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# Improving the Long-Term Performance of Bridge Decks through Full-Scale Accelerated Testing

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16. Abstract
This report looks at the stresses experienced by a bridge over its early life in an effort to better understand the progression of deck cracking and deterioration.

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## Introduction

It is desirable to limit tension stresses in concrete as it has very low tension strength and cracks can develop when even small tension stresses are experienced. Cracks in concrete decks provide pathways for chlorides to penetrate to the reinforcing leading to accelerated corrosion of the reinforcing steel. Therefore, it is important to understand the state of stress in a concrete deck when evaluating its performance and vulnerability to cracking.

The state of stress in a given region of a reinforced-concrete bridge deck changes over the life of the structure. The majority of tension stresses develop and cracks form due to volumetric changes in the early stages of the deck's life. These volumetric changes can be mainly attributed to shrinkage (autogenous and drying) and creep. Volumetric changes may also be induced by temperature changes. These changes result in stresses when there is restraint of the material, or when temperature differentials induce deformations in the structure.

This report examines the strains experienced during the life of a concrete deck by considering a 50-foot-long bridge that is being subjected to accelerated testing in Rutgers' BEAST Laboratory. Gauges were embedded in the deck of this full scale specimen, as well as attached to the four steel girders. The collected data includes the response of the structure to dead load and construction activities, live load, and environmental changes.

## Overview of the BEAST and Specimen

The BEAST laboratory was designed to accelerate the deterioration of a bridge by subjecting it to environmental and traffic loads. This is accomplished by enclosing the entire specimen within an environmental chamber. Within this chamber the bridge may be cooled or heated with the HVAC systems, inducing freeze-thaw cycles on a daily basis. Brine is applied to the deck periodically with an automatic sprayer system. Traffic loading is simulated with the loading carriage shown in Figure 1. This carriage is able to impart 60 kips of force into the bridge while moving back and forth over the bridge at 10 to 15 mph. This allows the live-load carriage to pass over the bridge approximately 15,000 times each day.

Figure 1: BEAST Live Load Carriage

The current specimen was designed according to the 7th Edition of the AASHTO LRFD Bridge Design Specifications (with 2015 revisions) and was constructed in 2019. It consists of a single 50-ft simply-supported span. It is composed of 4 rolled-steel girders spaced at 7.5 feet. An 8-inch-thick reinforced-concrete deck was cast in place. Stay-in-place forms were installed between beams 1, 2 and 3. Removable forms were used between beams 3 and 4. Diaphragm bracing is provided between the girders with channel sections.

The specimen is outfitted with a structural health monitoring system. This system consists of over 200 sensors including embedded strain gages (40), surface mounted strain gages (40), temperature sensors (80), hydrometers (10), displacement sensors (14), and accelerometers (12). These sensors provide an indication of structural behavior and characteristics.

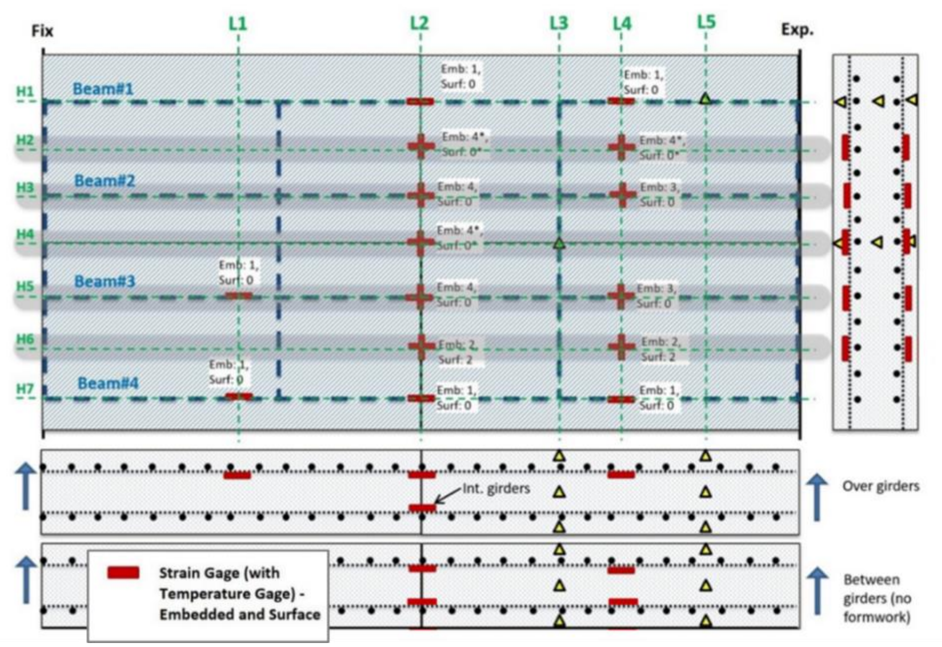


Figure 2: Embedded Sensor Layout in Deck

Displacement gauges were installed on the bottom of the girders to capture the vertical displacement of the bridge. Strain gauges were installed on the bottom flanges and the webs of girders. Strain gauges were also embedded within the deck. These gauges allowed the research team to capture the strain and temperature within the concrete deck as early as the deck pour through to operation. The diagram in Figure 2 depicts the distribution of sensors in the bridge deck.

The vibrating-wire strain gages measure strain by measuring the frequency of vibration of a known length of wire. Changes in temperature cause expansion or contraction of the material to which the gage is attached. The temperature change will also cause expansion or contraction of the vibrating wire. When the base material has the same coefficient of thermal expansion as the vibrating wire, the elongation of the wire will equal the elongation of the base material, and no change in strain would be detected. However, when the base material is different (e.g. concrete), the reported strain values must be corrected. Therefore, the concrete strain values reported in this report have been temperature corrected and all strain values represent stress-inducing strain and not strains arising from unrestrained thermal elongation/contraction which do not induce stresses in the material.

The specimen, testing, and all of the data presented herein, was made possible by the US DOT Federal Highway Administration, Office of Infrastructure Research and Development through the project entitled: Quantifying Long-Term Bridge Performance Through Full-Scale Accelerated Testing. The data produced from this project is publicly available at FHWA's InfoBridge (https://infobridge.fhwa.dot.gov/).

## Early-age: Deck Pour and Cure

The reaction of cement with water is an exothermic reaction releasing thermal energy, commonly referred to as the heat of hydration. For a poured-in-place concrete deck, the concrete is introduced to the structure having already begun the process of hydration. At the time of the deck pour, the concrete has no stiffness and cannot generate stress. As the concrete begins to set, the heat of hydration continues to drive up the temperature. The concrete reaches its maximum temperature after the concrete has achieved a final set. As the hydration reactions wane, the temperature declines and returns to ambient levels. The hydration process will continue for months or even years with minimal strength gain.

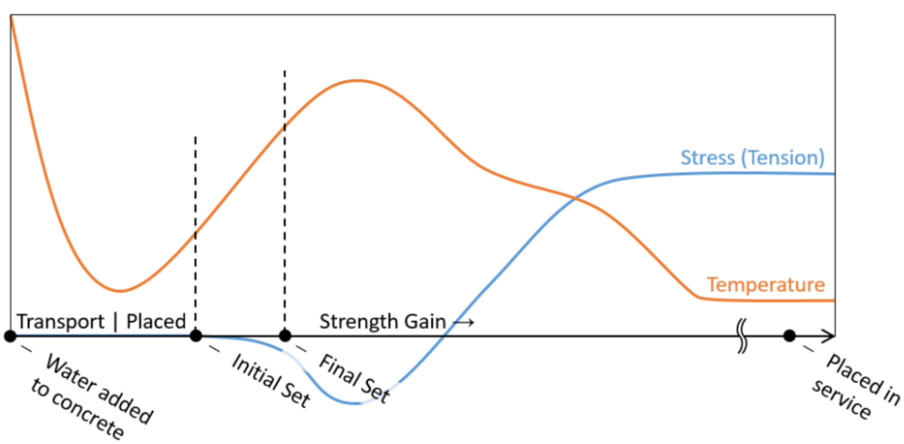


**Figure 3: Heat of Hydration During Concrete Setting**

The high temperatures during the hydration process may induce mechanical stresses due to thermal gradients (especially in larger pours) as well as influence the rate and nature of the chemical reactions that can result in expansion cracks. Therefore, to successfully track the state of concrete stress through construction, it is important to interpret temperature and strain data within the context of this hydration timeline.

The hardness of the deck concrete on the BEAST specimen during the hydration process is not known, and thus some assumptions must be made. The first is that the concrete has zero strength up until it sets, and that this occurs completely at a chosen "final set" point, which is commonly selected as the moment when the concrete reaches a maximum temperature.

Figure 4 portrays the temperature during and after the deck pour as recorded by several embedded gauges over beam 3. As can be seen from the plot, it takes several days for the heat of hydration to dissipate.

