several characteristics of the HPC used for the bridge and the constructed HPC bridge were then investigated, including hydration temperatures, mean bridge temperatures and thermal gradients.

HYDRATION TEMPERATURES

For hydration temperatures measurements, data acquisition systems were connected to the gauges during the main hydration cycle. Temperatures were generally recorded every 6 minutes.

Based on the data collected at the precast plant, a typical hydration curves for a HPC MoDOT Type II beam is shown in Fig. 3. Since all the casting beds were within a high-bay indoor environment, the temperature in the casting plant was relatively stable. Therefore, hydration curves are similar in shape to what might be expected under adiabatic conditions[1].

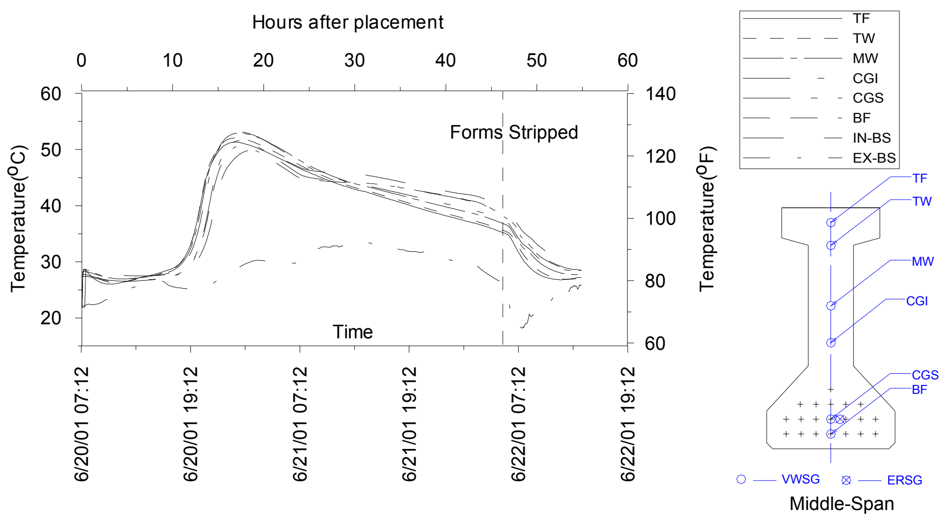


Fig. 3 Measured Hydration Temperatures in Mid-Span Section of Girder B23

The measurement results were summarized in Table 1. The peak temperature during hydration was 57°C (135°F) in beam B14 and beam B22. Maximum hydration temperatures in the six monitored HPC beams ranged from 53°C to 57°C (127°F to135°F). The peak temperature was recorded by gauges at TW or MW for most of the beams. Maximum temperature rise after the dormant period ranged from 26°C to 29°C (46°F to 53°F). Maximum gradients for most of the beams occurred immediately prior or right at the peak hydration temperature. A maximum gradient of 9°C (16°F) was observed between TW and BF gauges in beam B23. The effect of the removal of curing tarps and forms is apparent as shown in Fig. 3. A sharp temperature drop occurred across the depth of the section after removing the tarps and forms. The temperature variation within the member was not dramatic due to the size and shape of the member consistent with other studies on similar type I-shaped sections[4].

Hydration temperatures of the CIP deck were also monitored. The average placement temperature and average temperature at the end of the dormant phase were 26°C (79°F) and

28°C (82°F), respectively. Peak hydration temperature occurred 51mm (2 in.) below the top of deck surface with a value of 48°C (119°F). Maximum temperature rise after the dormant period was approximately 18°C (32°F).

For the HPC beams, equivalent maximum temperature rises ranged from 3.0 to 3.5°C per 59 kg/m^3 (5.4 to 6.2°F per 100 lb/yd^3) of cement. For the HPC CIP deck, maximum temperature rise after the dormant period was approximately 18°C (32°F). Equivalent maximum temperature rises of 3.2°C per 59 kg/m^3 (5.7°F per 100 lb/yd^3) of cement, or 2.7°C per 59 kg/m^3 (4.8°F per 100 lb/yd^3) of total cementitious material (cement and fly ash). These equivalent maximum temperatures are well below peak values observed in the ACI Committee 363 HSC State of the Art Document[5].

