

CONCRETE BRIDGE DECK BEHAVIOR UNDER
THERMAL LOADS

by

Jeffrey Keith Johnson

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Approved for the Department of Civil Engineering

Dr. Brett Gunnink

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I want to thank the people who have encouraged and inspired my pursuit of education and to share with them the following definition as it illustrates those impressions they have left for me to admire:

philomath \FIL-uh-math\, noun:

A lover of learning; a scholar.

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ABSTRACT

A major area of concern with concrete bridge decks is durability. The service life of bridge decks designed by traditional procedures is often shorter than desired. Typically, the decks crack under environmental loads, which lead to corrosion of the reinforcing steel and general deterioration of the concrete.

In this investigation, the response of three different bridge decks was studied in the field under environmental loads. The decks, located within a mile of each other on the same road, differ in their steel reinforcement and the type of concrete used; otherwise, they are identical. Therefore, they offer an opportunity to observe how these factors affect their behavior under environmental loads.

This investigation was divided into two tasks. In the first task, the basic response of the bridge deck concrete under changes in temperature was examined in the laboratory using specimens cast with the decks. The second task correlated the laboratory findings with measured responses in the bridges and examined the global thermal response of the decks.

The laboratory investigation showed that concrete has a distinct nonlinear strain response to temperature changes. This response is related to the concrete's nanostructure, which affects the behavior of water within the paste during the heating and cooling process. The deck concretes' coefficient of thermal expansion was generally found to range from 6 to 14 $\mu\epsilon/^\circ\text{C}$ as the temperature changed from -35°C to 40°C , and distinct differences were seen in the relationship between the strain and temperature for high performance versus standard concrete.

The bridge investigation showed that locally the thermal response of the bridges was similar to that measured in the laboratory. Globally, the transverse bridge response to changes in temperature was as expected (expansion and contraction with increasing and decreasing temperatures, respectively). Importantly, there were no significant differences in behavior between the three bridges. In reviewing the strain response to thermal events, it was discovered that anomalies in this response may be indicators of physical damage within the deck.

Because of the Saco bridges relatively young age (two years), continued investigation will be necessary as they mature and environmental degradation more readily presents itself.

CHAPTER 1

INTRODUCTION

Background

The overall objective of the Saco Bridge project is to evaluate the long term performance of three concrete bridge decks constructed with varying concrete type, strength and reinforcement configuration. Vehicle live load tests conducted at an age of two months showed only slight differences in the strength performance of the three bridges (Smolenski 2004). An equally important aspect of the decks' response is their behavior under long term environmental loads. The ability of bridges to withstand weathering is of special importance in Montana, because vehicular traffic is low and environmental conditions are harsh.

Bridge deck structures can undergo large material strains when exposed to environmental changes, namely large variations in temperature. Saco bridge deck temperatures during the first year of monitoring ranged from -40°C to 40°C . For this temperature range, thermal strains of up to $\pm 500\ \mu\epsilon$ could be produced, depending on the coefficient of expansion for the deck concrete (typically 7.0 to $14.0\ \mu\epsilon/^{\circ}\text{C}$) and the degree of fixity provided by the bridge superstructure. These thermally induced strains are of the same magnitude as the tensile cracking strains exhibited by plain concretes (from 100 to $200\ \mu\epsilon$) and are much larger than the maximum static strains measured during live load testing of the Saco Bridges (approximately $120\ \mu\epsilon$).

As a result of cracking produced by these thermally induced strains, it is likely that the service life of concrete bridge decks are shortened by: 1) the reduction of effective deck stiffness of the structure, which increases the strains within the rebar reinforcement, and 2) the access of chemicals used in deicing operations to the interior of the deck. Penetration of corrosive agents, such as deicing chemicals, can accelerate corrosion of the steel reinforcement. The corrosion byproducts greatly increase in volume (approximately 400 percent) and cause the concrete to crack and/or delaminate.

By measuring the strain levels across the concrete decks of actual bridge structures, it may be possible to obtain a better understanding of their strain response under environmental loads. Current design procedures can then be appropriately revised to improve bridge deck performance. Determining internal stresses and strains due to thermal loading requires accurate values of the coefficient of thermal expansion (CTE) for the bridge deck concrete. Notably, these coefficients may vary with the type of concrete used (i.e., standard or high performance concrete); thus they could play a critical role in understanding differences in performance between bridge decks made with different concretes.

Objective and Scope

The objective of this investigation was to study the response of the three Saco Bridge decks under environmental loads. The Saco bridge decks specifically differ in their steel reinforcement and the type of concrete used (see Table 1). Both of these factors are expected to affect their behavior under environmental loads. One deck, referred to as the

conventional (CON) deck, was designed following traditional Montana Department of Transportation (MDT) practices. A second deck, referred to as the empirical (EMP) deck, was designed using the AASHTO Empirical Deck Design procedure, which results in considerably less reinforcement in the deck than the conventional design. Finally, the third deck was constructed using high performance concrete (the HPC deck), coupled with a traditional reinforcement layout.

Table 1. Bridge Characteristics

Bridge Designation	Deck Reinforcing Configuration	Deck Concrete
Conventional (CON)	AASHTO LRFD	Conventional
Empirical (EMP)	AASHTO Empirical	Conventional
High Performance (HPC)	AASHTO LRFD	High Performance

The behaviors of the decks were continuously monitored using vibrating wire strain gages cast in them during construction. These gages measure concrete strain at various locations of interest throughout each deck. This analysis focused on determining the thermal strains that developed in the concrete during temperature changes. A necessary precursor to investigating the response of the decks to temperature changes was to determine the thermal behavior of the materials from which they were constructed. Specifically, CTEs were experimentally determined for each deck concrete using strain gage technology. The laboratory CTE values were then compared with bridge CTE values determined by strain gages. Ultimately, the curvatures and the axial in-plane

strains were calculated for each deck using the data from the vibrating strain gages. These aspects of the response were compared with and the expected temperature related deformations.

CHAPTER 2

LITERATURE REVIEW

The first comprehensive investigation into the thermal effects on bridge decks in the United States was National Cooperative Highway Research Program (NCHRP) Report 276, Thermal Effects in Concrete Bridge Superstructures, issued in 1985. An abridged version of this report was then published by the American Association of State Highway and Transportation Officials (AASHTO) as a recommended guide specification (rather than as a modification to the AASHTO design specifications). The current AASHTO LRFD Bridge Design Specifications now incorporate the recommendations of NCHRP Report 276 (with slight modifications).

Bridge Design for Thermal Events

The temperature behavior of bridges is typically viewed as two superimposed effects. The first effect is from a uniform change in temperature that occurs over the entire superstructure. This temperature event causes an overall length change for an unrestrained structure. If the structure is restrained, a uniform temperature change will produce internal stresses in the structure. The second event is a temperature gradient that occurs when the bridge superstructure heats unevenly (such as when the sun substantially heats the deck surface or a freezing rain falls on a warm day). The gradient temperature is only applied in the vertical direction, and no consideration is given to uneven heating longitudinally or transversely to the bridge deck.

The setting (installation) temperature, as defined by AASHTO, for the bridge or any of its components is determined by averaging the actual air temperature over the 24-hour period immediately before the setting event. The setting temperature is the reference temperature relative to the direction and magnitude of subsequent thermal behavior under temperature changes. The setting temperature is of specific interest when installing expansion bearings and deck joints. In addition to setting temperature, the range of temperatures the deck will cycle through is of interest in design.

AASHTO divides the country into two regions for determining uniform temperature ranges that bridges will experience based on the number of expected freezing days (temperatures less than 0°C). Moderate climates are defined as having fewer than 14 freezing days per year; conversely, cold climates have 14 days or more freezing days. This method is merely a general indication of the length of time a bridge deck could be expected to be at a colder temperature but not necessarily the actual temperature experienced. As expected, Table 2 indicates that cold climatic regions are to be designed for colder temperatures. The temperature range considered in design also depends on the bridge material. Metal bridges have quicker response to changes in temperature, where the thermal mass of the concrete bridges acts as a temperature regulator, reducing its ability to reach peak temperatures. Wood bridges have a further reduced temperature range because of their superior performance to temperature changes (critical strains for wood are much higher than those of concrete and steel and do not result in brittle failures).

Table 2. Uniform Temperature Ranges (from AASHTO 2000)

Climate	Steel or Aluminum (°C)	Concrete (°C)	Wood (°C)
Moderate	-18 to 50	-12 to 27	-12 to 24
Cold	-35 to 50	-18 to 27	-18 to 24

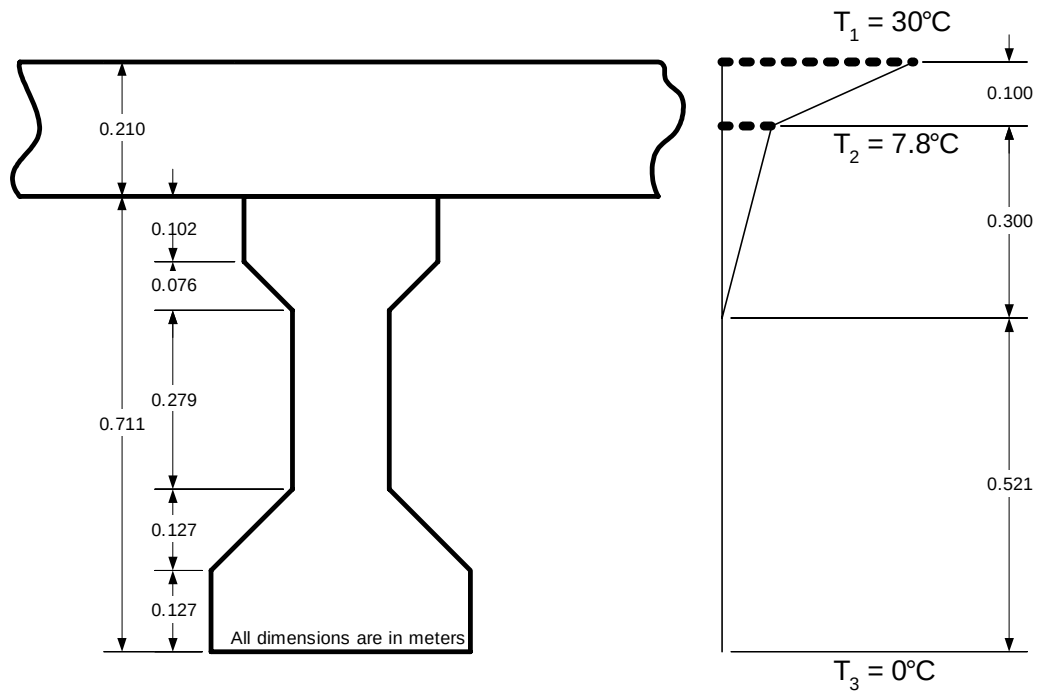
Knowing Saco, Montana has a cold climate; thus, referring to Table 2, a uniform temperature range of -18°C to 27°C would be used for design (concrete deck in a cold climate). The deck concrete setting temperature was measured to be approximately 25°C. These results imply that the Saco bridges will experience uniform temperature changes of:

$$\begin{aligned}\Delta T_{pos} &= 27 - 25 = 2^{\circ}\text{C} \\ \Delta T_{neg} &= 25 - (-18) = 43^{\circ}\text{C}\end{aligned}\tag{Eq.1}$$

Relative to temperature gradients, AASHTO divides the United States into four solar radiation zones, with zone 1 receiving the most and zone 4 the least solar radiation (Table 3). Zone 1 encompasses most of the Western United States, giving the Saco bridges the temperature gradient shown in Figure 1. Negative gradient temperatures (gradient under decreasing temperature) are obtained by multiplying the positive temperatures by -0.3. The lower magnitude negative temperature gradient reflects the different heating and cooling events for the bridge decks. The positive temperature gradients occur when during the summer the top of the deck is warmer than the soffit; the negative temperature gradient develops on winter nights when the deck temperature is cooler than the soffit. The summer event develops a much larger temperature gradient than the winter event.

Table 3. Gradient Temperatures (AASHTO 2000)

Zone	T_1 (°C)	T_2 (°C)
1	30	7.8
2	25	6.7
3	23	6
4	21	5

Figure 1. Saco Bridge Gradient Temperature Distribution
(for positive gradient temperature)

where:

T_1 = Temperature that develops at the top of the bridge deck

T_2 = Temperature that develops at location 0.1 meters below the top of the
bridge deck

T_3 = Temperature that develops in the bottom of the superstructure

Response Analysis

Several methods of analysis exist to determine the effect of temperature changes on structures. For simplification, the following assumptions are made when performing the thermal analysis of bridge structures following the bridge deck specifications (AASHTO 1989):

1. The material is homogeneous and exhibits isotropic behavior.
2. Material properties are independent of temperature.
3. The material has linear stress-strain and temperature-strain relations. Thus, thermal stresses can be considered independently of stresses or strains imposed by other loading conditions, and the principle of superposition holds.
4. The Navier-Bernoulli hypothesis that initially plane sections remain plane after bending is valid.
5. The temperature varies only with depth, but is constant at all points of equal depth (except those points over an enclosed cell).
6. Longitudinal and transverse thermal responses of the bridge superstructure can be considered independently and the results superimposed; i.e., the longitudinal and transverse thermal stress fields are assumed to be uncoupled.

AASHTO LRFD Bridge Design Specifications (Section 4.6.6) provides the following method for examining the force effects due to temperature deformations. The structure's response is divided into three effects: 1) axial expansion, 2) flexural deformation, and 3) self equilibrating stresses.

Axial expansion is caused by the uniform component of the temperature gradient. The uniform component of the temperature gradient is calculated as the average temperature across the cross section:

$$T_{UG} = \frac{1}{A_C} \iint T_G dw dz \quad (\text{Eq.2})$$

where:

T_{UG} = Temperature average across the cross-section

A_C = Transformed cross-section area

T_G = Temperature gradient

w = Width of element in cross-section

z = Vertical distance from centroid of cross-section

The uniform axial strain is:

$$\varepsilon_u = \alpha \cdot [T_{UG} + T_U] \quad (\text{Eq.3})$$

where:

α = Coefficient of thermal expansion

T_U = Uniform specified temperature

Flexural deformation is a result of plane sections remaining plane under a linear temperature gradient. This curvature is determined as:

$$\kappa = \frac{\alpha}{I_T} \iint T_G z dw dz \quad (\text{Eq.4})$$

where:

κ = Curvature

I_T = Transformed inertia of cross-section

Structures that are externally unrestrained develop no external forces. In a fully restrained structure, however, axial forces develop as:

$$N = E_C A_C \varepsilon_u \quad (\text{Eq.5})$$

where:

$$E_C = \text{Modulus of elasticity of the concrete}$$

Moment forces from the flexural deformation also develop as:

$$M = E_C I_C \kappa \quad (\text{Eq.6})$$

The internal “self-equilibrating” stresses are determined by first allowing the free expansion of the material fibers for the temperature load; from this analysis the axial force developed for the uniform temperature case, and the moment developed for the gradient temperature case are subtracted. The resulting stress distribution is termed the continuity stresses, which are required to keep plane sections plane.

$$\sigma_E = E_C [\alpha \cdot T_{UG} - \alpha \cdot T_G - \kappa \cdot z] \quad (\text{Eq.7})$$

Thermal Behavior of Concrete

The behavior of concrete during temperature changes is complex and has been shown to vary significantly between concrete mixtures. Concrete material models (including those that address thermal behaviors) are incomplete and subjective because concrete continues to change physically and chemically with, among other things, time, temperature, and stress. Furthermore, observed behaviors are related to the physical and chemical structure of the concrete on a nanometer scale. The various constituents of concrete that influence its behavior and their scale are illustrated in Figure 2.

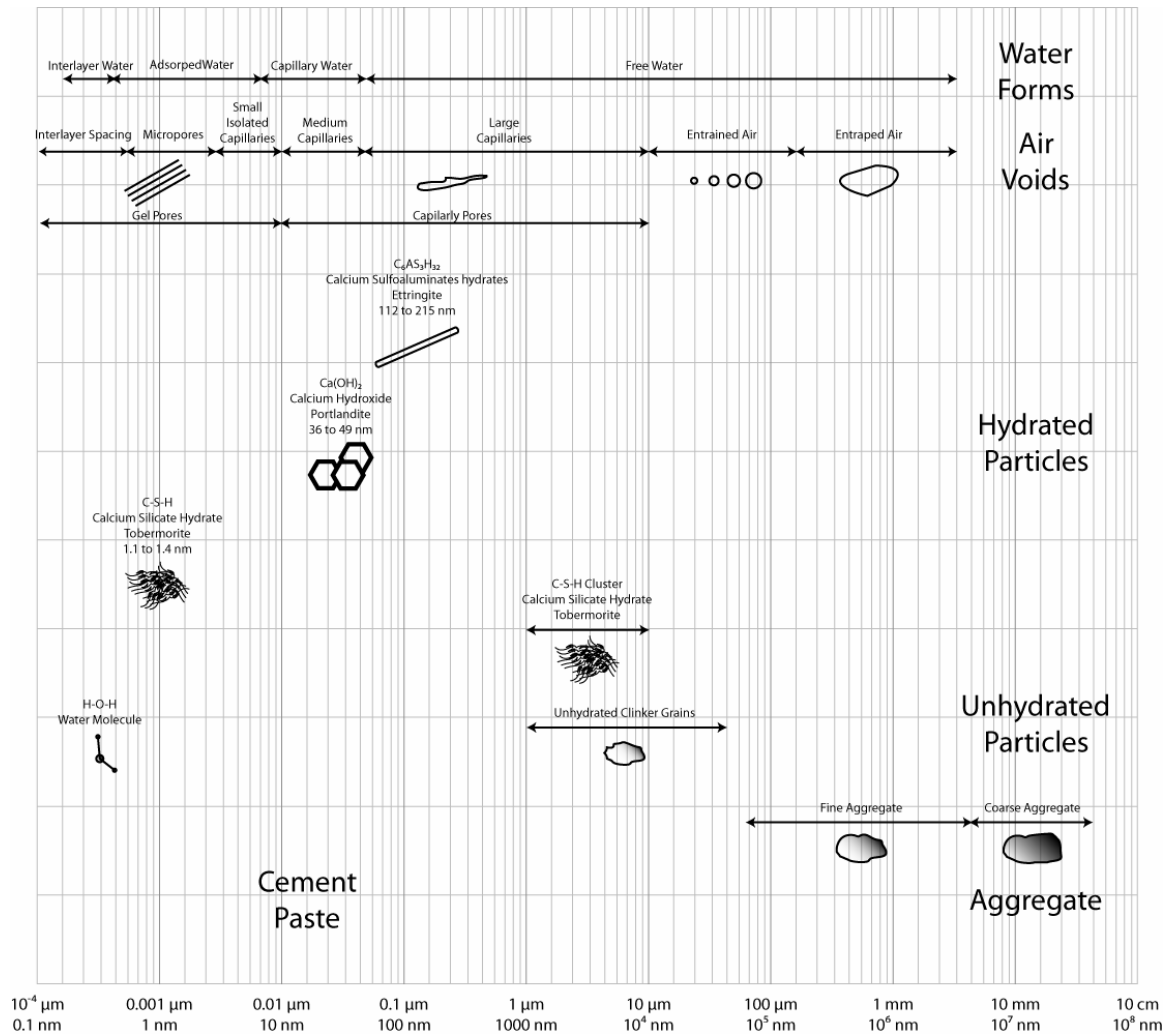


Figure 2. Concrete Structure (adapted from Mehta 1986)

Coefficient of Thermal Expansion

The thermal expansion of concrete varies with aggregate type, cement content, water-cement ratio, temperature range, concrete age and relative humidity (Kosmatka 2002). The CTE for concrete is highly dependent on the type of aggregate from which it is made. This situation is not surprising, since the aggregate typically makes up approximately 75 percent of the concrete's total volume. For design, the CTE value of

concrete is typically assumed to be constant, with a commonly reported value of $10.4 \mu\epsilon/^\circ\text{C}$ (approximately midway in the reported range of 7 to $14 \mu\epsilon/^\circ\text{C}$). This range is mostly a result of the aggregate used to make the concrete. A summary of concrete CTE values is shown in Table 4.

Table 4. Thermal Coefficients of Concrete (from NCHRP Report 276 1985)

Aggregate Type	Reported CTE $\mu\epsilon/^\circ\text{C}$			
	PCA (1982)	Emerson (1979)	Brown (1968)	Ontario (1979)
Quartzite	--	12.7	11.7-14.6	12.8
Quartz	11.9	--	9.0-13.2	--
Sandstone	11.7	11.7	9.2-13.3	9.5
Granite	9.5	9.6	8.1-10.3	9.5
Dolerite	--	9.6	--	9.5
Basalt	8.6	--	7.9-10.4	--
Marble	--	--	4.4- 7.4	--
Limestone	6.8	7.3	4.3-10.3	7.4

The aggregate source for the three decks in Saco was a gravel pit near Malta, Montana. The aggregate was believed to be primarily composed of quartzite, indicating a range of CTE values between 9.0 and $13.2 \mu\epsilon/^\circ\text{C}$. Table 5 shows common CTEs for all of the components of Portland cement concrete, as determined at the Turner Fairbanks Research Center (2005). The thermal properties of high performance concrete are within the same range as conventional concretes (Shah and Ahmad 1994). This result is not surprising, since the aggregate occupies the majority of the concrete's volume.

Table 5. Typical CTEs for Common Portland Cement Concrete Components
(from Turner Fairbanks Research Center 2005)

Material	CTE ($\mu\epsilon/^\circ\text{C}$)
Aggregate	
Granite	7 - 9
Basalt	6 - 8
Limestone	6
Dolomite	7 - 10
Sandstone	11 - 12
Quartzite	11 - 13
Marble	4 - 7
Cement Paste (saturated)	
w/c = 0.4	18 - 20
w/c = 0.5	18 - 20
w/c = 0.6	18 - 20
Concrete	7.4 - 13
Steel	11 - 12

While the general thermal behavior of concrete is dictated by the aggregate type, several secondary behaviors can be attributed to the thermal behavior of the water within the cement paste. These behaviors typically occur when the water within the cement paste changes phase. Berwanger (1971) identified a change in the strain slopes at around the freezing point of water for concrete specimens undergoing a temperature change. Powers (1947) showed that the freezing point of water in concrete was typically 5°C to 7°C below the standard freezing point. Powers associated this phenomenon with the alkali content of the water within the cement paste. In general, the water related

temperature behaviors are deeply affected by the nanostructure of the cement paste. Cement paste is composed of three primary constituents: 1) the cement gel, which consists of the hardened hydration products, 2) unhydrated cement particles and 3) pores of varying shape and size filled with water or air (excluding the gel pores, and interlayer spaces). Cement gel has the following components (Kumar Mehta 1986):

- Tobermite: Calcium silicate hydrate [C-S-H] ~50-60% Volume
- Portlandite: Calcium hydroxide [Ca(OH)₂] ~20-30% Volume
- Ettringite: Calcium sulfoaluminate hydrates [C₆A $\hat{S}$ ₃H₃₂] ~15-20% Volume

Where the following standard cement chemistry conventions have been employed:

C = CaO

S = SiO₂

A = Al₂O₃

F = Fe₂O₃

$\hat{S}$ = SO₃

H = H₂O

There are two generally acknowledged models for the cement gel, namely, the Powers-Brunauer model (Powers 1947) and the Feldman-Sereda model (Feldman 1968). Both models rely on the theory of adsorption for determining the specific area of the cement gel. Adsorption is a method of determining the surface area of a material by the volume of a gas that is adsorbed by the material's surface at a constant temperature and pressure. As the pressure is increased, the gas penetrates into smaller voids. A relationship can be developed between surface area and void size at a given pressure.

The Powers-Brunauer model used water vapor for adsorption while the Feldman-Sereda model used nitrogen gas. Following the water vapor approach, the cement gel's specific surface was estimated to be $200 \text{ m}^2/\text{g}$ to $500 \text{ m}^2/\text{g}$, which was from three to five times larger than that estimated using nitrogen gas ($50 \text{ m}^2/\text{g}$). The Powers-Brunauer model explains this difference by noting that the water molecule has a smaller diameter (0.325 nm) than the nitrogen molecule (0.405 nm) (Popovics 1998). The Feldman-Sereda model contends that the difference is due to the interaction of the water vapor with the interlayer water formed within the C-S-H formations. Because direct observation of the cement gel structure is not possible (scanning electron microscopes only have a resolution in the micrometer range while the gel structure is defined at the nanometer range), the true structure must be inferred by empirical experiments. Popovics (1998) suggests that the Feldman-Sereda model best explains the current observations.

The Feldman-Sereda model (see Figure 3) leads to the existence of water in the following states: 1) capillary water, 2) adsorbed water, 3) interlayer water, and 4) chemically combined water (Kumar Mehta 1986). As will be discussed in Chapter 3, the behavior of the water within each physical state and its movement between states may help explain dimensional changes in the cement paste associated with changes in its temperature.

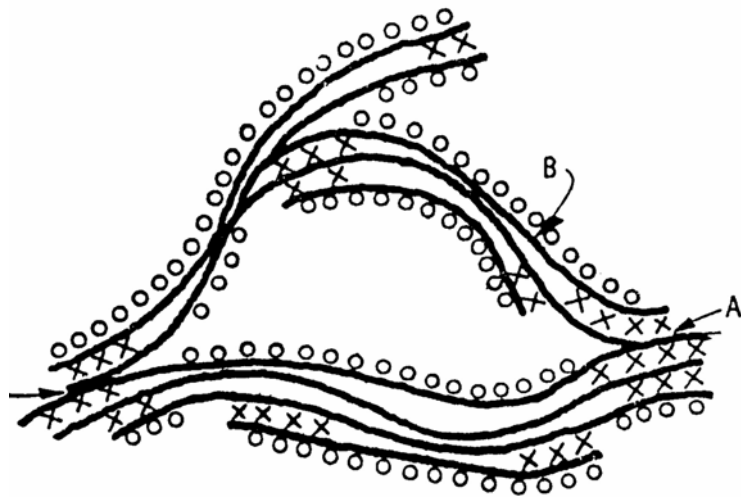


Figure 3. Simplified Feldman-Sereda Model for the Structure of Cement Gel:

A - interparticle Bonds: x – interlayer water:
 B – tobermorite sheets: o – physically adsorbed water
 (from Feldman and Sereda 1968)

Capillary water is the water present in voids greater than 5 nm in size. A portion of the capillary water is given the distinction of free water, which is the water in the large capillaries and other larger voids (> 50nm in size). It is called free water because its removal from the hydrated cement paste does not cause a dimensional change. Conversely, when water is removed from small and medium capillaries, shrinkage of the hydrated cement paste may result.

