CASE STUDY OF A TENNESSEE BRIDGE WITH HIGH-STRENGTH AND NORMAL-STRENGTH CONCRETE BEAMS

X. Sharon Huo, Ph.D., P.E., Tennessee Technological University, Cookeville, TN
Pingsheng Zhu, M.S., University of Southern California, Las Angels, CA
Francis Ung, M.S., EIT, United Consulting Group, Ltd., Norcross, GA

ABSTRACT

Tennessee Department of Transportation (TDOT) has sponsored several research projects to experimentally study the short-term and long-term behavior of HPC bridges. This paper presents the research results of a case study on a HPC bridge in Tennessee. The bridge is a twin-bridge over Pistol Creek in Blount County, Tennessee. The beams in one bridge lane are of high-strength concrete (HSC) with a compressive strength of 13,000 psi and the ones in the other lane are of normal-strength concrete (NSC) with a compressive strength of 7,000 psi. The bridge was instrumented for the measurements of concrete strains and temperatures of precast beams, cast-in-place decks, and diaphragms during various stages. The cambers of prestressed beams were also measured. Test results presented in the paper include material properties of concretes, time-dependent strains of precast beams as well as cast-in-place deck slabs and diaphragms, and cambers of prestressed concrete beams. The results obtained from HSC and NSC beams are compared and the differences between HSC and NSC bridge decks and diaphragms are discussed. The recommendations for design and construction of HPC bridges are also presented in the paper.

Keywords: Bridge, Prestressed concrete beam, High-strength concrete, Normal strength concrete, Instrumentation, Concrete strains, Camber, Time-dependent behavior

INTRODUCTION

The use of high-performance concrete (HPC) in highway bridges has grown significantly in recent years. HPC is the concrete that is optimized for a specific application and often possesses qualities such as high strength, low permeability, and excellent long-term durability. Recent researches have shown that the material properties of high strength concrete (HSC), especially time-dependent properties, are different with those of normal strength concrete (NSC) due to different mixture proportions[1]. The time-dependent properties such as shrinkage, creep, and modulus of elasticity directly affect the design of HSC members[1]. It is important to understand the actual behavior of HSC bridge members in order to design and construct the HSC members properly. Experimental study of actual bridge is an essential method to obtain the information of the behavior of HPC bridges [2,3,4,5]. With more knowledge of HSC bridge members, the state Department of Transportation would be able to specify more appropriate provisions on design and production of HSC beams, as well as construction of HPC bridges. As a result, the HPC Bridge will have the best performance in service[6].

HPC has been used in Tennessee's highway bridges for a number of years. The Tennessee Department of Transportation (TDOT) has sponsored several research projects on HPC bridges. The case study on NSC and HSC beams of Pistol Creek Bridge in Blount County is one of these research projects. The primary objective of the case study is to experimentally study the short-term and long-term behavior of prestressed HSC and NSC bridges during various stages of construction and service. To achieve this goal, several HSC and NSC bridge beams and portions of cast-in-place deck and diaphragm in the selected bridge were instrumented.

DESCRIPTION OF INSTRUMENTED BRIDGE

The instrumented bridge is Pistol Creek Bridge located on State Route 162, over Pistol Creek, in Blount County, Tennessee. The bridge is a twin-bridge with an eastbound right lane and a westbound left lane. The beams in right lane of the bridge are HSC beams with specified compressive strengths of 5,500 psi at release and 10,000 psi at service, and the beams in the left lane are NSC beams with specified compressive strengths of 5,500 psi at release and 6,000 psi at service The bridge has five spans and five lines of prestressed concrete beams. The total length of the bridge is 373.2 ft and the width of the bridge is 51.21 ft. Prestressed concrete beams are AASHTO Type III beams and the spacing of beams is 10.6 ft center to center. The thickness of cast-in-place concrete deck is 8.75 in. In each prestressed beam, there are thirty 0.5 in. diameter low-relaxation prestressing strands located in the bottom flange of the beam. The strands are straight along the beam and are debonded at both ends. The elevation and the typical cross-section of the bridge are shown in Fig. 1.

