Connecticut Permanent Long-Term Bridge Monitoring Network
Volume 1: Monitoring of Post-Tensioned Segmental Concrete Box-Girder Bridge – I-95 over the Connecticut River in Old Saybrook (Bridge #6200)

Prepared by: Adam Scianna, Stephen Prusaczyk,
Zhaoshuo Jiang, Richard E. Christenson, John T. DeWolf

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Prepared for:
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James A. Fallon, P.E.
Manager of Facilities and Transit
Bureau of Engineering and Construction

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16. Abstract

This report describes the instrumentation and data acquisition for an eleven span segmental, post-tensioned box-girder bridge in Connecticut. Based on a request from the designers, the computer-based remote monitoring system was developed to collect temperature data to provide engineers with information for use in the evaluation of the long-term behavior and performance of the bridge. The system was used over a five year period to determine maximum and minimum temperatures through the box girder cross section and to develop thermal gradients in both the vertical and transverse directions. Comparisons have been made with design specification provisions and with recommendations proposed by previous researchers. In addition, software has been developed to determine the relationship between the daily maximum temperature differences and the air temperatures inside the box girder. This approach is also used to develop the relationship between the maximum stresses due to temperature differences and the air temperature inside the box girder. The monitoring system is currently being upgraded.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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INTRODUCTION

Researchers at the University of Connecticut and in the Connecticut Department of Transportation have been using field monitoring to explore the behavior of bridges during the past two and a half decades (Lauzon and DeWolf, 2003). This report is based on the research project that was developed to place long-term monitoring systems on a network of bridges in the state (DeWolf, Lauzon and Culmo, 2002; Olund and DeWolf, 2007; DeWolf, Cardini, Olund and D'Attilio, 2009). The first system was installed in 1999, and since then five other bridges have been added to the network. The bridges have been selected because they are important to the state's highway infrastructure and because they are typical of different bridges types. Each monitoring system has been tailored to the particular bridge, using a variety of sensors, and all data is collected remotely. As with many of our busier highways, it is not possible to close a bridge for monitoring, and thus all systems collect data from normal vehicular traffic. The goal of this research has been to use structural health monitoring to learn about how bridges behave over multi-year periods, to provide information to the Connecticut Department of Transportation on the behavior of the state's bridges, and to develop structural health monitoring techniques that can be used to show if there are major changes in bridges' structural integrity.

The current four-year phase in this long-term project has focused on: installation and implementation of monitoring systems on two new bridges; substantial upgrading of the monitoring equipment, with addition of video collection; and, development of techniques for long-term structural health monitoring. Specifically for this bridge, during the current project the monitoring system was replaced, which included removal of the previous data acquisition system and replacement with National Instruments CompactDAQ hardware connected to a Small Form Factor PC. The new data acquisition system allows for enhanced capabilities, including improved sensor resolution, anti-aliasing of accelerometer signals, internet connectivity for viewing and archiving of data, and flexibility for future expansion. These efforts are documented within the report.

This report involves the Baldwin Bridge (Inventory Number 6200). The bridge crosses the Connecticut River in the southern part of Connecticut. It carries the Interstate I-95 over the Connecticut River, pictured in Figure 1. An elevation is shown in Figure 2.

Figure 1. Baldwin Bridge

Figure 2. Bridge Elevation

The bridge spans in the east west direction. The bridge is 2522 feet in length from abutment to abutment, and it consists of eleven continuous spans with span lengths varying from 177 feet to 275 feet. There are two separate post-tensioned bridges, each with a single cell, one carrying the eastbound traffic and the other carrying the westbound traffic. Figure 3 shows the cross section, and Figure 4 shows a view of the interior of bridge.

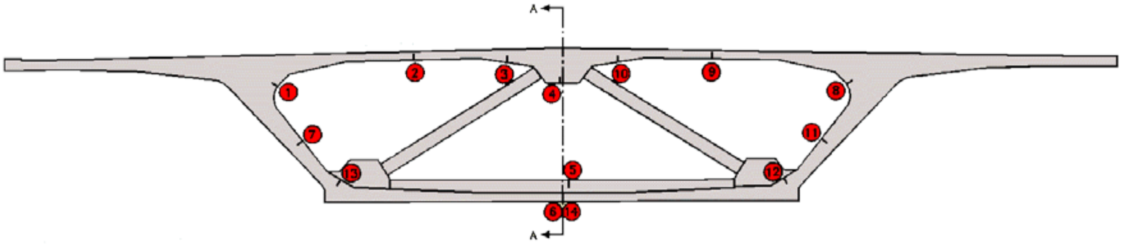


Figure 3. Bridge Cross Section

Figure 4. View of Bridge Interior

Each bridge has four lanes with shoulders on each side. There is a 2-inch asphalt layer on the top of the box girder section. An elevation of the full bridge is shown in Figure 5.

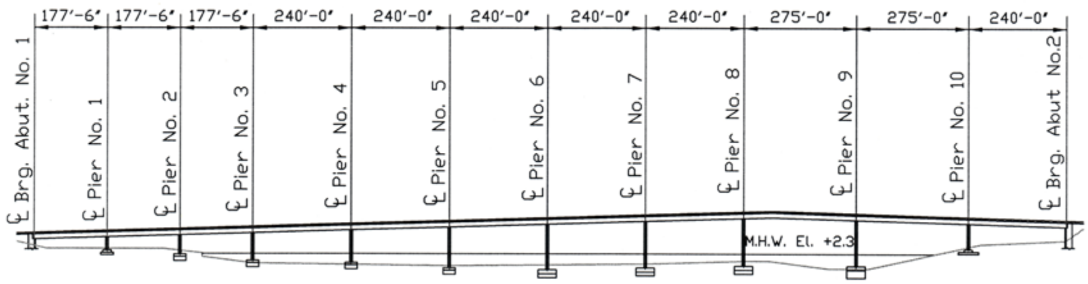


Figure 5. Bridge Elevation

Prior to the installation of the monitoring system, visual inspection determined that there were large cracks in the web near the interior piers. They extended primarily in the diagonal direction. The designers of the bridge recommended that temperature monitoring of the bridge would be useful to explore the cause of this cracking. This need was used as the basis in the design of the long-term monitoring system on this bridge.

This report is based on the M.S. thesis work of Paramita Mondal (Mondal 2004) and papers by Mondal (Mondal and DeWolf, 2004; Mondal and DeWolf, 2007).

OBJECTIVES AND SCOPE OF STUDY

The primary interest in monitoring this bridge has been to address the observed cracking behavior and to evaluate its long-term influence on the overall behavior of the bridge. The cracking was thought to be due to temperature variations, and thus the monitoring system was designed to evaluate these variations over the cross section in both horizontal and vertical directions. The data would be collected over a multiple-year period in order to provide information on how the temperature changes influence the cracking behavior. Of interest were differences due to location in the cross section and due to annual changes.

The evaluation of the field data would include the development of computational techniques to explore the behavior of the bridge and to provide a continuous picture of the daily maximum temperature differences in both the vertical and transverse directions. The temperature differences would be compared with the approaches given in the AASHTO Guide Specifications and with those proposed by other researchers. The data collected would also be used to determine the influence of temperature differentials on the bridge's structural behavior.