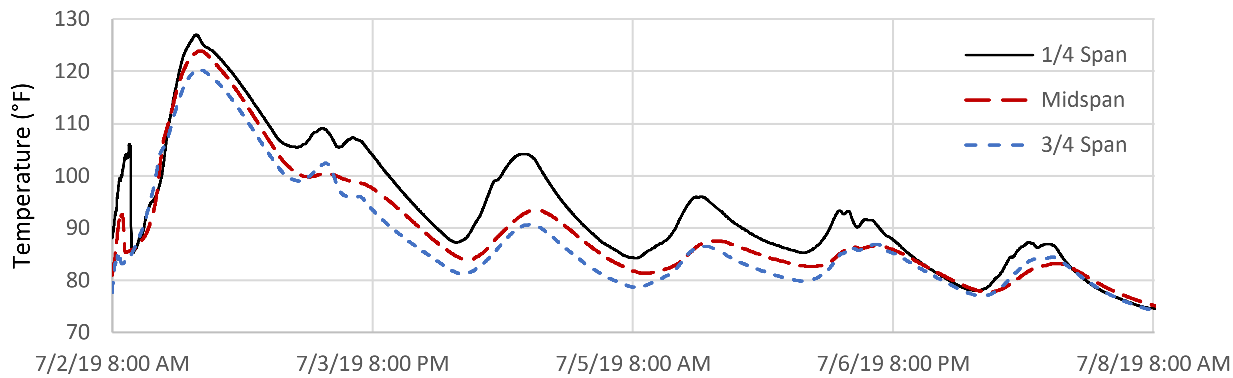


Figure 4: Internal Deck Temperature after the Pour

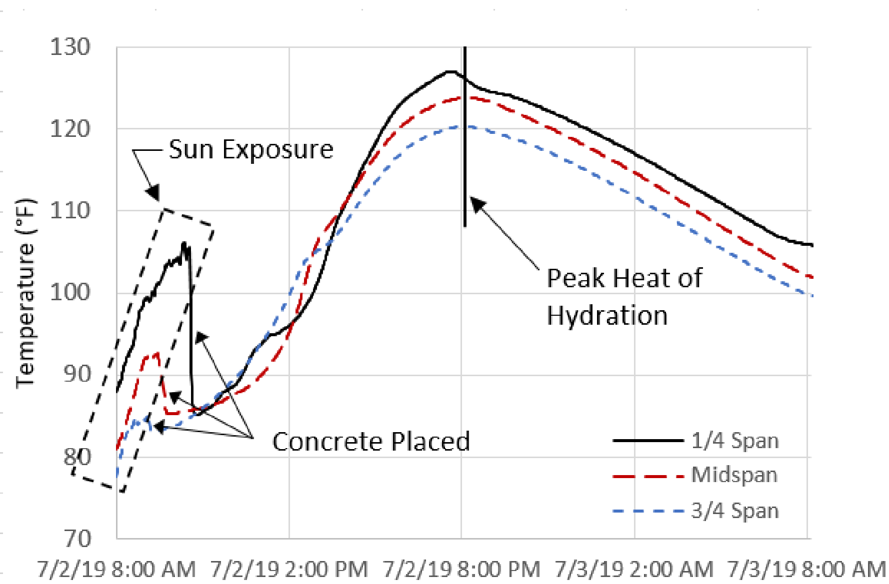


Figure 5: Internal Deck Temperature the Day of the Pour

Figure 5 illustrates the concrete temperature timeline on the day of the deck pour. The morning of the deck pour, the girders and reinforcement and the gages, which are tied to rebar, were heated by morning sun. When the wet concrete was poured, it was cooler than the gauges and caused an immediate drop in temperature. As the hydration process continued, the temperature of the concrete increased until it reached a maximum approximately 10 hours after placement. Figure 6 shows the deck strain when zeroed at the peak heat of hydration.

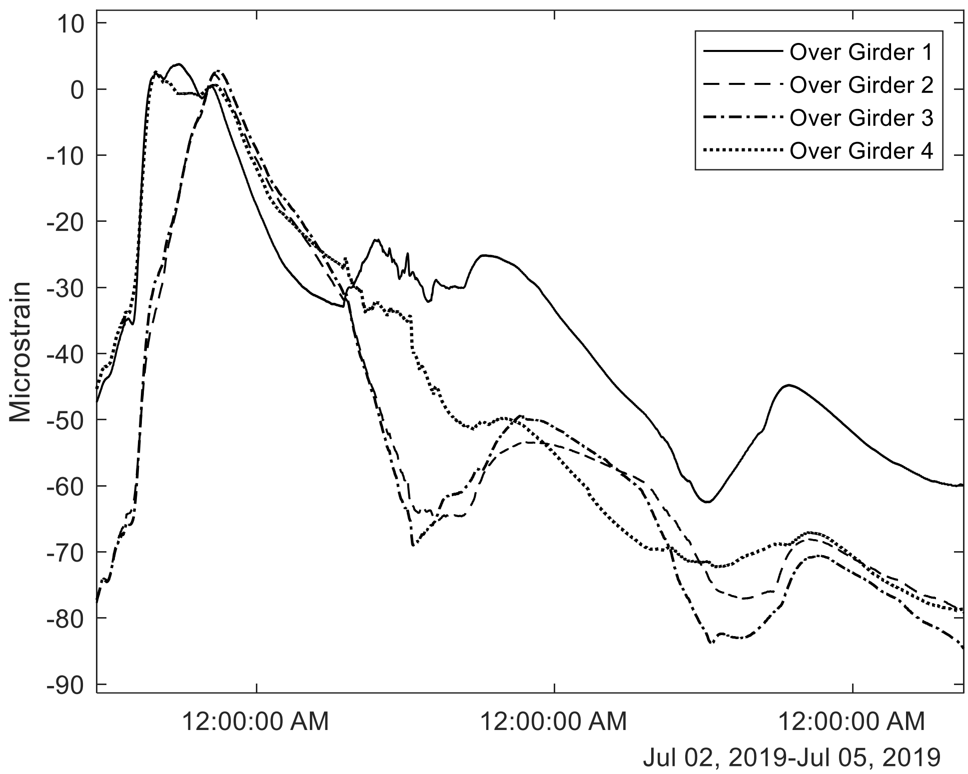


**Figure 6: Strain at Top Longitudinal Reinforcement at Midspan**

As the concrete temperature increased, it heated the reinforcing causing thermal elongation. The concrete also heated the girders by conduction at their interface. This caused the upper flanges to elongate more than the bottom flanges, and resulted in negative curvature (positive camber) and thus upward deflection. As the heat of hydration dissipated and the deck cooled, the deck appears to go into compression (in the longitudinal direction).

When considering total strain in the concrete deck, the unstressed state of the gauges must be assumed. Strain readings are subsequently zeroed by subtracting the unstressed values from the raw values. Therefore, at the “zero” point, it is assumed that the deck has no stress, but any changes in the state of the structure, (e.g. changes in loading or geometry) will induce stress within the concrete. Due to the nature of wet concrete and the thermal effects of the hydration reaction, this “zero” point can be impossible to exactly identify. Even though the approximate time of concrete set may be deduced from the strain and temperature data, there is no single moment that the deck attains stiffness, and the stress/strain relationship (elastic modulus) will evolve as the concrete gains strength. Therefore, when estimating the total strain experienced by the concrete, the effect of the “zero” point assumption on results must be considered.

One method of assessing the early stiffness of concrete is to compare nearby gauges that are embedded in the concrete. When their readings begin to behave similarly, the concrete has reached a point of stiffness such that the concrete is bonded to the gage. In our case, the gage is attached to reinforcing, thus correlation between gages indicates that the concrete has bonded with the reinforcing. The relative behavior of adjacent gages can be assessed by looking at the curvature between the gauges, or, in other words, the difference between gage readings divided by the distance between the gages. For a rigid cross section, we expect a linear strain profile between gauges and for the girder and deck to assume the same strain profile. Figure 7 provides the curvature between deck gauges as well as the curvature between girder gauges for midspan of beam 3.