Table 1 Summary of Measured Hydration Temperatures for HPC Beams

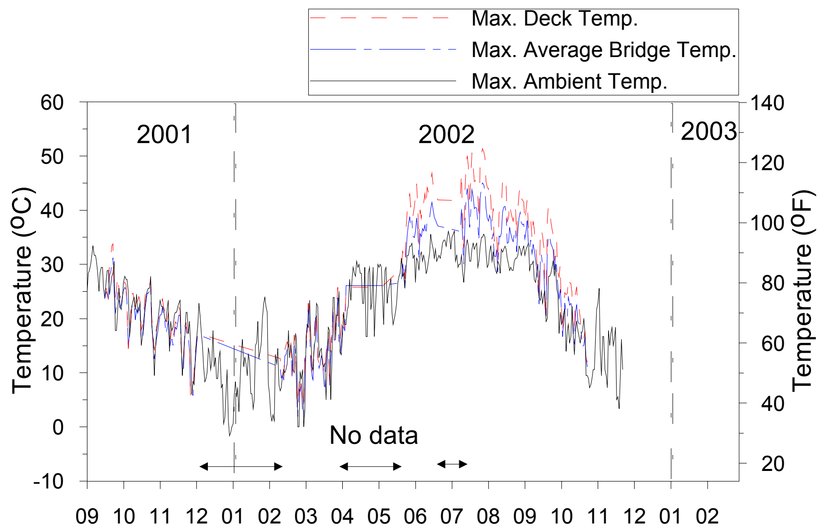
Beam	B21	B22	B23	B24	B13	B14
Placement Time (2001)	7:30am, 6/13	7:45am, 6/13	7:10am, 6/20	7:30am, 6/20	7:20am, 7/3	7:00am, 7/3
Avg. Placement Temp.	85°F	85°F	82°F	81°F	81°F	80°F
Avg. Temp. at End of Dormant Phase	86°F	86°F	82°F	82°F	83°F	82°F
Peak Hydration Temp.	134°F	135°F	127°F	134°F	133°F	135°F
Location of Peak Hydration Temp.	MW	MW	CGI	MW	TW	TW
Maximum Temp. Rise after Dormant Period	48°F	50°F	46°F	51°F	49°F	53°F
Maximum Gradient	9°F	13°F	16°F	13°F	15°F	15°F
Temperature: °C = (°F - 32)/1.8; Temp. Change: °C = °F/1.8						

Cracking was observed on almost all instrumented beams as a result of the restraint provided by the bed against contraction due to cooling and drying shrinkage since the strands were not released until nearly the 48 hour point after casting. However, these cracks closed entirely after release. No structural impact was observed as a result of this cracking and none should be expected. Based upon previous observations by the authors, it is likely that many of these early-age cracks self-healed due to on-going hydration processes of the concrete after release of prestressing.