Adsorbed water is the water located close to the solid hydrated cement paste surface. This water is attracted to the solid surface by hydrogen bonding (water tension) and is mostly removed when the relative humidity of the paste is below 30 percent. It is thought that most of the drying shrinkage exhibited by concrete is associated with the removal of the adsorbed water. Figure 4 shows the physical shrinkage in the cement gel structure as water molecules are removed.

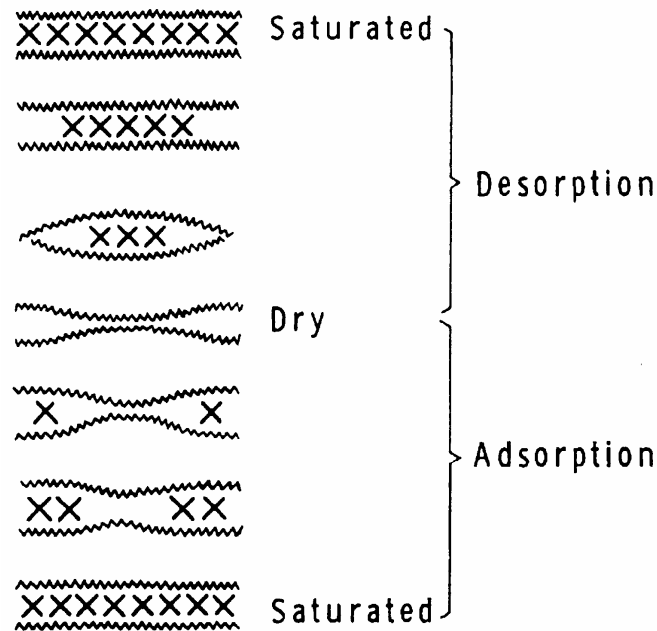


Figure 4. The State of the Cement Gel Structure as Water Molecules (x) are Removed and Added to the Paste Structure (from Fieldman and Sereda 1968)

Interlayer water is the water the Feldman-Sereda model uses to account for errors in the Powers-Brunauer model. It is thought to form hydrogen bonds between the layers of the C-S-H matrix. Interlayer water can be partially removed with drying below 11 percent relative humidity. The removal of interlayer water causes a large amount of drying shrinkage. Because the water exists in the structure as a single layer, it takes considerable time to exit between the C-S-H layers. The presence and gradual expulsion of interlayer water from the cement gel may be responsible for the phenomenon of creep; as the paste matrix undergoes a sustained load, the interlayer areas are compressed. Due to its confined nature, the water takes a considerable time to exit the structure. When the load is removed, the water rapidly infiltrates the C-S-H due to its strong affinity to the paste under capillary conditions allowing some recovery of the creep strain.

Chemically combined water strongly bonds within the hydrated paste and forms part of the structure. This water is not lost on drying and reforms with the hydrates at high temperatures (i.e., nonevaporable water).

Frost Action

Water in the cement paste affects the overall behavior and performance of a concrete on macroscopic level, and is associated with concrete's response to freezing and thawing cycles. Freeze-thaw damage is directly associated with the phase change of the water in the cement paste. There are three mechanisms that contribute to freeze-thaw damage: 1) hydraulic pressure, 2) osmotic pressure, and 3) capillary effect (Kumar Mehta 1986).

The first mechanism, hydraulic pressure, is thought to typically develop in the capillaries and other larger voids. When water freezes, it expands approximately 10 percent. If the water content of the void is above the critical saturation level (91.7 percent water), the expansion will cause the void to enlarge and/or cause some of the water to exit the void. The pressure associated with this phenomenon is called the hydraulic pressure, and it is proportional to the distance the water must travel to an 'escape boundary'. This boundary is typically provided by air entrained into the concrete. The required amount of air entrainment is determined by limiting the provided 'escape boundary' distance to a maximum value of 0.2 mm (as suggested by the American Concrete Institute 2005).

Another phenomena that is believed to occur as the larger voids start to freeze, is that the salts (alkalis, chlorides, and calcium hydroxide) naturally occurring in the hydrated

cement paste migrate into the adjacent water. The alkali concentration in the nearby water greatly increases as a result, which lowers its freezing point. Osmotic pressures develop from the differences in alkali concentrations, and adjacent water tries to flow towards the high concentration. This pressure may become high enough to rupture the cement paste near the surface, causing scaling.

The final mechanism that is believed to develop in the paste under freeze-thaw action results from the thermodynamic imbalance between the frozen water in the larger voids and the unfrozen water located in the gel pores. The interlayer and adsorbed water remains unfrozen at super cooled temperatures (perhaps as cold as -78°C in the smallest voids), due to its bonding with the C-S-H matrix, which greatly reduces its ability to rearrange into ice. As the temperature decreases, the forces drawing the unfrozen water towards the ice formation increase, causing a transmittal of water into more ice. Damage is done when the concrete's permeability is too low to allow the water transition needed to meet external demand. When the forces become large enough, the cement gel can rupture.

Air entrainment has historically shown to be the best remedy for concrete's susceptibility to freeze-thaw. The entrained air (typically assumed to be unsaturated) reduces pressures within the matrix by allowing expansion of ice within the voids and by providing an escape boundary for water towards the lower energy state. Its effectiveness has been inferred from the differences observed in the temperature deformations of normal concrete and air entrained concrete. Normal concrete, when cooled, typically undergoes an expansion due to the formation of ice within the voids. Air entrained

concrete when cooled typically contracts at a different rate, a result of the combination of the expansion of ice formations and the shrinkage of the C-S-H due to the removal of water.

High performance concretes have characteristically lower permeabilities and porosities compared to standard concretes. The strength gains in high performance concretes are specifically attributed to the reduction in pore numbers and size. High performance concretes typically have less available water for freeze-thaw than conventional concrete, due to their relatively lower initial water to cement ratios. In addition, their lower permeability reduces the ability for water to infiltrate into the concrete from the external environment. Low permeability, however, also reduces the ability of the water to move through the cement paste under the mechanisms stated above. This situation could make high performance concretes more susceptible to damage from rapid freezing and thawing (Zia et al. 2005). The two characteristics work to offset each other. Research has suggested that non-air-entrained high performance concrete has good frost resistance (Shah and Ahmad 1994) (Zia et al. 2005).

Hammer and Sellvold (1990) proposed much of the damage in concretes caused by freeze-thaw could be attributed to thermal incompatibility of its components, instead of the formation of ice, based on calorimeter data that suggested little ice formed at temperatures above -20°C .

CHAPTER 3

CTE INVESTIGATION

This investigation was divided into two tasks. The first task consisted of determining the specific CTEs of the concretes used in the three bridge decks at the Saco site. The second task consisted of analyzing data collected from the bridges in Saco on their response to environmental loads. Naturally, the results of the first task are critical to the second task. Evaluation of the concrete used in the Saco bridges requires knowledge of the laboratory measured CTEs (task one) and their unique features, as they relate to the concrete microstructure (described in the previous sections).

CTE Methodology

A technique for determining CTE's published by the MicroMeasurements Group (Raleigh, North Carolina) was used to determine accurate CTE values for the bridge deck concrete. This technique utilizes foil strain gage technology, which has the distinct advantage of being easy to implement in a laboratory equipped to perform stress analysis. A strain gage is installed on a reference material for which the CTE is well known. An identical gage is then installed on the material of interest. The temperatures of the specimens are changed in a controlled environment to induce thermal strain. Subtracting the reference output from the sample output and dividing by the change in temperature gives the difference in CTEs between the two materials:

$$\alpha_S - \alpha_R = \frac{(\epsilon_{T/O(G/S)} - \epsilon_{T/O(G/R)})}{\Delta T} \quad (\text{Eq.8})$$

where:

α_R = CTE of the reference material

α_S = CTE of the sample material

$\varepsilon_{T/O(G/R)}$ = Strain measured on the reference material (including thermal gage sensitivities)

$\varepsilon_{T/O(G/S)}$ = Strain measured on the sample material (including thermal gage sensitivities)

ΔT = Temperature change the materials undergo

In this investigation, concrete specimens from each bridge were instrumented with strain gages and placed in an environmental chamber. The specimens were cycled through several temperature regimes, while strain and temperature were measured. This data was subsequently used in the equation above to evaluate CTEs as a function of temperature and cycle history.

Specimens

Three concrete specimens from each bridge were used in this investigation. The samples measured 406.4 x 101.5 x 76.2 mm. The samples were cast during deck construction from concrete taken from different truckloads used at or close to the areas of the bridge decks that were heavily instrumented.

The specimens used for this investigation were stored at the bridge site for the first year and then move to a laboratory on the Montana State University campus for several months. The laboratory is known to have a relative humidity of approximately 15 to 25

percent year round. The average humidity of the bridge deck structures during the past two years was approximately 60 percent.

The reference material chosen for this experiment was titanium silicate, a specially formulated glass developed for high precision optics. It was specifically chosen for this purpose because of its extremely low CTE ($0.03 \mu\epsilon/^{\circ}\text{C}$) and the consistency of its CTE between different samples. A sample measuring $25.4 \times 152.4 \times 6.4$ mm was obtained from MicroMeasurments.

To further evaluate the accuracy of the instrumentation and testing methodology, a precision ground bar (± 0.0254 mm) of low carbon 1018 steel was also included in this investigation. This material was chosen because it was used to determine the thermal output curve provided by MicroMeasurments for the strain gage temperature corrections. An annealed sample measuring $50.8 \times 609.6 \times 3.2$ mm with a Rockwell hardness of B61-B62 was obtained through McMaster-Carr (Los Angeles, California). Similar to titanium silicate, the CTE of 1018 steel is relatively constant, with a reported value of $12.1 \mu\epsilon/^{\circ}\text{C}$.

Instrumentation

The strain gages (type N2A-06-40CBY-350) used for this investigation were purchased from MicroMeasurments (see Figure 5). These gages were chosen because of their accuracy and ability to lay flat during installation. They were also used because concrete's non-homogeneous nature requires long gages to "average" the response. According to the manufacturer, the thermal response of these gages was matched to approximately $12.1 \mu\epsilon/^{\circ}\text{C}$, and therefore the thermal output was expected to be relatively small over the temperature range of interest for the concrete and reference specimens.

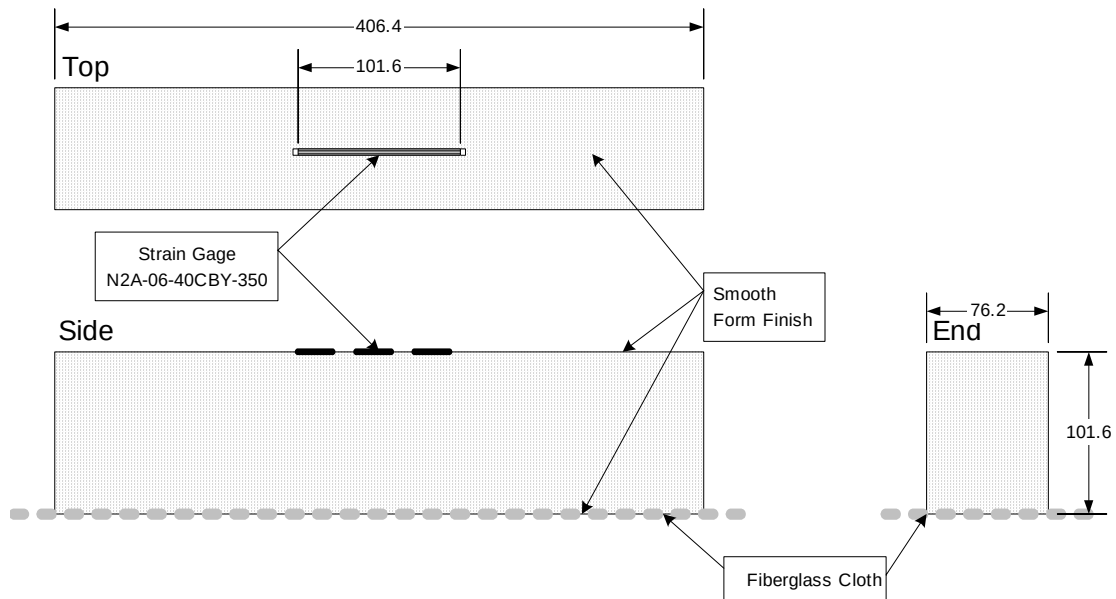


Figure 5. Specimen Dimensions and Layout

Minimal temperature corrections were necessary because the same type of strain gages were used throughout. Hence, any variation in results due to measurement error was carried uniformly across all specimens and canceled out in the calculations. Nonetheless, to increase the accuracy of this investigation, two corrections were applied to the strain data: 1) a transverse gage sensitivity correction (for the measurement error due to any transverse dimensional changes), and 2) a gage factor correction (to adjust for changes in the strain gage factor with temperature). These corrections resulted in less than a two percent change in the strain measurements.

The specimens were prepared following the recommendations of MicroMeasuments (1993). The gages were glued to the specimens using M-Bond AE-10, the adhesive recommended by MicroMeasuments. Special care was exercised to ensure the gage installations were as similar as possible across the specimens. The thickness of the glue

between the gages and specimens was kept small, because of the effect this thickness can have on measurement results. The room temperature at the time of installation was documented, and the adhesive was not cured using elevated temperatures. This procedure was followed to ensure a more repetitive installation quality and to reduce internal stresses between the gage, specimen, and glue. A typical strain gage installation is shown in Figure 6.



Figure 6. Typical Concrete Strain Gage Installation

After installation, the gages were visually and electronically checked for quality. The gage installations were qualitatively judged and ranked based on this inspection. These

rankings were then used to decide if any disparate results from a specific specimen should simply be discarded due to a suspect gage installation. The gages were thoroughly coated with silicon adhesive to protect them from moisture. Two strain gages were installed on one of the specimens from each bridge to evaluate the repeatability and accuracy of the gages.

Type T thermocouples purchased from Omega (Stamford, Connecticut) were used to measure the temperature of each specimen. The T type thermocouples were chosen because of their temperature range (-250°C to 350°C) and accuracy (the greater of 1.0°C or 0.75 percent of the measurement). The thermocouples were fastened to the specimens directly adjacent to the strain gages using silicon adhesive. An Omega thermocouple-to-analog converter (Model SMCJ-T) was used to determine the junction temperature at the multiplexer for the thermocouples used on the specimens.

Measuring the necessary strains on all specimens required 14 Wheatstone bridges. They were built using printed circuit boards with quarter bridge layouts. Three strain gage lead wires were used to minimize temperature effects. The bridges were completed using 350 ohm ultra precise resistors (0.01 percent). Three circuit boards were built, with each board having the ability to accommodate six strain gages (see Figure 7).

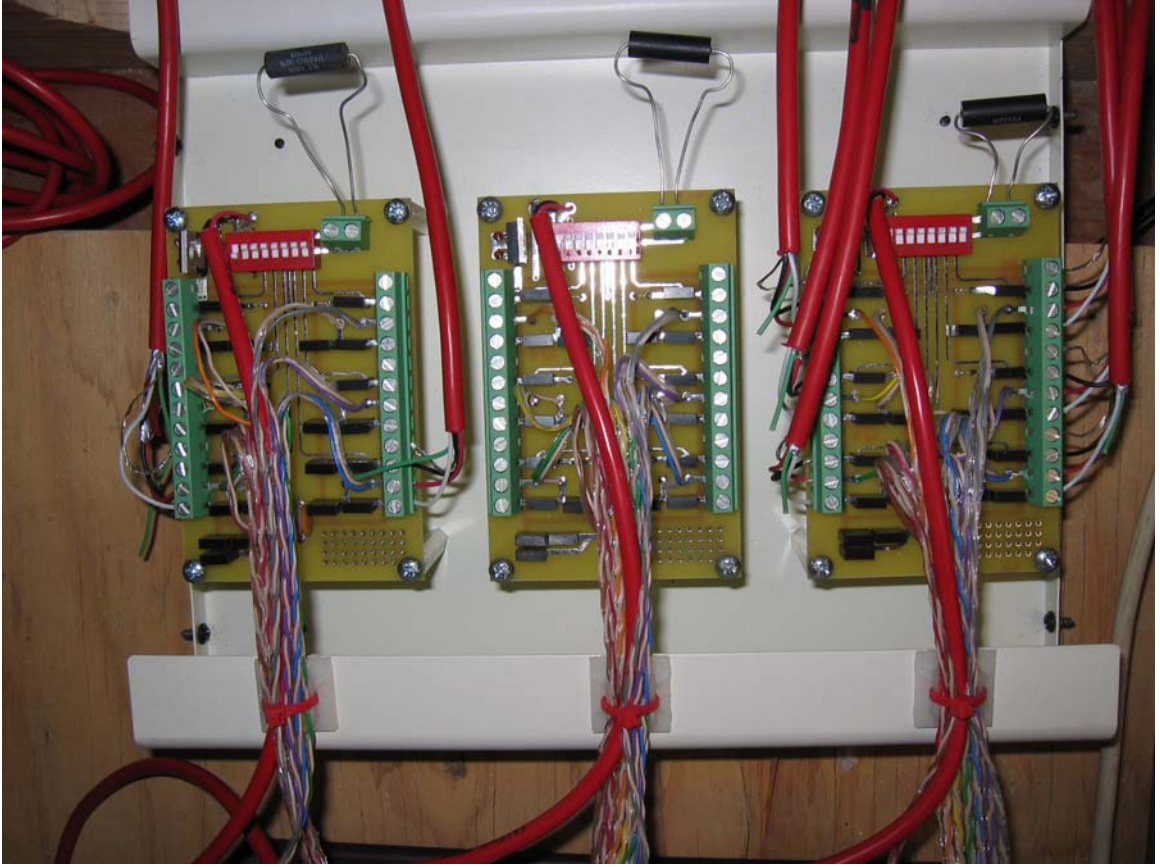


Figure 7. Circuitry used for the Strain Measurements

The thermally induced strain ($\epsilon_{T/O}$) in Equation 9 was determined using Wheatstone bridges. It can be shown for a Wheatstone bridge in the quarter bridge configuration with three lead wires that the strain output is related to the measured voltages as:

$$\epsilon_{T/O} = K_1 \left\{ \frac{2[E_{so}(-E_i E_{zo} + E_o E_{zi}) + E_{si}(E_i E_{zo} - E_o E_{zi})]}{F(2E_o + E_i)(E_{zi} E_{so} - E_{zo} E_{si})} \right\} \quad (\text{Eq.9})$$

where:

K_1 = Result of the specified nominal resistor values used within the Wheatstone bridge circuitry and for this investigation was equal to - (12/17477).

E_i = Measured input voltage from the strain gage during testing.

E_o = Measured output voltage from the strain gage during testing.

E_{si} = Measured input voltage from the strain gage during the shunt calibration process. This term is used to determine the additional resistance imparted to the system from the lead wires and connections.

E_{so} = Measured output voltage from the strain gage during the shunt calibration process. This term is used to determine the additional resistance imparted to the system from the lead wires and connections.

E_{zi} = Measured input voltage from the strain gage in the unstrained state. This term accounts for changes in gage resistance during the installation process and is meant to keep the Wheatstone bridge balanced mathematically.

E_{zo} = Measured output voltage from the strain gage in the unstrained state. This term accounts for changes in gage resistance during the installation process and is meant to keep the Wheatstone bridge balanced mathematically.

F = Strain gage factor, provided by manufacturer

Measurements were recorded using a Campbell Scientific CR10 data logger. The capacity of this logger was expanded with a Campbell Scientific MX416 multiplexer. The multiplexer was used to measure the temperature and strain for each specimen. The multiplexer was located outside the environmental chamber because of its sensitivity to temperature and humidity.

Procedure

To conduct the cyclic temperature testing, the test specimens were placed on a table in a 2 x 2 x 2 m environmental chamber. Internal fans were used to ensure a uniform temperature throughout the area of the chamber containing the specimens and to increase the heat transfer between the air and specimens. Figure 8 shows the specimens within the environmental chamber, and Figure 9 shows the specimen layout and labeling nomenclature. The specimens were placed on fiberglass cloth to reduce surface friction between them and the table as they expanded and contracted during testing.



Figure 8. Specimens placed in environmental chamber.

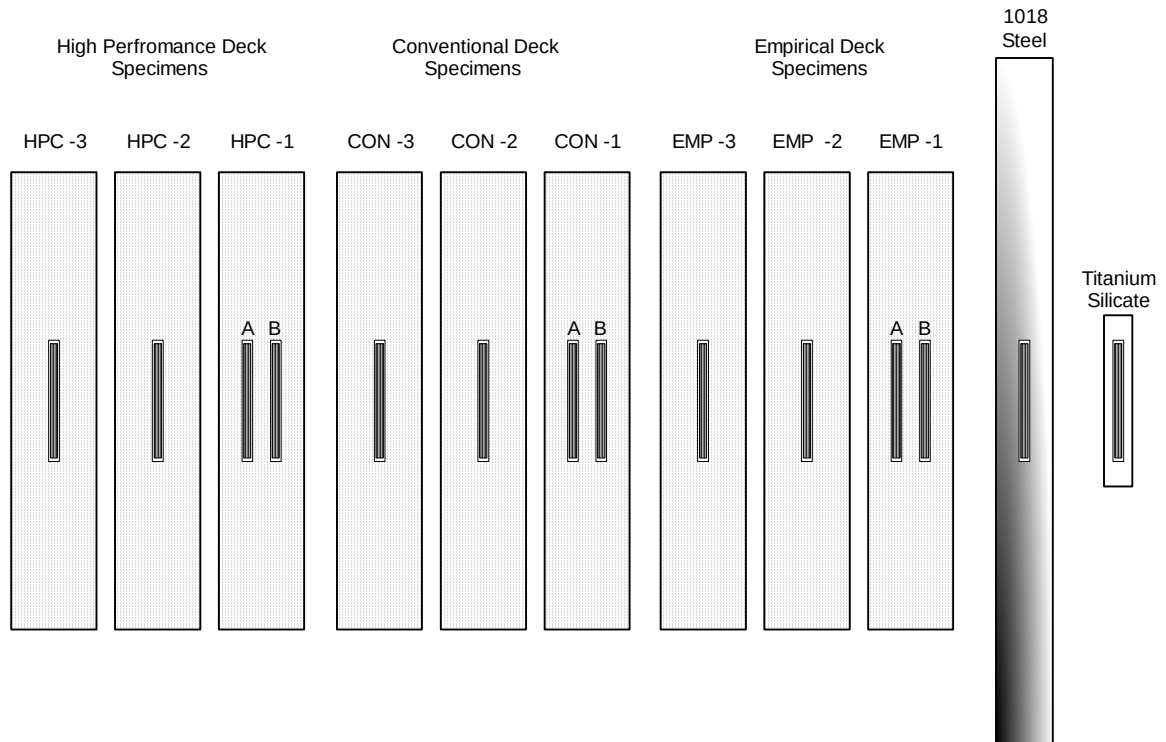


Figure 9. Specimen Layout within the Environmental Chamber and Labeling Nomenclature

The temperature range for testing was -40°C to 45°C with the primary focus of the investigation on the -40°C to 40°C range previously observed in the Saco bridge decks. The wider temperature range for the shakedown cycles was recommended by MicroMeasurements to stabilize the specimens for better results.

During testing, the temperature in the environmental chamber was changed at a maximum rate of 5°C over fifteen minutes. This rate was determined to be acceptable based on a finite difference analysis of the heat transfer between the concrete specimens and the surrounding air. This analysis showed that the thermal strain differential between the outer surface and the interior of the specimen was less than $5\text{ }\mu\epsilon$. At the beginning of each step, the temperature was changed 5°C over a period of at least 15 minutes (the

ramp duration). It subsequently was held constant for at least 180 min. (the soak duration), to allow the specimens to fully acclimate to the surrounding air temperature. Six cycles from 20°C to the coldest temperature (-45°C or -40°C) to the hottest temperature (50°C or 45°C) and back to 20°C (with 34 steps/cycle) were run extending over one month (see Table 6). The first two cycles were used to stabilize the testing system and evaluate the methodology used, while the last four were used to investigate the repeatability of the experiment and obtain CTE values.

Table 6. CTE Testing Regime

Cycle	Ramp Duration	Soak Duration	Temperature Range	Total Cycle Time
	(min)	(min)	(°C)	(hr)
P1-P2†	15	180	20, -45, 50, 20	123.5
1	15	180	20, -40, 45, 20	114.5‡
2	15	240	20, -40, 45, 20	146.5‡
3	15	300	20, -40, 45, 20	178.5
4	60	300	20, -40, 45, 20	204.0

† Shakedown cycles

‡ Includes an extended 300 minute soak at -40°C and 45°C

Average values of strain and temperature were recorded every two minutes throughout the testing process. Measurements were made by sampling each gage in succession ten times within a twenty-second time interval. The data logger then calculated and stored the average value and the standard deviation of the measurements. The data was immediately inspected for any hysteretic behavior in the recorded strain versus temperature response. Non-hysteretic behavior generally represents repeatable,

accurate data, thus validating the methodology employed in this investigation. Figure 10 shows typical data taken over the last four thermal cycles, specifically cycles one through four.