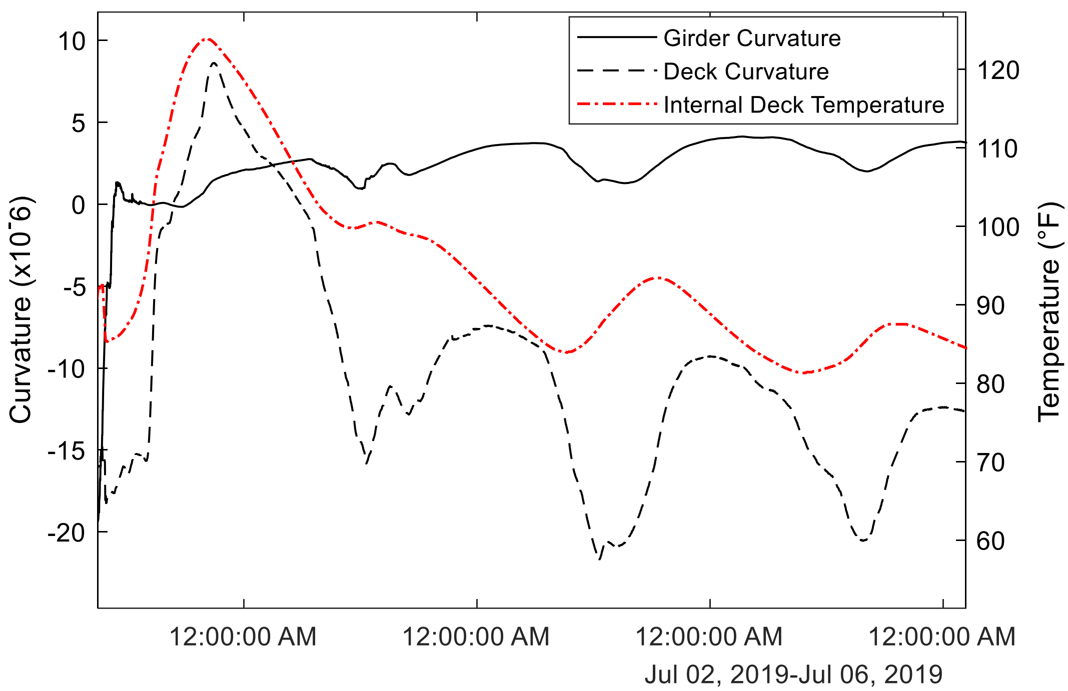


Figure 7: Time History of Temperatures and Curvatures During Deck Pour

As can be seen in Figure 7, the girder experiences a jump in curvature that may be attributed to the addition of deck dead load at the time of pour. After the placement, the concrete temperature continues to rise, and as the top flanges heat up, curvature in the girders slowly decline (as positive camber is thermally induced). At approximately 8:00 PM, the maximum heat of hydration is reached, and we can expect the concrete to be gaining stiffness. At approximately 7:00 AM the next morning, the deck and girder curvature begin to correlate as seen in Figures 7 and 8, indicating the composite cross section is deforming with a linear strain profile and therefore behaving compositely.

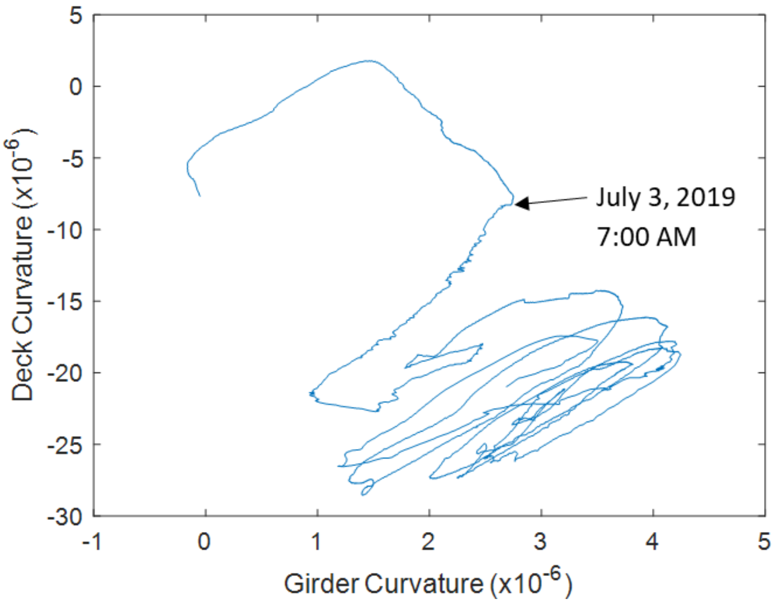


Figure 8: Correlation Plot of Girder and Deck Curvature

The strain in the top of the deck over the three-month cure time is provided in Figure 9. Strain values have been zeroed at the peak heat of hydration. This assumption results in a lower bound estimate of strain values (conservative when considering tension).

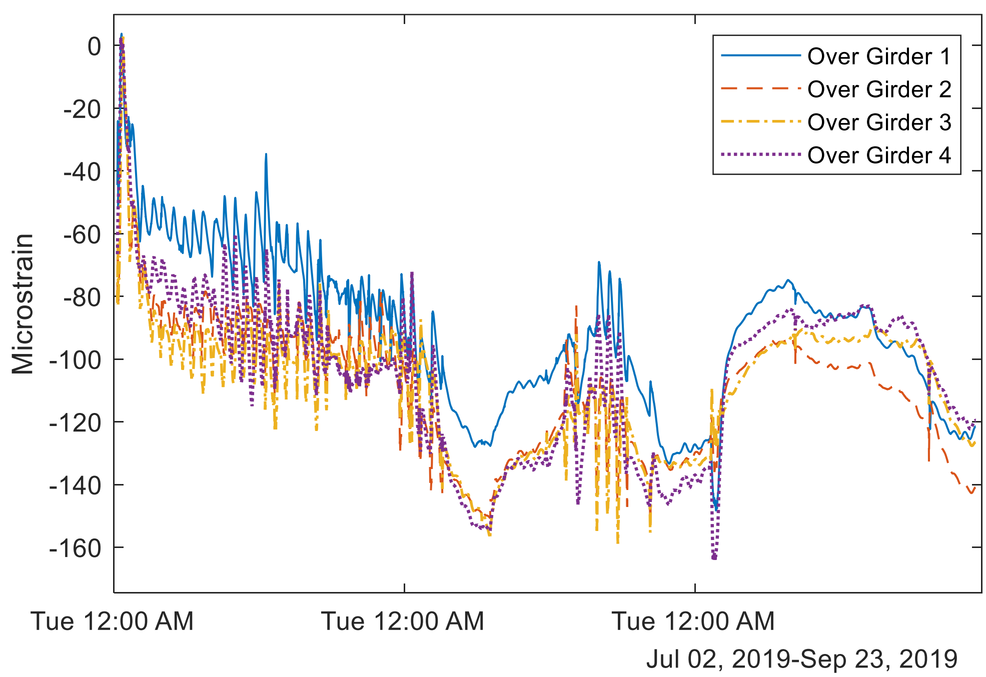


Figure 9: Longitudinal Strain at Top Reinforcement at Midspan

The daily maximum compression strain (negative strain) values are summarized in Figures 10 for longitudinal gages located at midspan.

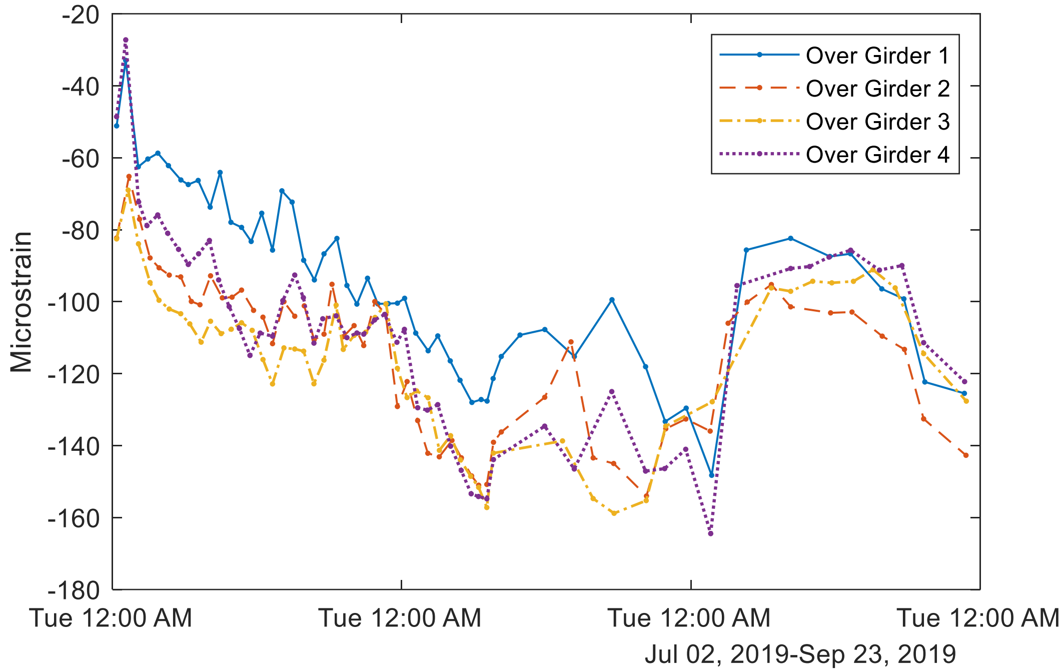


Figure 10: Maximum Daily Longitudinal Compression Strain During Deck Cure.

A similar trend is observed in the transverse deck strains. Figure 11 provides the maximum transverse compression strains at the top of the deck at midspan over the duration of the cure.