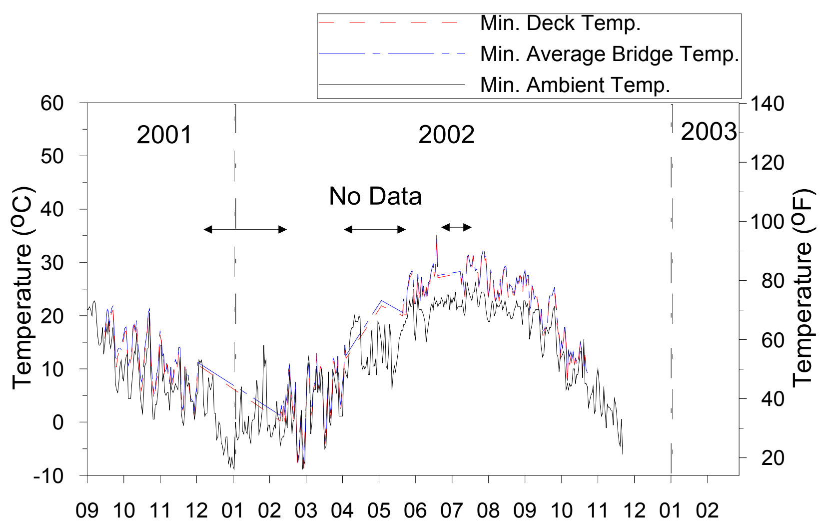
MEAN BRIDGE TEMPERATURES

Girder and deck bridge temperatures were measured using thermocouples and thermistors after the bridge was constructed. Temperatures were generally recorded every 15 minutes or once per hour. For each set of readings at a composite section of a girder and corresponding deck portion, an average bridge temperature was calculated. The average bridge temperature can be defined as a weighted mean of the temperatures at different depths of the composite cross-section, and is computed as the sum of the products of each measured temperature within the cross-section and its given weights[1]. It was found that total contribution of the deck temperatures to the average bridge temperatures was approximately 70 percent.

Daily maximum and minimum temperatures for girder B13 are shown in Fig. 4 from September 2001 to September 2002. General trends can be seen in these figures, especially the difference in maximum and minimum deck, average bridge and ambient temperatures during the summer. Average daily maximum and minimum temperatures were computed for each calendar month and shown in Fig. 5, since the daily variations in temperatures make some trends difficult to find.



a.) Maximum Temperature

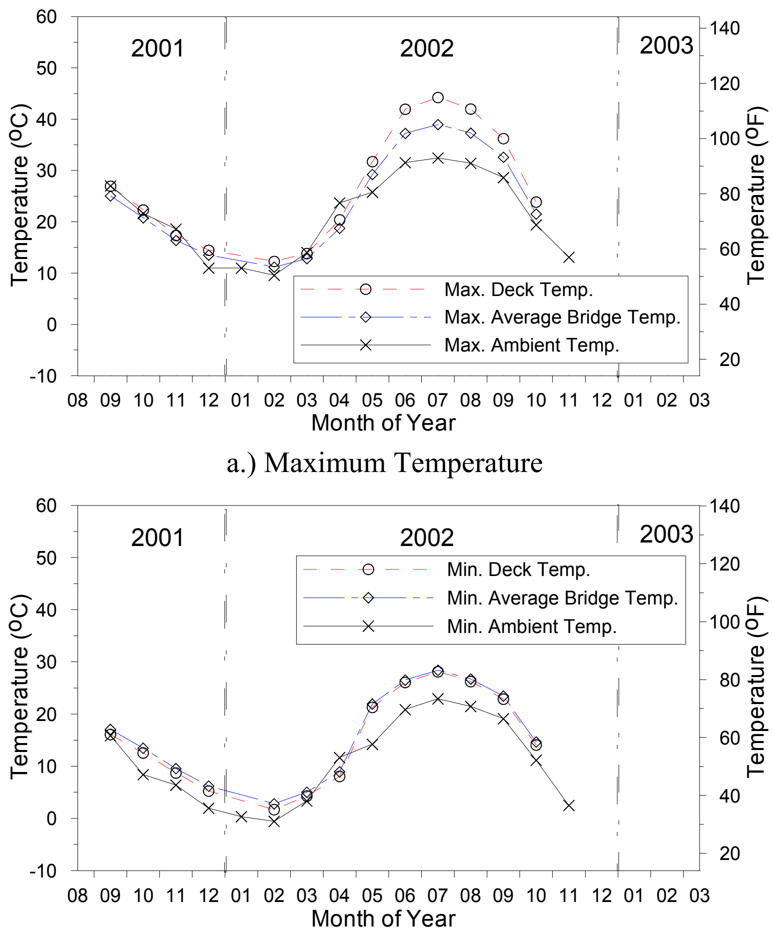


b.) Minimum Temperature

Fig. 4 Extreme Daily Temperature of Composite Girder B13

As illustrated in Fig. 4, the maximum temperatures tend to occur during the middle of the summer, typically in July. During this period, the maximum average bridge temperature was approximately 7°C (12°F) warmer than the maximum ambient temperatures. However, during the winter months from December to February, there was essentially no difference between the maximum average bridge temperatures and the maximum ambient temperatures. The average daily minimum temperature tended to remain higher than the minimum ambient temperature. On average, the difference was approximately 6°C (10°F) during summer month, and approximately 4°C (8°F) during the winter month.