INSTRUMENTATION AND DATA ACQUISITION

The initial monitoring system used sixteen thermocouples, with fourteen spread over the cross section. One was placed inside the box girder to record the internal air temperature and one was placed in the instrument cabinet to assure that the monitoring equipment is not subject to temperatures outside the normal working range. The sensors were installed by drilling approximately 4-inch holes in the box girder on the inside of the box section. The thermocouple sensors are approximately 2 inches in length and are held in each hole by grout. Figure 6 shows one of the sensors next to a quarter.

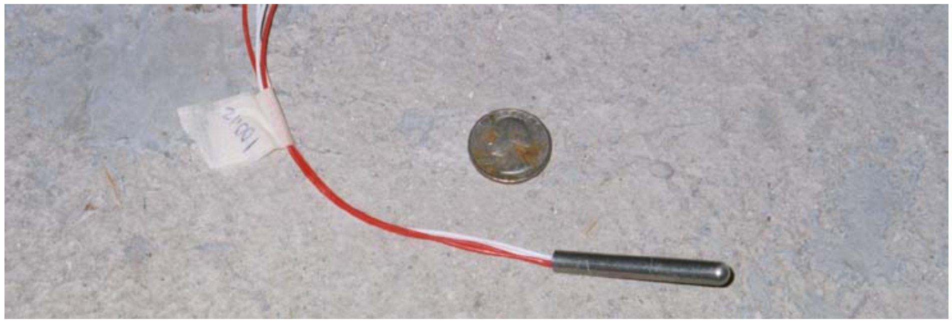


Figure 6. Thermocouple for Embedment into Concrete

Figure 7 shows the location of the sensors over the box section. All sensors were embedded in the interior of the box girders.

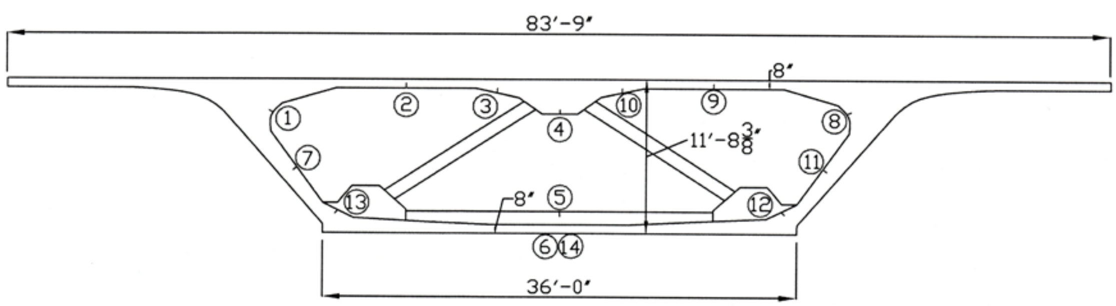


Figure 7. Bridge Cross Section with Location of the Thermocouples

Since the bridge extends in the east-west direction, the temperatures were assumed to be constant in the longitudinal direction; i.e., the only variations in the temperature would be in the vertical and horizontal directions. Thus the monitoring system was placed between two piers, numbered two and three in Figure 5. The extensive data collected is expected to fully define the influence of temperature on the full bridge's performance.

All the thermocouples were connected to an on-site computer through a data logger. The onsite laptop computer was connected to the monitoring system through a GPIB PCMCIA card. The computer could control the monitoring system using a program written in HP VEE. The bridge monitoring system was remotely accessed using a dial-up modem with a commercial software package. Recently, the system has been converted to a DSL connection. The monitoring system was first operational in September 1999.

The monitoring system is capable of scanning up to 60 channels per second. Since only temperature data is collected, data was normally collected at fifteen-minute intervals, beginning in September 1999. Temperature changes occur over relatively long time periods, and the fifteen-minute intervals are considered fully adequate for evaluating the behavior of the bridge due to temperature variations.

Temperature data can be analyzed on a daily, monthly or yearly basis and saved accordingly. It was decided to collect and store data in separate files on a monthly basis in the monitoring computer at the University of Connecticut. It was felt the monthly basis would provide a reasonable division in the extensive data collected for long-term comparisons. When reading the monthly data file, the software analysis program first validates data and discards any erroneous data.

The software was developed to calculate the maximum, minimum and average temperatures recorded by each sensor over specific time periods. The software also calculates vertical and horizontal temperature differences, and it calculates stresses due to the temperature differences.

When the system was designed and built for initial installation in 1999, it was necessary to develop extensive software to collect the data, conduct the analysis, and then transmit information to remote monitoring sites. The monitoring system is now undergoing extensive upgrading, as reported later in this report.

DATA ANALYSIS

The following data analysis shows what was learned from the first five years of monitoring. This includes the basic temperature variations with respect to the cross section and air temperature, how thermal stresses influence the structural behavior, and the relation between the maximum daily compressive stress and the box girder and air temperatures. A more extensive evaluation of the data and the analysis of the data have been presented by Mondal in her M.S. Thesis at the University of Connecticut (Mondal, 2004) and in related papers by Mondal and DeWolf (2004; 2007). What follows summarizes what has been learned from the temperature monitoring.

Temperature Variations in the Box Girder Related to Longitudinal Behavior

This section compares data from two typical months: January, which is representative of the colder part of the year and July, which is representative of the hotter part of the year in Connecticut. The months chosen were from 2002. Additional studies demonstrated that the overall results for these two months are comparable to data recorded in other years.

Figure 8 shows temperatures recorded by the fourteen thermocouples embedded in the concrete box girders for January, and Figure 9 shows the temperatures for July. Typically, the data ranges from upper 30s to low 50s in January and between 70s and 100 degrees in July.