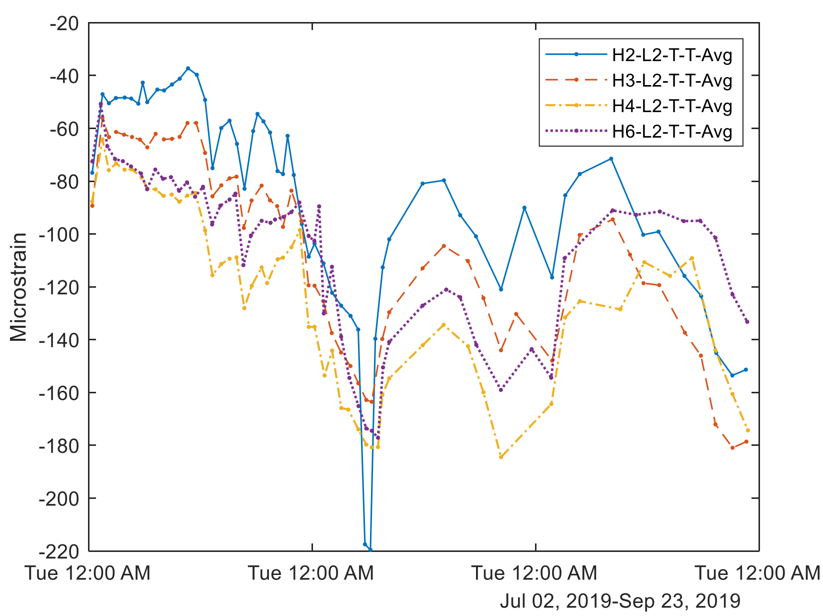


Figure 11: Maximum Daily Transverse Tension Strain During Deck Cure

From the deck strain time histories, it is evident that the strain levels fluctuate greatly. During this time, the only appreciable load applied to the specimen was from the accumulation of standing water from rain. The majority of this fluctuation can therefore be attributed to temperature induced effects.

As the superstructure cools, the contraction of the girders, along with the loss of the thermal uplift, induces compression in the deck. Furthermore, drying shrinkage causes a reduction in concrete volume, which is restrained by the reinforcement (and girders), thereby inducing tension in the surrounding concrete, and compression in the reinforcement. Therefore, the general increase in compression strain (more negative) over the cure time as measured by the reinforcement gages may be attributed to volumetric changes due to temperature changes and drying shrinkage.

Samples of the concrete used in the deck were gathered at the time of the pour for testing. These included 10 inch prisms that were cast for measuring drying shrinkage. The prisms were stored at 50 percent relative humidity (±4%) and 73 degrees Fahrenheit (±3%). The majority of this shrinkage occurred in the first 30 days. The specimens experienced an average of 0.07% change in length (700 $\mu\varepsilon$ compression). However, in the actual bridge deck, the concrete was restrained from shrinking by the rebar (as well as girders). Furthermore, the deck cured under different conditions than the specimens (e.g., covered with soaked burlap during the cure). Therefore, the magnitude of actual shrinkage strain experienced by this deck is unknown.

In addition, any net compression in the deck after the pour may result in creep of the concrete and a reduction of net compression. The creep therefore results in an increase in compression strain while the stress remains constant or decreases. This creep occurs in tandem with shrinkage, and thus gage readings report the combination of rebar compression due to concrete shrinkage and the increase in compression strain (in both concrete and rebar) due to creep. However, these terms are not easily separated, and since they have opposite effect on the concrete stress, the gage readings during this cure period cannot reliably indicate the stress state of the concrete deck.

In Figure 12, the girder curvature, computed from the bottom flange and web gage readings, is used to extrapolate the maximum compressive strain at the top layer of reinforcement. The extrapolated values represent the expected strain in the top reinforcement if the cross section has a linear strain profile (i.e.; plane sections remain plane). While this assumption is false, the values still serve as an estimate of the strain induced by global deformation.

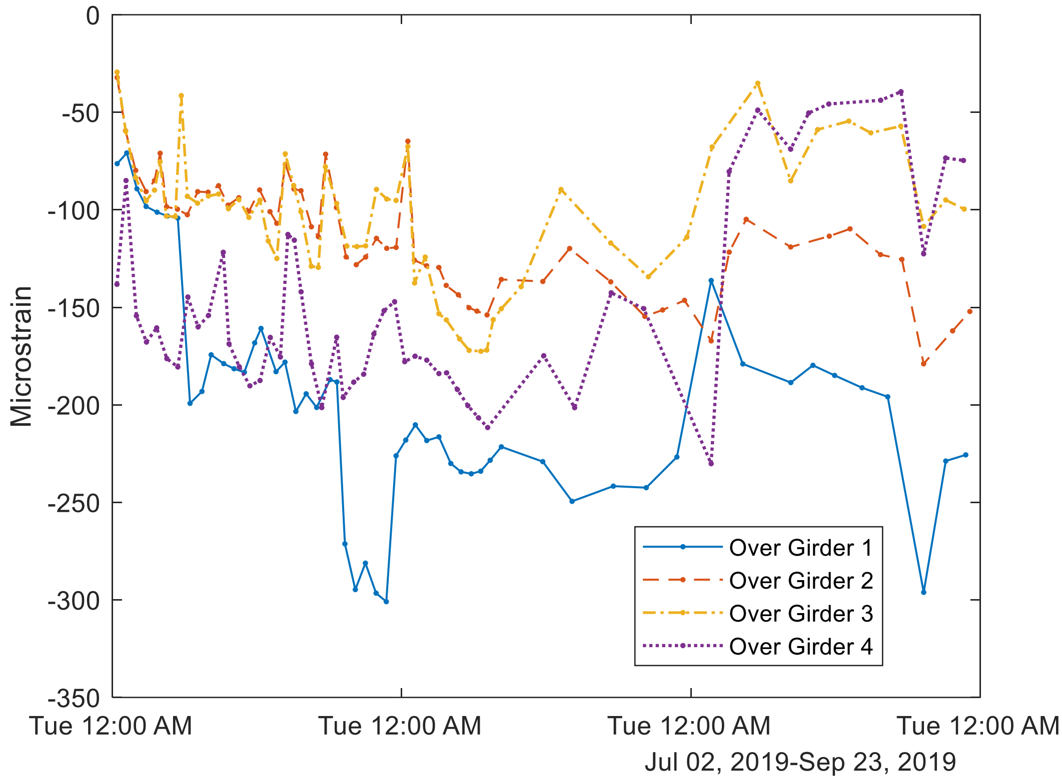


Figure 12: Top Reinforcing Extrapolated Strain at Midspan

A comparison of Figure 10 and 12 reveals that much of the strain measured by the embedded gages may be attributed to global deformation of the structure. In fact, the extrapolated readings exceed the observed strain values, which is due to the discrepancy between the assumed linear strain profile and the actual strain profile.

In summary, this bridge experienced a number of loads at the early stages in the life of the deck. The concrete deck in particular experienced strain from creep, shrinkage, thermal elongation and contraction, and global deformations due to thermal gradients. Based on the measured data, and our assumptions of when the concrete has set, it is postulated that global deformations due to temperature during and after the concrete pour left the concrete in a state of compression. This compression was gradually reduced by creep and shrinkage. However, the magnitude of creep and shrinkage strain cannot be deduced from the data. However, it is expected, that given the stability of the girder curvature over the cure time, the deck remained in compression, carrying some of the dead load.

# Operational Demands

During accelerated testing the specimen is subjected to live loads with the moving carriage and environmental loads through changes in ambient temperature. The effect of these loads are first examined separately.

The live load carriage applies approximately 60 thousand pounds of force to the deck. The position of the carriage on the deck has been varied over the duration of accelerated testing. Figures 13 and 14 summarize several of the load positions.