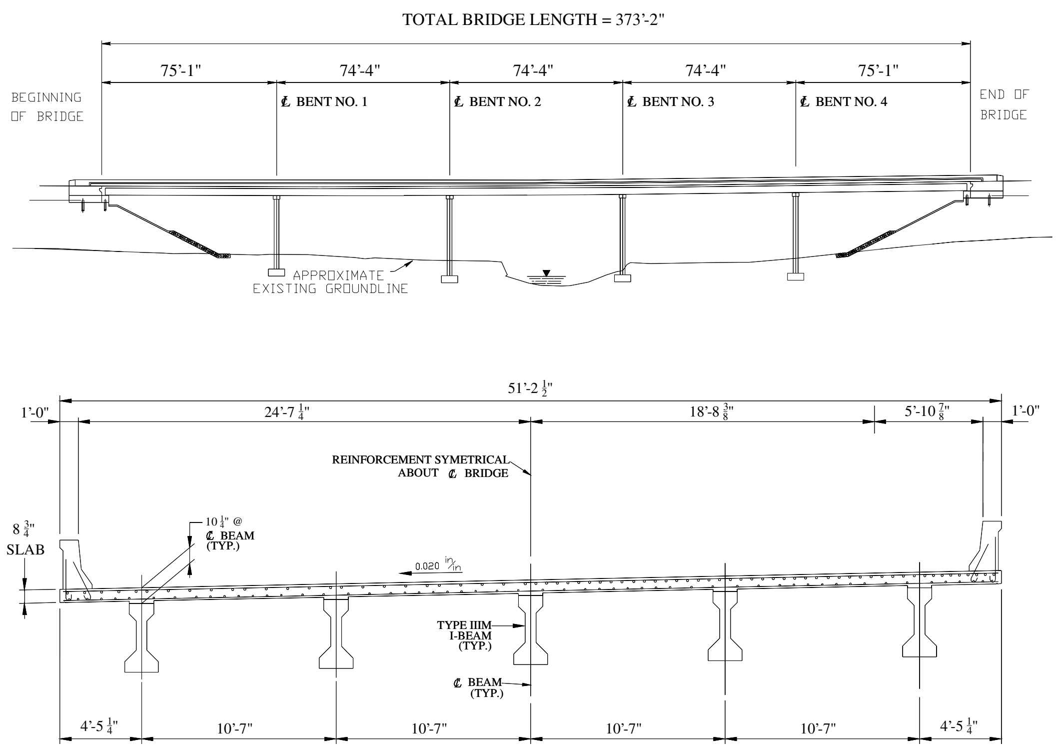


Fig. 1 Elevation and Typical Cross-section of Pistol Creek Bridge

INSTRUMENTATION PLAN

The bridge was instrumented for the measurements of temperatures of concrete, strains of prestressed beams, strains of cast-in-place concrete decks and diaphragms, and cambers of prestressed beams. In each bridge lane, sixteen vibrating wire strain gauges (VWSG) were installed in two prestressed beams and nine strain gauges were installed in portions of deck and a diaphragm. The strain gauges in beams were placed at four different sections including 0.4 span-length section in the first span, and end section, quarter span-length section and midspan section in the second span. The vibrating wire strain gauges were used to measure the strains and temperature of concrete. Each vibrating wire strain gauge consisted of a strain gauge and a thermistor, which could measure both strain and temperature of concrete at the same time. Fig. 2 shows the plan view of bridge instrumentation. The measurement of strains and temperature began immediately before beam casting and continued during curing, storage, and different construction stages. At bridge site, vibrating wire strain gauges were installed in bridge deck at 0.4L section in the first span and midspan section in the second span. Additional gauges were installed in bridge deck between beam lines. The gauge positions in beam and deck slab are shown in Fig. 3. The diaphragm that connected two instrumented beams was also instrumented, as shown in Fig. 4.