Maximum average bridge temperatures measured on any singe day were 45°C (118°F), 44°C (117°F), 42°C(108°F) and 41°C(107°F) for B13, B14, B23 and B24, respectively. Meanwhile, minimum average bridge temperature measured for all locations in the bridge was -8°C (18°F).



a.) Maximum Temperature

b.) Minimum Temperature

Fig. 5 Average Extreme Daily Temperature by Month of Composite Girder B13

In Table 2, the measured average bridge temperatures in this study are compared to the design temperatures suggested by AASHTO standard specification, AASHTO LRFD, NCHRP Report 276 and a method suggested by Gross and Burns[1] specific to HSC beams. The designed temperature increases and decreases suggested in the AASHTO Standard

Specifications[6] underestimated the maximum increases and decreases that were observed in the bridge. The philosophy of these code provisions is questionable since the extreme temperatures are independent on the setting temperature. When the concrete set temperature is very high or very low, this method could underestimate the corresponding temperature increase or decrease. The Methods suggested in the LRFD Specification[7] and NCHRP Report 276 Method[8] are also inappropriate for the bridge locations monitored.

Table 2 Summary of Mean Bridge Temperatures

	MAX. TEMP.		MIN. TEMP.	
	B13	B14	B13	B14
HISTORICAL CLIMATE DATA (AMBIENT TEMPERATURES) (weather.com, 2002)				
Extreme Ambient Temp. Ever Recorded	111	111	-15	-15
Avg. Extreme Ambient Temp. for Peak Month	91	91	27	27
MEASURED AVERAGE BRIDGE TEMPERATURES (9/01 - 10/02)				
Extreme Average Bridge Temp.	113	112	18	20
Avg. Daily Bridge Temp. for Peak Month	102	100	37	38
Setting Temperature	71	71	71	71
Temp. Changes Relative to Setting Temp.	42	41	-53	-51
DESIGN TEMPERATURES				
AASHTO Standard Specifications (AASHTO, 1996)	ST+35	ST+35	ST-45	ST-45
AASHTO LRFD (AASHTO, 2002 Interim Revision)	80	80	0	0
NCHRP Report 276 Method (AASHTO, 1989)	92	92	30.5	30.5
Suggested By Gross and Burns (1999)	106	101	6	6
Suggested By the Authors, Equations (1) and (2)	111	111	6	6
All temperatures in ^{o}F. °C = (°F - 32)/1.8				

A simple approach is suggested for the determination of maximum and minimum design temperatures for the analysis of uniform axial effects by Gross and Burns[1] after monitoring four HPC bridges in Texas. The designed maximum temperatures using this method underestimated the maximum temperatures that were observed in the bridge. However, the minimum temperatures suggested by this method appears appropriate for the bridge monitored in this study because there is expected to be lower bridge temperatures during the lifetime of the bridge.

A modified approach based on the method suggested by Gross and Burns[1] is developed to provide more realistic design temperatures than the current methods previously discussed. It is not intended to exactly predict the extreme average bridge temperatures that may occur in the lifetime of a bridge structure. Obviously additional experimental data is necessary to determine the validity of this approach. The design maximum and minimum temperature can be calculated by the following equations:

$$T_{design,\max} = \frac{1}{2}(T_{July,\max} + T_{all-time,\max}) + 10^{o}F \quad \text{Equation (1)}$$

$$T_{design,\min} = \frac{1}{2}(T_{Jan,\min} + T_{all-time,\min}) \qquad \text{Equation (2)}$$