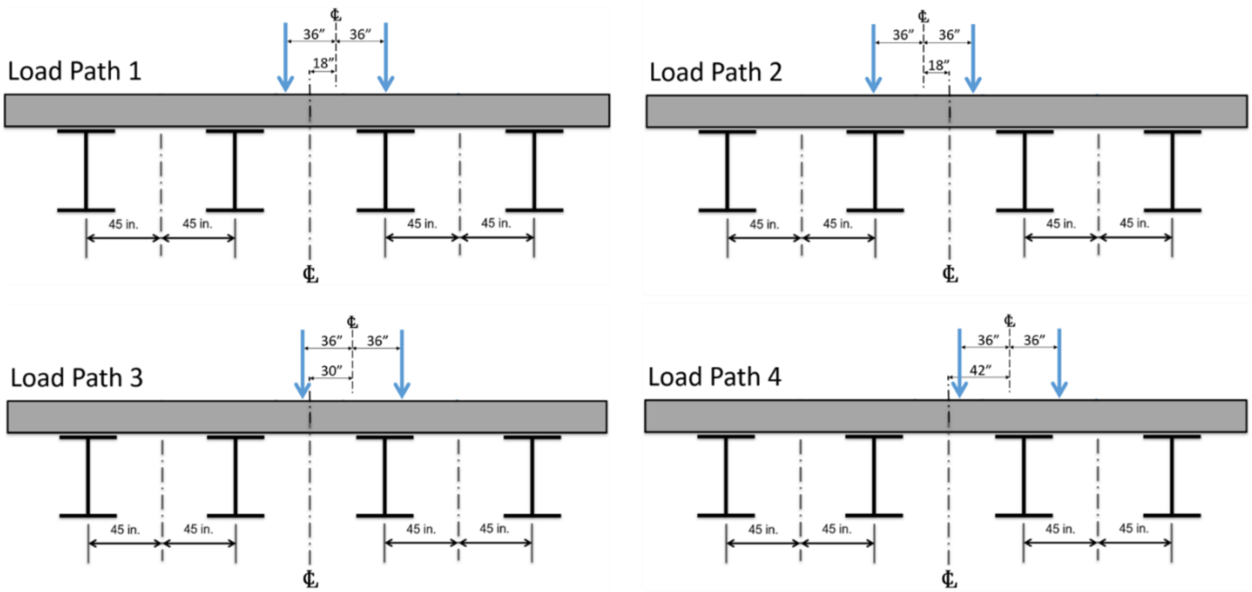


Figure 13: Load Paths 1 through 4

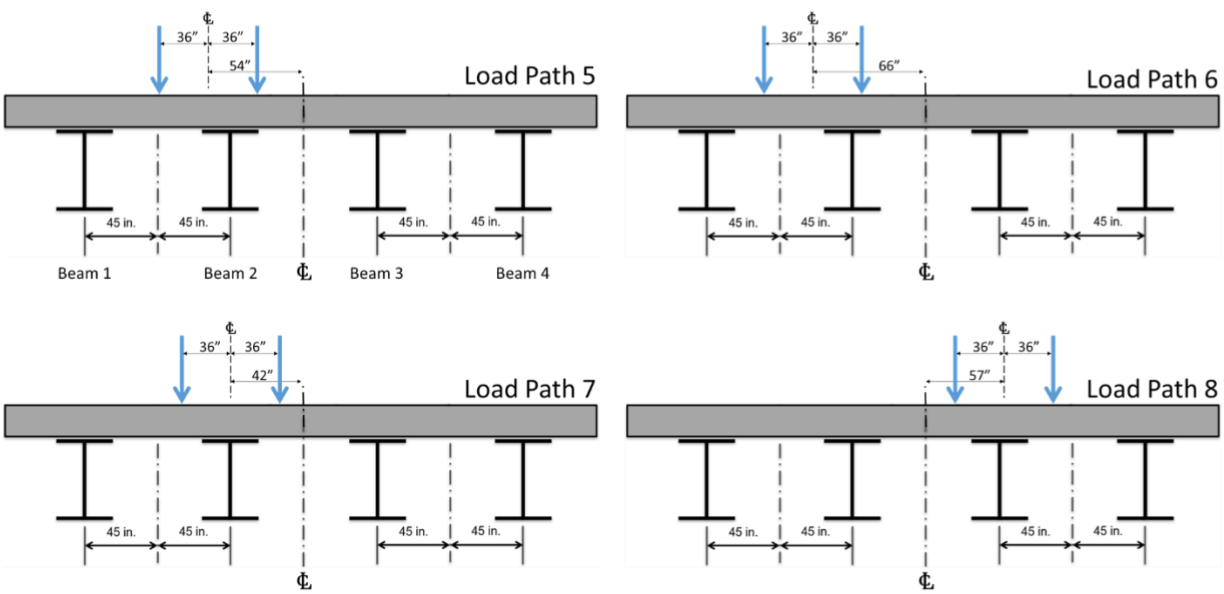


Figure 14: Load Paths 5 through 8

The peak longitudinal strain values in the deck under various loading configurations are provided in Figure 15.

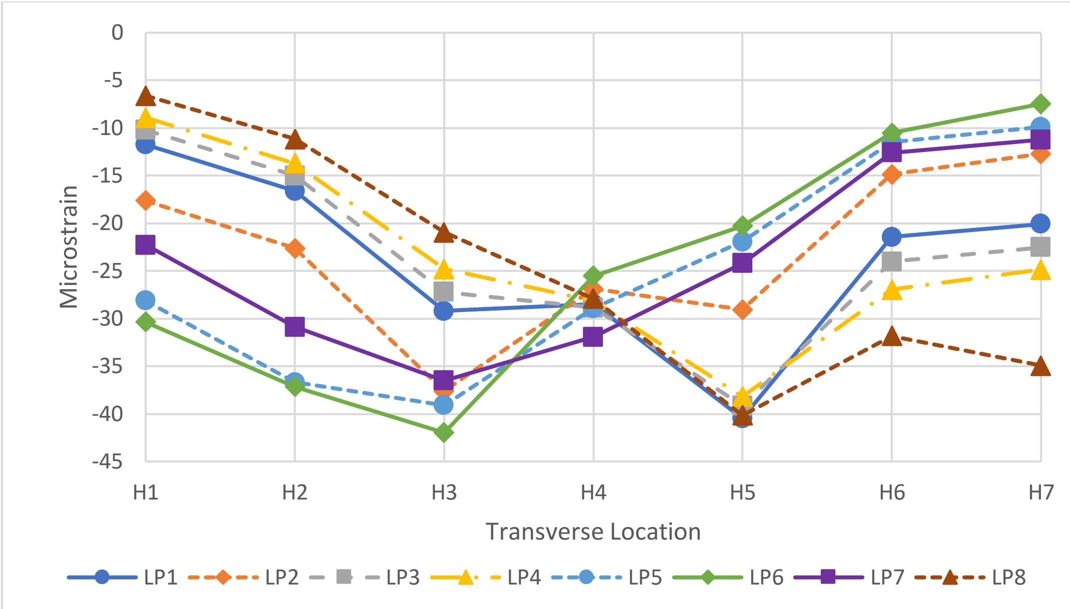


**Figure 15: Peak Longitudinal Strain at Top Reinforcement at Midspan**

From Figure 15 we may observe that the live load results in compression strains in the top of the deck in the longitudinal direction. However, the strain in the transverse direction can be positive (tension) depending on the position of the load as observed in Figure 16.

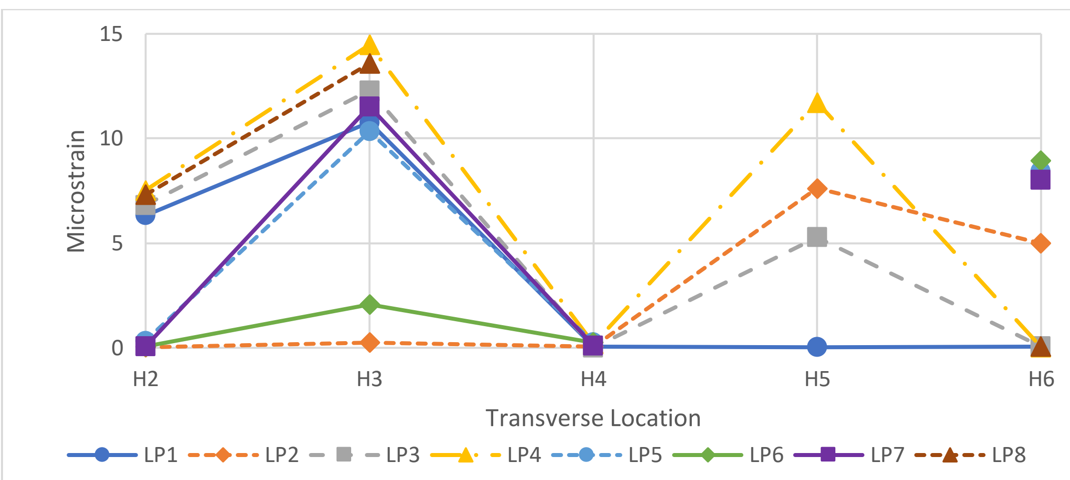


**Figure 16: Maximum Transverse Strain at Top Reinforcing at Midspan**

This is due to the shape of deck deformation over the girders (illustrated in Figure 17). With the load positioned on either side of the girder, the deck is forced to deform similar to a continuous beam with reverse curvature over the girders. However, the longitudinal flexibility of the girders permits downward deflection under load, and therefore reduces the curvature demanded of the deck. Even though the deflection of the girders reduces the deck demands, tension strains are still developed. It should be noted that only the maximum strain values are provided in Figure 16 and the live load may also result in compressive strains in the deck.