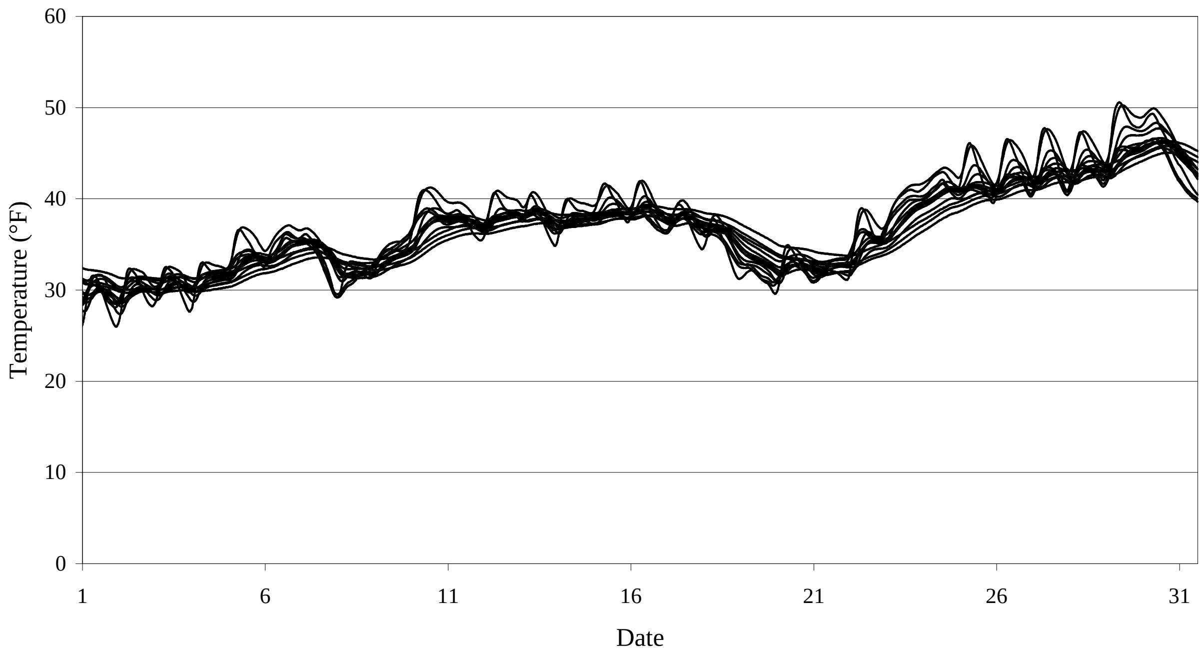


Figure 8. Temperatures Recorded in January

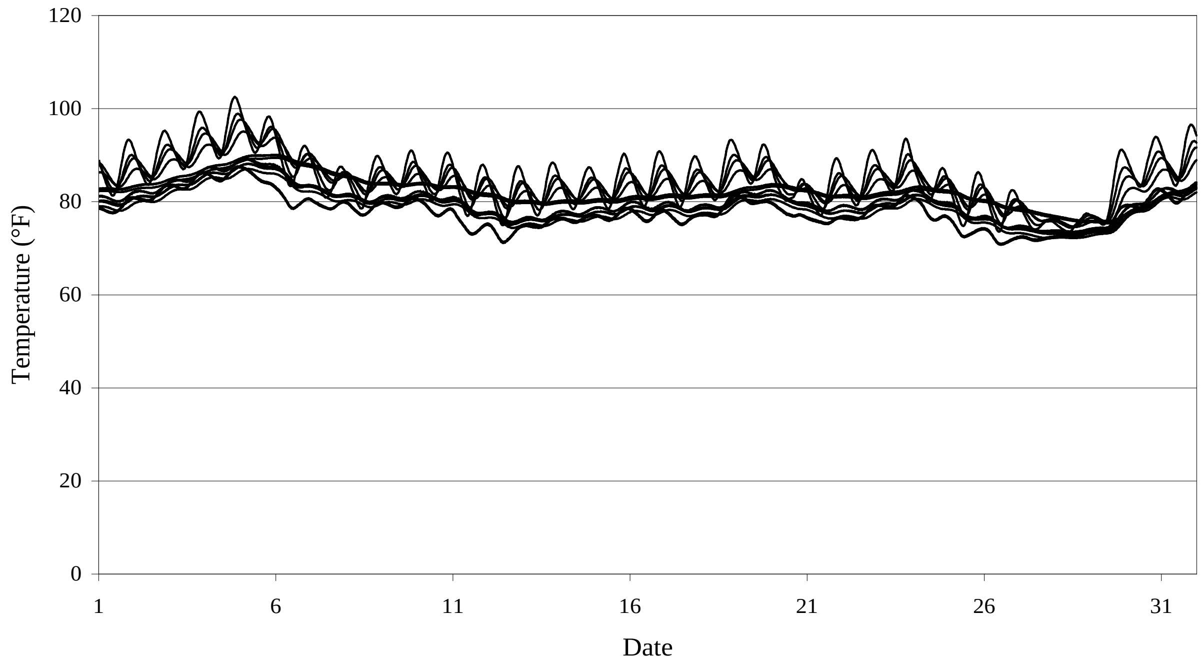


Figure 9. Temperatures Recorded in July

Typically, the higher temperatures are those from the thermocouples at the top of the box girders closest to the deck. The comparison of the data for January and July shows that the daily temperature variations are larger in the summer. Normally, all of the sensors reach their minimum or maximum temperatures simultaneously. Figure 10 shows the frequency distributions for the time associated with the maximum temperature for the months of January and July.

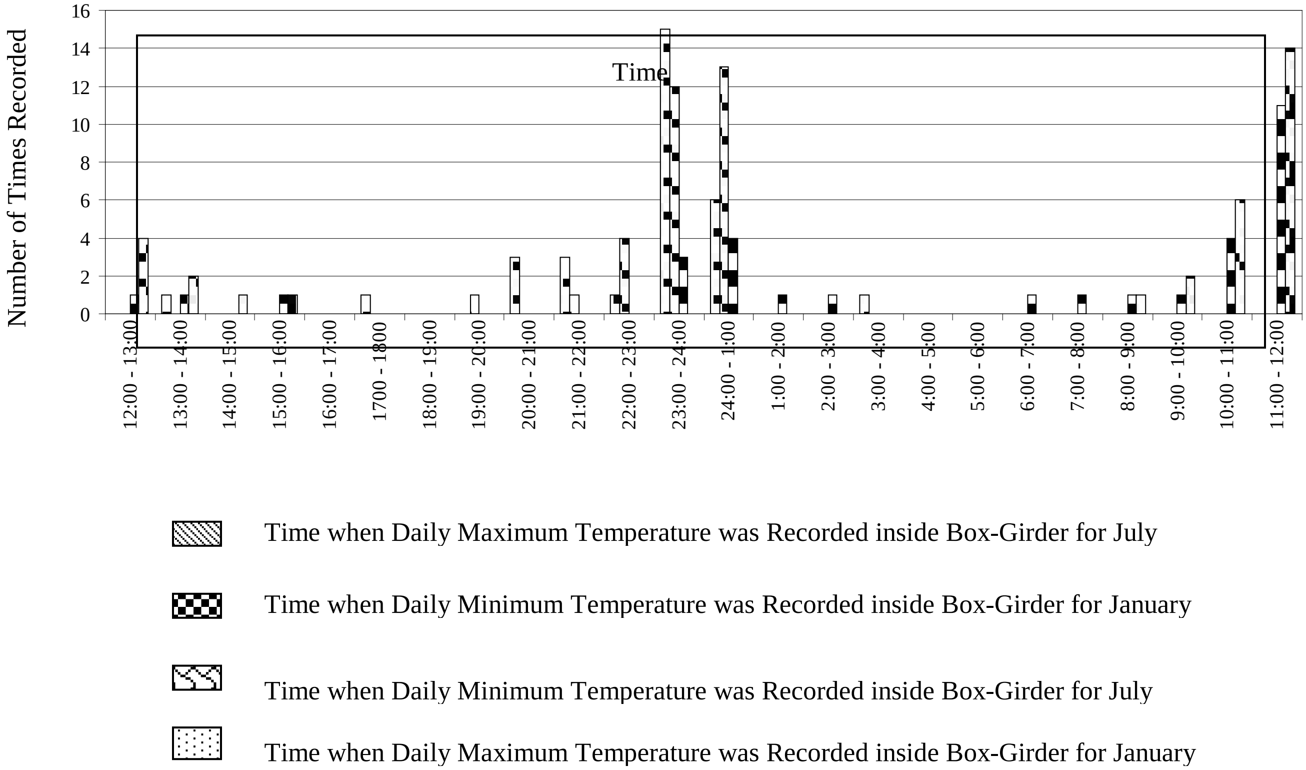


Figure 10. Frequency Diagram for Time Associated with Daily Maximum or Minimum Temperature

Figure 10 shows that the maximum temperature in the box girder, corresponding to the maximums in the thermocouples, is reached typically between 11 pm and 1 am. This is true for individual thermocouples, and it is generally true at all other times during the year. These maximums are reached approximately 10 to 12 hours after the time when the outside air temperature is at a maximum.