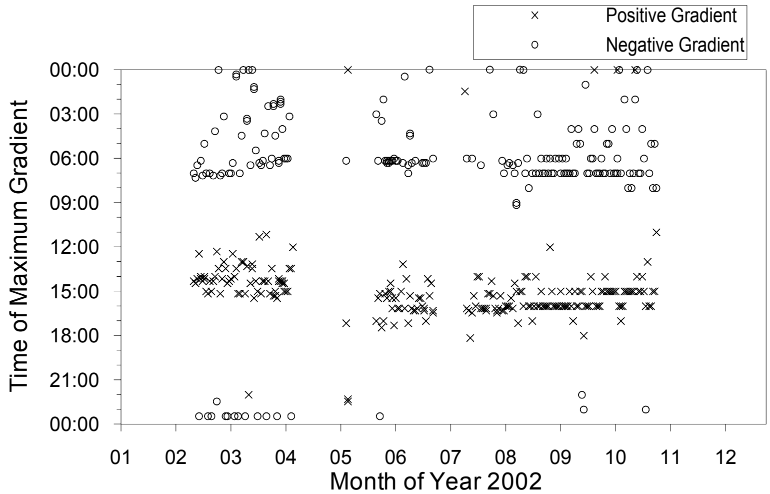
Where $T_{July,\max}$ is the average daily maximum ambient temperature in July at the bridge location; $T_{Jan,\min}$ is the average daily minimum ambient temperature in January at the bridge location; $T_{all-time,\max}$ and $T_{all-time,\min}$ are the maximum ambient temperature and minimum ambient temperature ever recorded at the bridge location, respectively. The calculated temperatures using above method are listed in Table 2 and found to be close to the extreme average bridge temperatures monitored in this study.

THERMAL GRADIENTS

The daily temperature cycle will lead to thermal gradients in a structure. During a sunny day, the bridge deck heats up much more quickly than the underside of the bridge and thus positive thermal gradient results. When a bridge superstructure that had obtained a high temperature during the day experiences a reduction in temperature caused by a cool night, a negative gradient (deck cooler than underside) may develop. Because the surface area of bridge deck is typically much larger than the rest of the superstructure, the deck loses heat more quickly. Its temperature may drop below the temperature of the rest of the superstructure, resulting in a negative gradient. A positive thermal gradient is defined as a gradient in which the maximum temperature occurred at a location (typically in the deck) higher than the location of the minimum temperature. In a similar way, a negative gradient is defined as a gradient in which the maximum temperature occurs at a location lower than the minimum temperature (typically in the deck). The magnitude of either gradient is defined as the difference between the maximum and minimum temperatures.

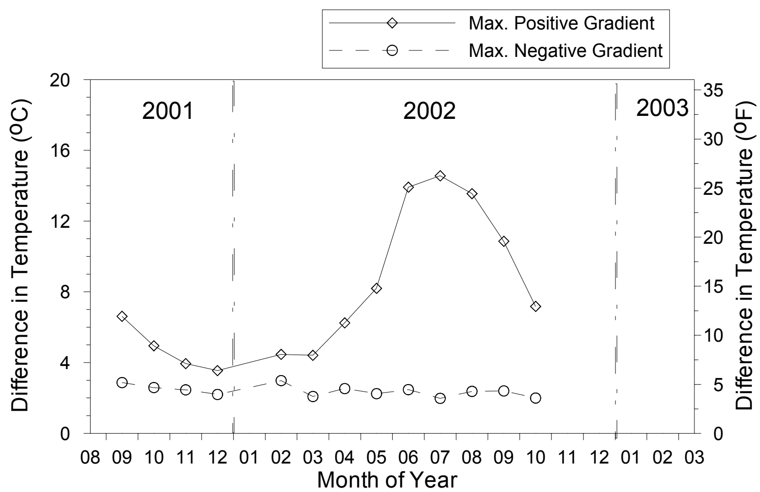
The magnitude of the maximum positive gradient varies substantially from day to day. Maximum gradients tended to be higher during summer months because of the intense solar radiation and high ambient temperatures. During fall and winter months, maximum positive gradients could be either high or low, depending on the ambient conditions. It has been noted that on a few winter days there was no positive gradient at any time during the day. It can be noted in Fig. 6 that the maximum positive gradient almost always occurred between 2:00 PM and 5:00 PM during the summer, and between 1:00 PM and 4:00 PM during the winter.