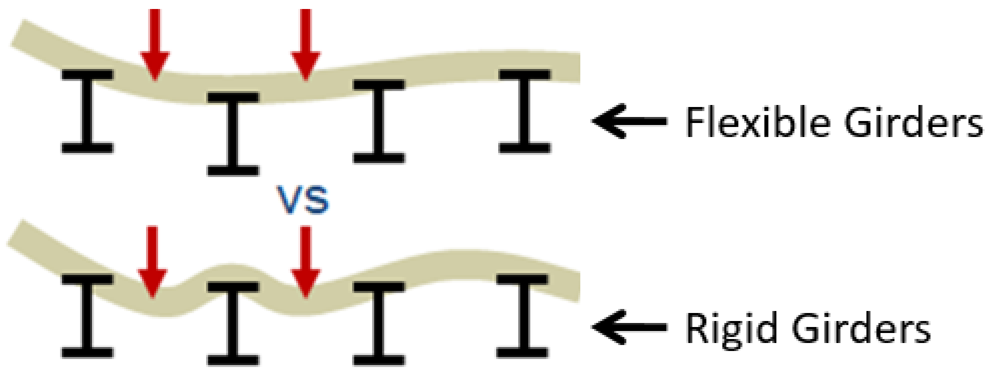


Figure 17: Deck Deformation over Girders

The bottom of the deck is similarly influenced by the transverse deformation of the deck over the girders. In this case, the curvature of the deck over the girders reduces the tension strains that are developed from the global downward deflection. Negligible compressive strains are produced by the live load.

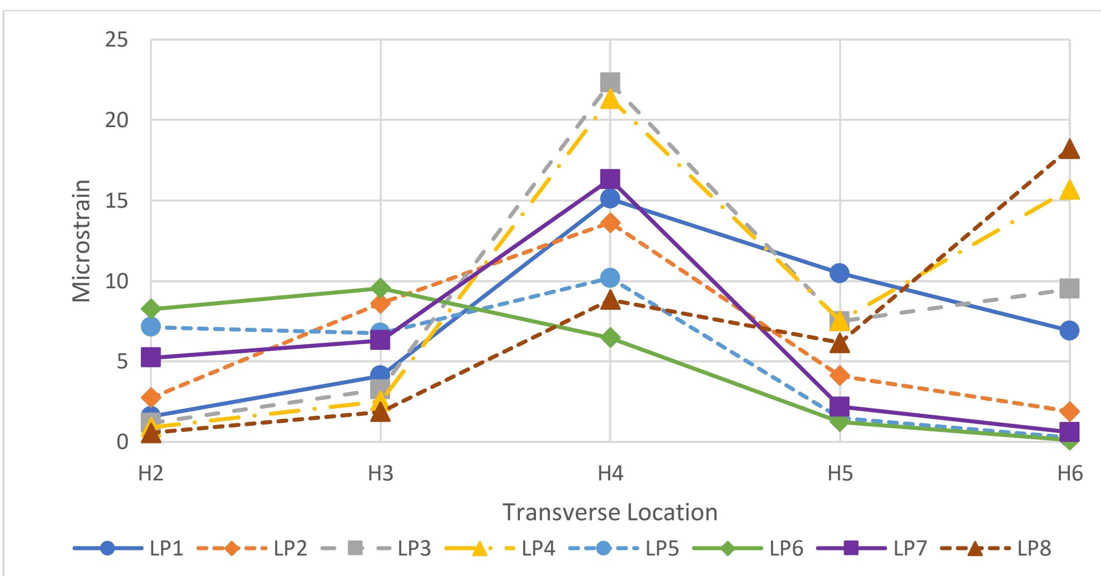


Figure 18: Maximum Transverse Strain at Bottom Reinforcing at Midspan

From Figures 15, 16, and 18, it can be seen that live load induces compressive deck strains in the longitudinal direction, and compressive or tension deck strains in the transverse direction depending on the location of interest and the position of the load.

However, this specimen is subjected to environmental loads in addition to the live loads. The specimen is heated and cooled to attain a freeze-thaw cycle every 24 hours. Figure 19 provides the longitudinal strain at the top of the deck over a period of several days and through multiple freeze-thaw cycles. For these gages, the strains due to unrestrained thermal expansion have been negated so only stress inducing (mechanical) strains are plotted. Furthermore, the values presented were zeroed near the beginning of the record since the relative change in strain is of interest and the state of zero stress is unknown.

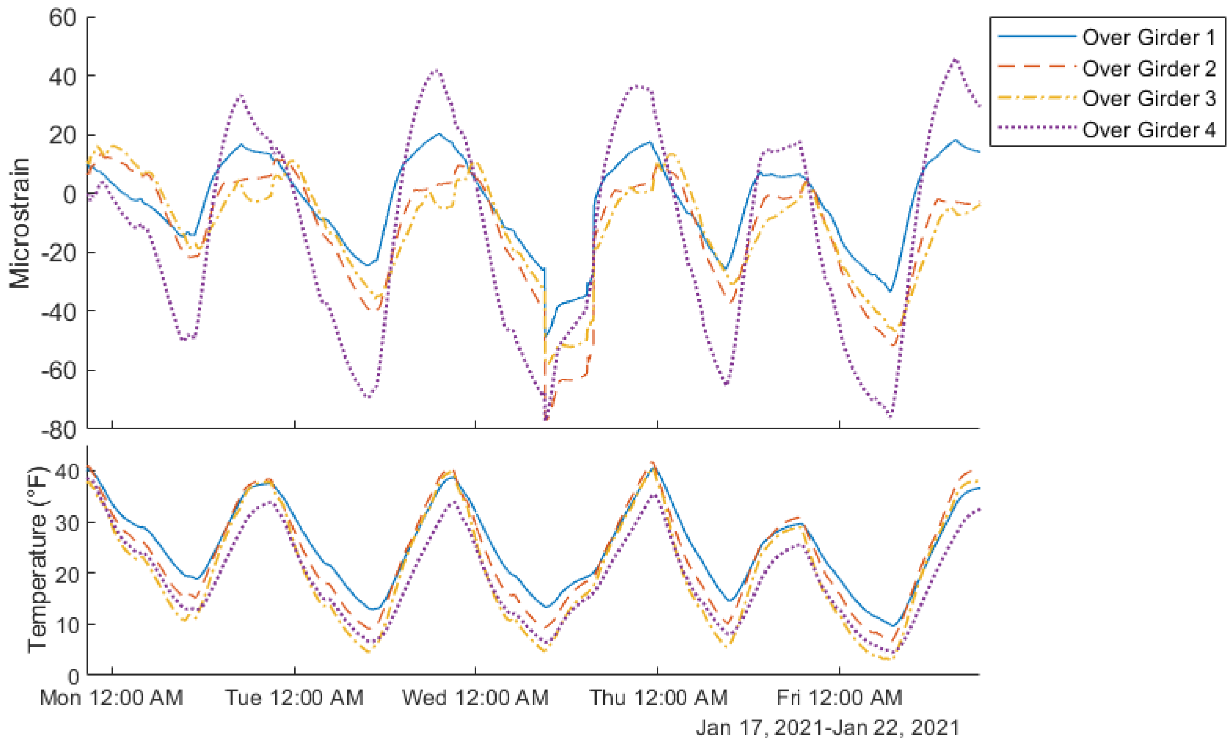


**Figure 19: Longitudinal Strain Response and Temperature Time History**

As can be seen, an increase in temperature generally results in an increase in strain. The deck over girder 4 experiences the largest change in strain, often experiencing a swing of more than 100 microstrain due to temperature effects alone. In comparison, the applied live load only resulted in strains of less than 50 microstrain (compression).

Similar trends are observed for transverse strains at the top and bottom of the deck at midspan. Warming of the bridge increased strains toward tension. A 30 to 40-degree temperature swing induced a change in strain of 60 to 80 microstrain.