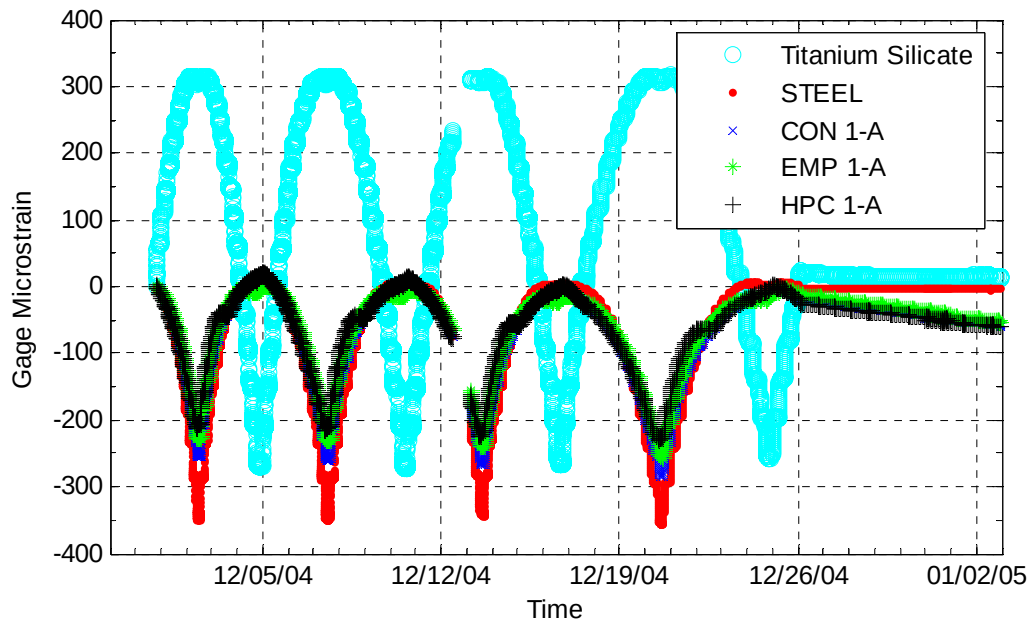


Figure 10. Measured Gage Strain with Time (Cycles 1 through 4)

CTE Findings

True Strain

Calculation of CTE values requires a period of thermal steady state for each measurement. Thermal steady state conditions were assumed to occur at the end of each temperature step. This assumption was believed to be reasonable based on the strain and temperature measurements taken over a 32 minute interval at the end of each step, (see Figures 11 and 12).

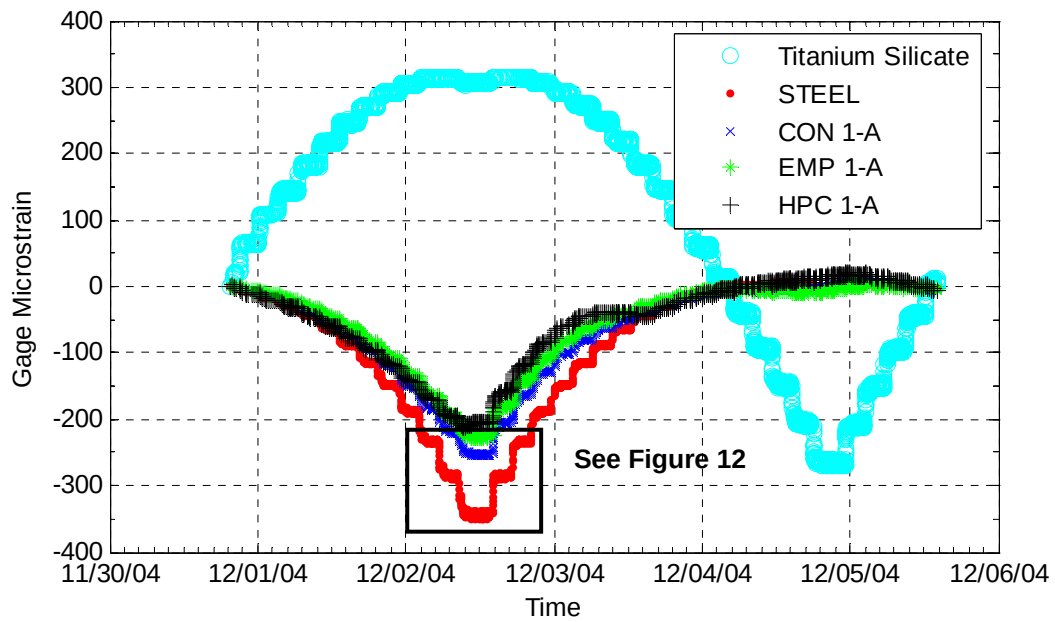


Figure 11. Measured Gage Strain with Time (Cycle 1)

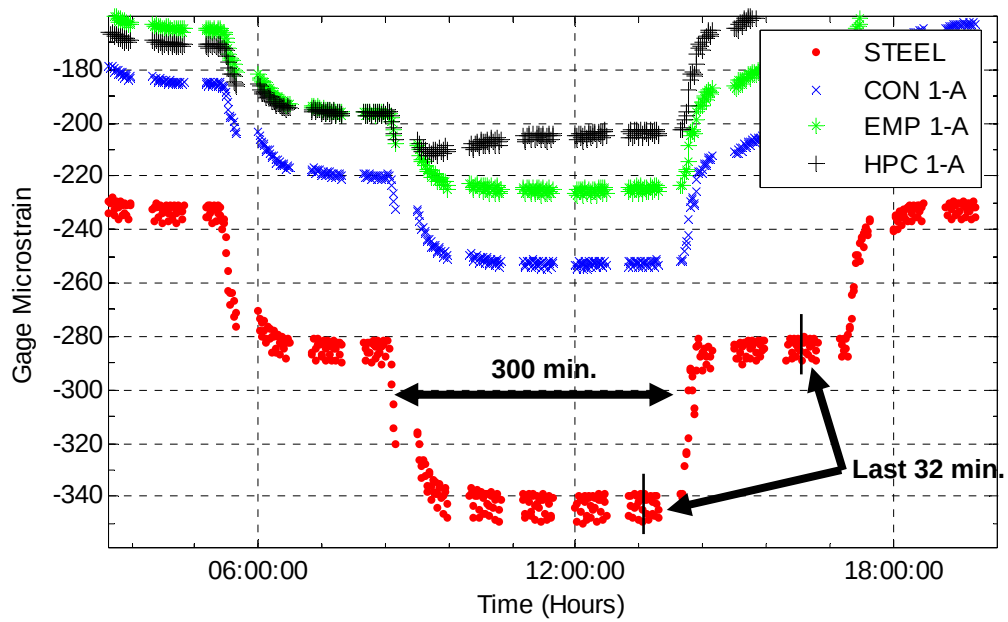


Figure 12. Measured Gage Strain with Time (Close Up)

The gage strains as measured at the ends of each temperature step during a typical temperature cycle from -40°C to 45°C (and back) are plotted as a function of temperature in Figure 13. There is a slight amount of horizontal spreading in the values reported at each temperature step due to temperature measurement and control errors.

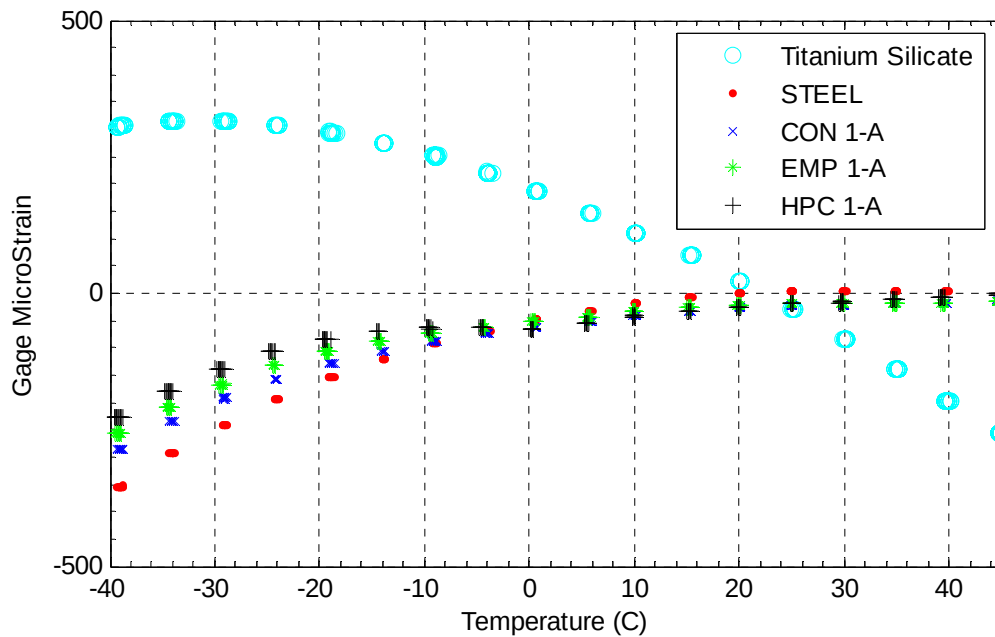


Figure 13. Gage Strain (Thermal Gage Effects Included)

A linear, least-square-mean fit was applied to the strains recorded for the titanium silicate at the end of each temperature step. The equation for this fit was then evaluated for each concrete specimen temperature measurement within the same temperature step. This process gave an equivalent titanium silicate gage strain for each specimen temperature measurement. Recall that the coefficient of thermal expansion of the titanium silicate was very close to zero. Therefore, the apparent strain measured on the titanium silicate represents thermal gage effects, not true strain of the material. The

titanium silicate strain measurements were subtracted from the measured specimen strains to obtain the true strain experienced by the specimen (recall Equation 8). The true specimen strains obtained in this fashion are presented in Figure 14.

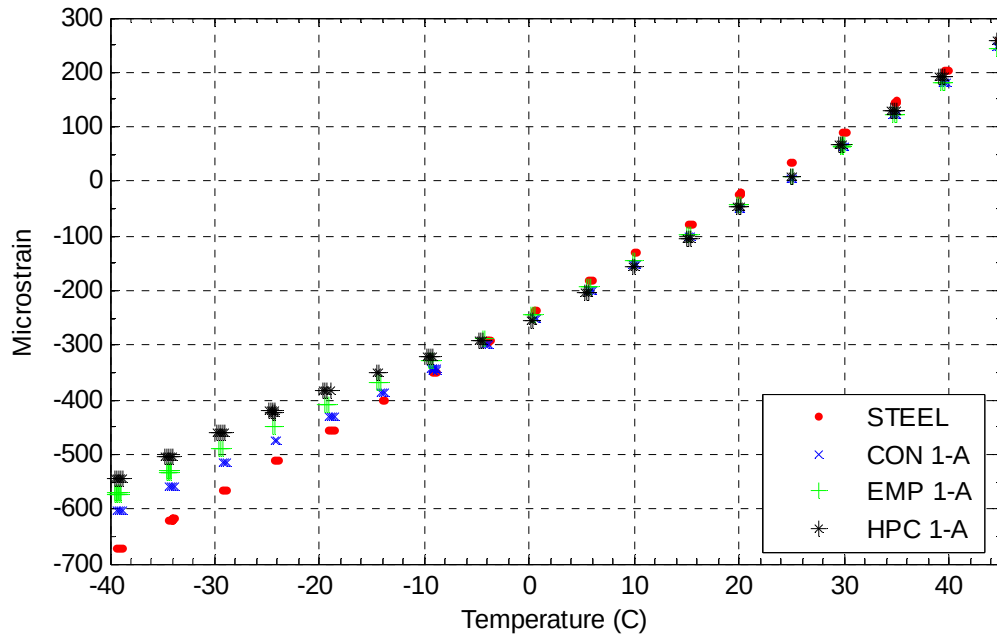


Figure 14. True Concrete Strain during Heating Cycle 4
(Thermal Gage Effects Subtracted Out)

CTE

The CTE is the change in strain as a function of change in temperature, and it can be calculated as the first derivative of strain with respect to temperature as shown in:

$$CTE = \frac{d\varepsilon}{dT} \quad (\text{Eq.10})$$

where:

ε = True strain

T = Temperature

A piecewise linear, least-square-mean fit was first performed through a windowed subset of the strain data with respect to temperature for each heating and cooling cycle. The derivative of the fit was then numerically evaluated at the central temperature of the moving window. Good results were obtained when the window extended over three temperature steps (48 data points) because of the temperature measurement errors within each step.

CTE Results for 1018 Steel

The 1018 steel showed a nearly constant CTE at a value of approximately $11.1 \mu\epsilon/^\circ\text{C}$, as shown in Figures 15 and 16. A typically reported value of the CTE for 1018 steel is $12.1 \mu\epsilon/^\circ\text{C}$. The difference between the measured and reported CTE values could be associated with the variability in the CTEs among different ‘heats’ of 1018 steel. The nonlinear change in CTE values across the temperature range is probably due to very slight differences in the thermal characteristics of the steel and titanium silicate strain gages. Note that the 1018 steel specimen was relatively thin (with a thickness of 3.2 mm), and it may have experienced some bending distortions as it “dragged” across the surface on which it rested. This would account for the hysteretic CTE values for different cycles. Note that the dispersion of the results for the 1018 steel CTE values (Figure 15) suggests an error band of only $\pm 0.2 \mu\epsilon/^\circ\text{C}$.

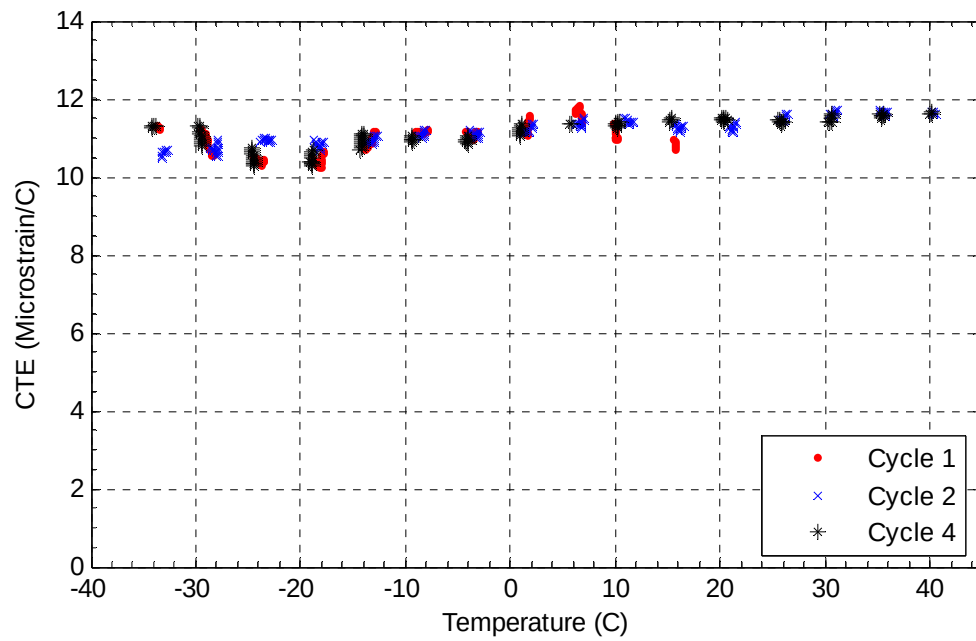


Figure 15. 1018 Steel Cooling All Cycles

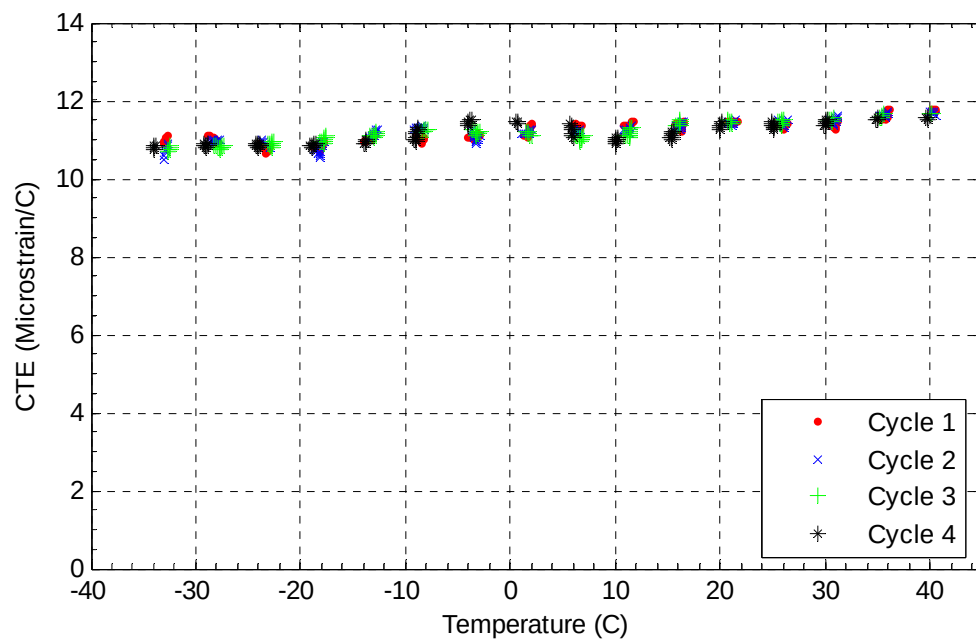


Figure 16. 1018 Steel Heating All Cycles

CTE Results for Concrete

The concrete CTE values, when plotted as a function of temperature, exhibited a nonlinear behavior. The conventional and empirical deck concretes showed similar behaviors for both the cooling and heating cycles, as might be expected, as these decks were made with concretes of the same design. The high performance concrete exhibited a distinctly different thermal behavior from the standard concrete used in the conventional and empirical decks. In general, this difference may be attributed to the properties of the cement paste, as all the concretes were made with the same aggregate.

The cooling CTEs of the standard concrete specimens have a positive near linear slope, with all the strain gages reporting similar values (see Figure 17). Generally, the decrease in the CTE as the specimens cooled can be attributed to the behavior of the water in the paste. Four distinct regions are evident in the CTE response shown in Figure 17: 1) 40°C to 30°C, 2) 30°C to -5°C, 3) -5°C to -15°C and 4) -15°C to -35°C.

Region 1 shows a near constant CTE of between 11.9 and 12.2 $\mu\epsilon/^\circ\text{C}$. As the specimens were cooled, the CTE values decreased nearly linearly in Region 2 to values between 9.4 and 10.0 $\mu\epsilon/^\circ\text{C}$. This phenomenon could be explained as follows: The specimens shrank as the temperature was lowered, until a critical temperature was reached relative to the pore structure and moisture content. At this temperature, the water in the cement paste began to be compressed and resisted further shrinkage, which reduced the CTE values of the concrete.

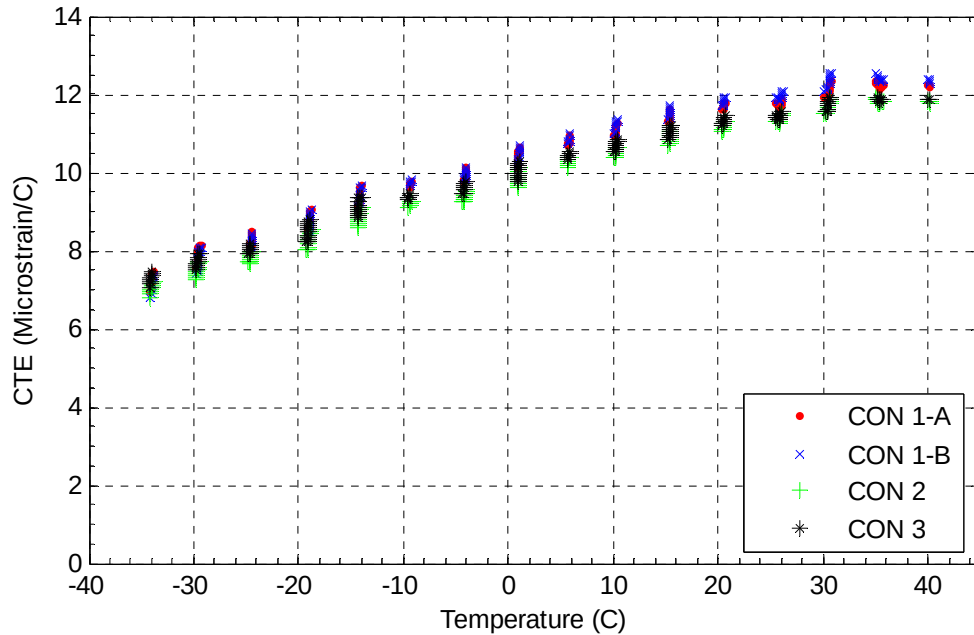


Figure 17. Standard Concrete CTE Cooling - All Gages - Last Cycle

As the specimen continued to shrink, an increasing fraction of the water in the cement paste reached a confined state. Despite the increase in pressure on the water, it was unlikely to be transported away from the high-pressure regions (small voids) due to the affinity between the water and the cement paste structure. This explanation of the observed behavior is supported by Figure 18, which shows that even at long soak times, the CTE does not increase, as would be expected if the water began to migrate to lower pressure zones under the influence of the thermal compression forces. Referring to Figure 18, cycle 4 had almost twice the soak time as cycle 1, but the CTE values are virtually identical.

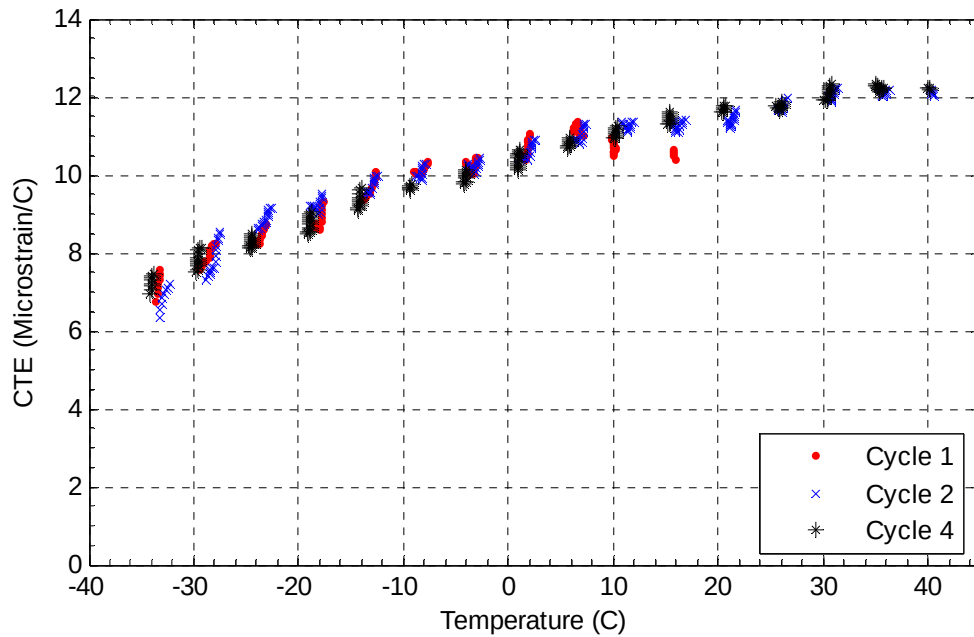


Figure 18. Standard Concrete CTE Cooling – Gage CON 1-A - All Cycles

At the end of Region 2 (-5°C), the water in the medium size pores began to freeze. The freeze point is thought to be suppressed to -5°C for two primary reasons: 1) the pore water has an alkalinity that lowers its freezing point, and 2) the pore water is under some external pressure from the cement paste structure, which also lowers the freezing point. The frozen pore water created a thermodynamic imbalance between the frozen and unfrozen pore water. This energy imbalance helped break the small pore water bonds with the cement paste and caused the migration of unfrozen water in the small pores towards the frozen water. The data suggests that these two actions nearly cancel each other out in Region 3. As small pore water was removed from the cement paste nanostructure, the paste shrank, but at the same time, the freezing water expanded (resulting in a constant CTE across this temperature range). After the bulk of the water

froze, the further contraction of the specimen was increasingly resisted by the increasingly large ice crystals (Region 4), due to ice formation expansion (molecular structure) and additional water freezing. Region 4 ended with CTE values ranging between 7.4 and 8.6 $\mu\epsilon/^\circ\text{C}$.

The heating CTE as a function of temperature of the standard concrete specimens has a generally positive slope, similar to the cooling CTE behavior, with all the gages reporting similar values (see Figure 19). However, there are several subtle differences between the heating and cooling CTE behaviors. Generally, the increase in the CTE as the specimens were heated can be attributed to the expansion of the cement paste as it absorbed water. Figure 19 shows four distinct regions: 1) -35°C to -15°C , 2) -15°C to 0°C , 3) 0°C to 10°C and 4) 10°C to 40°C , that are similar to, but do not exactly correspond to the regions delineated during the cooling CTE evaluation.

Region 1 shows a constant CTE value of approximately 8.3 $\mu\epsilon/^\circ\text{C}$. As the specimens heated, they expanded. While it would be expected that this expansion would be resisted by the ice in the larger pores of the paste (depressing the CTE values), the frozen water was simultaneously contracting, and the combination of these two events caused no internal pressures on the cement matrix, allowing it to expand freely. Region 1 does exhibit a slight decrease in CTE values, which may be associated with the small pore water contracting as the temperature increased. At the beginning of Region 2 (-15°C), the frozen water located in the smaller pores began to thaw. This water was absorbed into the C-S-H gel, resulting in its expansion and an associated increase in the CTE

values. The CTE values level off after Region 2 because nearly all of the ice has transformed into water.

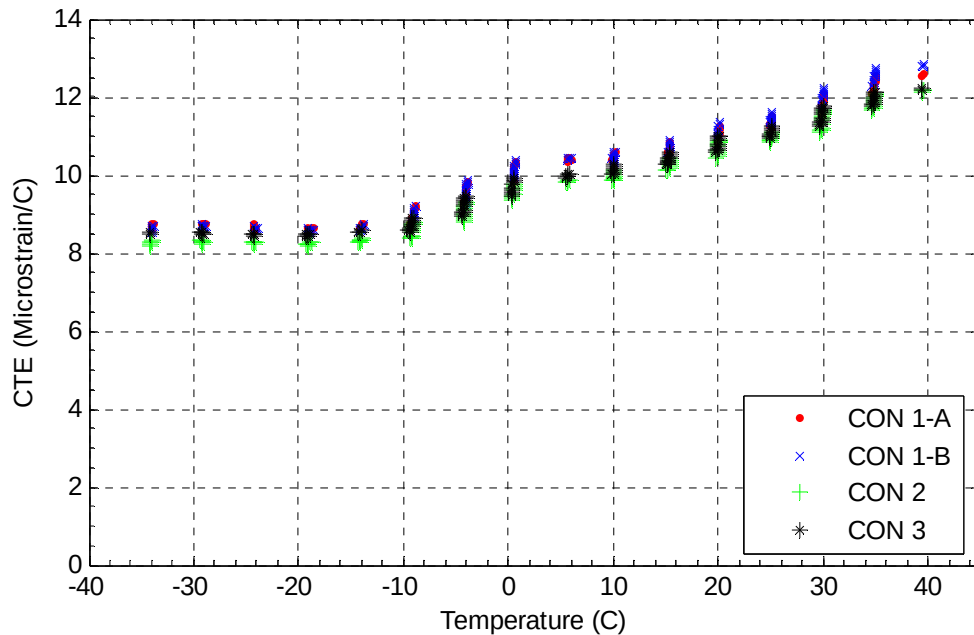


Figure 19. Traditional Concrete CTE Heating - All Gages - Last Cycle

The change in CTE during the soak time at each temperature seen in Figure 20 is indicative of the expansion of the paste as water was absorbed by the C-S-H gel. At about 10°C, the specimens exhibited increasing CTE values as they were further heated. This expansion could be associated with two phenomena: 1) as the specimen expanded, water was able to enter smaller openings in the pore structure, hydrating more of the cement paste and/or 2) the water within the pore structure changed its structure as it was heated, becoming less dense.