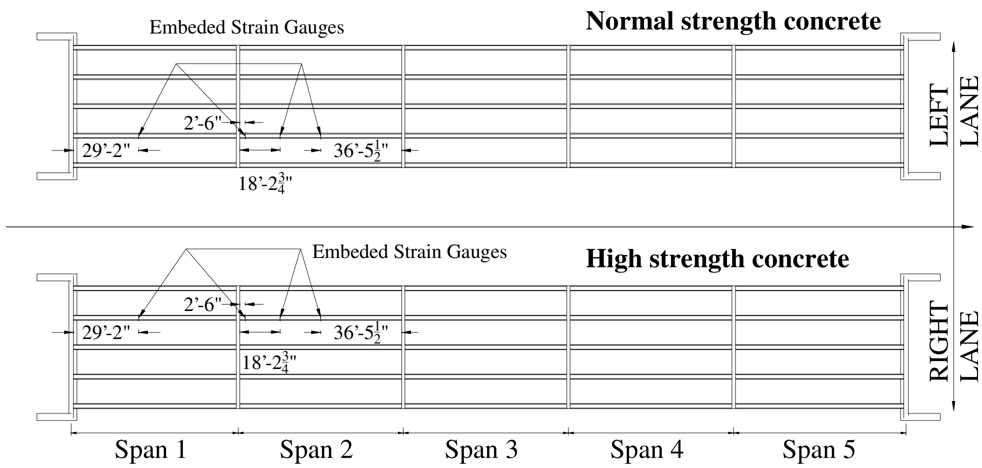


Fig.2 Plan View of Bridge Instrumentation

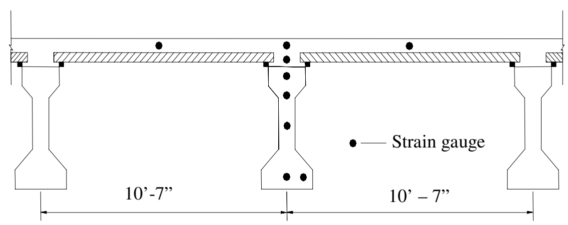


Fig. 3 Gauge Positions in Precast Beam and Deck Slab

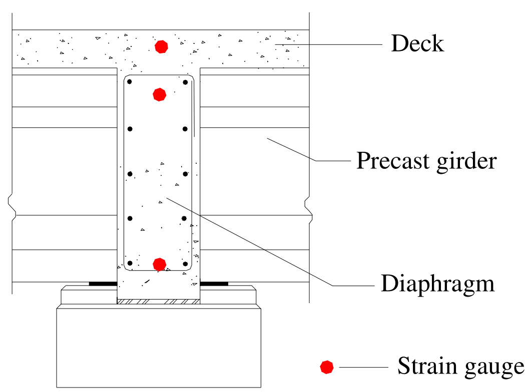


Fig. 4 Gauge Positions in Diaphragm

The cambers of prestressed beams were measured using survey method. The elevations of three points along a beam, two at ends and one at midspan, were measured using autolevel. Then the camber at the midspan was calculated based on measured data. The camber measurement began right after releasing of prestressing steel and was continued afterward.

PRODUCTION AND INSTRUMENTATION OF PRESTRESSED BEAMS

The beams were fabricated at the plant of Ross Prestressed Concrete Inc. in Knoxville, Tennessee. The mixture proportions of NSC and HSC are shown in Table 1. The Ross Prestress Concrete Inc. developed the mixture of HSC. In the HSC mix, Silica fume and high range water reducer were used. The water/cementitious ratio of NSC is 0.37 and water/cementitious the ratio of HSC is 0.32.

Table 1 Mixture Proportions

Material	NSC	HSC
Type I Cement (pcy)	752	800
Silica fume (pcy)	-	50
Fine aggregate (pcy)	1,433	1,368
Coarse aggregate (limestone) (pcy)	1,820	1,780
Water (pcy)	275	275
WRDA/Retarder (oz/cy)	22	-
Rheobuild-1000 (oz/cy)	100	-
RE 3000 FC (oz/cy)	-	85
W/C	0.37	0.32

The instrumentation of strain gauges was completed prior to beam concrete casting. Gauges were tied to pre-made mounting frames and were placed at the locations as designed. Fig. 5 showed the portions of instrumented beams, at the end and at the midspan, with vibrating wire strain gauges in positions. Both NSC and HSC beams were steam cured.