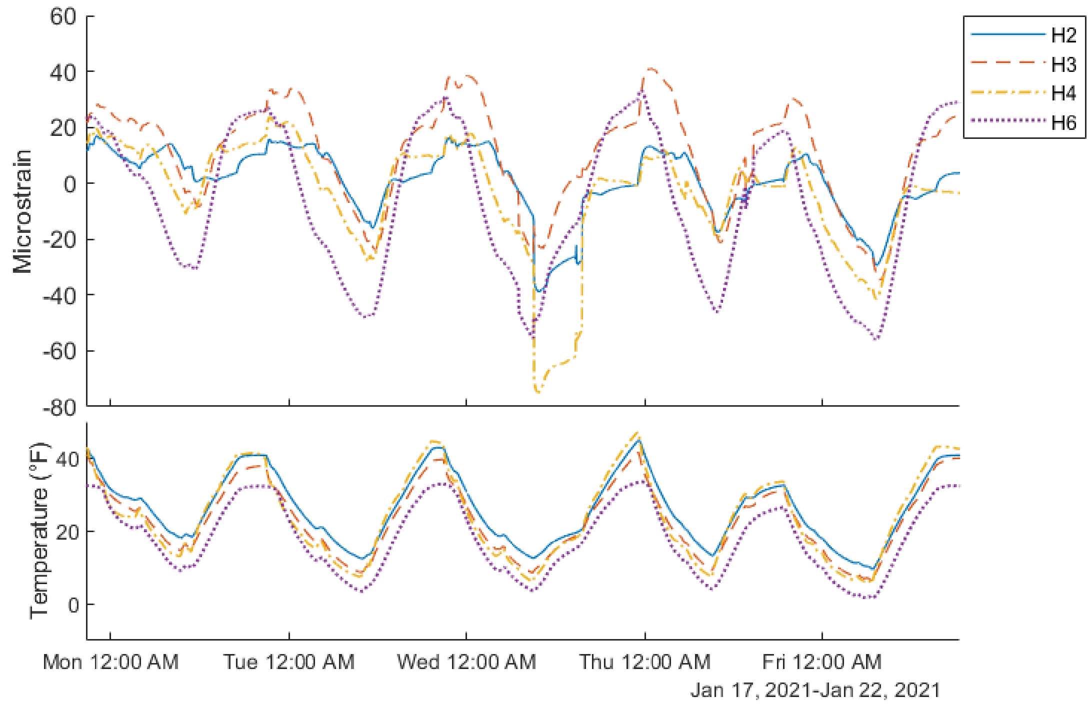


**Figure 20: Transverse Strains in the Top of the Deck at Midspan**

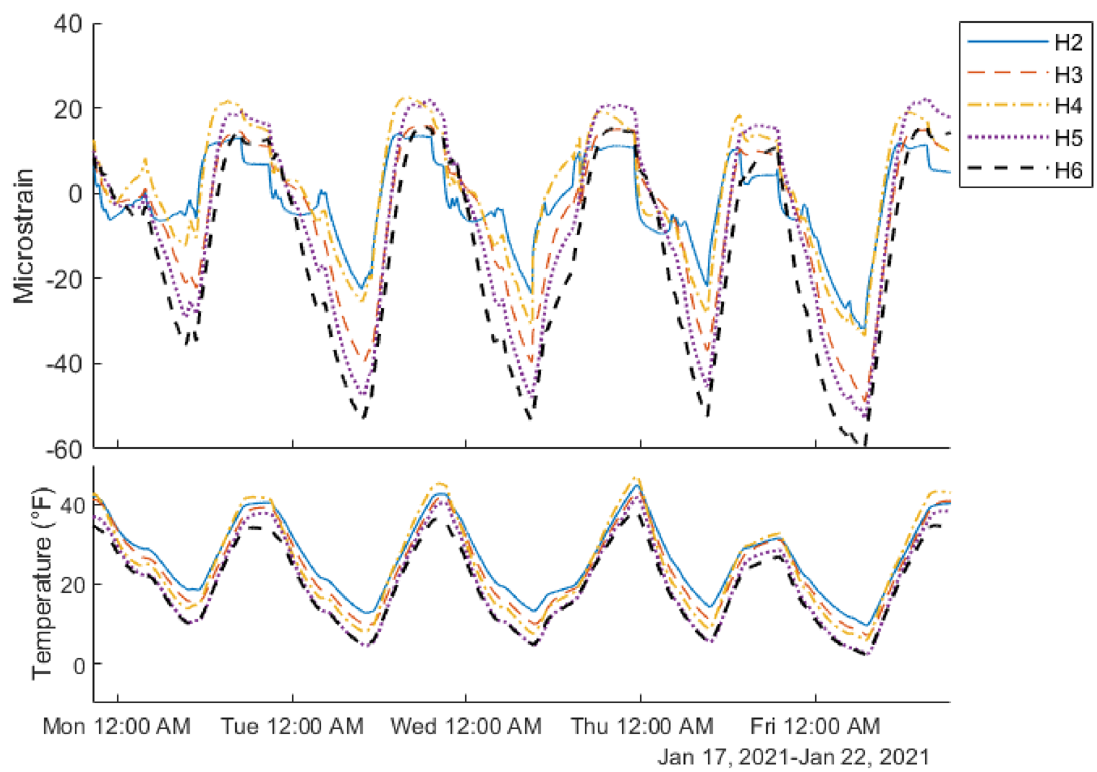


**Figure 21: Transverse Strains in the Bottom of the Deck at Midspan**

In conclusion, the operational demands induce compressive and tension stresses in the deck. Strains from temperature effects are generally much greater than those from live-load. Stresses from live-loads are generally compressive, except for transverse strains under some load cases. While the daily fluctuations in deck strain due to accelerated testing may not be at levels that cause material failure, their cumulative effect may still lead to cracking and deck deterioration.

## Long-term Strain Distribution

During the deck cure, concrete stresses are dominated by thermal effects and shrinkage effects. After the deck has cured, the drying shrinkage has largely ceased and the concrete stresses are due to thermal effects and gravity loads. Furthermore, environmental and live loads, applied as part of the accelerated testing plan, may cause deterioration or changes in load distribution. Figure 22 provides the strain of the top longitudinal reinforcement at midspan over each of the four girders over eighteen months. The daily maxima of stress inducing strains are reported (unrestrained thermal strains are negated). Since these gages experience an increase in compression under live load (negative strain), the maximum provides a fair estimate of the strains when the bridge is not loaded.

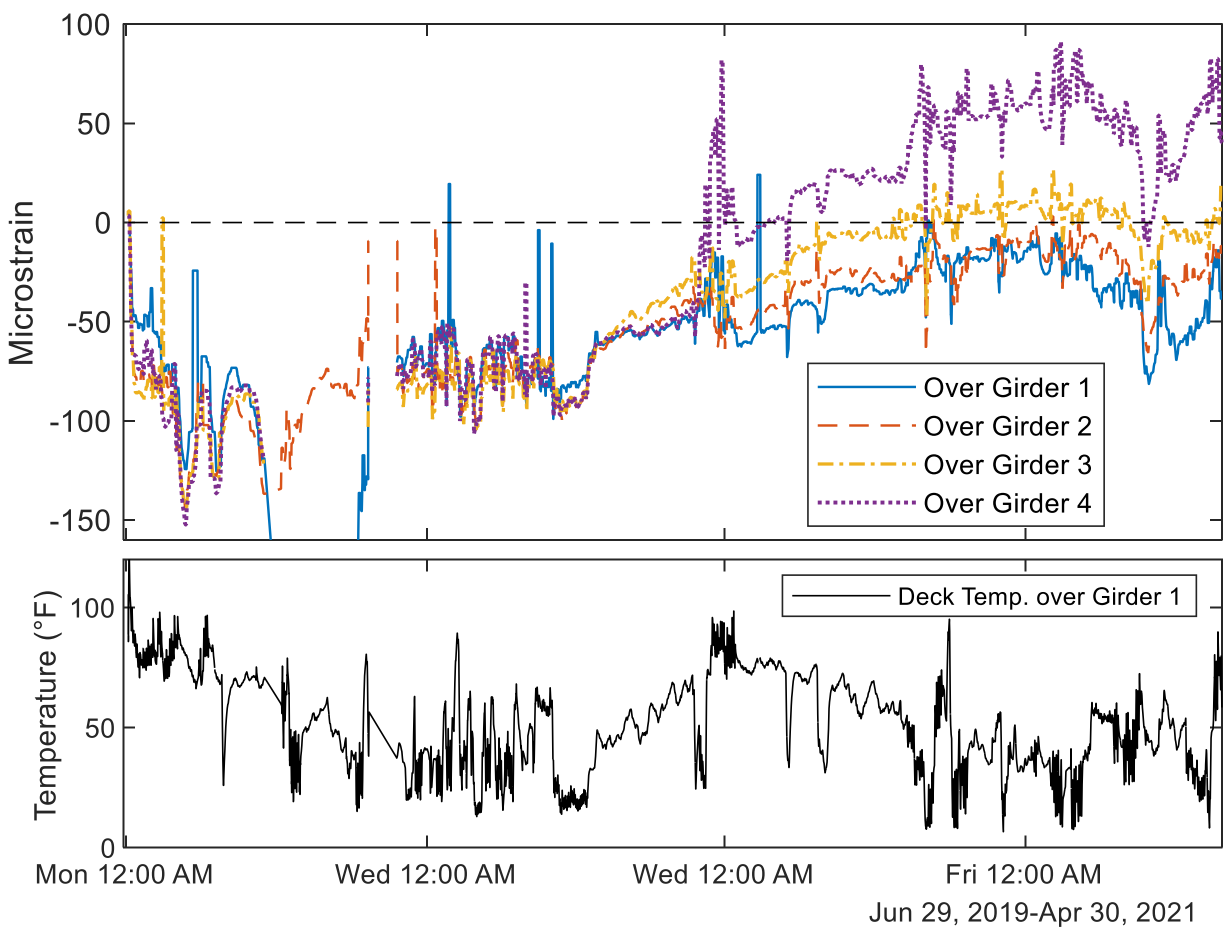


Figure 22: Maximum Longitudinal Strains of Top Reinforcement over 18 Months

The time history of the reinforcement strains indicate that the reinforcement experiences an initial decrease in strain (compression) that may be attributed to the manner in which the deck set and cured as discussed in the previous section. However, the strain gradually increases over nearly 18 months, as shown in Figure 22. While it may be tempting to attribute these changes to drift of the gages, since the longitudinal gages all display a similar trend, it is unlikely the observed phenomenon can be attributed to sensor deficiencies. It should be stressed that these gages are tied alongside reinforcement and therefore, report the elongation of the reinforcement rather than the concrete. The strain in the reinforcing should equal that in the adjacent concrete once the concrete sets and for strains induced by externally applied loads. However, once the concrete strain exceeds the cracking limit, the concrete will crack and strain right next to the crack would equal zero. Therefore, the reinforcement gage readings may not accurately represent the strain experienced by the concrete at a material level.