Temperature Variations in the Box Girder Related to the Bending Behavior

Variations in effective bridge temperatures in the longitudinal direction result in expansion and contraction of the bridge superstructure. These movements had to be taken into account in the design of the bridge. Excessively large design movements would result in large displacements at the joints and bearings, resulting in increased maintenance expenses. In addition to the longitudinal displacements, the differences in the temperatures in the vertical and horizontal directions result in bending of the box girders, and the resulting stresses must be added to the girder stresses due to the dead and live loads.

The American Association of State Highway and Transportation Officials Guide Specifications (1989) that were used at the time of design has recommendations that can be used to estimate the thermal behavior based on an approach that has been in use since the 1920s. Roeder (2003) developed an improved approach to estimate the thermal movement. He considered the relationship between the effective bridge temperature and climatic conditions for different bridge types, and then he used this to develop design recommendations. Other studies that have used temperature data collected in the field on different concrete bridge types have been carried out by Roberts-Wollman, et al. (2002), Shushkewich (1998), Elbadry and Ghali (1983) and Inaudi et al. (2002).

The *AASHTO Guide Specifications, Thermal Effects in Concrete Bridge Superstructures* (1989) relates the effective bridge temperature to the normal daily air temperature and the type of bridge. The effective bridge temperature is needed to determine the influence of temperature variations on the longitudinal displacements and resulting stress levels. The extreme value for the maximum effective bridge temperature for concrete bridges in Connecticut determined from the AASHTO specification is 86° F. For the same bridge type and location, Roeder (2003) has proposed a value of 99° F. Figure 11 shows the average maximum effective bridge temperature T_{AvgMax} calculated during a four-year period with the extreme values given by AASHTO and Roeder.

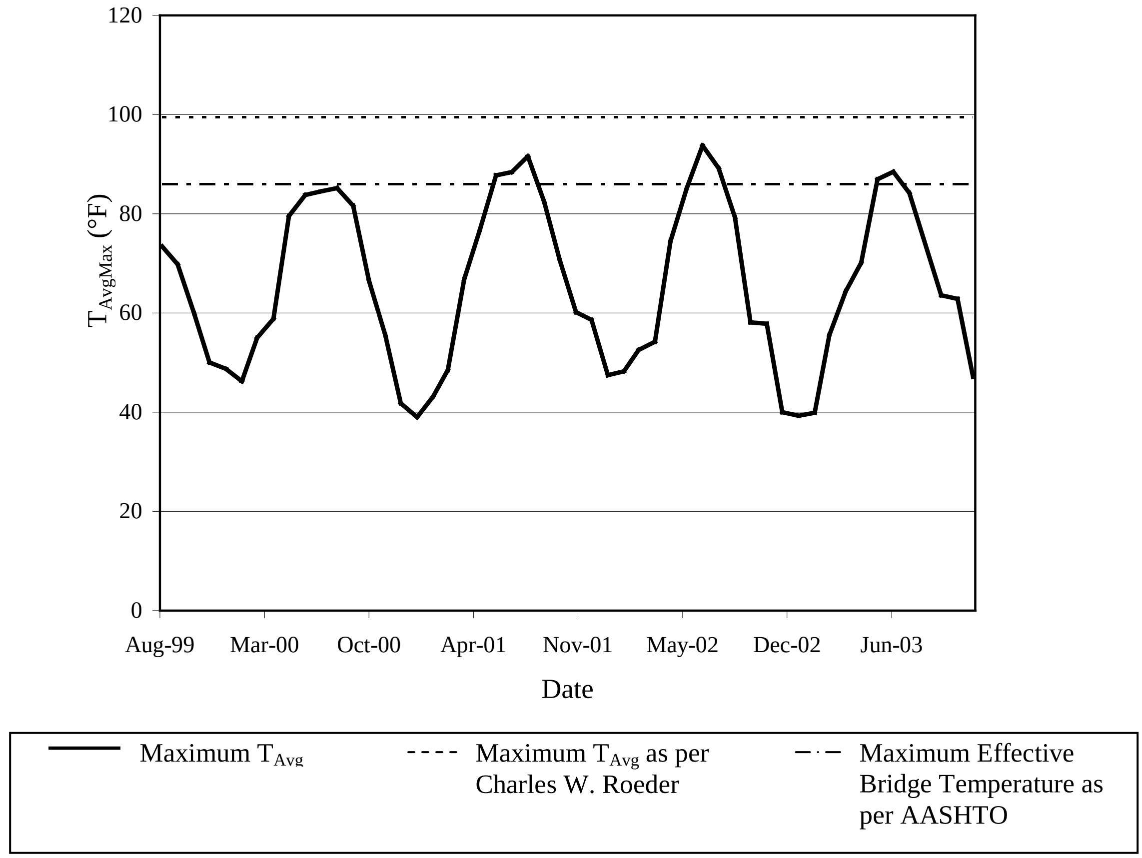
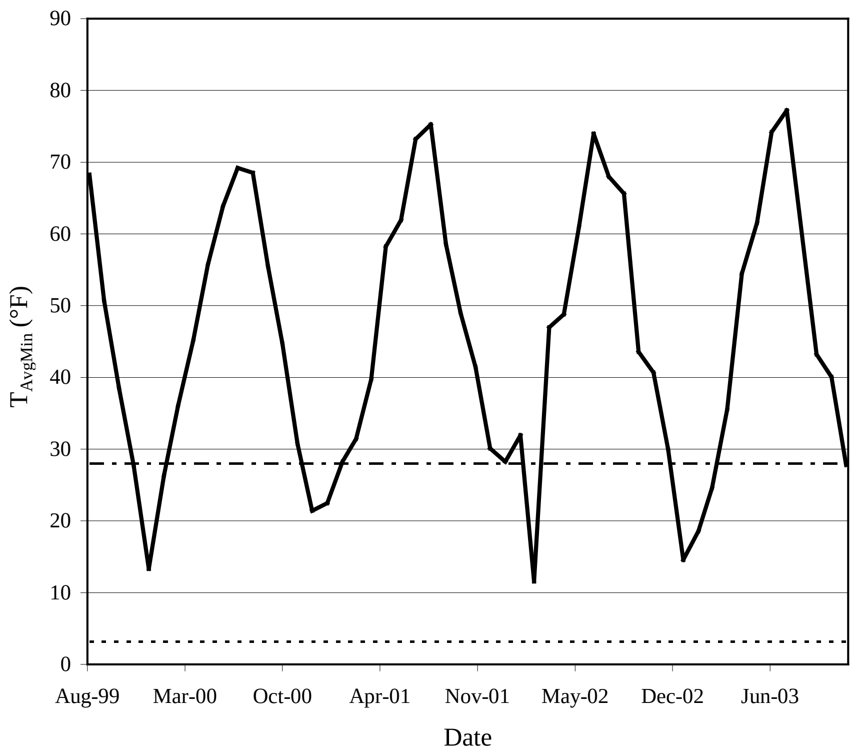


Figure 11. Average Maximum Bridge Temperatures

During the summer, the effective bridge temperature T_{AvgMax} exceeded the value suggested by AASHTO for considerable periods. The maximum value of the effective bridge temperature recorded was 94° F. This signifies that during the summer the bridge elongates more than it would using the values specified by AASHTO.