Similarly, maximum daily negative gradients also varied from day to day. Negative gradients were generally not higher during any part of the year. Negative gradients occurred during the early morning but the exact time varied substantially during this time frame from day to day. The average maximum positive gradients are highest during the summer months and lowest during the winter months. However, the average maximum negative gradients remain relatively constant during a year.



Time of Maximum Positive and Negative Gradients

a.) Maximum Temperature



Average Maximum Daily Gradients By Month

b.) Minimum Temperature

Fig. 6 Time of Maximum Gradients and Average Gradients by Month

Maximum thermal gradients for four monitored beams are summarized in Table 3. The maximum positive gradient ranged from 13 to 20°C (23 to 36°F) and the peak negative gradients ranged from 4 to 6°C (7 to 10°F). It was also noted that thermal gradients in interior and adjacent exterior beams can be quite different due to the effect of handrail, which was built partly over exterior beams.

In Fig. 7, maximum and minimum gradients measured are compared to the NCHRP 276 method, AASHTO LRFD and a methodology suggested by Gross and Burns[1] specific to HSC beams. It can be observed that the maximum measured positive gradients are quite different from those specified by NCHRP, AASHTO, and Gross and Burns. Temperature at the lower deck gauge was underestimated by the design gradients using all other methods. The shape of the negative measured gradients is reasonably similar to the design negative gradients specified by AASHTO LRFD. Therefore, only a modified design positive thermal gradient is recommended as shown in Fig. 8. Note that the temperature at depth of 0.36 m (14 in) from the top is defined as 2.5 °C (4.5 °F) and at 0.10 m (4 in) from the bottom it is zero. Clearly further study is required prior to implementation to check or modify the model, which has been developed from this study alone.

Table 3 Maximum Thermal Gradients and Time of Occurrence

	MAXIMUM MEASURED THERMAL GRADIENTS				HIGHEST AVERAGE MEASURED THERMAL GRADIENTS FOR A CALENDAR MONTH			
Girders	B13	B14	B23	B24	B13	B14	B23	B24
Positive Gradient	36.0	23.1	25.6	19.6	26.2	16.7	18.2	14.3
	7/25/02	7/25/02	7/25/02	7/25/02	Jul-02	Jul-02	Jul-02	Jul-02
Negative Gradient	9.0	10.1	7.1	9.3	5.4	6.6	5.4	6.4
	11/19/01	2/12/02	10/6/01	2/14/02	Feb-02	Feb-02	Jun-02	Feb-02
All temperatures in °F. °C = (°F - 32)/1.8								