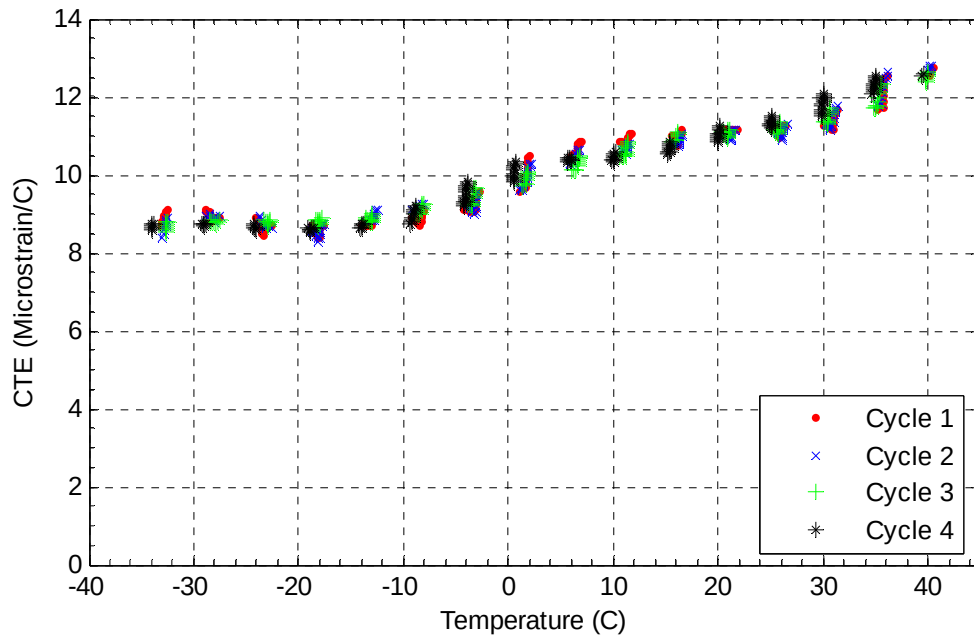


Figure 20. Traditional Concrete CTE Heating – Gage CON 1-A - All Cycles

The relationship between the cooling CTEs of the HPC specimens and temperature has a positive near linear slope (see Figure 21), again with four distinct regions of behavior: 1) 40°C to 20°C, 2) 20°C to 10°C, 3) 10°C to -10°C and 4) -10°C to -35°C.

Region 1 shows a near constant CTE of between 11.6 and 12.6 $\mu\epsilon/^\circ\text{C}$. As the specimens cooled, the CTE values decreased nearly linearly to values between 9.0 and 10.0 $\mu\epsilon/^\circ\text{C}$. This behavior is similar to that of the standard concrete specimens, except it began at a lower temperature and ended at a higher temperature than was observed for the conventional concrete. This difference could be attributed to different moisture contents between the conventional and high performance concretes.

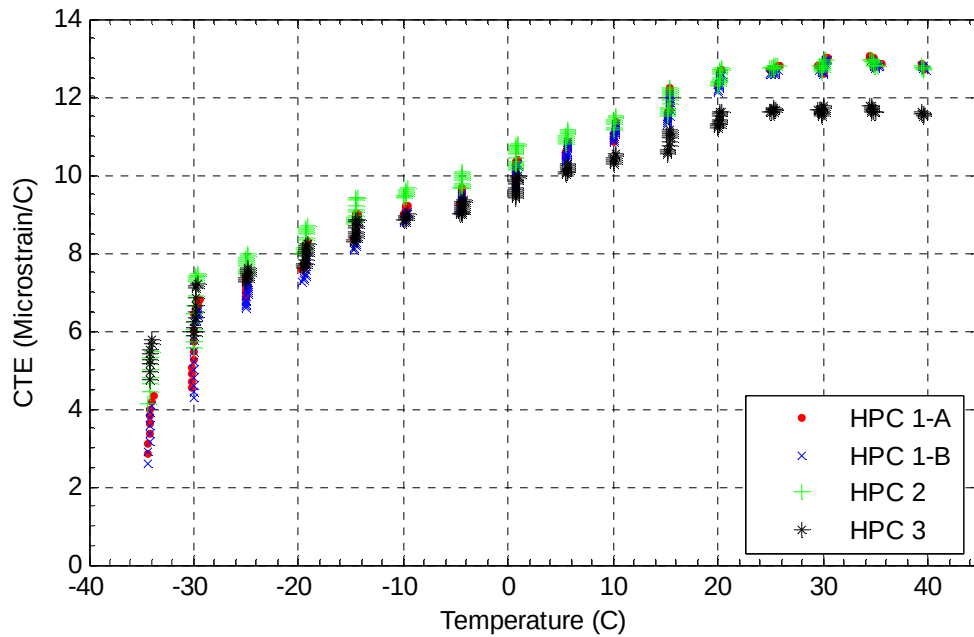


Figure 21. High Performance Concrete CTE Cooling - All Gages - Last Cycle

Again, the behaviors in cycle 1 and cycle 4 are similar, despite differences in their soak times (see Figure 22), which would suggest no migration of water occurred. The HPC concrete does not show a change in CTE values as it passes through 0°C within Region 3, as was seen for the standard concrete. This situation could be attributed to the lower porosity of the HPC, which does not allow the water in the paste structure to migrate and expand, therefore offering no phase change and rapid expansion. At the end of Region 3 (the beginning of Region 4), the water in the medium pores began to freeze, similar to what is believed to have happened to the standard concrete specimens. The lower “freezing” point compared to the standard concrete is most likely due to the smaller pore structure of the HPC, with less connections between the entrained air voids. Large pressures developed in these pores as the water tried to expand through the lower porosity

pores, which suppressed the freezing point of the water. The HPC specimens showed a similar decrease in CTE values in Region 4 to that seen in the standard concrete, but with a steeper slope. This difference in behavior again is probably related to the lower porosity of HPC. As the supercooled water expanded, it resisted the contraction of the cement paste structure. Due to the closed nature of the cement paste structure, the pressure was not relieved by migration of the water to larger voids with a lower energy state. Region 4 ends with CTE values between 2.5 and 6.0 $\mu\epsilon/^\circ\text{C}$.

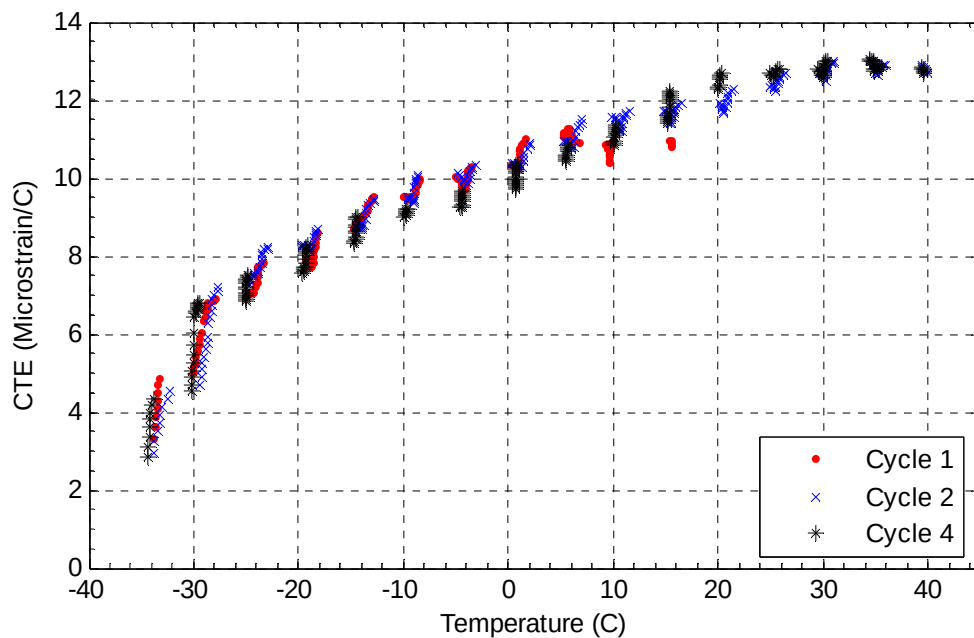


Figure 22. High Performance Concrete CTE Cooling – Gage HPC 1-A - All Cycles

The heating CTE as a function of temperature for the HPC specimens has a positive nonlinear slope. As seen previously, the relationship between CTE and temperature exhibits four distinct regions during heating: 1) -35°C to -30°C , 2) -30°C to -10°C , 3) -10°C to 0°C and 4) 0°C to 40°C (see Figure 23).

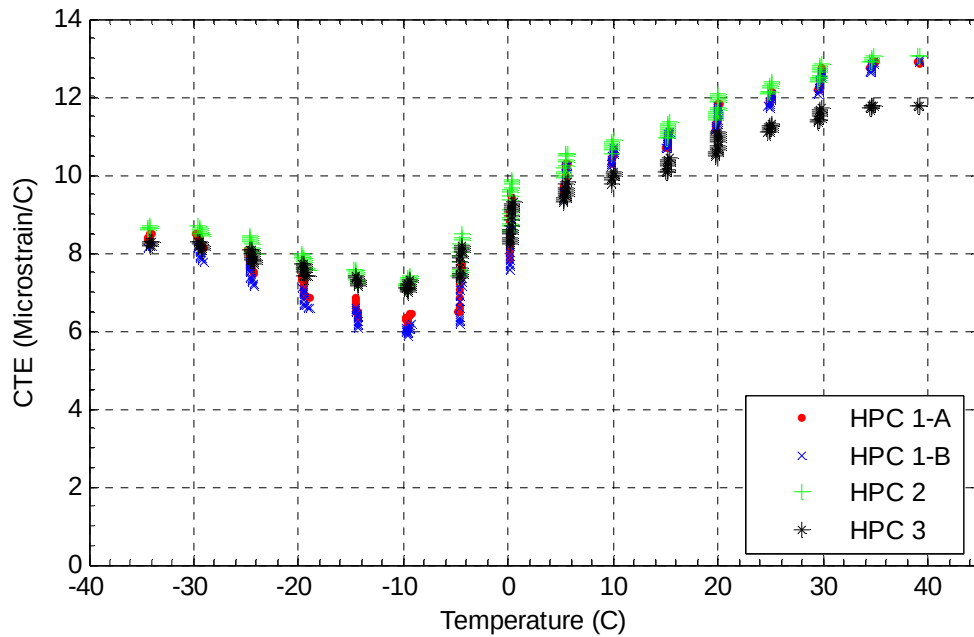


Figure 23. High Performance Concrete CTE Heating - All Gages - Last Cycle

Once again, these regions do not precisely coincide with the regions identified during the cooling cycle. Region 1 shows a constant CTE value of approximately $8.5 \mu\epsilon/^\circ\text{C}$, which is similar to the standard concrete CTE value. As the specimens heated, they expanded. Simultaneously with this expansion, the water frozen in the large pores also contracted, and the combination of these two events caused no internal pressures on the cement matrix, allowing it to expand freely. The slight decrease in CTE with temperature increase seen in the heating cycle for standard concrete in Region 1 is much more dramatic for the HPC specimens. This behavior is probably associated with the large amount of water “trapped” in the smaller pores of the HPC cement paste. When the specimens warmed, the water contracted, relieving the pore pressure and suppressing the CTE values. Because this behavior begins at a low temperature, it suggests that water is

trapped in the smaller pores and is not able to migrate to the entrained air. This behavior is consistent with some research that has been done that suggests air entrainment has little benefit for improving the freeze-thaw resistance of high performance concrete (Zia et al. 2005). During Region 3, most of the small pore water had thawed and began to be absorbed by the cement paste, causing expansion. Figure 24 shows that paste hydration happens within cycle 1's temperature soak time because the CTE values do not change appreciably for cycle 4's longer soak time. Region 4 shows continually increasing CTE values as the specimens were further heated, for similar reasons as discussed for the standard concrete.

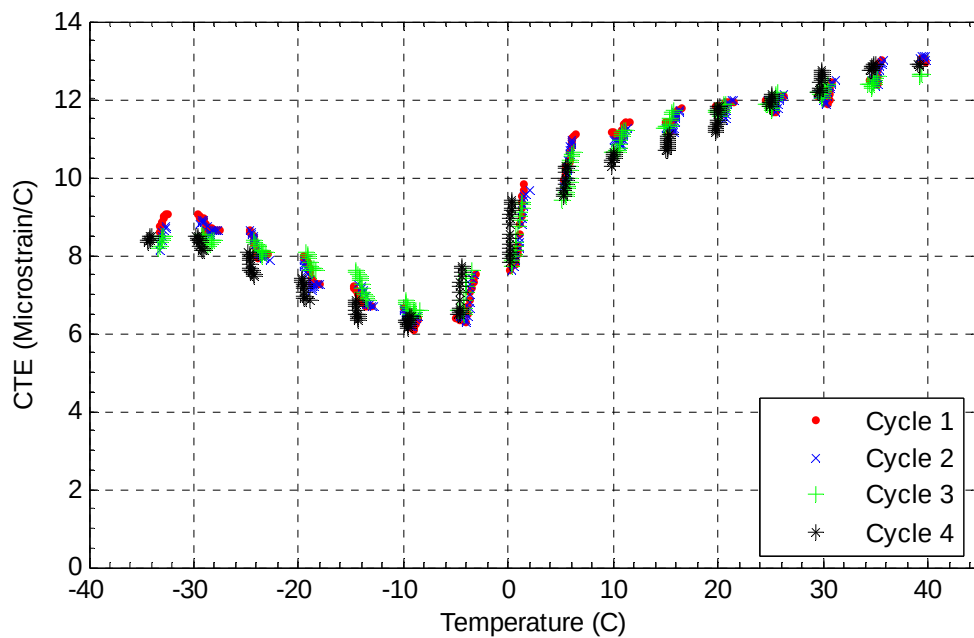


Figure 24. High Performance Concrete CTE Heating – Gage HPC 1-A - All Cycles

CTE Conclusions

The CTE investigation showed that concrete has a varying CTE value with temperature, which was attributed primarily to the behavior and interaction of the water in the paste with the pore structure. The amount of variation of the CTE with temperature (ranging by as much as $6 \mu\epsilon/^\circ\text{C}$ at -10°C to $13 \mu\epsilon/^\circ\text{C}$ at 40°C) merits reconsideration of the assumption of the constant CTE used in all analysis methods. In addition, the investigation showed that the cement paste structure has a measurable effect on the CTE values, as shown by the differences in CTE values and behavior between the standard and high performance concretes. Figures 25 and 26, respectively, show the cooling and heating CTE values for all the specimens. The similarities in the conventional and empirical deck concrete specimens are evident, as is expected since they are the same concrete mix.

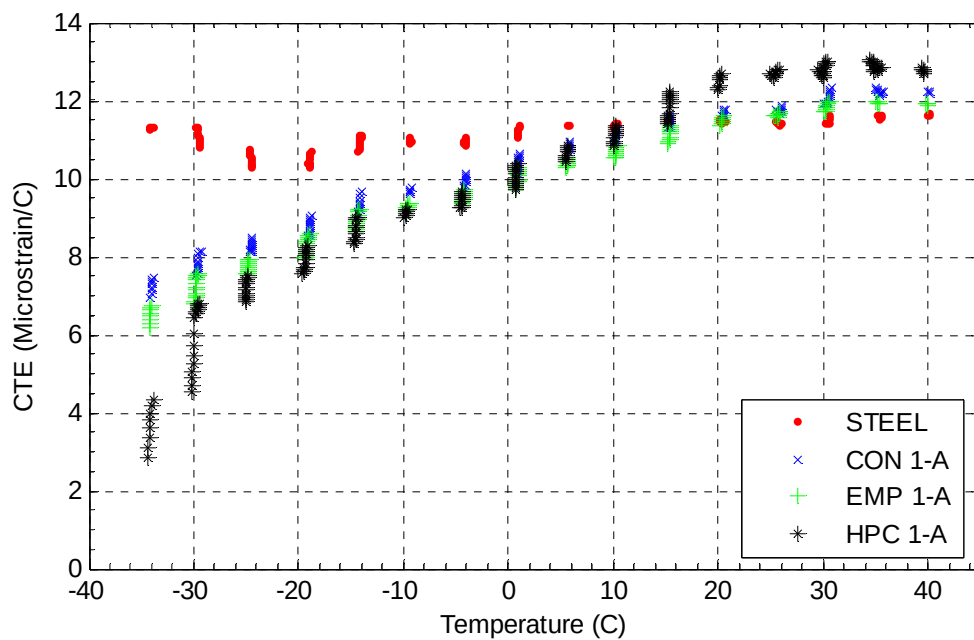


Figure 25. Coefficients of Thermal Expansion for the Last Cooling Cycle

The distinctive nature of the HPC specimens, notably in the heating cycle (-20°C to 0°C) are also evident. It should be noted that these values are probably influenced to some extent by the specific testing regime that was used in these analyses. If different temperature ranges, soak lengths, and cycling were used, the results would differ somewhat from these.

In general, the results showed that as the temperature decreased, so did the concrete CTE values. This behavior has a significant advantage for concrete bridge decks, in that as the temperatures decrease: 1) the concrete does not shrink as much as would be anticipated by traditional concrete CTE values, and 2) as the temperature decreases from about 15°C and below, the steel reinforcement begins to shrink at a greater rate than that of the concrete. This situation is of specific interest for the case when a bridge is fully restrained and the temperature decreases. Traditional analyses would suggest that as the temperature decreases, forces would develop that create tension in the concrete, facilitating cracking at a time when corrosive products (i.e., deicing salts) are most likely to be present (winter). But because of concrete's reducing CTE value as temperatures decrease, the steel reinforcement tends to force the concrete into compression, hence reducing its susceptibility to cracking. Correspondingly, concrete's higher CTE value at higher temperatures has the same effect. Hence, matching the setting temperature to where the nonlinear concrete CTE value crosses that of steels will cause the steel to compress the concrete over the entire temperature range. Note that the lower high performance concrete CTE values indicate that greater compressive forces would develop in the high performance concrete compared to standard concrete. This would have an

advantage of increasing serviceability due to the larger ‘post-stressed’ forces in the concrete.

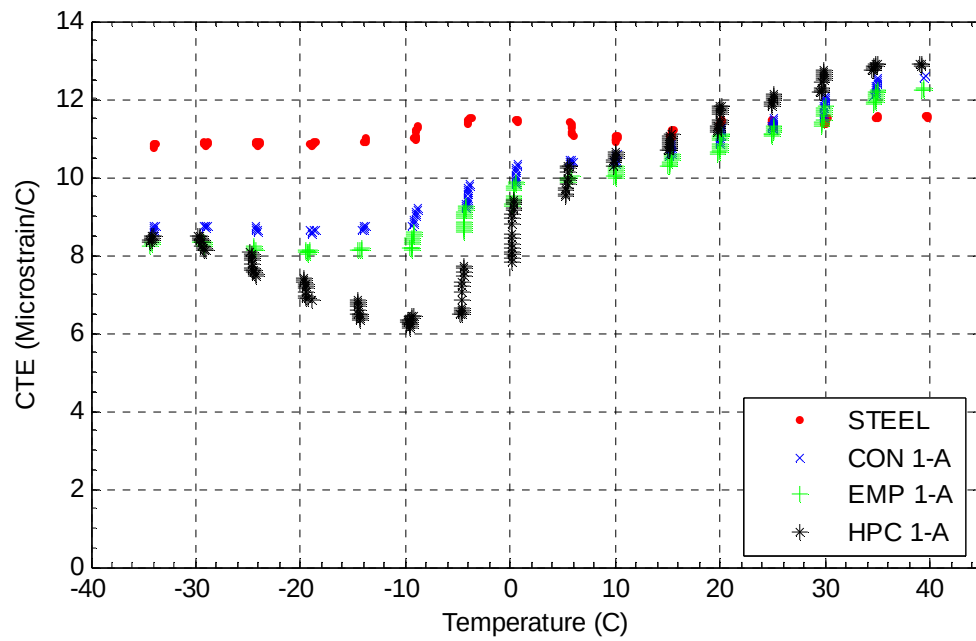


Figure 26. Coefficients of Thermal Expansion for the Last Heating Cycle

CHAPTER 4

BRIDGE INVESTIGATION

The primary objective of this part of the project was to investigate the behavior of the three different bridge decks under environmental (notably thermal) loads. The CTE information obtained in the laboratory investigation on the bridge concretes (Chapter 3) was used in the subsequent analysis of the strain data collected from the bridge decks as they experienced daily temperature fluctuations over the past two years (May 2003 to May 2005). Presented below is a brief description of the bridges and their construction, followed by an analysis of their behavior under environmental loads.

Description of the Bridges

Three new bridges with identical geometries were constructed in the summer of 2003 on Route 243 approximately one mile north of Saco, Montana. Route 243 is a local route that carries an average annual daily traffic of 220 vehicles (1998), with 11.5 percent being trucks (MDT Bridge Plans and Quantities, 2003). The environmental conditions can be quite harsh in this region of the United States, with winter temperatures of -40°C and summer temperatures exceeding 45°C .

A typical bridge and deck are shown in Figure 27. All three bridges are 44.5 meters long, divided into three approximately equal spans, as shown in Figure 28. The bridges, 9.1 meters wide, are supported by four lines of girders spaced at 2.4 meters on center, as shown in Figure 29 and Figure 30. The girders that support the deck are standard, Type-I prestressed concrete I-beams. The thickness of each deck is approximately 210 mm.



Figure 27. Completed Bridge Deck

The decks on the three bridges were constructed with different concretes and different reinforcing steel configurations. Two decks, the conventional (CON) deck and the empirical (EMP) deck, were constructed with standard concrete with a specified 28-day compressive strength of 21 MPa; one deck, the high performance deck (HPC), was constructed with high performance concrete with a specified strength of 28 MPa.

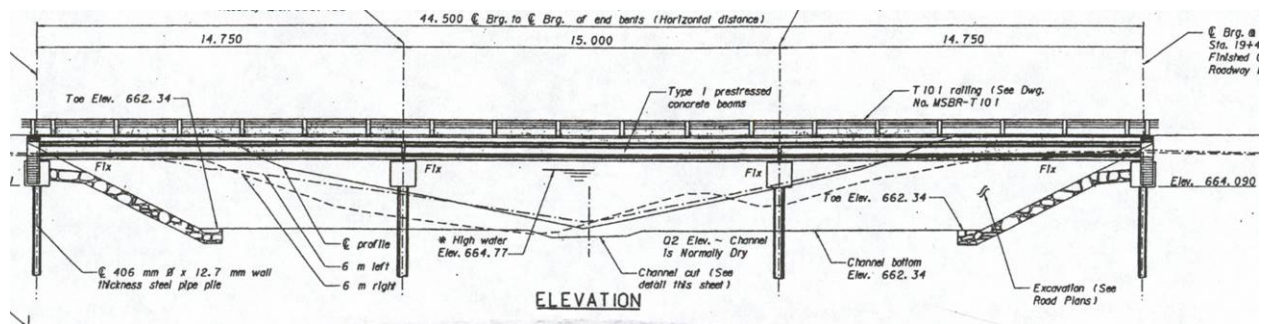


Figure 28. Elevation View of Typical Saco Bridge

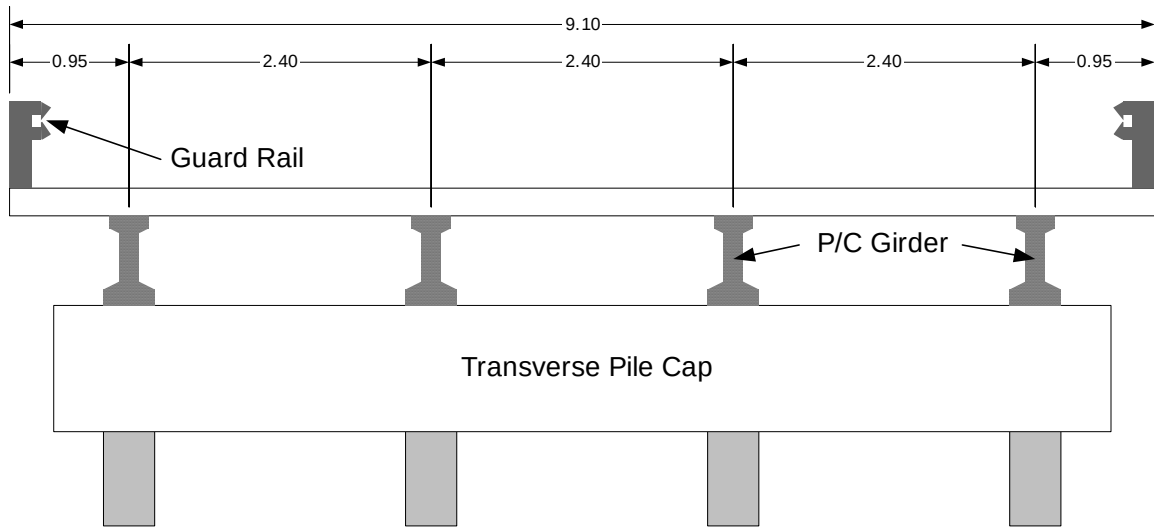


Figure 29. Cross Sectional View

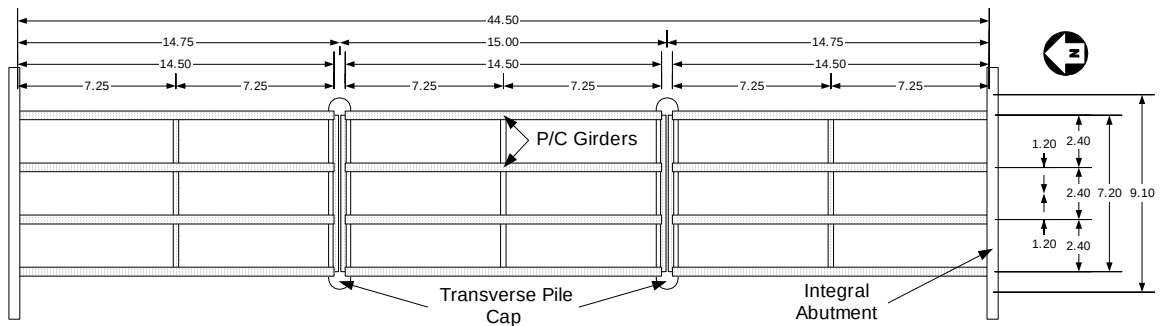


Figure 30. Plan View of Bridge Decks

The layout of the reinforcing steel varied between the three decks. These layouts are summarized in Figure 31 and pictured in Figure 32. The Conventional and HPC decks were designed using the traditional strength approach described in the AASHTO bridge design specification (AASHTO 2000). The traditional strength design method treats the deck as if it were a beam in flexure spanning the girders. This design results in the primary reinforcement oriented transversely to the length of the bridge.