AASHTO (1989) specifies that the extreme value of the minimum effective bridge temperature for concrete bridges in Connecticut is 28° F. Roeder has proposed using a value of 3° F. Figure 12 shows the average minimum bridge temperature calculated during the four-year monitoring period with the extreme values suggested by both AASHTO and Roeder.



Note: See Figure 11 for legend

Figure 12. Average Minimum Bridge Temperatures

It is clear from Figure 12 that the minimum effective bridge temperatures values are considerably less than the AASHTO values during the winter, with a minimum value of 12° F.

In addition to the effective bridge temperatures which are related to the longitudinal displacements, it also is necessary to look at the influence of the vertical and horizontal temperature differences in the determination of the thermal stresses. The maximum vertical temperature differences result in bending stresses that add or subtract from the dead and live load bending stresses. Vertical temperature differences were determined on each side of the box girder cross section and are plotted for July in Figure 13.

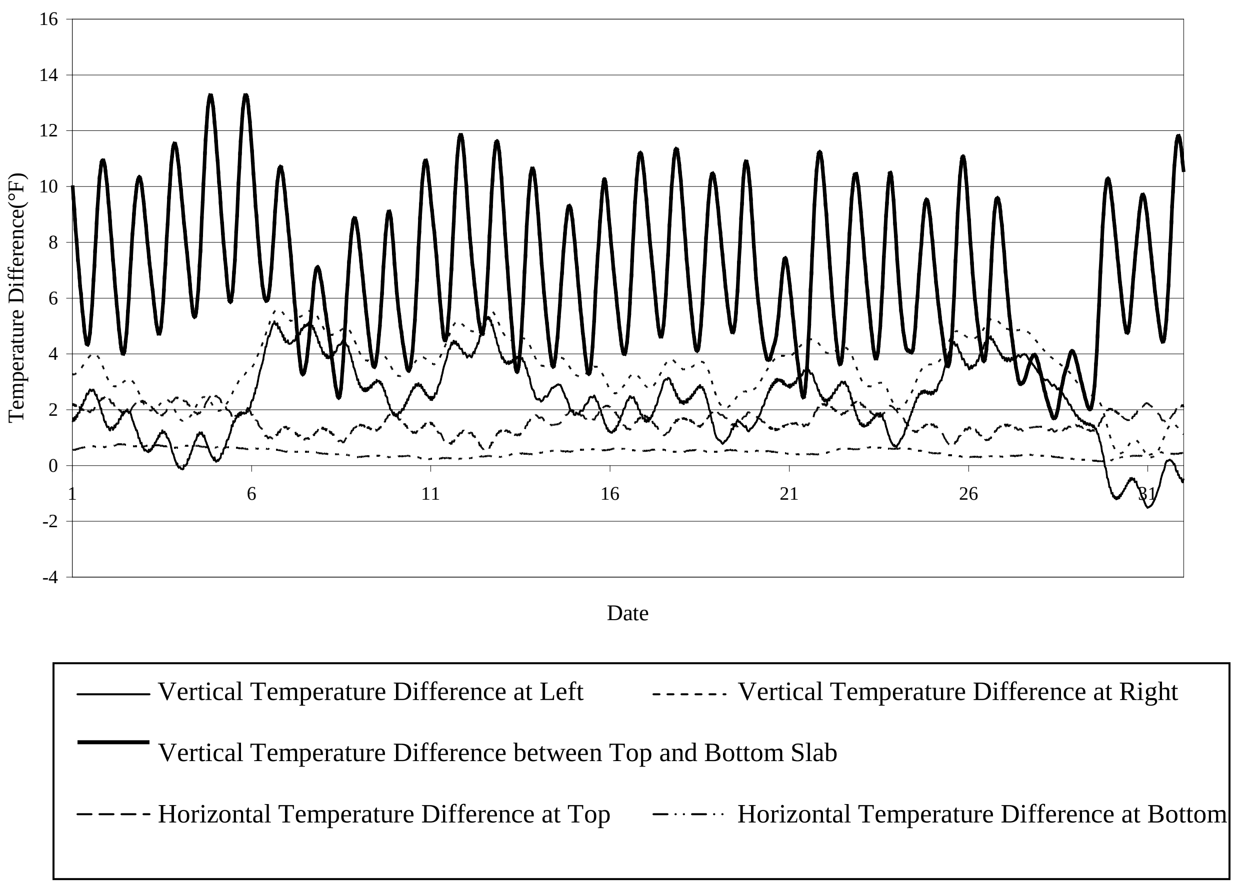


Figure 13. Temperature Differences in July

As shown, the vertical temperature difference is generally positive; i.e., temperature at the top is higher than at the bottom. The graph for the vertical difference shows discrete peaks and valleys corresponding to the daily cycle. Further analysis of the data has shown that the maximum vertical temperature difference generally occurs between 7 pm and 9 pm. Figure 14 shows the vertical temperature differences for January 2002.