The gage over beam 4 (H7-L2-T-L) indicates that the reinforcement in this region experienced a large jump in tension strain compared to gages over other girders, nearly 1 year after the deck is placed (Figures 22 and 23). As time continues, the strains above beam 4 continue to increase. The exact reason for this change is difficult to determine. One possible explanation is that shrinkage had left the concrete in a state of tension near cracking limits and thermal expansion and contraction, along with live loading, caused a crack to form at the gauge. This would result in a higher local strain at the gauge.

However, even if the sudden jump in the readings of one gage may be attributed to a crack development, the long term trend of longitudinal reinforcement towards tension remains unexplained. The structural response around the time of this observed shift is explored further and plotted in Figures 23 and 24.

On June 15$^{th}$ at approximately 9:30 PM, a heating cycle was initiated that brought the deck from 29 degrees to 50 degrees Fahrenheit. This resulted in an increase in tension strain in the deck and a decrease in tension strain in the girders. Over the next three days, the specimen was subjected to four freeze-thaw cycles. During each heating phase, a consistent increase in strain is observed. However, after each cooling phase, the deck strains over beam 4 (H7) do not completely return to the previous baseline, instead gaining 8 to 10 microstrain (of tension). On June 19$^{th}$, over the course of approximately 16 hours, the deck experienced a temperature increase of more than 50 degrees, resulting in a significant increase in baseline microstrain. After these several days of freeze-thaw cycles, the deck over beam 4 had experienced an increase of approximately 30 microstrain in baseline readings.

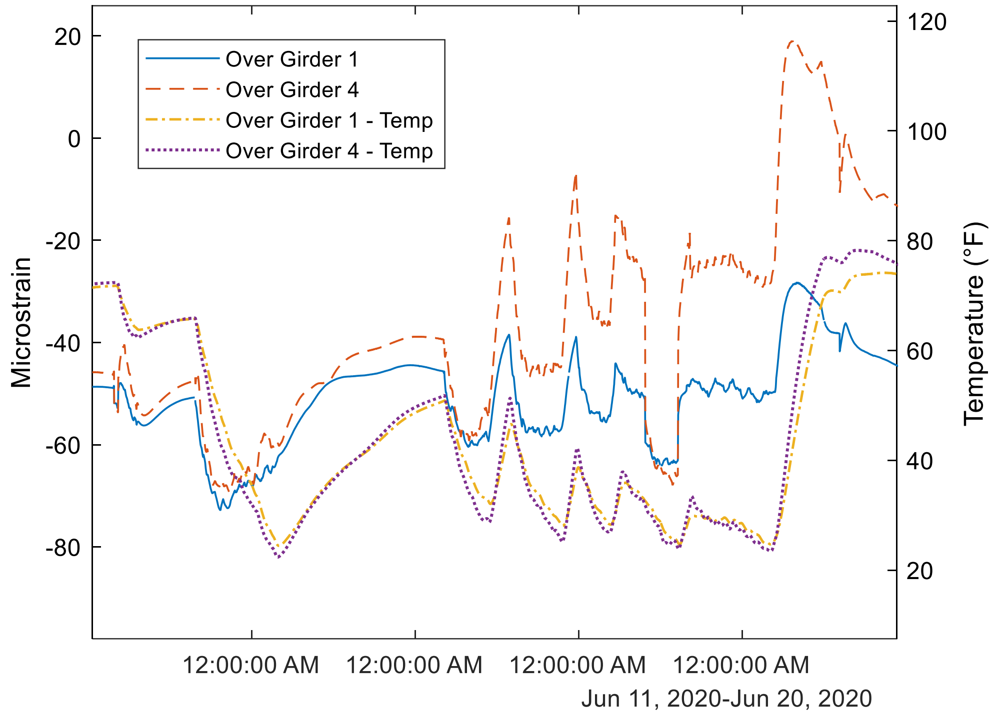


Figure 23: Longitudinal Deck Strain at Top Reinforcement over Girders 1 and 4

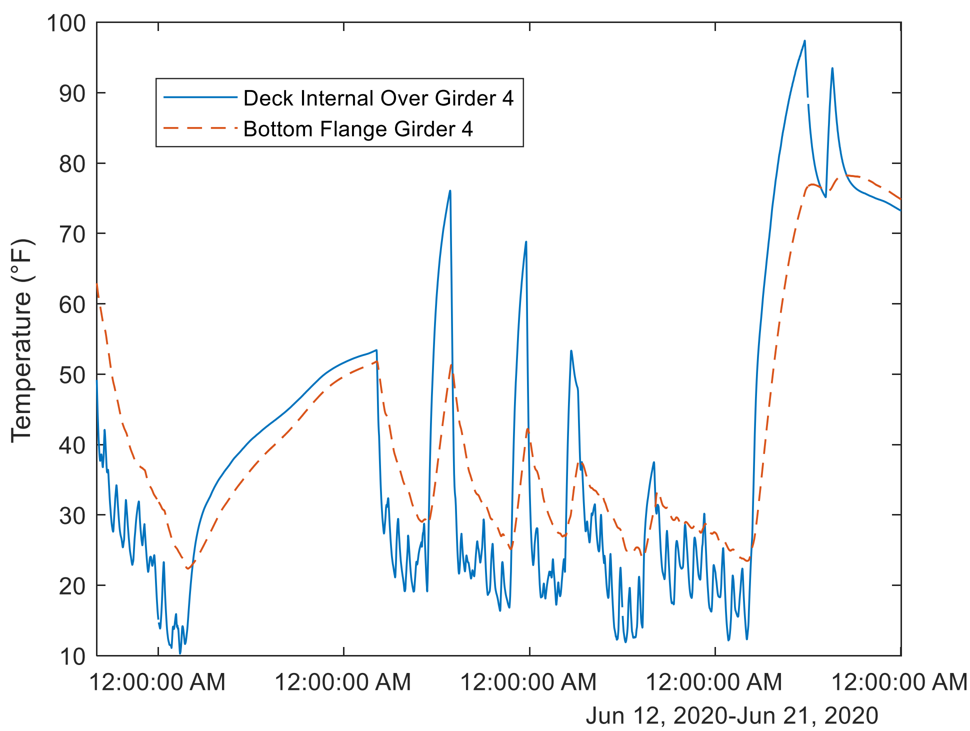


Figure 24: Comparison of Girder Temperature and Internal Deck Temperature

The rapid heating of the specimen was accomplished by heaters located at the exterior girders. The heating during this period also occurred while the live load carriage was positioned over girders 3 and 4. As can be seen in Figure 24, the rapid heating resulted in significant difference between deck and girder temperatures. The thermal expansion of the girders would therefore have been restrained by the deck, thereby increasing the (tension) strain in the deck.

Regardless of the exact mechanisms that caused shifts in baseline deck stress levels, the role of rapid temperature changes is undeniable. Permanent shifts of deck stress toward tension occurred as a result of freeze-thaw cycles.

## Conclusions

During the set and cure of the cast-in-place deck, the deck experienced a general increase in compression strain (more negative) as a result of global deformation due to temperature differentials between the girders and fresh concrete. The exact level of strain at the end of the cure is dependent on the temperature and geometry (camber) of the structure when the concrete gained strength. However, over time, shrinkage and creep served to reduce the locked-in compression in the deck.

During testing, freeze-thaw cycles and rapid heating resulted in permanent shifts in the deck toward tension. The cumulative effects of these mechanisms seem to result in net tension in some regions of the deck after 18 months of testing.

The results presented herein have illustrated that the strains experienced in a concrete deck over its life are largely a function of the structure's response to temperature changes. Temperature differences between fresh concrete and the girders may cause camber and result in a final condition that is favorable with net compression in the deck. However, this case study showed that the combination of early-age shrinkage and thermal responses can still lead to tension strains in both the longitudinal and transverse directions. While these tension strains do not exceed the concrete capacity, their cumulative effect may certainly lead to fatigue cracking of the deck and accelerate deterioration.

## Recommendations

The specimen investigated for this report was subjected to heating and cooling at an accelerated rate compared to natural condition. In the case of real bridges, heating and cooling is likely to occur over a longer time period, and thus temperature differentials that induce global deformations would not be as great. Furthermore, heating of real bridges is likely to be driven by solar exposure, and thus the deck would heat first. Although expansion of the deck would be restrained by the girders, the resulting deformation would cause an increase in strain (toward tension). Therefore, temperature induced deformation of the structure should be considered when predicting deck demands.