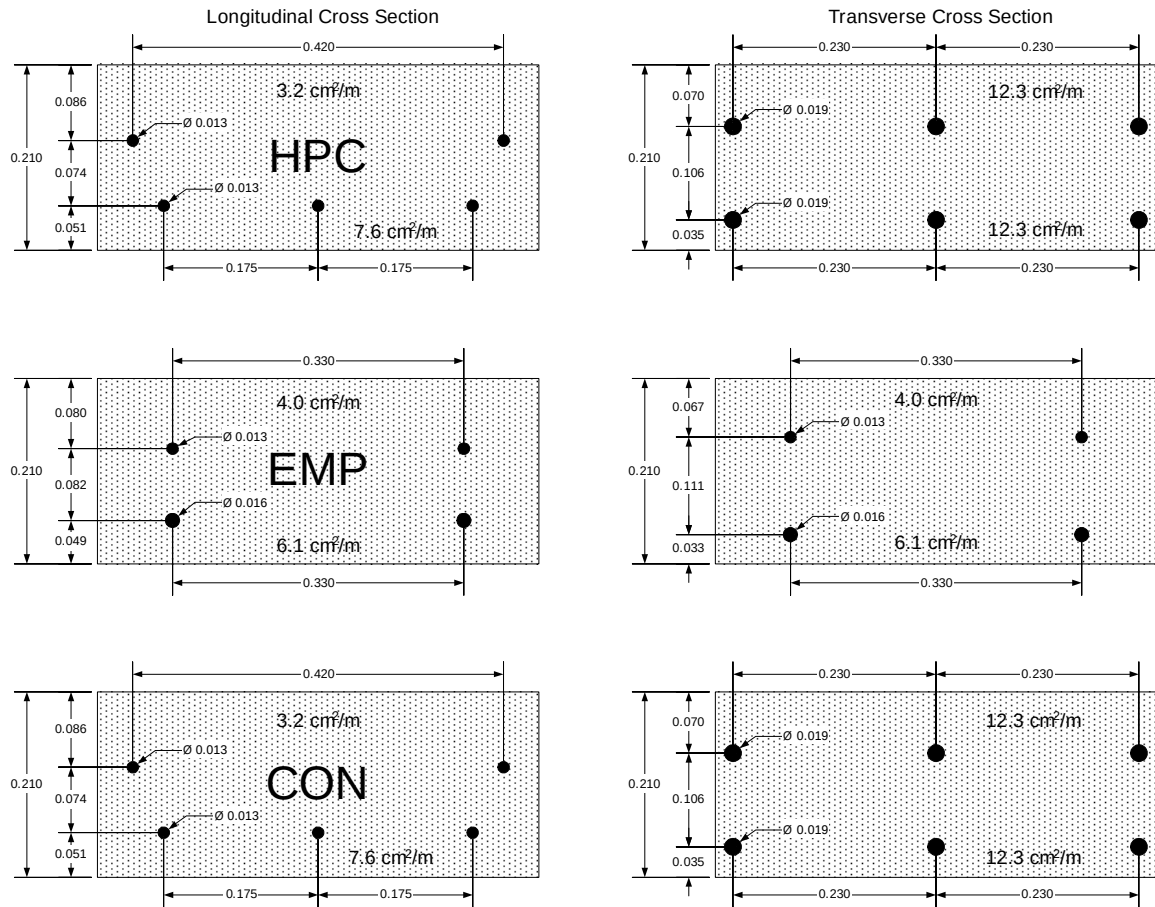


Figure 31. Typical Bridge Deck Cross-Sections and Steel Amounts



a) Conventional and HPC Decks



b) Empirical Deck

Figure 32. Steel Reinforcement Layout in the Saco Bridge Decks

The empirical design approach, which requires no formal analysis, is permitted by the AASHTO specifications for monolithic concrete bridge decks that satisfy specific conditions. Using this design method, reinforcement ratios for the top and bottom mat in both the longitudinal and transverse directions are simply functions of the depth of concrete and length of the span. AASHTO specifies a minimum reinforcement ratio equal to $3.8 \text{ cm}^2/\text{m}$ in each direction in the top mat, and $5.7 \text{ cm}^2/\text{m}$ in each direction in the bottom mat. The reason for the increased amount of steel in the bottom mat is for better crack control in the positive bending regions of the slab. Comparatively, the conventional and high performance decks used $14.5 \text{ cm}^2/\text{m}$ more steel than the empirical deck in the transverse direction with nearly the same amount of steel in the longitudinal direction (a difference of only $0.7 \text{ cm}^2/\text{m}$). Reducing the amount of steel in the empirical deck, especially in the top layer, decreases the opportunity for, and the effects of reinforcement deterioration.

Bridge Deck Construction

Concrete was poured into the decks using a two-yard bucket and a crane, and was leveled using an automatic screed (Figure 33). Sixteen to eighteen truckloads of concrete (approximately 95 cubic meters) were used to cast each bridge deck. During construction of the decks, MDT performed air content tests, slump tests, and collected test specimens from selected concrete trucks to make compression strength test cylinders. Average results from the slump and air content tests are presented in Table 7. In addition, concrete test specimens were collected from a representative portion of the concrete used to cast the instrumented section of each deck. Sufficient samples were collected to be

tested at three different times: 28-days, at the first live load test (approximately 90 days), and at the second live load test (approximately 810 days). All material sampling and testing was conducted in substantial accordance with ASTM specifications.

Table 7. Average Slump and Air Content from Bridge Deck Concrete

Bridge Deck	Average Slump (mm)	Average Air Content (%)
Conventional	79.4	6.3
Empirical	92.1	6.5
HPC	168.3	5.1



Figure 33. Placing the Deck Concrete

Selected concrete specimens were cured in a moist cure room at Montana State University (for 28-day testing) while the remaining samples were cured near the bridge decks. These specimens were used to determine uniaxial compressive strength, elastic modulus, splitting tensile strength, modulus of rupture, and shrinkage of the bridge deck concrete.

Average strength values and elastic moduli values from specimen tests at 28-days and 90-days (approximately the time of the live load tests) are provided in Tables 8 and 9, respectively.

Table 8. Average Compressive Strengths of Deck Concrete

Bridge	Compressive Strength	
	28-day (MPa)	90-day (MPa)
Conventional	28.0	38.1
Empirical	27.4	32.9
HPC	46.0	49.3

Table 9. Average Modulus of Elasticity of Deck Concrete

Bridge	Modulus of Elasticity	
	28-day (MPa)	90-day (MPa)
Conventional	29.3	30.2
Empirical	27.9	24.8
HPC	31.5	30.8

Steel Reinforcement

Epoxy-coated, Grade 60 steel rebar was specified for each of the bridge decks. Mill test reports were obtained from MDT for the rebar used in the project. The average yield strength of the steel is 478.5 MPa, the average ultimate tensile strength is 740.1 MPa, and the elongation at failure is 14.5 percent.

Prestressed Girders

Type I prestressed concrete beams were used to support the bridge decks. The dimensions and section properties of these beams are shown in Figure 34. The girders were cast near Billings, Montana, and therefore have a different aggregate source than the concrete used for the bridge decks. The results of the concrete compressive strength tests for the prestressed girders (obtained from MDT) were averaged and are summarized in Table 10.

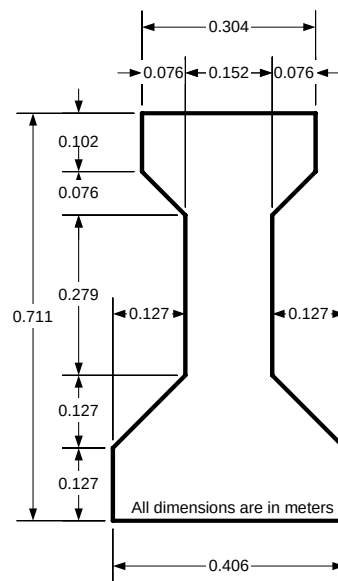


Figure 34. Dimensioned Cross-Section of Saco Bridge Girders

Table 10. Average 28-day Prestressed Girder Concrete Compressive Strengths

Bridge	Compression Strength (MPa)
Conventional	63.0
Empirical	61.0
HPC	64.2

Long Term Strain Monitoring

Vibrating wire strain gages were located in specific areas of the bridge decks to characterize their behavior under long-term environmental loads. These gages were installed adjacent to the top (T) and bottom (B) reinforcing mats at the locations shown in Figure 35. Relative to nomenclature, gage “TV-D-3-B” is a transverse (T) vibrating wire gage (V) at grid point D-3, in the bottom (B) of the deck.

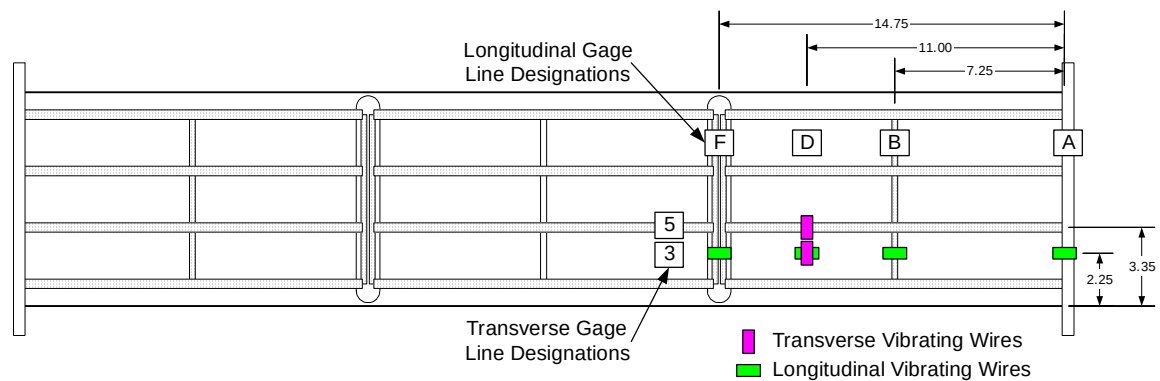


Figure 35. Vibrating Wire Locations

The 20 vibrating wire strain gages that were embedded in each bridge deck for long term monitoring were purchased from Geokon (model VCE-4200). This gage, shown in Figure 36, has a 153 mm gage length, 3000 $\mu\epsilon$ range, and 1 $\mu\epsilon$ sensitivity. They are

designed to be embedded directly in concrete and are typically used to monitor long-term strain and temperature in structures such as foundations, piles, bridges, dams, containment vessels, and tunnel liners.

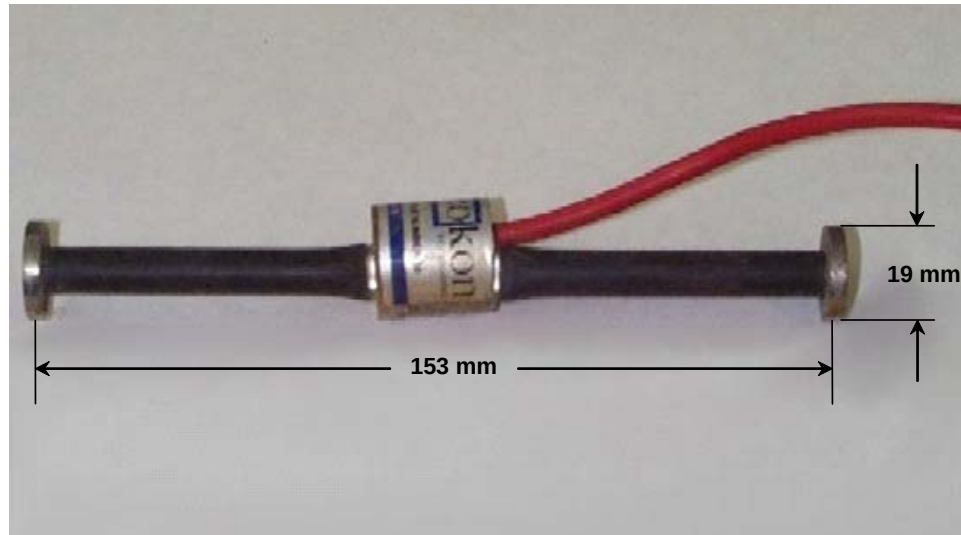


Figure 36. Vibrating Wire Strain Gage (VCE-4200)

The gages use a steel wire in tension between two circular end plates to measure length changes in the concrete. As the concrete contracts or expands, the wire responds accordingly, thereby changing its resonant frequency of transverse vibration. The wires are excited by electromagnetic coils, which also detect their resonant frequencies of vibration. Frequencies detected by the coil are converted to a DC voltage (using a Campbell Scientific, Inc. AVW1 unit), which are then recorded using a Campbell CR5000 data logger. The AVW1 unit will accommodate a single vibrating wire gage or, if a multiplexer is used, up to 16 vibrating wire gages may be sequentially converted and recorded. Each vibrating wire gage is also equipped with a thermistor to record temperature. Temperature measurements are used to apply temperature corrections to

measured strains. Differences in the thermal expansion coefficients of steel and concrete necessitate these temperature corrections.

Using plastic coated steel wire, each vibrating wire strain gage was suspended in the concrete between the reinforcing bars (Figure 37). As suggested by Geokon, a thin rubber pad was placed between the gage body and the tying wire to damp resonant vibrations in the steel wire and rebar cage. Strain and temperature measurements have been recorded hourly in the Saco bridge decks starting soon after they were poured up to the present.

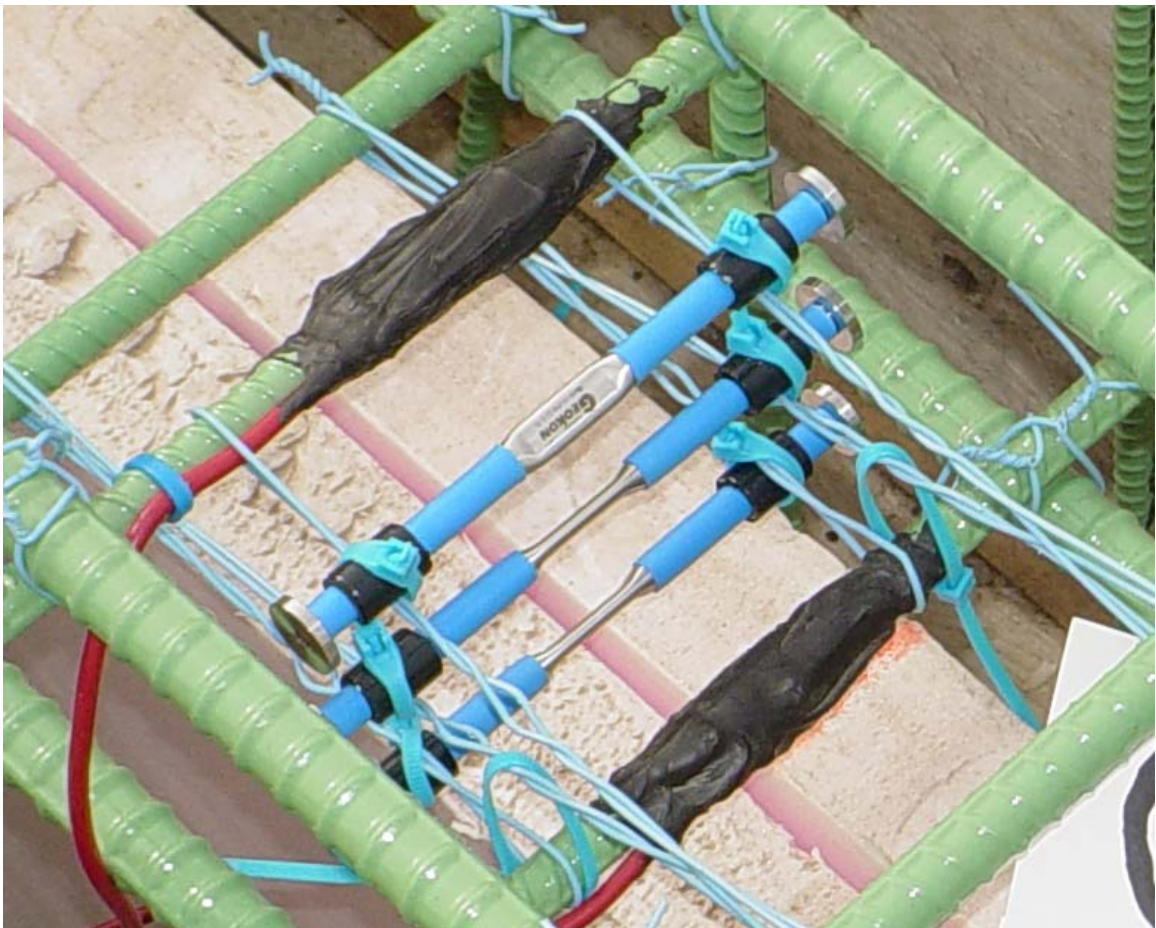


Figure 37. Vibrating Wire Installation

Data Analysis

Two “types” of strain are of primary interest in this investigation, the total strain experienced by the decks as they undergo thermal changes and the part of the total strain that has internal stress associated with it. The total strain is of interest because it reflects the deformed physical shape of the bridge decks. The internal stress strain is of importance because it reflects the internal material stresses and forces that could cause damage to the decks. The intent of a typical strain gage installation is to collect information on the stress related strain, and the concrete strain data from the vibrating wire gages was initially corrected to produce “stress” strain using the equation provided by Geokon for this purpose (see Equation 11).

$$\varepsilon_{\sigma} = \varepsilon_M + (\alpha_S - \alpha_C) \cdot (T_M - T_R) \quad (\text{Eq.11})$$

where:

- ε_{σ} = Stress related strain
- ε_M = Measured strain
- α_S = CTE of steel
- α_C = CTE of concrete
- T_M = Measured temperature
- T_R = Reference temperature

Stress strain is the strain that results from a stress in the concrete. This condition typically results from external loads applied to the structure, and the associated strains can be negative and positive. The external load applied to the structure can be generated by its supports, if they interfere with dimensional movements associated with temperature

changes, shrinkage, or creep. For example, when a structure with fixed supports experiences an increase in temperature, the associated expansion is resisted by the supports, generating compression throughout the structure. Since unrestrained thermal strains produce no stress in a structure, the measured strains typically are adjusted to remove this part of the strain response.

Thermal strain is the strain developed by the change in a material's length due to a change in temperature:

$$\varepsilon_{Th} = \int_{T_R}^{T_M} \alpha_c \cdot dT \quad (\text{Eq.12})$$

where:

$$\varepsilon_{Th} = \text{Thermal related strain}$$

This strain is positive when the temperature increases and negative when the temperature decreases. Thermal strain occurs without any stress, if the structure is free to change shape and size under the effect. If the strain sensing elements used in the decks were made of concrete, they would not register any strains in the deck as they freely expand and contract under temperature changes, as the gages would have exactly the same strain as the deck, itself. Since the gage sensing elements are made of steel, a correction factor (effectively the second term in Equation 11) is necessary to compensate for the difference in the CTEs of the steel gage components and the surrounding concrete. Equation 11 assumes constant CTEs across the entire temperature range for both the steel and concrete. The laboratory CTE analysis, however, showed that the CTE of the deck

concrete varies significantly (approximately 50 percent) over the temperature range of interest (-40°C to 27°C).

Thus, a more accurate stress strain was determined by using the concrete CTEs obtained from the laboratory portion of this investigation, specifically, the CTE curve from the conventional specimen 1-A during cooling cycle four, in Equation 11. The total strain was subsequently obtained by adding the thermal strain to the corrected “stress” strain:

$$\varepsilon_T = \varepsilon_\sigma + \alpha_C \cdot (T_M - T_R) \quad (\text{Eq.13})$$

where:

$$\varepsilon_T = \text{Total strain}$$

Note that the total strain in concrete is comprised of four main components: 1) thermal strain, 2) stress strain, 3) creep strain, and 4) shrinkage strain. The corrected “stress” strain generated by Equation 11 actually consists of the stress strain, creep strain, and shrinkage strain in the concrete:

$$\varepsilon_T = \varepsilon_{Th} + \varepsilon_\sigma + \varepsilon_{Cr} + \varepsilon_{Sh} \quad (\text{Eq.14})$$

where:

$$\varepsilon_{Cr} = \text{Creep strain}$$

$$\varepsilon_{Sh} = \text{Shrinkage strain}$$

Creep and shrinkage are time dependent phenomenon, and in many experiments of short duration they are small in magnitude and thus neglected. Creep strain is a phenomenon associated with sustained loading of concrete. Concrete under sustained loading will try to redistribute the internal stress to a lower state of equilibrium, similar to

how clayey soils will consolidate under long-term loads. The Feldman-Spears Model explains this phenomenon by the removal of interlayer water from the C-S-H gel. The external load on the concrete places a pressure on the pore water. Over time, the pore water relieves the pressure by exiting the concrete's microstructure. The result is similar to shrinkage, in that the concrete shrinks as the water's resistance is removed. Creep strain was neglected in this investigation primarily because it is small in magnitude for lightly loaded members.

Shrinkage strain develops as the concrete cures and dries, and it typically occurs within the first six months after it was cast. Generally, shrinkage strains are not recoverable, except under extraordinary circumstances. Shrinkage is associated with the use (by curing) and/or removal (by drying) of pore water from the concrete microstructure and produces a negative (shrinking) strain. The AASHTO bridge design specifications suggest using the following shrinkage equation (2000):

$$\varepsilon_{sh} = -k_s \cdot k_h \left(\frac{t}{35 + t} \right) \cdot 510 \quad (\text{Eq.15})$$

where:

$$k_s = \left(\frac{\frac{t}{26 \cdot e^{0.36 \cdot VIS} + t}}{\frac{t}{45 + t}} \right) \left(\frac{1096 - 94 \cdot VIS}{923} \right) \quad (\text{Eq.16})$$

$$k_h = \frac{(140 - H)}{70}, \text{ for average annual relative humidity's } (H) < 80\% \quad (\text{Eq.17})$$

$$k_h = \frac{3 \cdot (100 - H)}{70}, \text{ for average annual relative humidity's } (H) \geq 80\% \quad (\text{Eq.18})$$

H = Average annual relative humidity's as a percentage

t = Time in days since the concrete was removed from curing

VIS = Volume to surface area ratio expressed in inches

The design manual suggests an average annual relative humidity of about 60 percent for the Saco bridge decks with a VIS ratio of approximately 101.25mm (3.99 inches).

Shah and Ahmad state that “laboratory and field studies have shown that shrinkage of higher strength concrete is similar to that of lower-strength concrete” (1994). This observation, along with the absence of any parameter in the AASHTO equation related to type of concrete, implies that there should be no appreciable differences in the shrinkage strains between the three bridges. Figure 38 shows the shrinkage strain calculated for the Saco bridge decks using Equation 15.

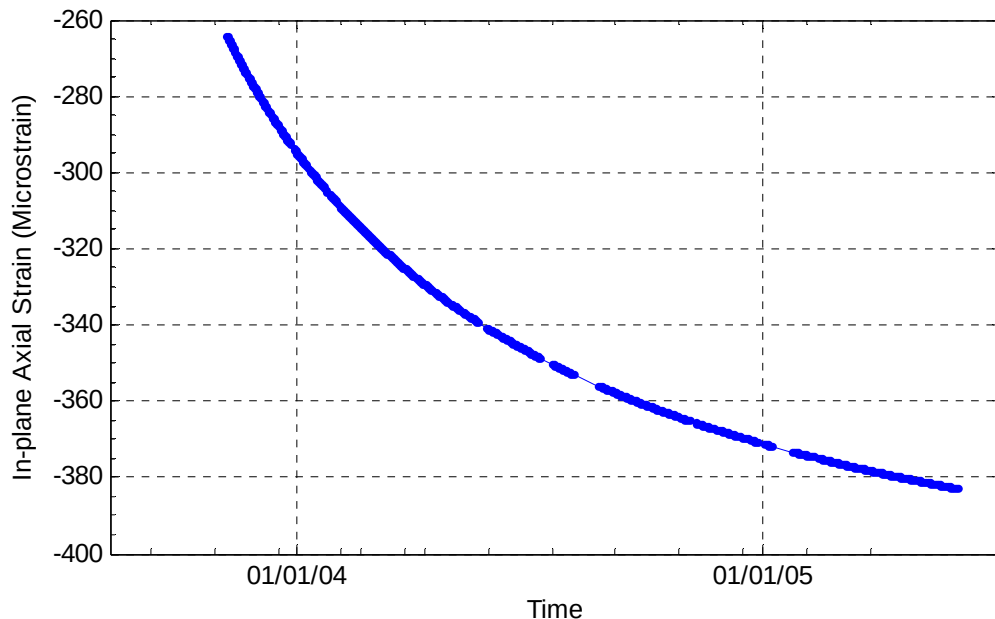


Figure 38. Typical Shrinkage Strain for Saco Bridges

The following section draws on this strain discussion to determine stress strains. Stress strains, where referred to, were calculated as:

$$\varepsilon_{\sigma} = \varepsilon_T - \varepsilon_{Th} - \varepsilon_{Sh} \quad (\text{Eq.19})$$

Bridge Findings

Thermal Strain and CTE

An algorithm, similar to the one used to determine the CTEs for the laboratory specimens from strain and temperature data, was used to estimate CTEs of the actual bridge decks from the long term strain data. As part of this analysis, it was assumed that at the center of the bridge span, the deck is relatively free to expand and contract in the transverse direction. This assumption was justified because of the large in-plane stiffness the bridge decks compared to the weak axis stiffness of the bridge girders. Note that any transverse resistance offered by the girders should act to reduce the apparent CTE values of the decks by imposing compression under a temperature increase and tension under a temperature decrease. Following these assumptions, the change in total strain in the transverse direction as a function of temperature is approximately the CTE of the composite bridge deck section (including the effects of both the concrete and reinforcing steel).