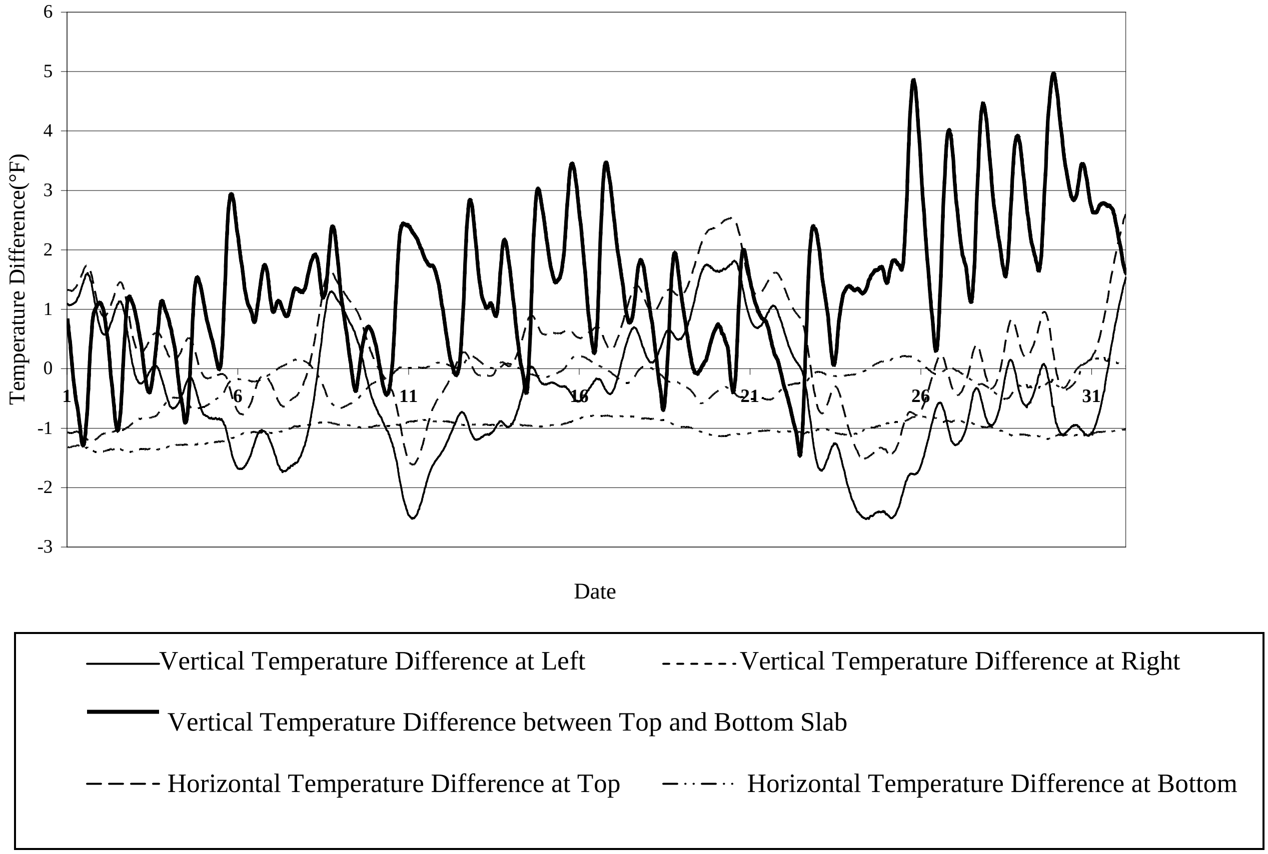


Figure 14. Temperature Differences in January

During the winter, the vertical temperature difference has values both positive and negative during several consecutive days. Also, as shown, the maximum temperature difference is considerably smaller than that in the summer. During the summer, the maximum daily vertical temperature difference typically is in the range of 9° F to 13° F, with a maximum of 15° F recorded during the monitoring period.

Extensive analysis of the horizontal temperature differences across the width of the box girder has shown that they are considerably smaller than those in the vertical direction, and so it is only necessary to look at the vertical differences to determine the maximum stresses resulting from the temperature variation over the cross section.

Using the information in Figure 10, the daily maximum vertical temperature difference at the centerline and the daily maximum temperature inside the box girder generally occur at approximately the same time each day. Further study shows that there is a relationship between the maximum vertical temperature difference between the top and bottom of the slab and the air temperature inside the box girder. A statistical analysis has been carried out for year 2002, which is typical of the full monitoring period. Figure 15 shows the plot of daily maximum vertical temperature difference as a function of the daily average air temperature inside the box-girder.

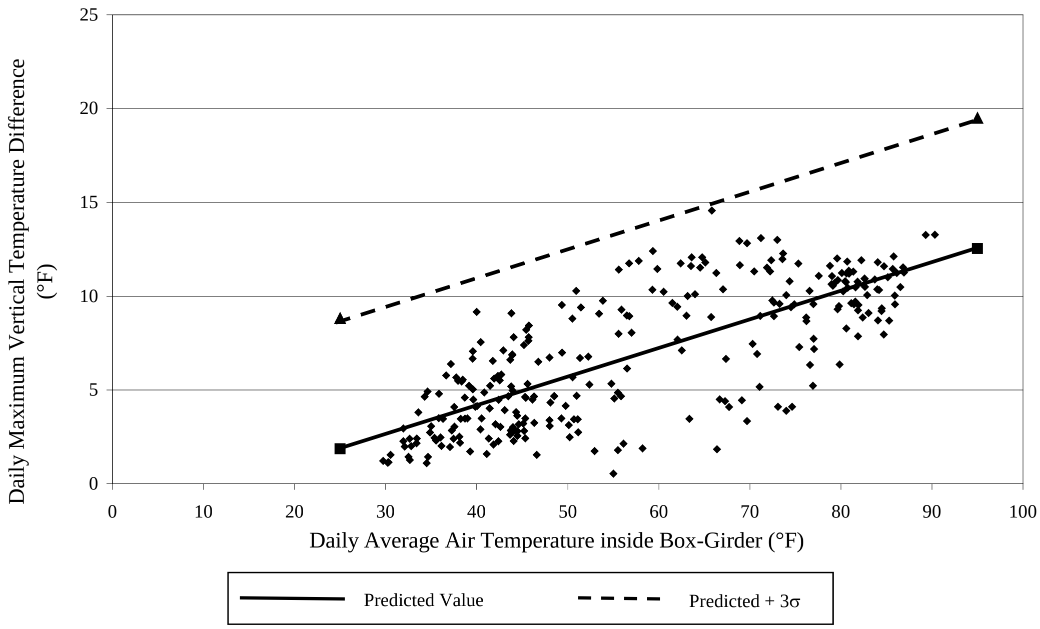


Figure 15. Maximum Daily Vertical Temperature Difference versus Daily Average Air Temperature inside Box Girder for Full Year

The maximum positive thermal gradient is shown in Figure 16.

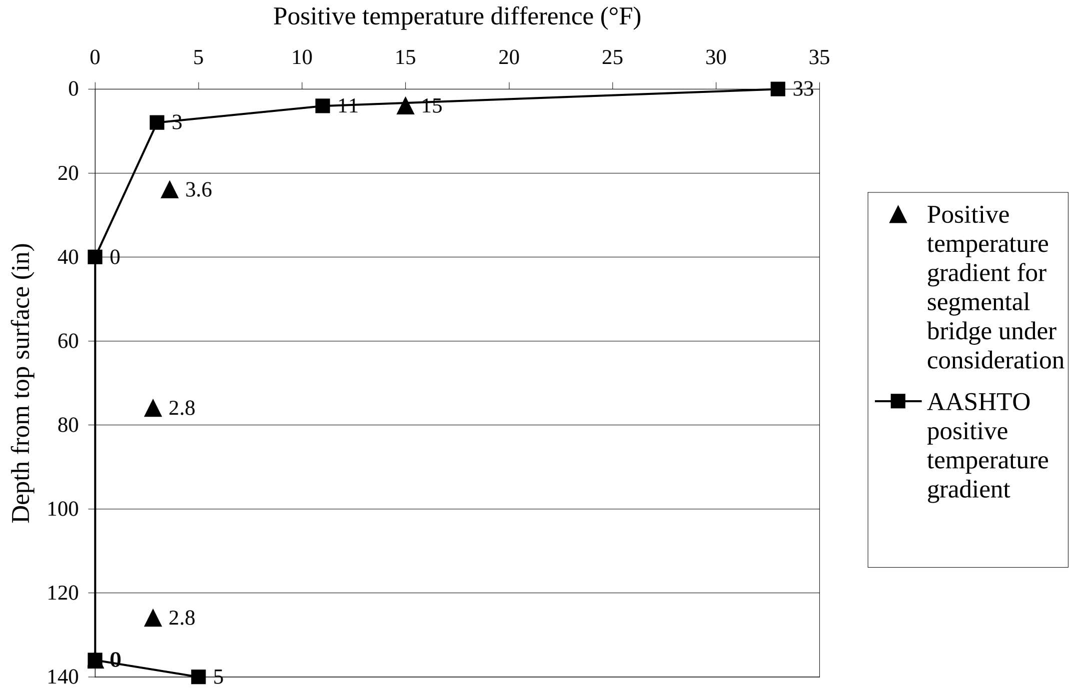


Figure 16. Maximum Positive Thermal Gradient

The thermal gradient is drawn using the thermocouples in the top slab, the web and the bottom slab. Since the sensors were installed on the interior of the box girder, they do not give the extreme top and bottom temperatures, so it was necessary to use a linear interpolation. The maximum temperature difference observed between the top-most sensor and that closest to the bottom is 15° F.