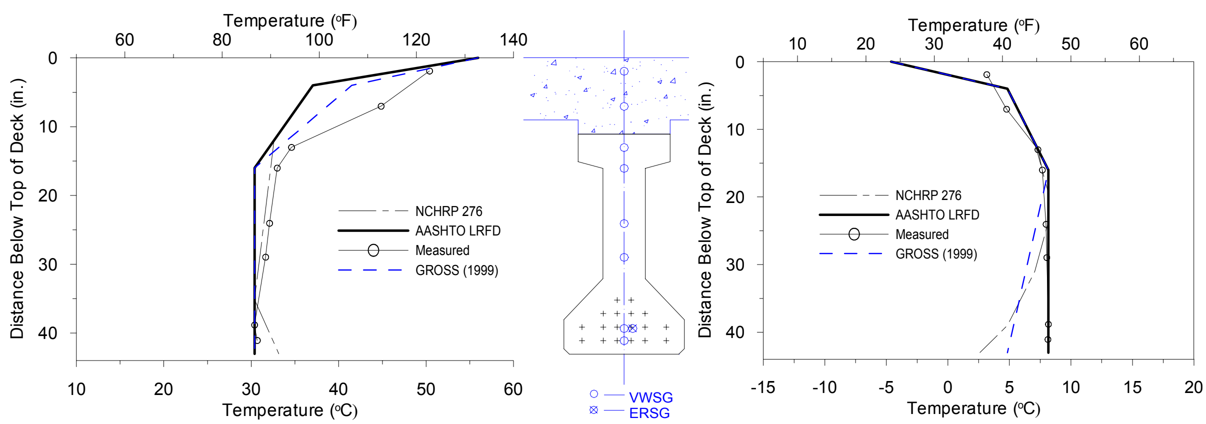


Fig. 7 Design Thermal Gradients and Maximum Measured Gradients

Using the method suggested in AASHTO guide specification on thermal effects in concrete bridge superstructures[8], the theoretical thermal stresses and strains resulting from the maximum measured positive gradient in composite beam B13 are shown in Fig. 8. It can be seen that the calculated strains correlate reasonably well with the measured strains. Here temperature and strain differences were taken between the 8:00 AM reading on the day of the maximum positive gradient and the 4:00 PM reading. Self equilibrating thermal stresses using different design gradient shapes were compared and illustrated in Fig. 8. It can be found that thermal stresses below the CIP deck using any design methods were very close to those calculated using measured thermal gradients. However, the stresses using measured thermal gradients were quite different from those calculated using known design methods in the deck except the method suggested by the authors. Stresses resulting from unfactored live load and impact are also shown for comparison. It can be clearly seen the thermal stresses are relatively small in magnitude in this specific project and unlikely to cause any distress.

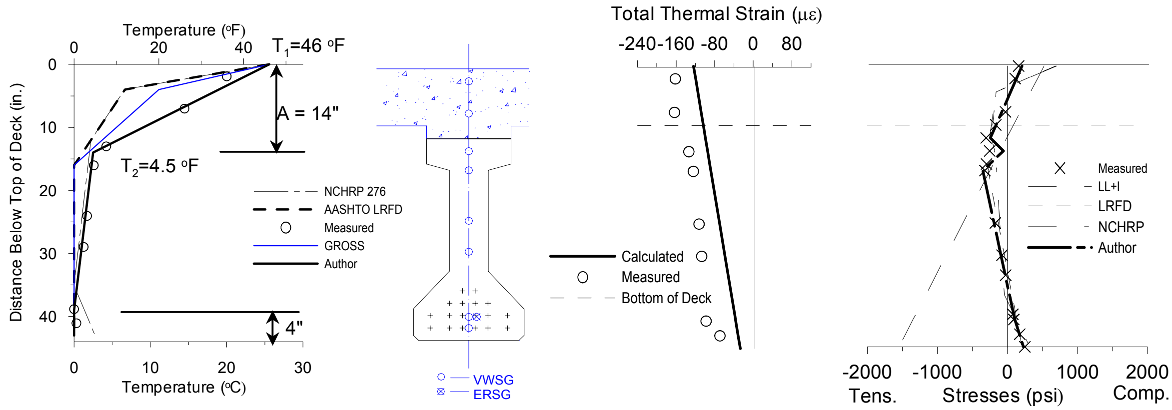


Fig. 8 Recommended Positive Thermal Gradients and Thermal Strains & Stresses

CONCLUSIONS

The following conclusions are drawn based on the test results and analysis:

1. The maximum measured hydration temperature in the HPC precast / prestressed beams was 57.2°C (135°F). Equivalent maximum temperatures in both HPC beams and the CIP deck are well below peak values suggested by ACI Committee 363[5] largely due to the mix constituents and bridge sections utilized.

2. The methods for effective bridge temperature suggested in the AASHTO Standard Specification, LRFD Specification and NCHRP Report 276 Method are inappropriate for the bridge locations monitored in this study. A modified approach is developed to provide more realistic design temperatures.

3. Maximum measured positive gradients are quite different from those specified by NCHRP, AASHTO, and Gross and Burns. Temperature at the lower deck gauge was underestimated by the design gradients using all other methods. The shape of the negative measured gradients is reasonably similar to the design negative gradients specified by AASHTO LRFD.

4. Thermal stresses below the CIP deck using any design methods were very close to those calculated using measured thermal gradients. However, the stresses in the deck using measured thermal gradients were quite different from those calculated using known design methods except for the method suggested by the authors. Thermal stresses for this structure were relatively small in magnitude compared to stresses resulting from unfactored live load (plus impact) and unlikely to cause any distress.

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