Figure 39 and Figure 40 show the results of these CTE calculations as a function of absolute temperature for the same location on the conventional and empirical decks, respectively. The CTE values are similar in magnitude to those observed in the laboratory (8 to 14 $\mu\text{ε}^{\circ}/\text{C}$), and the trend in the values relative to absolute temperature

(increasing CTE with increasing temperature) is similar to the trend seen for the laboratory specimens (Figure 17). The spreading of the bridge CTE values is due in part to the algorithm used in their calculations and the presence of stress strain within the data.

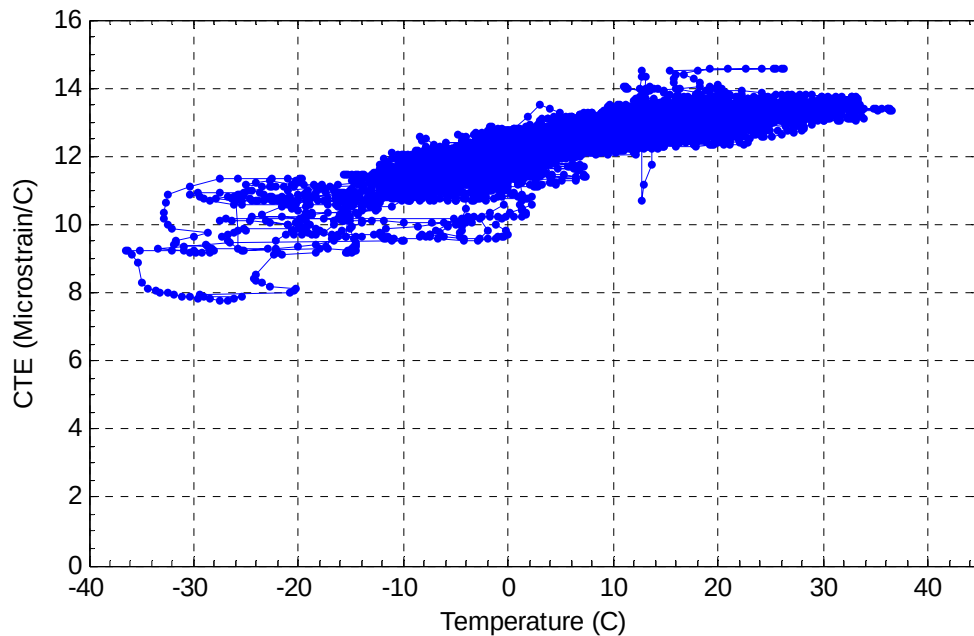


Figure 39. Conventional Deck CTE Values at Location TV-D-3-B

Figure 41 shows the CTE values calculated for the same location as above for the high performance deck. These CTE values are radically different (-60 to $20 \mu\epsilon/^{\circ}\text{C}$) from the CTE values calculated for the conventional and empirical decks (10 to $14 \mu\epsilon/^{\circ}\text{C}$) (as well as from the values from the laboratory specimens for the HPC concrete). In an effort to understand what might be occurring with the high performance deck, CTE values were also calculated in the transverse direction at an adjacent location in the deck (see Figure 42).

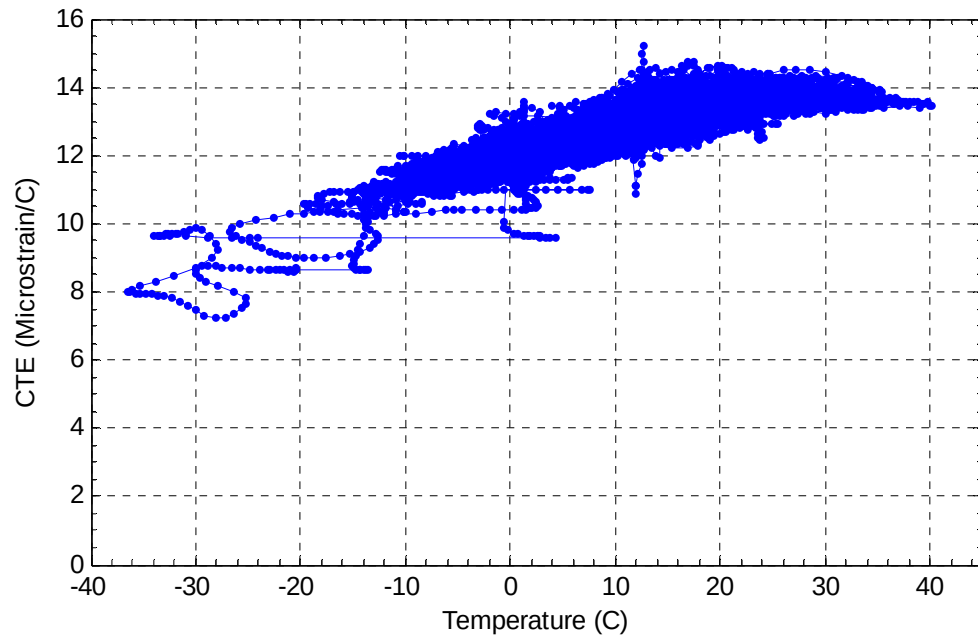


Figure 40. Empirical Deck CTE Values at Location TV-D-3-B

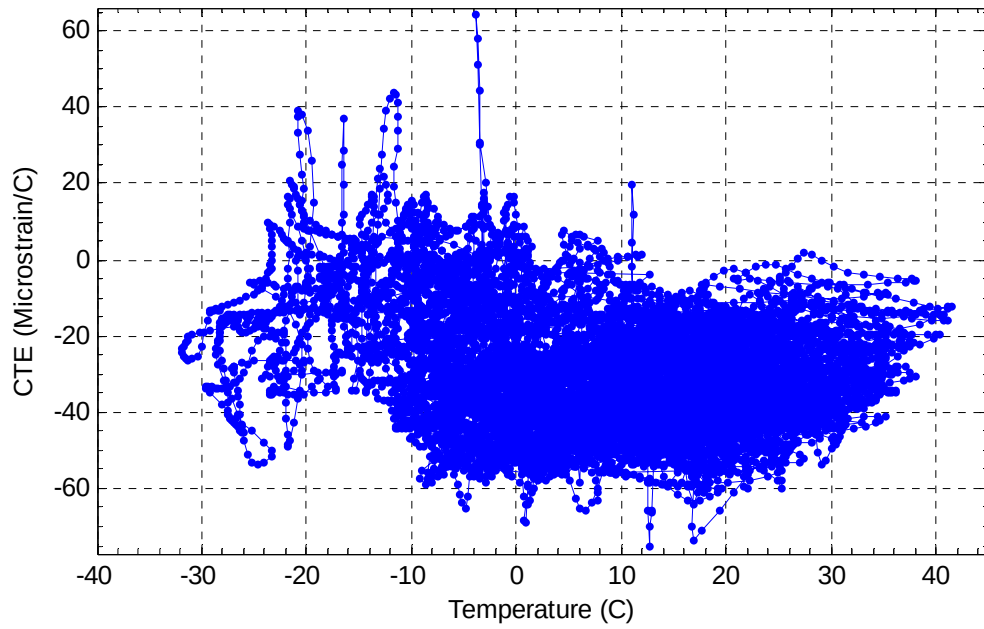


Figure 41. High Performance Deck CTE Values at Location TV-D-3-B

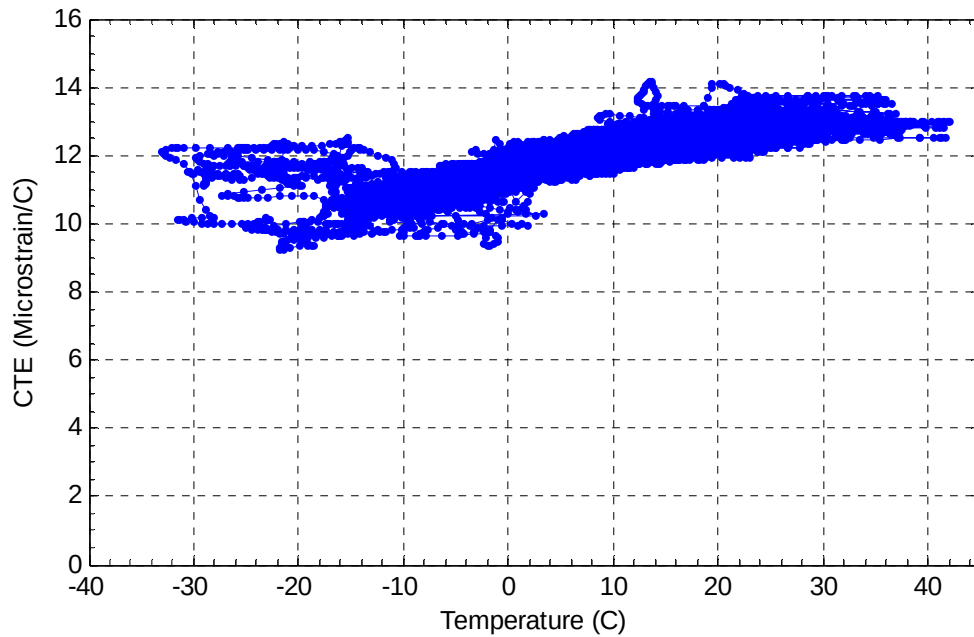


Figure 42. High Performance Deck CTE Values at Location TV-D-5-B

The CTE values at this location are very similar to those calculated for the conventional and empirical decks, indicating that the unusual behavior observed in the CTE values at the first location may be indicative of some unusual localized behavior adjacent to the gage location. This location (centered between the girders) is of specific interest because it is the location at which the greatest tensile stresses are expected in the decks in the transverse direction under vehicle loads. The unusual CTE behavior at this location could be indicative of tensile cracking. The negative CTE values may represent events where during an increase in temperature the crack closes, producing a negative strain and conversely for decreasing temperatures the crack opens, producing positive strains. Notably, a similar CTE behavior was observed at another location in the decks that is known (from visual inspection) to be cracked. The CTE behavior at this cracked location

is shown in Figure 43. A field inspection will be conducted in the future to see if a longitudinal tension crack has formed in the high performance deck between the girders.

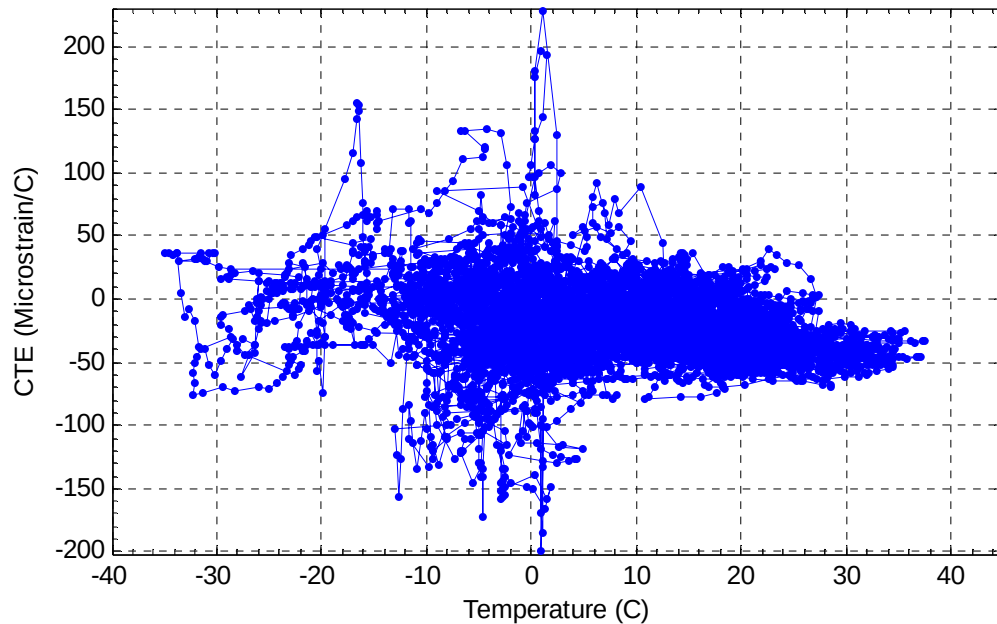


Figure 43. Conventional Deck CTE Values at Location LV-F-3-T
(Which is Known to be Cracked)

If a crack is found in the high performance deck, it may be valuable to investigate changes in CTE behavior (as determined from the strain data) as an indicator of the occurrence of physical distress in the decks. It may be informative to analyze data recorded earlier at this location to determine if any features in the response can be identified as indicative of the future cracking behavior of the deck. Predicting future distress from present response data is an elusive problem, and this type of CTE analysis may contribute to its solution.

Composite Behavior

The CTE values determined from the field data and presented in Figures 39, 40, and 42 are approximately $2 \mu\epsilon/^{\circ}\text{C}$ higher than the CTE values measured in the laboratory. The probable explanation for this difference is that the steel within the concrete is acting with the concrete to increase the CTE of the composite material. The *AASHTO Guide Specifications Thermal Effects in Concrete Bridge Superstructures* states that “It is thought that the maximum effect on the concrete’s thermal coefficient by the presence of steel reinforcement in no more than 10 percent” (1989). The effect of the steel in this case appears to be somewhat larger than that suggested by AASHTO (approaching 20 percent at lower temperatures).

Structural Behavior under Temperature Changes

The behaviors of the Saco bridge decks under temperature changes are significantly influenced by the composite nature of their construction. The bridge decks behave compositely in two respects. Following the fundamental behavior of composite cross-sections, the unsymmetrical layout of the reinforcement causes internal stresses and moments in the decks when they undergo a temperature change. More globally, the decks are rigidly attached by shear studs to the prestressed girders underneath, and thus act compositely with the girders in longitudinally carrying applied loads into the supports. Force equilibrium and strain compatibility are imposed to analyze composite systems. Force equilibrium requires that the internal forces must balance at any cross-section. Strain compatibility requires that there must be continuity in the deformations

over the depth of cross-section. The second condition is typically simplified to the assumption of a linear strain profile across the depth of the cross-section.

An analysis of the first composite behavior mentioned above, that is, the behavior of the reinforced deck cross-section under a change in temperature, was done using the strain data collected in the transverse direction of the decks. Typical deck cross-sections in the transverse direction were previously presented in Figure 31. The behavior of these deck sections when subjected to temperature changes was analyzed using an approach developed from the methodology used by Park and Paulay (Park and Pauley 1975) to investigate shrinkage related stresses in unsymmetrically reinforced members. The modified analysis addresses the stresses and deformations introduced in the section by the differences in the CTE values between concrete and steel. The analysis uses the transformed section properties and assumes the principle of superposition is valid. The temperature effects are evaluated for two cases, a uniform temperature change and a linear temperature gradient, and the results are superimposed.

The uniform temperature change analysis begins with the calculation of the uniform concrete thermal strain for the temperature change under consideration, assuming unrestrained conditions. The reinforcement is then assumed to develop a force based on the difference between the concrete and steel thermal strains for this temperature change.

The basic composite cross-section was assumed to have the generalized geometry described in Figure 44.

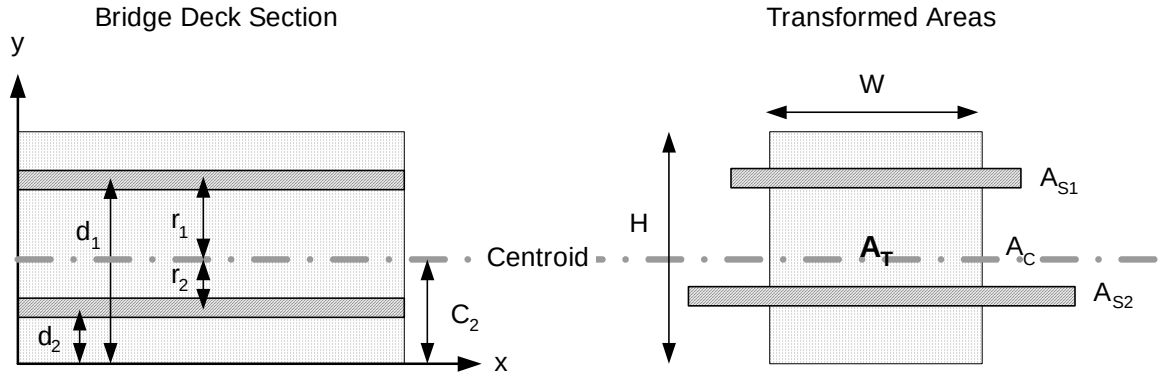


Figure 44. Composite Action Diagram

The associated section properties were calculated as:

$$n = \frac{E_S}{E_C} \quad (\text{Eq.20})$$

$$A_T = H \cdot W + (n-1) \cdot A_{S1} + (n-1) \cdot A_{S2} \quad (\text{Eq.21})$$

$$C_2 = \frac{A_C \cdot \frac{H}{2} + (n-1) \cdot A_{S1} \cdot d_1 + (n-1) \cdot A_{S2} \cdot d_2}{A_T} \quad (\text{Eq.22})$$

$$I_T = \frac{H^3 \cdot W}{12} + A_C \cdot \left(\frac{H}{2} - C_2 \right)^2 + (n-1) \cdot A_{S1} \cdot (r_1)^2 + (n-1) \cdot A_{S2} \cdot (r_2)^2 \quad (\text{Eq.23})$$

where:

E_S = Modulus of elasticity for the reinforcement

E_C = Modulus of elasticity for the concrete

A_{S1} = Area of reinforcement at location 1

A_{S2} = Area of reinforcement at location 2

r_1 = Distance from the centroid to location 1

r_2 = Distance from the centroid to location 2

Due to the difference in the CTE values for the steel and concrete, an external force would have to be applied to the reinforcing steel if the composite cross-section was to actually experience a uniform strain across its depth (see Figure 45a). The magnitude of the required forces can be calculated based on the difference in the CTE values for steel and concrete. Once the magnitudes of the forces are known, equal and opposite forces can be applied to the cross-section, and the strains across its depth can be calculated using elastic principles (see Figure 45b). The final or net behavior of the cross-section (Figure 45c) is obtained by adding the effects of the uniform strain case (Figure 45a) to the effects of the forces that actually develop between the steel and concrete (Figure 45b).

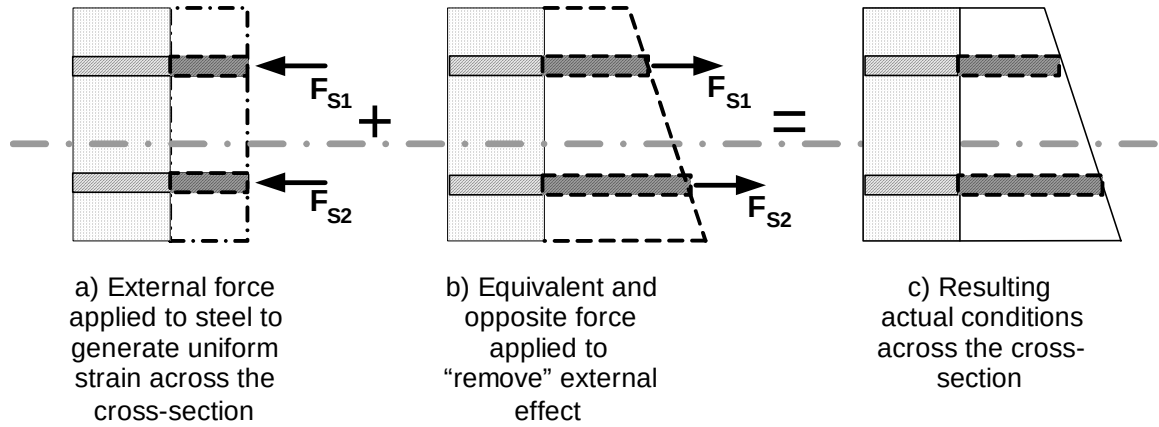


Figure 45. Composite Action for a Uniform Temperature Change

The specific operations that accompany this sequence of calculations are summarized below.

$$T_U = T_2 + r_2 \cdot \frac{(T_2 - T_1)}{(r_2 - r_1)} \quad (\text{Eq.24})$$

where:

T_U = Uniform temperature determined at the centroid

T_1 = Temperature measured at location 1

T_2 = Temperature measured at location 2

and

$T_1 = T_2$ for the uniform temperature case

Strains associated with the case of uniform strains across the depth of the cross-section are calculated as:

$$\varepsilon_{C_{thermal\ 1}} = \varepsilon_{C_{thermal\ 2}} = \int_{T_R}^{T_M} \alpha_C \quad (\text{Eq.25})$$

$$\varepsilon_{C_{thermal\ 1}} = \varepsilon_{C_{thermal\ 2}} = \int_{T_R}^{T_M} \alpha_S \quad (\text{Eq.26})$$

The external forces that have to be applied to the steel to generate the uniform strain case can then be written as:

$$F_{S1} = A_{S1} \cdot E_S \cdot (\varepsilon_{S_{thermal\ 1}} - \varepsilon_{C_{thermal\ 1}}) \quad (\text{Eq.27})$$

$$F_{S2} = A_{S2} \cdot E_S \cdot (\varepsilon_{S_{thermal\ 2}} - \varepsilon_{C_{thermal\ 2}}) \quad (\text{Eq.28})$$

The axial force and moments from these forces are:

$$P = F_{S1} + F_{S2} \quad (\text{Eq.29})$$

$$M = F_{S1} \cdot r_1 + F_{S2} \cdot r_2 \quad (\text{Eq.30})$$

The strains associated with these internal force resultants are:

$$\varepsilon_{C_{Steel}}(y) = \frac{P}{A_T \cdot E_C} + \frac{M \cdot y}{I_T \cdot E_C} \quad (\text{Eq.31})$$

Finally, the true strain across the cross-section can be calculated by superimposing these two results as:

$$\varepsilon_{C_{Total}}(y) = \varepsilon_{C_{thermal}} + \varepsilon_{C_{Steel}}(y) \quad (\text{Eq.32})$$

For a gradient temperature across the depth of the deck cross-section, the general analysis approach is similar to that used for the uniform temperature case; the complexity of the analysis is simply increased to accommodate variable concrete strains across the cross-section, as a function of changing temperature (see Figure 46).

Temperature at any depth in the cross-section can be calculated in terms of T_1 and T_2 assuming a linear gradient as:

$$T(y) = T_2 + \frac{T_1 - T_2}{r_1 - r_2} \cdot y \quad (\text{Eq.33})$$

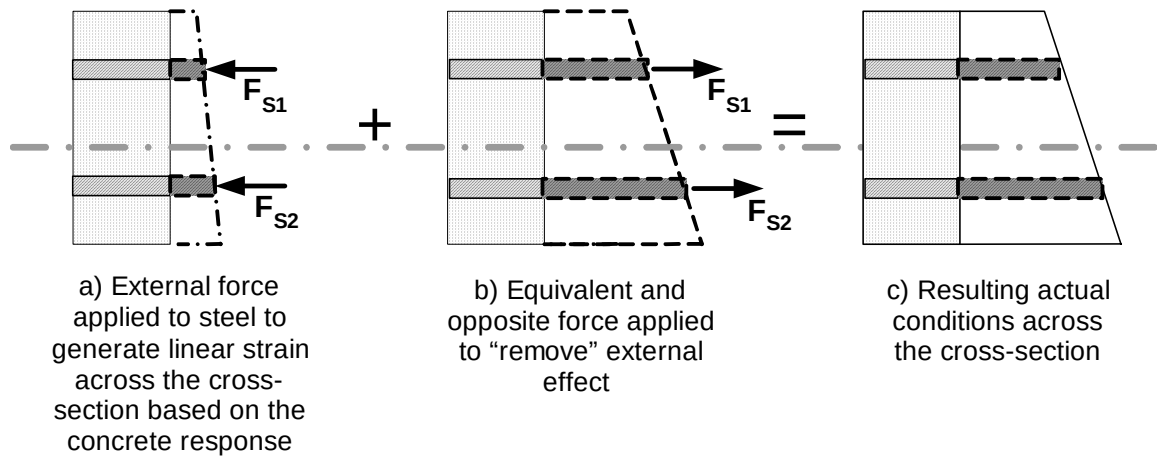


Figure 46. Composite Action for a Gradient Temperature Change

Thermal strains of interest can then be independently calculated at locations 1 and 2 through the depth as:

$$\varepsilon_{C_{thermal}}(y) = \int_{T_R}^{T(y)} \alpha_C \quad (\text{Eq.34})$$

$$\varepsilon_{C_{thermal1}} = \int_{T_R}^{T_{M,1}} \alpha_C \quad (\text{Eq.35})$$

$$\varepsilon_{S_{thermal1}} = \int_{T_R}^{T_{M,1}} \alpha_S \quad (\text{Eq.36})$$

$$\varepsilon_{C_{thermal2}} = \int_{T_C}^{T_{M,2}} \alpha_C \quad (\text{Eq.37})$$

$$\varepsilon_{S_{thermal2}} = \int_{T_C}^{T_{M,2}} \alpha_S \quad (\text{Eq.38})$$

The equivalent external forces that must be applied in this case (see Figure 46) can then be calculated as:

$$F_{S1} = A_{S1} \cdot E_S \cdot (\varepsilon_{S_{thermal1}} - \varepsilon_{C_{thermal1}}) \quad (\text{Eq.39})$$

$$F_{S2} = A_{S2} \cdot E_S \cdot (\varepsilon_{S_{thermal2}} - \varepsilon_{C_{thermal2}}) \quad (\text{Eq.40})$$

From equilibrium considerations:

$$P = F_{S1} + F_{S2} \quad (\text{Eq.41})$$

$$M = F_{S1} \cdot r_1 + F_{S2} \cdot r_2 \quad (\text{Eq.42})$$

Relating strains to internal forces:

$$\varepsilon_{C_{Steel}}(y) = \frac{P}{A_T \cdot E_C} + \frac{M \cdot y}{I_T \cdot E_C} \quad (\text{Eq.43})$$

Finally, superimposing the resulting strains:

$$\varepsilon_{C_{Total}}(y) = \varepsilon_{C_{thermal}}(y) + \varepsilon_{C_{Steel}}(y) \quad (\text{Eq.44})$$

Using these equations, and the actual temperatures experienced by the decks in the field, the theoretical increase in thermal strains due to composite behavior were

calculated (i.e. the second term in the Equations 32 and 44). These strains were then analyzed as a function of temperature to obtain an equivalent CTE value representing the contribution of composite action to the thermal behavior of the decks. Figures 47, 48, and 49 show the CTE values corresponding to these additional composite strains for each deck type. These CTE values can be added directly to the previously calculated concrete CTE values to obtain the net CTE values for the deck configurations. As expected, the composite contribution to the CTE values increase with decreasing temperatures, reflecting the higher CTE value of steel ($12.1 \mu\epsilon/^\circ\text{C}$) relative to concrete at lower temperatures. The composite effect is greatly diminished as the concrete CTE and steel CTE values approach each other as the temperature increases (the CTE values for the two materials are almost equal at a temperature of 25°C).