The field data have also been compared to the temperature variation given by the AASHTO Guide Specifications Thermal Effects in Concrete Bridge Superstructures (1989). Using this Specification, the temperature gradient at the top surface for a concrete bridge with 2 inches of asphalt in Connecticut should be 33° F, and at 4 inches below the top surface, it should be 11° F.

Thus, the observed temperature difference at 4 inches below the top (15° F) is higher than the specified AASHTO value.

Influence of Thermal Stresses on the Structural Behavior

Mondal (2004) used an approach given by Shiu and Tabatabai (1996) and the *AASHTO Guide Specifications, Thermal Effects in Concrete Bridge Superstructures* (1989) to estimate stress levels due to the temperatures variations. This is briefly described in the following.

When a simply supported beam that is free to expand and contract is subjected to a linear positive vertical thermal gradient, the beam will expand in the longitudinal direction, and it will bend upward without thermal stresses. Due to the vertical thermal gradient, the concrete at different depths will try to expand according to this temperature difference. When this thermal gradient is non-linear, as shown in Figure 16, there are constraints at these different levels that lead to stresses that are known as Eigen stresses. The final stress at a particular section in the beam cross section is the summation of the Eigen stress and the stresses developed due to the support conditions. The stresses developed due to the support conditions are known as secondary stresses. In a continuous beam, the secondary stresses develop due to the intermediate supports, and they are known as continuity stresses.

Because it is not possible to determine the amount of longitudinal constraint in the bridge (i.e., how much the bridge is free to expand and contract), the two extreme cases were considered in calculating the stresses developed due to the vertical temperature difference. These two cases give the two extreme stress values. The first is based on the assumption that the bridge is free to expand longitudinally, and the second is based on the assumption that the bridge is fully restrained in longitudinal direction. In the actual case, the stresses developed in the bridge cross section are somewhere in between these two extremes.

If the bridge is considered to be free to elongate and contract in the longitudinal direction, the total stress at a particular cross section is the summation of the Eigen stress and the continuity stress at the section. Mondal (2004) and Mondal and DeWolf (2007) have shown that in this case the maximum value of the compressive stress at the top sensor location is 0.23 ksi, and the maximum value of the tensile stress at the bottom sensor location is 0.19 ksi. Using the maximum vertical temperature gradient specified by AASHTO (1989), the compressive stress at the top sensor location would be 0.07 ksi, which is lower than the actual stress. The AASHTO specification distribution can also be used to get the stress at the top of the girder. This stress, interpolated from the field stress at the sensor location, is 0.56 ksi. Using the AASHTO specification, the tensile stress at the bottom sensor location would be 0.17 ksi, which compares with the value of 0.19 ksi as determined from test data.

If the bridge is considered constrained at the supports in the longitudinal direction (i.e., if the expansion joints allow expansion and contraction), the resistance to the expansion and contraction induces additional strains and stresses. In the extreme situation, the bridge is fully prevented from longitudinal movement, resulting in an increase in the compressive stress at the top. In this case, variation of stress through the depth of the cross section is same as the nonlinear temperature gradient through the depth. The resulting compressive stress calculated in this way at the top sensor location is 0.33 ksi, based on the extreme temperature variations recorded during the monitoring period. Using the AASHTO specification stress distribution, this stress at the top sensor location would be 0.24 ksi, and the stress at the top surface would be 0.73 ksi. Thus, assuming that the bridge is not free to expand and contract, the resulting stress at the location of the top sensor is higher than predicted by AASHTO.

Relation of the Daily Maximum Compressive Stress to the Average Box-Girder Air Temperature

In this section an approach is developed to predict the extreme stress values developed at the top and the bottom sensor locations due to the vertical thermal gradient, using the air temperature inside the box girder.

The relationship between the daily maximum compressive stress at the top sensor location for the maximum positive vertical temperature difference each day and the daily average air temperature inside the box girder is shown in Figure 17. This is based on a statistical analysis of the data from 2002, which is typical of other years.

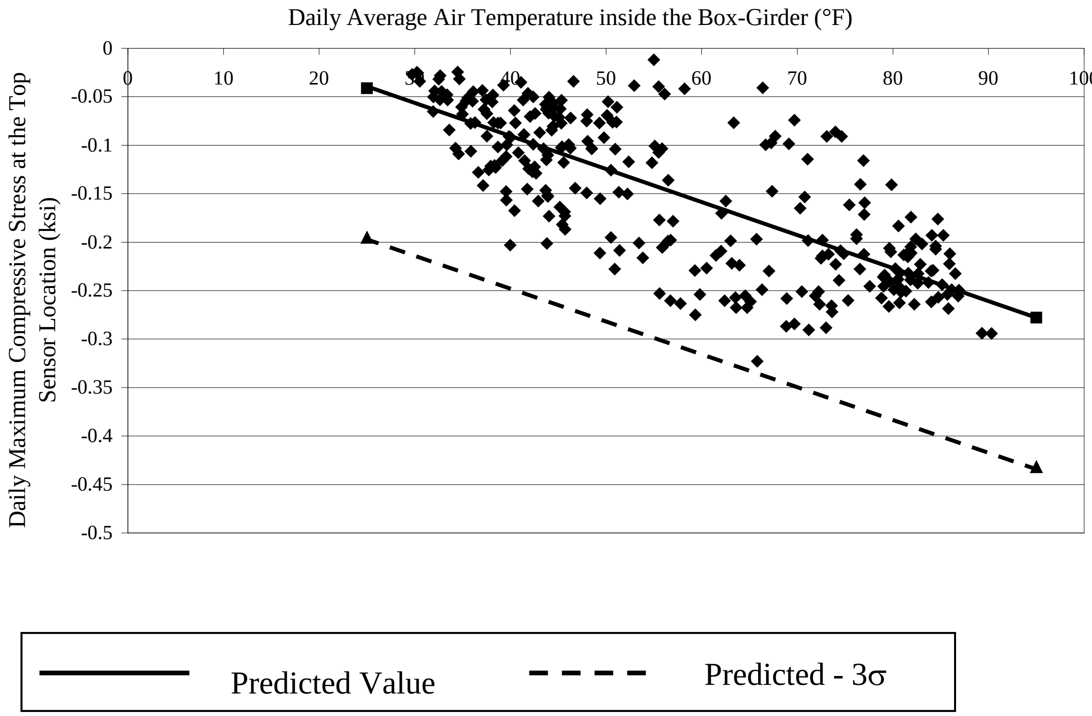


Figure 17. Daily Maximum Compressive Stress at the Top Sensor Location versus the Daily Average Air Temperature in the Box Girder for Full Year

From Figure 17, the equation of the regression line that can be used to predict the compressive stress developed at the top sensor location due to maximum vertical temperature difference is

$$\sigma_{Comp} = -0.0022T_{Avg} + 0.026$$

where σ_{Comp} is the predicted maximum compressive stress at the top sensor location due to maximum vertical temperature difference and T_{Avg} is the average air temperature inside the box girder. This has a standard deviation of σ = 0.045 ksi. Figure 17 also shows a dashed line, located 3 times σ = 0.045 ksi below the value given by the equation. This equation has been further verified using data for the other years during the monitoring.

Using the same approach, the relationship between the daily maximum tensile stress at the bottom sensor location and the daily average air temperature inside the box-girder is shown in Figure 18.

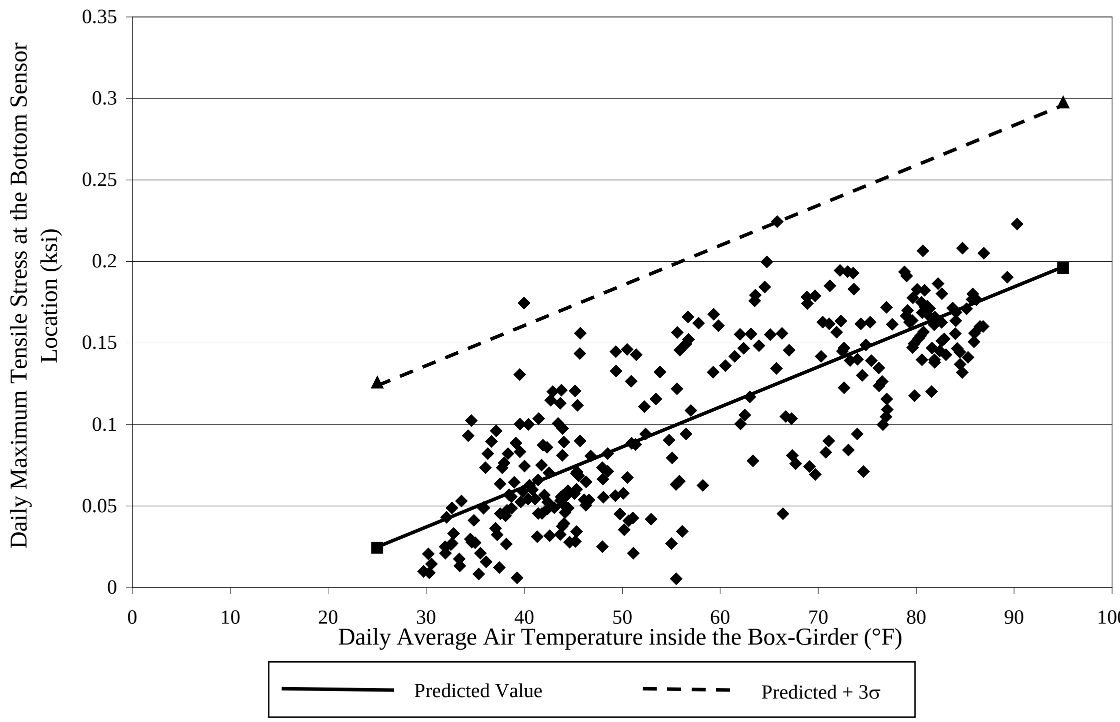


Figure 18. Daily Maximum Tensile Stress at the Bottom Sensor Location versus the Daily Average Air Temperature in the Box Girder for Full Year

From Figure 18, the equation of the regression line that can be used to predict the tensile stress developed at the bottom sensor location due to maximum vertical temperature difference is

$$\sigma_{Ten} = 0.00245T_{Avg} - 0.037$$

where σ_{Ten} is the predicted tensile stress at the bottom sensor location due to the maximum vertical temperature difference and T_{Avg} is the average air temperature inside the box girder. This has a standard deviation σ = 0.034 ksi. Figure 18 also shows a dashed line, located 3 times σ = 0.034 ksi above the value given by the equation.

An additional study was made to review the relation between the atmospheric temperature and the stresses developed in the bridge (Mondal, 2004). This was done using the maximum, minimum and average daily atmospheric temperatures for 2002 and 2003. The equation for the daily maximum compressive stress at the top sensors is

$$\sigma_{Comp} = -0.0026T_{Max} + 0.061$$

where σ_{Comp} is the predicted compressive stress at the top sensor location due to the maximum vertical temperature difference and T_{Max} is the daily maximum atmospheric temperature. The standard error is σ = 0.040 ksi.

The equations for the daily maximum tensile stress at the bottom sensor is

$$\sigma_{Ten} = 0.00224T_{Max} - 0.035$$

where σ_{Ten} is the predicted tensile stress at the bottom sensor location due to the maximum vertical temperature difference of a day and T_{Max} is the daily maximum atmospheric temperature. The standard error is σ = 0.039 ksi.

DESIGN OF NEW MONITORING SYSTEM

The initial monitoring system ran continuously from September 1999 to December 2003. Data was also collected between June 2004 and August 2005. The preceding data analysis has been based on this monitoring period. As was shown, the stress levels are low. Along with the fact that the propagation of the cracks declined during this period, it was concluded that no new information would be learned by continuing to collect only temperature data. Thus data collection was stopped. In 2009, the decision was made that the system would be upgraded with new hardware and additional sensors, based on what was learned from upgrading of the monitoring systems on other bridges in the long-term research project.

Consistent with efforts to upgrade the monitoring systems and capabilities on other bridges in the project, the monitoring system was replaced in July 2010. This included removal of the previous data acquisition system, with replacement with National Instruments CompactDAQ hardware connected to a Small Form Factor PC. This CompactDAQ has four modules installed that provide power to the sensors and collect data measurements from the sensors previously installed on the bridge. These modules not only support the input of RTDs, but they can measure resistance, voltage, and current as well. This combined with the remaining four expansion slots on the CompactDAQ will enable researchers to add a wider variety of sensors on the bridge for the purposes of structural health monitoring. The updated bridge monitoring system at the Baldwin Bridge provides:

- improved resolution of the sensor measurements with the 24-bit system;
- connectivity to the Connecticut Department of Transportation computer network over the internet, allowing for full access to the bridge monitoring computers;
- potential for real-time remote viewing of the bridge monitoring data from any PC on the ConnDOT network using a java-based Real-Time Data Viewer (RTD);
- capability for automated data archival to an offsite FTP server; and
- flexibility to expand the current system to new sensors.

Work is now underway to expand the system so that in addition to temperature, sensors will be added to monitor deformations. Of interest are monitoring of the cracks, monitoring of potential deformations at the box girder joints, and movements at the supports.

CONCLUSIONS

The results from five years of data collection were used to develop an automated system for evaluation of the extensive data from the temperature monitoring of this large, continuous multi-span, concrete box-girder bridge. The results of the study have been used to compare the field data to that predicted by the AASHTO specification and guidelines suggested by other researchers.

Comparisons have been made with the AASHTO specification, demonstrating that the specification underestimates the stress levels due to temperature variations. Nevertheless, the estimated stress levels using the field temperature data were consistently below 1 ksi.

A procedure was developed to estimate the compressive stress at the top and the tensile stress at the bottom of the girders, using either the average air temperature inside the box girder or the atmospheric temperature.

The bridge monitoring system has recently been redesigned to upgrade the control system to make it consistent with other bridges used in the overall research project. The redesign has been based on what has been learned from the analysis of the field data to date.   It is expected that some sensors will be moved and others will be replaced with different sensors. This will lead to further changes to the software developed for this bridge.

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