The conventional and high performance decks have similar composite CTE results (Figures 47 and 49); whereas the composite contributions to the CTE values for the empirical deck are approximately 40 percent less (see Figure 48). This difference is directly attributable to the difference in the reinforcing steel in the empirical deck compared to the conventional and high performance decks. The magnitude of the composite effect would be expected to be proportional to the amount of reinforcing steel in the deck, and the empirical deck has 70 percent less steel in the transverse direction than the conventional and high performance decks.

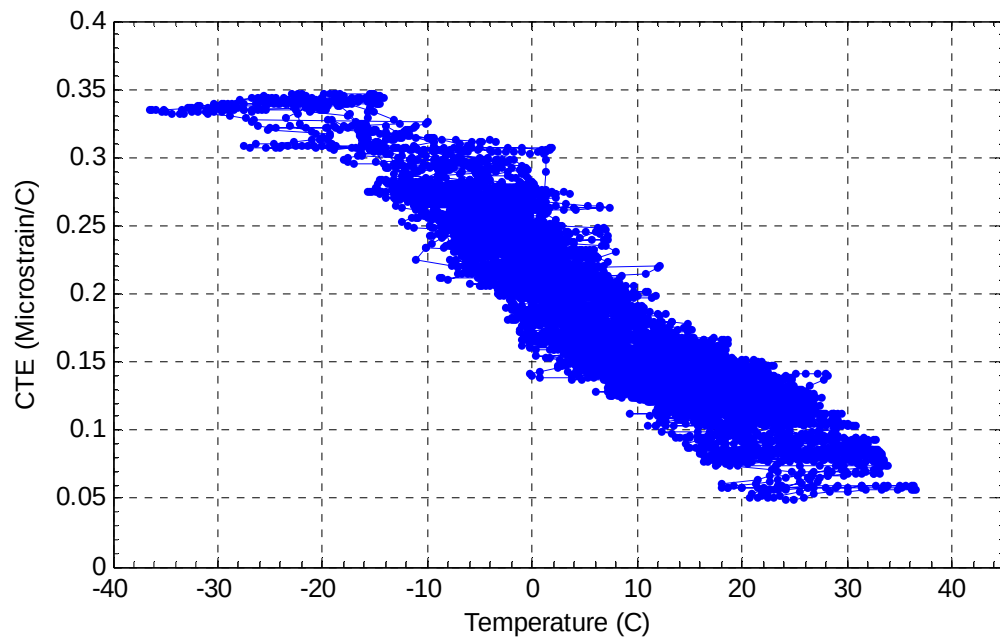


Figure 47. Conventional Deck Composite CTE Values at Location TV D 3 B

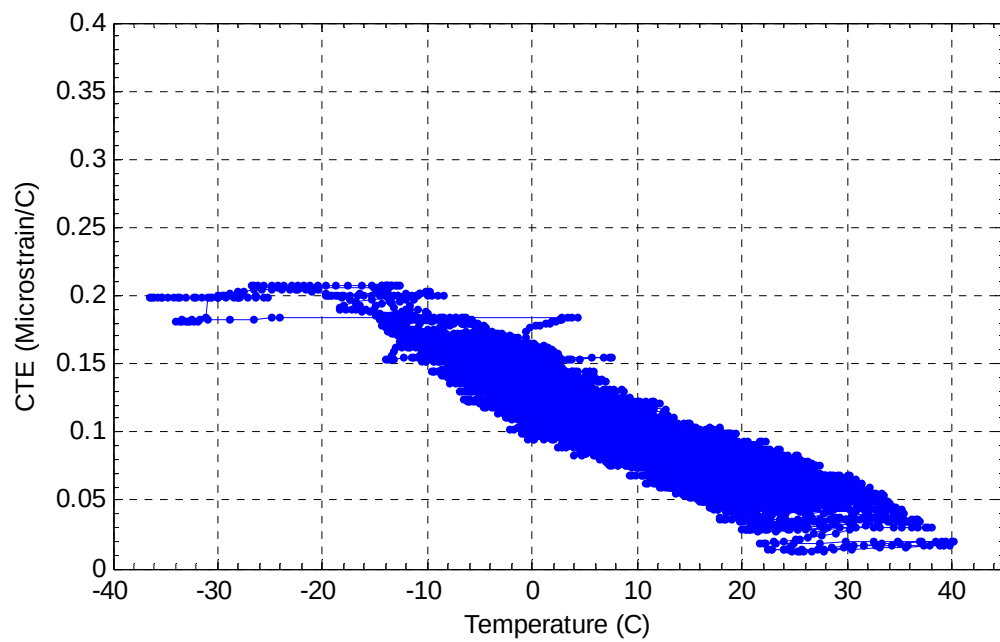


Figure 48. Empirical Deck Composite CTE Values at Location TV D 3 B

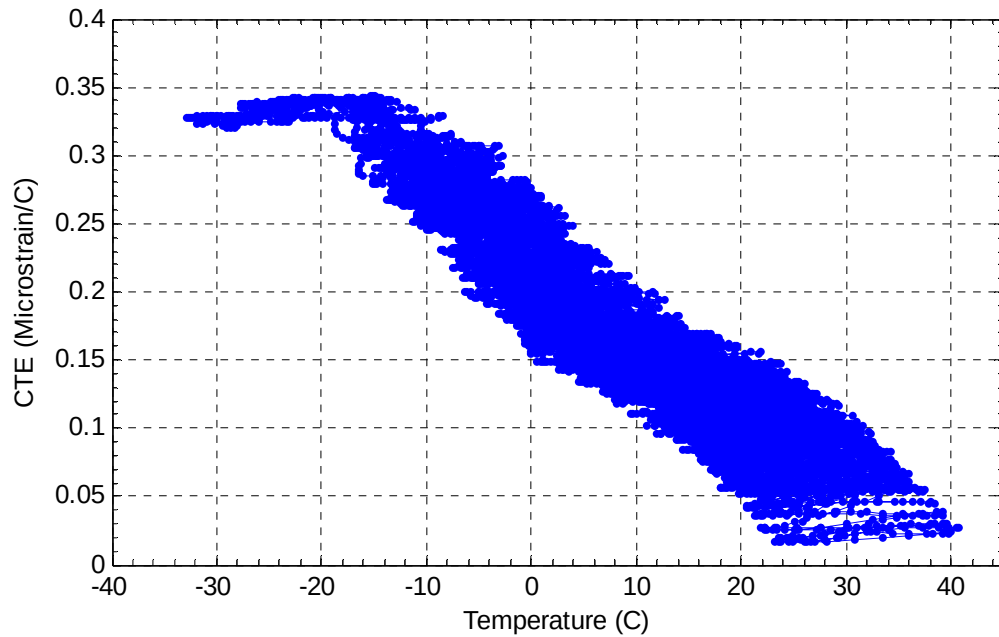


Figure 49. High Performance Deck Composite CTE Values at Location TV D 5 B

The strain contributions of composite action with respect to the CTE values are not large enough (approximately $0.35 \mu\epsilon/^\circ\text{C}$) to fully account for differences between the bridge CTE values and the pure concrete CTE values (approximately $2.0 \mu\epsilon/^\circ\text{C}$). This discrepancy suggests that the in-situ bridge deck concretes have slightly higher CTE values than the concrete specimens evaluated in the laboratory, even though they were cast from the same batch of concrete. This behavior may result from varying environmental conditions between the bridge deck concrete and the laboratory specimens, notably relative to their moisture conditions. As previously discussed, CTE values for concrete appear to be affected by the behavior of the water in the hardened paste matrix, and the amount of water in the hardened paste is influenced by the relative

humidity experienced by the concrete (which was not the same in the laboratory and at the bridges).

Bridge Deck Deformations

Another way to view the effects of environmental conditions on the Saco bridge decks is to more globally consider their bending and in-plane axial deformations. Curvatures can be conveniently used to look at the bending deformations of the decks, because they are easily visualized and can be readily calculated from the strains directly measured in the field. It can be shown that curvature is related to the axial strain at a distance (y) from the centroid of a cross section as (Gere 2001):

$$\kappa = \frac{-\varepsilon_x}{y} \quad (\text{Eq.45})$$

where:

κ = Curvature

ε_x = Axial strain

It can also be shown geometrically (see Figure 50) that the internal strain angle is related to the axial strain at a distance from the centroid as:

$$\tan \varphi = \frac{-\varepsilon_x}{y} \quad (\text{Eq.46})$$

where:

φ = Internal strain angle

y = Distance from gage location to the cross-section centroid

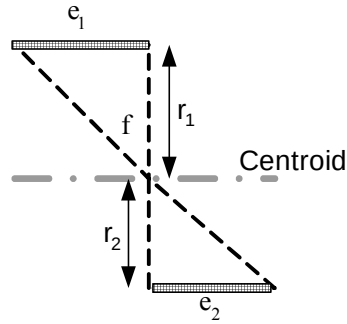


Figure 50. Internal Strain Angle

Equating the two previous equations gives:

$$\kappa = \tan \varphi \quad (\text{Eq.47})$$

Finally, the internal strain angle can be calculated using strains measured at two locations a distance apart in the deck as:

$$\kappa = \tan \varphi = \frac{\varepsilon_2 - \varepsilon_1}{r_2 - r_1} \quad (\text{Eq.48})$$

where:

ε_1 = Strain at location 1

ε_2 = Strain at location 2

r_1 = Distance from location 1 to the centroid of the cross-section

r_2 = Distance from location 2 to the centroid of the cross-section

Relative to in-plane deformations, these deformations are proportional to the normal axial strains experienced at the centroid of the deck cross-sections. Strain at the centroidal axis in the cross-section can be readily calculated from the strain measurements made at two locations across the depth of the deck as:

$$\varepsilon_{Centroid} = \varepsilon_2 + r_2 \cdot \frac{\varepsilon_2 - \varepsilon_1}{r_2 - r_1} \quad (\text{Eq.49})$$

The assumption of an uncracked concrete cross-section was used to determine the bridge deck centroid by the ‘transformed area’ method presented earlier.

The transverse response of the decks under different thermal conditions was once again viewed in terms of two components:

1. The net change in average temperature across the depth of the decks was assumed to predominantly generate in-plane deformations. The previously discussed curvature that develops under such a temperature change due to the unsymmetrical reinforcing pattern was neglected as small in magnitude.
2. The temperature gradient across the depth of the deck was assumed to primarily result in bending curvature of the decks, with a negative gradient (greater temperature at the top of the deck relative to the soffit) generating negative curvature.

Following this approach, typical bending and axial deformations of the bridge decks during diurnal temperature cycles were determined using the strain data from July 3, 2004 to July 5, 2004. These days were chosen for this analysis because of their even heating and cooling cycles which are generally representative of the bridge decks’ daily behavior. Using the equations derived above, curvature and total in-plane normal strain in the transverse direction were calculated from the field data as a function of time. These deformations are plotted from midnight on July 2 to midnight on July 4 in Figures 51, 52, and 53 for the conventional, empirical and high performance decks, respectively.

The response shown is for a location in the decks three-quarters along the span, and midway between the girders (Location D 3 in Figure 35). Also shown in Figures 51, 52, and 53 are the average temperatures across the depth of the decks, as well as the temperature gradients, as a function of time. Note that deformations in the transverse direction were analyzed in this investigation due to the relatively simpler boundary conditions in the transverse compared to the longitudinal direction of the decks.

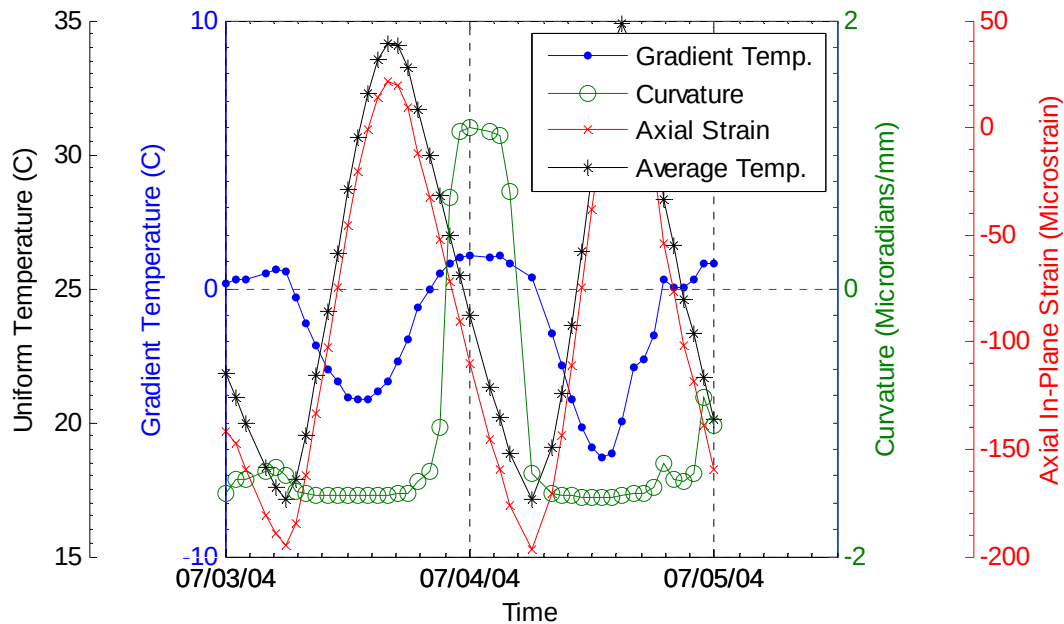


Figure 51. Conventional Deck Behavior at Location TV D 3 (Total Response)

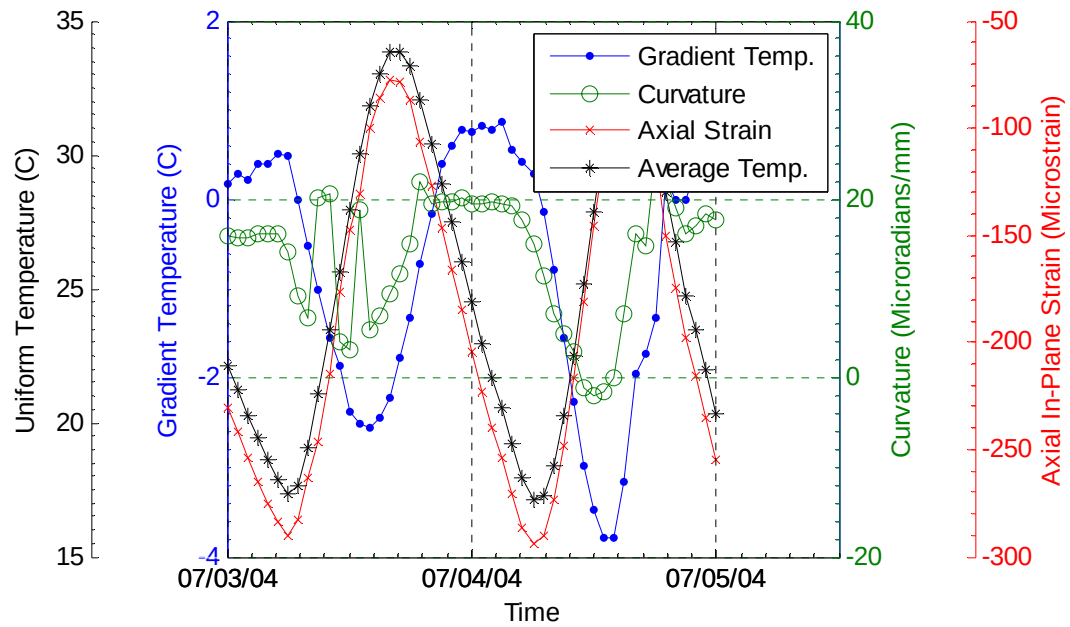


Figure 52. Empirical Deck Behavior at Location TV D 3 (Total Response)

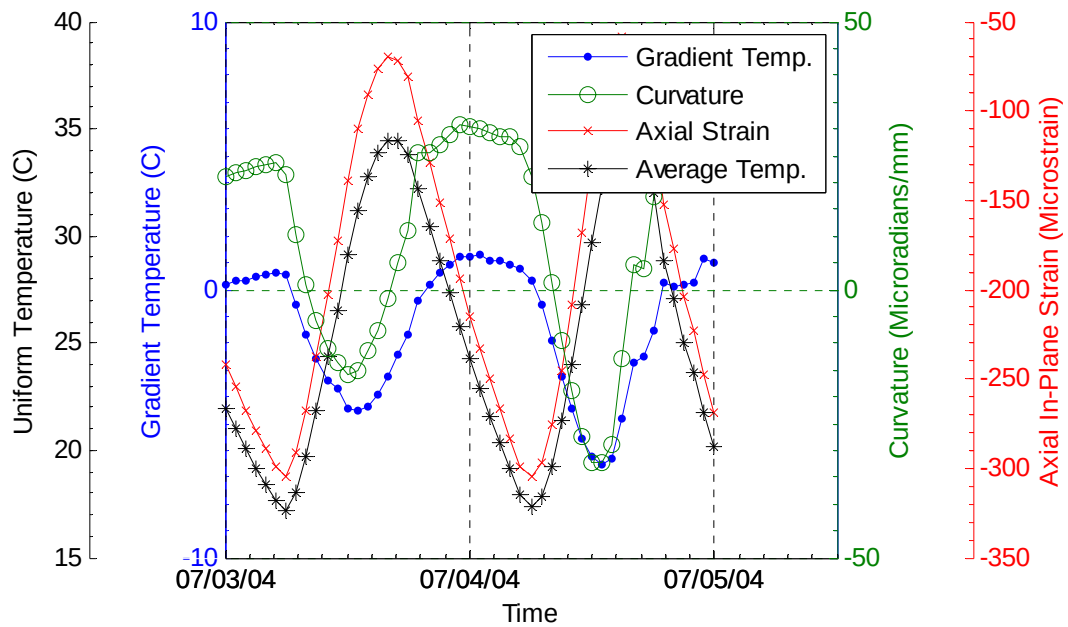


Figure 53. High Performance Deck Behavior at Location TV D 3 (Total Response)

Referring to Figures 51, to 53, the deformations are remarkably similar in all three decks; therefore, the behavior of only one of the decks, specifically the conventional deck (Figure 51), will be discussed in detail below. Further note that a more comprehensive view of the deck behavior in the transverse direction can be obtained by simultaneously looking at the data available at the face of a girder, as well as midway between the girders. The deformations of the conventional deck in the transverse direction at the face of a girder (location D 5 in Figure 35) are shown in Figure 54.

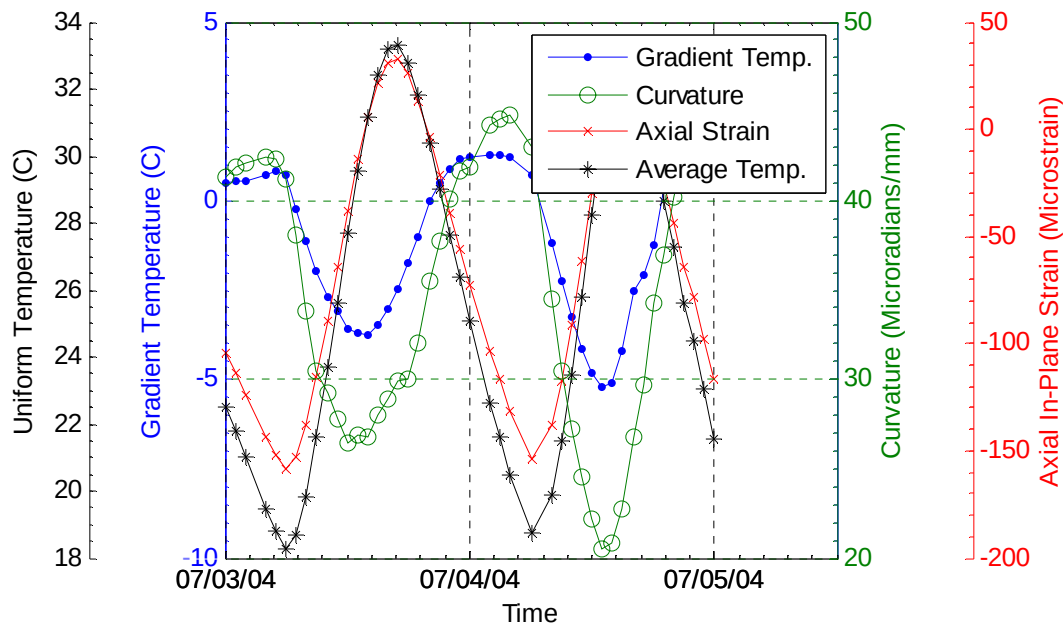


Figure 54. Conventional Deck Behavior at Location TV D 5 (Total Response)

Referring to Figures 51 and 54, the average temperature in the deck is lowest in the early morning (approximately 5:00am), with a value of approximately 17°C; it climbs during the day to a peak of approximately 34°C in the late afternoon (approximately 4:00pm). Note that the temperatures at the strain gage located in the middle of the span

between the girders (Figure 51) are approximately one degree cooler at the low point and one degree higher at the high point than the temperatures at the face of the girder (Figure 54). This behavior results from the thermal mass of the girder nominally moderating the temperature extremes in the deck immediately adjacent to it.

The temperature gradient experienced by the deck also varies throughout the day. The gradient, less than 1°C at 5:00am, becomes increasingly negative throughout the morning, as the top surface of the deck heats up more rapidly than the bottom surface. The peak negative gradient occurs around noon and is approximately -6°C for the location within the deck span and is -4°C at the face of the girder. The peak positive gradient temperature occurs near midnight, as the top surface of the deck cools more rapidly than the bottom surface, possibly because the girders and the ground below the bridge act as a heat sink, storing the day's heat and radiating it to the bottom of the deck in the evening.

Relative to the coincident deformations experienced by the deck under these thermal conditions, the total in-plane axial transverse strains are positive as the temperature increases, which is consistent with the deck expanding as the temperature increases. The magnitude of the change in the strain as the temperature moves between its extreme values is approximately $215\ \mu\epsilon$ for the location between the girders and $190\ \mu\epsilon$ for the location at the face of the girder. This difference of $25\ \mu\epsilon$ is due to the difference in the extreme temperatures at the two locations. The two degree difference in the extreme temperatures corresponds to a strain difference of $25\ \mu\epsilon$ using a CTE of $12.5\ \mu\epsilon/^{\circ}\text{C}$.

Therefore, the in-plane response at the two locations under the thermal change is nearly identical.

The significance of the absolute magnitudes of the in-plane strains reported in Figures 51 through 54 is somewhat uncertain, as it is possible that some baseline shifts may have inadvertently occurred in the data as a result of some rewiring that had to be done for the data acquisition system. Nonetheless, the absolute strain magnitudes are uniformly negative, which is consistent with the expected unrestrained shrinkage of the deck concrete over time. Furthermore, the general level of the absolute strains (150 to 300 $\mu\epsilon$) is consistent with the magnitude of the shrinkage strain estimated by Equation 15 (see Figure 38).

The curvature deformations of the decks are consistent with the thermal gradients that they experienced. A negative temperature gradient in the middle of the day, for example, during which the top of the deck expands more than the bottom, corresponds with a negative curvature (see Figure 55). Correspondingly a positive temperature gradient in the evening during which the soffit of the deck expands more than the top, corresponds with a positive curvature (see Figure 56).

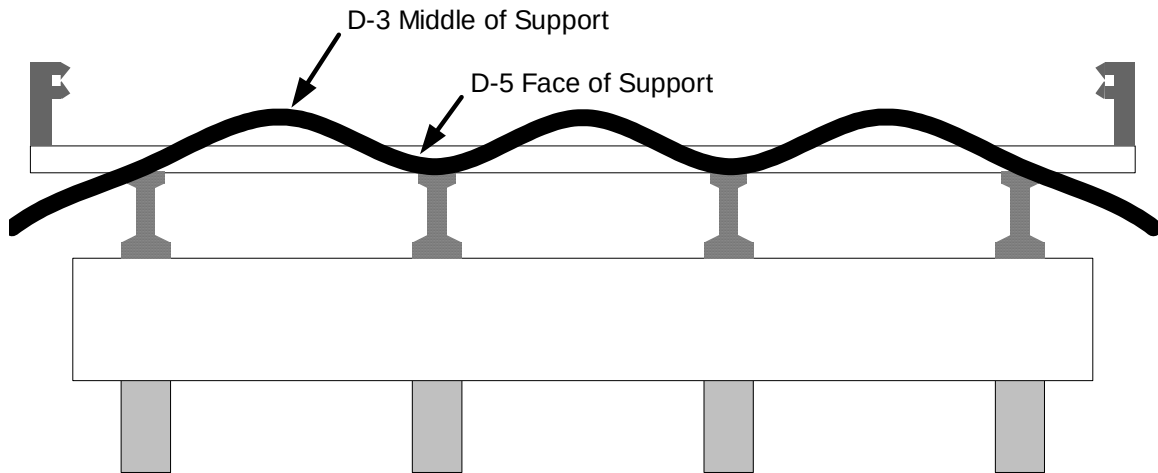


Figure 55. Expected Physical Bridge Deck Deformations
(for a Negative Temperature Gradient)

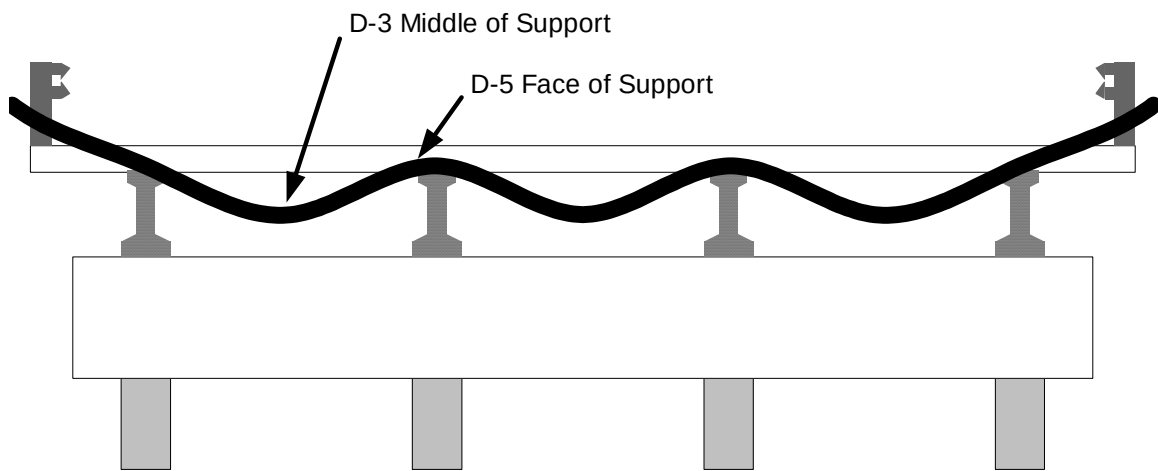


Figure 56. Expected Physical Bridge Deck Deformations
(for a Positive Temperature Gradient)

One anomaly in this response is that the changes in curvature observed in the interior of the spans (between the girders) are in the same direction as the changes in curvature at the face of the girders. The deck was expected to act in the transverse direction somewhat like a continuous beam over several supports, with opposite curvatures in the interior of the spans relative to the curvatures at the supports (i.e., at the girders). One

explanation for the observed response is that the expected point of contraflexure in the curvature occurs closer to the girders than the location of the strain gage at the face of the girders. In this case, the strain gages in the middle of the span and at the face of the girder would indicate the same direction of curvature. Relative to the absolute magnitudes of the reported curvatures, their significance is uncertain due to possible baseline shifts in the strain data. Nonetheless, the magnitude of the curvatures calculated at the location between the girders in the conventional deck (see Figure 51) are dramatically smaller than those of the other curvatures ($2.5 \mu\kappa/\text{mm}$ compared with an average value of $19 \mu\kappa/\text{mm}$ at other locations). One explanation for this apparent anomaly is that the deck has experienced some physical damage at this location, possibly cracking, that has interrupted the continuity of the structure. This deck will be inspected to determine if it has sustained any physical damage consistent with this departure in its curvature response. It may be possible that changes in curvature of this kind can be developed as an indicator of the occurrence of physical distress in the decks.

Close examination of Figures 51 and 54 reveals that the extreme deformation response occurs slightly ahead of the extreme temperatures in time. One possible explanation for this behavior is that the strain compatibility and strain boundary conditions are driven primarily by in-plane conditions in the longitudinal and transverse directions; while the thermal compatibility and thermal boundary conditions are primarily in the out-of-plane direction. Thus, the local bridge deformations are influenced by what is happening over a much larger area of the bridge, compared to the local absolute

temperatures experienced in the decks. The significance of one occurring before the other is uncertain.

The discussion above considered the total deformations experienced by the deck as a function of temperature; the part of this deformation specifically associated with internal stresses in the decks is also of interest in this investigation. Referring to Figure 57, the stress curvature is inversely proportional to the temperature gradient, as expected. The axial in-plane stress strain, however, is directly proportional to the uniform temperature. That is, as the temperature increases, the in-plane stress strain also increases, indicating that the deck is in tension in the transverse direction. This result is counter intuitive, in that the girders are expected to offer at least nominal resistance to the expansion of the deck under an increase in temperature, placing the deck in compression. This situation may result from the manner in which the stress related contribution to the total strain has been calculated. The results of this calculation are sensitive to the estimated magnitude of the shrinkage strains, and additional research is needed on this matter before the stress strains are accepted as reliable.

The stress strain range between the extreme low and high values is approximately 25 $\mu\epsilon$, which corresponds to about 1.5 $\mu\epsilon/^\circ\text{C}$ for the measured temperature range. This difference suggests, as did the bridge CTE measurements, that the CTE values obtained from the laboratory investigation are lower than the actual in-situ CTE values (by at least 1.5 $\mu\epsilon/^\circ\text{C}$). This phenomenon is consistent between the decks and among the locations within each deck (Figure 58).

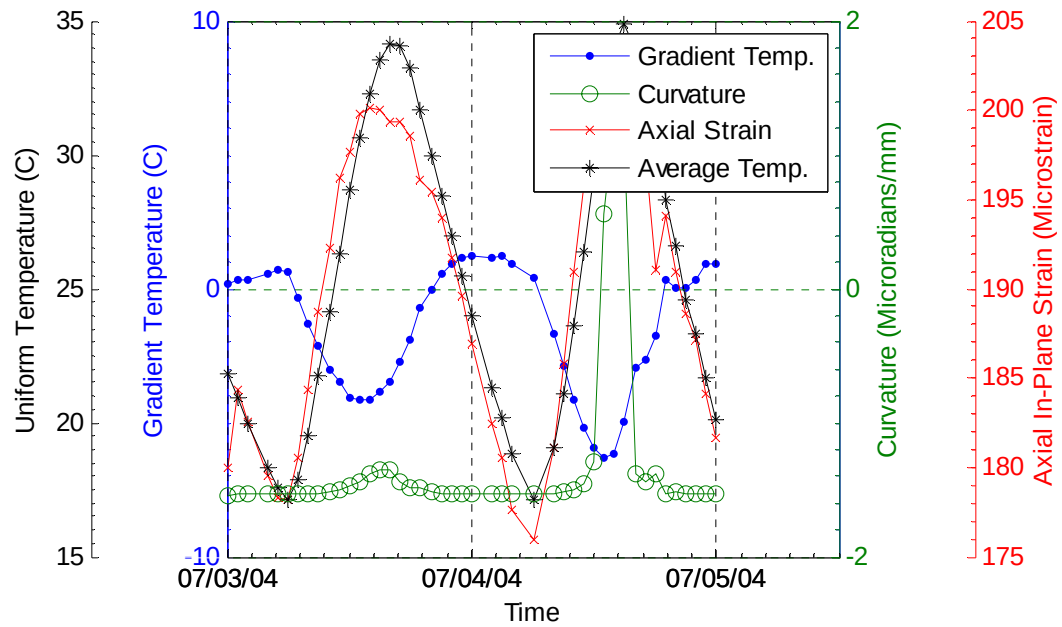


Figure 57. Conventional Deck Behavior at Location TV-D-3 (Stress Related Response)

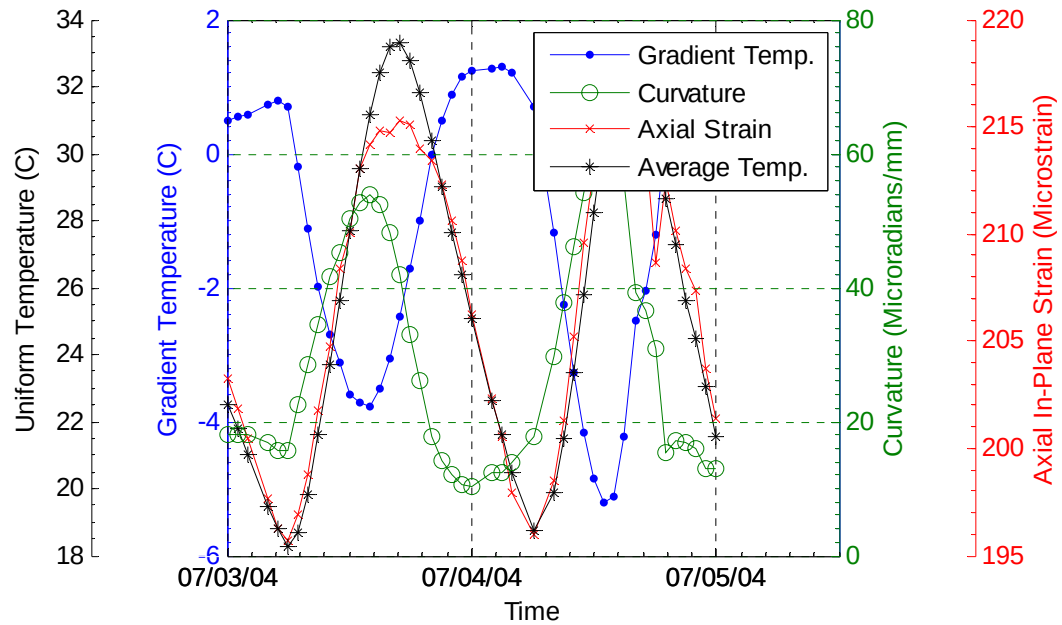


Figure 58. Conventional Deck Behavior at Location TV-D-5 (Stress Related Response)

Longitudinal Thermal Behavior

The behavior of the bridge decks in the longitudinal direction is complicated by their composite behavior with the prestressed girders and by the complex boundary conditions longitudinally. Relative to boundary conditions, the decks are continuous in the longitudinal direction, while the girders terminate at each support. Note also that the deck, the girders, and the abutment were cast integrally at the north and south ends of each structure. Despite these complexities, the general thermal response of the decks in the longitudinal direction is similar to that in the transverse direction. Typical deformations responses are shown in Figures 59 through 62 as a function of time over the same two-day period used above in the transverse response analysis.

Once again, the axial in-plane strains vary directly as a function of the absolute temperature (e.g., an increase in temperature correlates with an absolute expansion of the deck), while the curvatures vary directly as a function of the temperature gradient (e.g., a negative temperature gradient corresponding to the top surface being warmer than the bottom surface, corresponds to a negative curvature). The boundary conditions mentioned above may have the effect of moving the peak axial in-plane strain and curvature responses relative to the peak temperature effects that are associated in some manner with them..

The longitudinal total strain response at location F 3 (see Figure 62) is of particular interest. The strain values recorded are approximately three times those measured at other longitudinal and transverse locations. This location is known to have a transverse

crack, as explained and supported by the CTE values in the previous section, thus explaining the higher strain values.

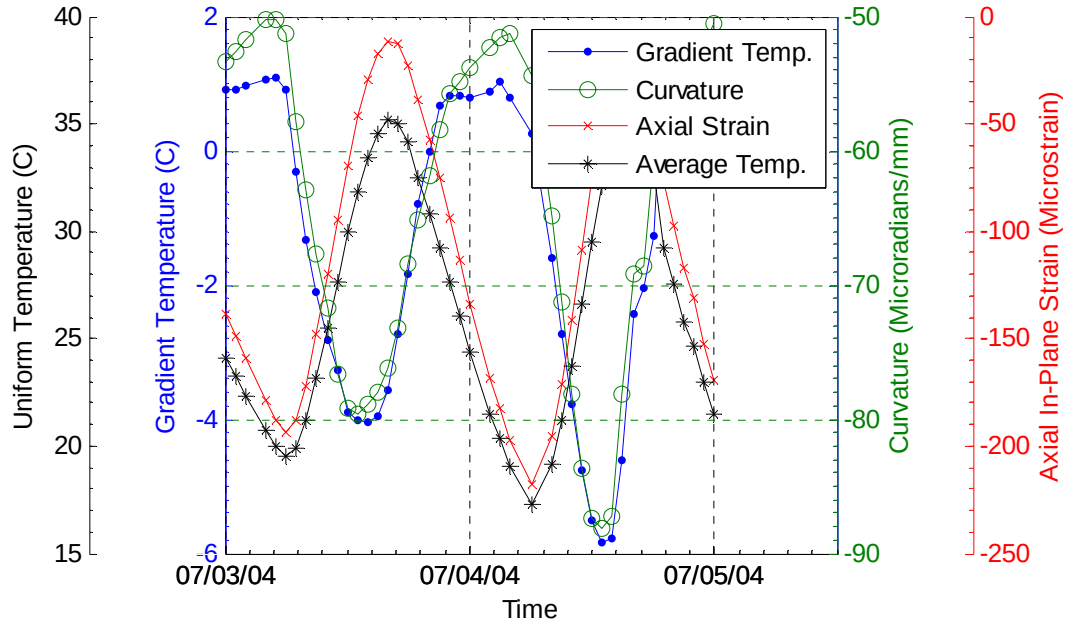


Figure 59. Conventional Deck Behavior at Location LV-A-3 (Total Response)

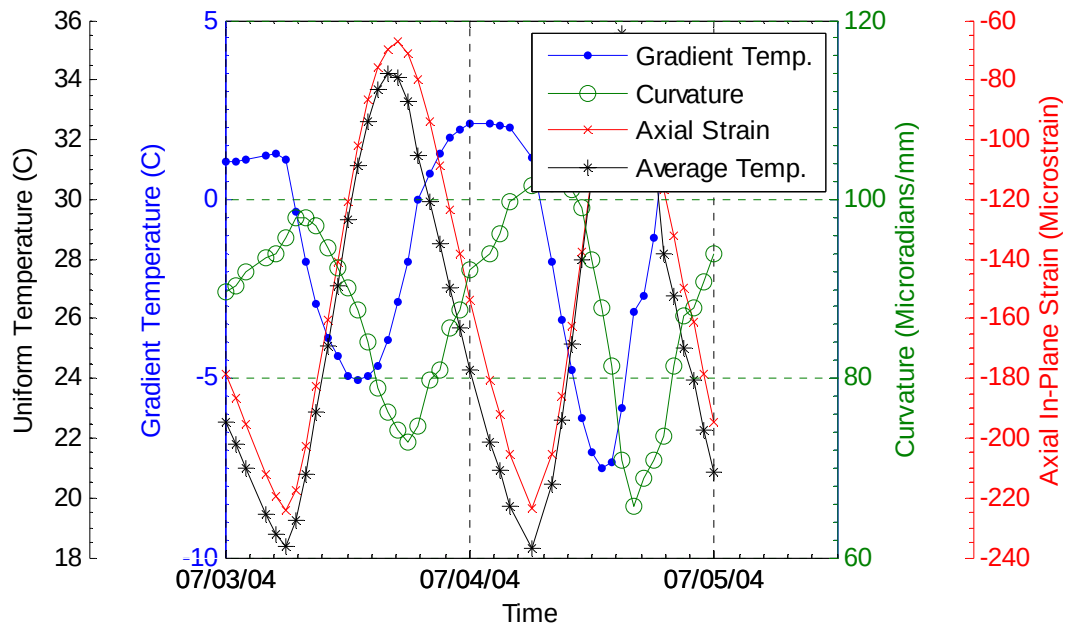


Figure 60. Conventional Deck Behavior at Location LV-B-3 (Total Response)

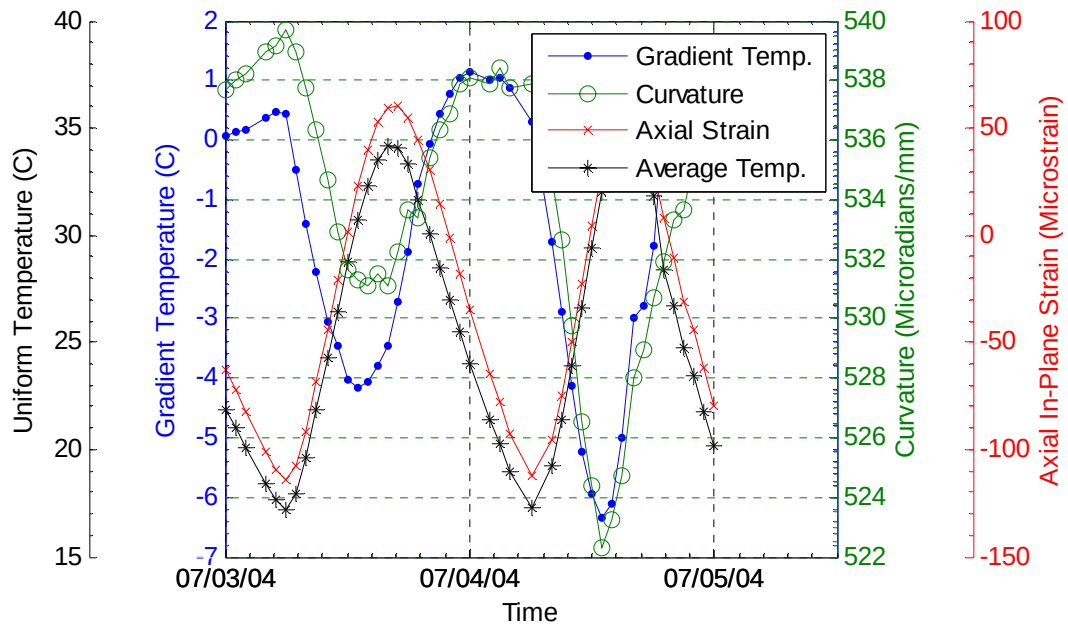


Figure 61. Conventional Deck Behavior at Location LV-D-3 (Total Response)

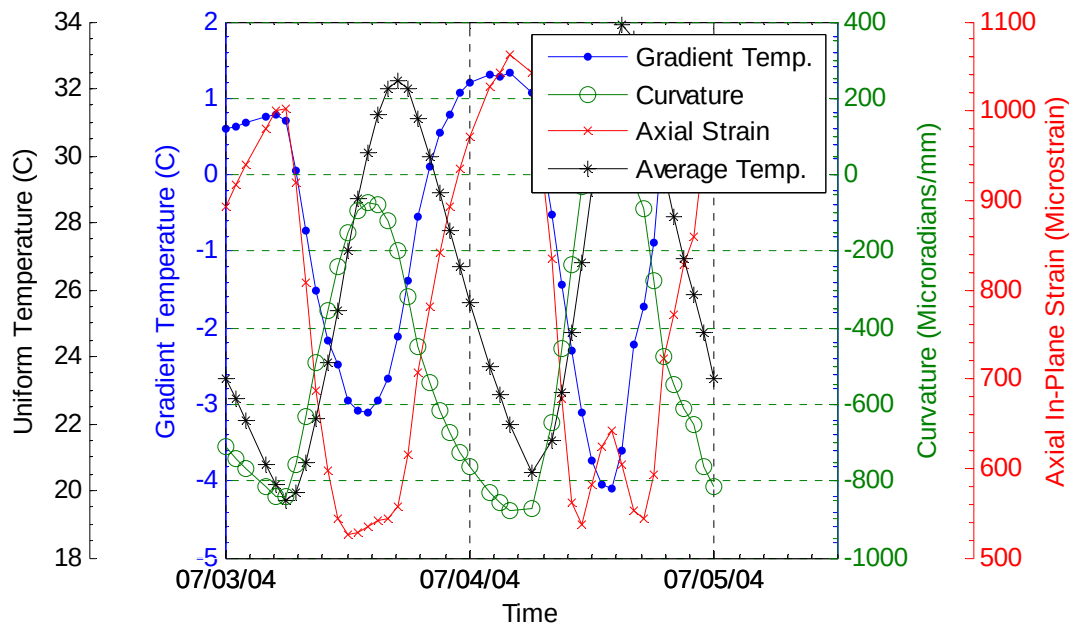


Figure 62. Conventional Deck Behavior at Location LV-F-3 (Total Response)

Temperature Loading

The temperature measurements made within the bridge decks during the previous two years offer a comparison with design values given by AASHTO. Typical temperature gradients measured through the depth of the cross-section (with a negative gradient representing increasing temperature from the bottom of the deck towards the top) and average temperatures of the cross-section calculated at the centroid are shown in Figure 63 for approximately a year and one-half period. The extreme gradient temperatures are approximately 2°C and -6°C . These gradients are much less than the approximate 20°C temperature gradient proposed by AASHTO (Figure 1). However, AASHTO's recommendation of multiplying their recommended temperature gradient by -0.3 for the inverse temperature gradient is supported by the measured temperatures. This inverse gradient, with greater temperatures at the bottom surface of the bridge deck relative to the top surface, seems to occur regularly throughout the year. The gradient temperatures are largest during the summer months, probably in response to larger amounts of solar radiation received at the bridge site during the summer months relative to the winter months.

The measured minimum and maximum average temperatures of approximately -38°C and 40°C , respectively, exceed the proposed AASHTO recommendations of -18°C to 27°C (Table 2). AASHTO's recommended values do encompass the majority of the temperature measurements, excluding rare extreme events. Thus, AASHTO's values may represent only the most probable and persistent temperatures for design stresses, and

they may rely on inherent safety factors to keep stresses within reason for extreme thermal events.

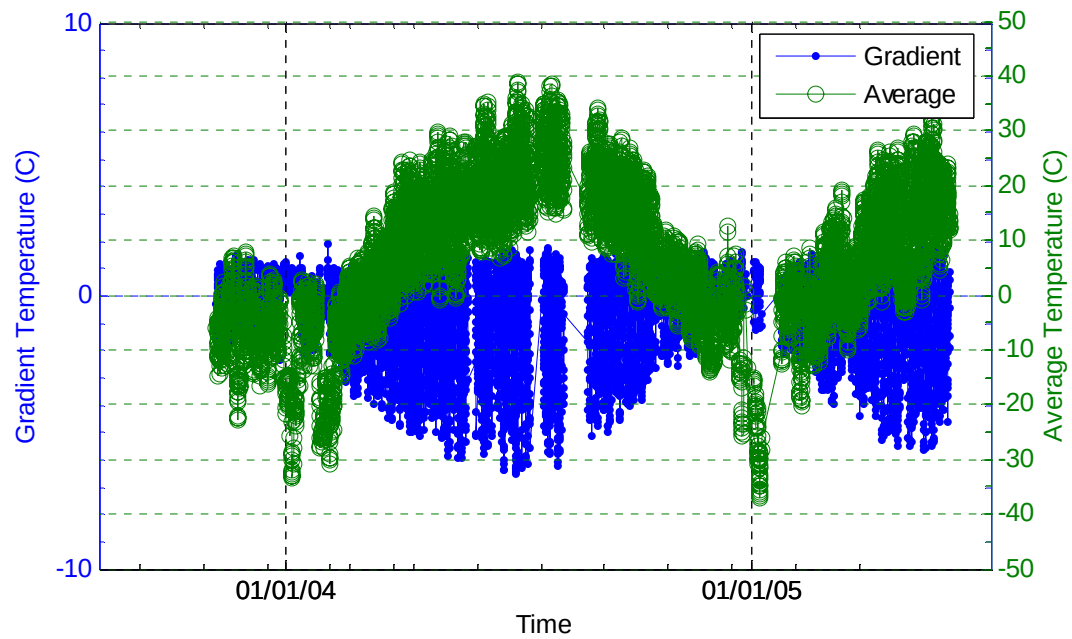


Figure 63. Conventional Deck Temperature Measurements at Location TV-D-3

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

The objective of this project was to investigate the response of three bridge decks made with different concretes and reinforcing steel configurations under environmental (notably thermal) loads. The investigation began with a laboratory study of the thermal behavior of the concretes used in the decks, followed by an analysis of the measured deck behavior in the field. Two different concretes were considered for the decks, a standard bridge deck concrete typically used by MDT, and a high performance concrete. Two reinforcing arrangements were used in the decks, a conventional design based on a traditional transverse load transfer model and a more lightly reinforced design based on empirical observations of bridge deck performance.

CTE Investigation

The laboratory investigation showed significant variations in the CTEs of concrete as a function of temperature. In general, as temperature decreases, CTE values also decrease. This variation is believed to be related to behavior of the water in the concrete's nanostructure. This finding contradicts the traditional assumption that CTE values for concrete are constant and that their values are similar to those of steel. Data suggest that this assumption is only valid for temperatures approaching 25°C. At lower temperatures, concrete's CTE is up to 30 percent less than the reported CTE of steel.

The fact that the CTE value of concrete is less than that of steel does offer some benefits to concrete performance. Notably, at lower temperatures, the higher thermal shrinkage of the steel tends to compress the concrete, reducing its tendency to crack.

While generally the CTEs of traditional concrete and those of the high performance concrete are similar, some distinct differences in the behavior of high performance concrete exist. Notably, the high performance concrete has much lower CTE values at approximately 0°C and below. These differences in CTE behavior are related to the nanostructure (notably pore size of the high performance concrete), which affects the behavior of the water within the cement paste.

Bridge Investigation

Analysis of the strain data collected in the field found that the CTE values for the bridges were similar to those determined in the laboratory investigation. The difference between the laboratory and in-situ CTE values are explained in part by the composite action of the steel reinforcement within the deck, which increases the CTE values compared to the plain concrete. A second factor that may contribute to this difference is that the relative humidities experienced by the specimens tested in the laboratory (approximately 20 percent) were considerably lower than the average relative humidity recorded in Saco (approximately 60 percent). This difference would have a noticeable effect on the concrete CTE values because they are dependent on the presence of physical water in the cement paste structure.

At one of the strain gage locations analyzed, the in-situ bridge deck CTE values were dramatically different than the laboratory values and values at other locations in the deck. This unusual response may correlate with cracking in the deck, particularly since it was at a location that is expected to experience the greatest tensile strains under vehicular loads, mid-span between girders. Note that the possibility that cracking had occurred was not as obvious in the raw strain data, which “looks” to be acceptable and within reason. The absence/presence of cracking will be investigated in the field in the near future. If indeed the deck is cracked, the occurrence of unusual CTE behavior may be useful in determining damage in a structure for infrastructure health monitoring.

In addition to the CTE values, the field strain data showed that the decks in the transverse direction were behaving as expected with regard to curvature and in-plane axial strain while undergoing thermal loads. Importantly, there were no significant differences among the conventional, empirical, and high performance bridge decks at this time. A slight anomaly in the curvature in the transverse direction may suggest damage adjacent to the strain gage location. Only a cursory examination was made of the deck response in the longitudinal direction. This examination indicated that longitudinal deformations are similar in all three decks, and at least generally consistent with expected behaviors.

Because of the relatively young age of the Saco bridges (two years), continued investigation should take place as the bridges mature; the most noticeable damage typically takes place in the range of 10 to 15 years.

Recommendations

Some recommendations for further research are:

- Collect more temperature and strain profile data to correlate the two with environmental weather.
- Evaluate laboratory specimens for CTE results under different temperature regimes and humidities, including the effects of solar radiation.
- Cast instrumented specimens located at the bridge site to record absolute shrinkage and temperature characteristics of the concrete.

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