(a) Gauges in midspan section

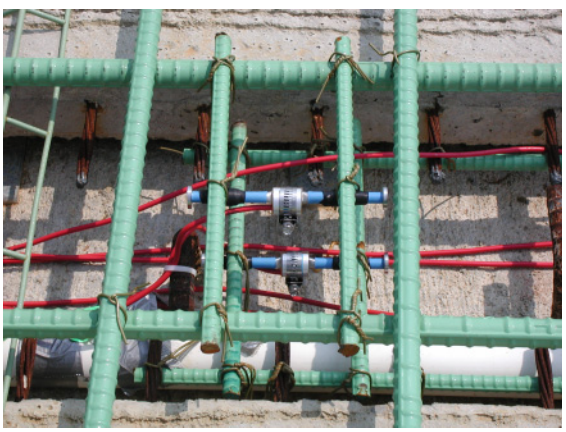
(b) Gauges in end section

Fig. 5 Instrumented Prestressed Concrete Beams

For NSC beams the steam curing temperature was 145^{o}F and the curing duration was 40 hours. The temperatures at different points of a cross section were very close and were higher than the steam curing temperature because of the hydration heat of concrete. The average temperature of NSC was 160^{o}F. The steam curing temperature for HSC beams was around 110^{o}F, which was much lower than that for NSC beams. The curing duration for HSC beams was 20 hours. Similar to the results from NSC beams, the temperatures at different points of a cross section of HSC beams were very close. The average temperature of HSC was 125^{o}F. The steam temperature for HSC was set lower than that for NSC to ensure the strength and other qualities of HSC as recommended by many researchers.[2,5,7]

INSTRUMENTATION OF CAST-IN-PLACE CONCRETE DECKS AND DIAPHRAGMS

The concrete in cast-in-place deck is the standard TDOT Type D concrete. The specified compressive strength of concrete is 4,000 psi. As described in the instrumentation plan, the cast-in-place concrete decks in each bridge lane were instrumented with six vibrating wire strain gauges. Fig. 6 shows the instrumented cast-in-place concrete deck. A diaphragm connected two instrumented beams were instrumented with three vibrating gauges cross the depth of the diaphragm. Fig. 7 shows the instrumented diaphragm. The data collection was started before the concrete placement in cast-in-place deck and continued afterward.



(a) Strain gauges over beam

(b) Strain gauges over precast panel

Fig. 6 Instrumented Cast-In-Place Concrete Deck

Fig. 7 Diaphragm Instrumentation

TEST RESULTS

COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY OF HSC AND NSC

The test results of compressive strength and modulus of elasticity of NSC and HSC are listed in Table 2. The results were obtained from 6 in. diameter cylinders. The compressive strength of HSC at 28 days was 13,400 psi that was much greater than the required strength of 10,000 psi. The compressive strength of NSC at 28 days was 7,170 psi that was greater than the specified strength of 6,000 psi too. It can also be seen from Table 2 that the modulus of elasticity of HSC was higher than that of NSC. These results were consistent with other researchers' observations that higher modulus of elasticity would normally come with higher compressive strength of concrete.[7] Although in general the modulus of elasticity of HSC is higher than that of NSC it may not be true for concrete during the very early age. Based on the initial camber observation, the modulus of elasticity of NSC at 40 hours of concrete age might be higher than that of HSC at 20 hours of concrete age. It seems that the maturity of concrete has a lot to do with the growth of modulus of elasticity. The test results also indicated that using lower steam curing temperature in HSC beam production was proper and successful.

Table 2 Material properties of concrete

Age of Concrete (days)	Normal-Strength Concrete		High-Strength Concrete	
	Compressive Strength (psi)	Modulus of Elasticity (ksi)	Compressive Strength (psi)	Modulus of Elasticity (ksi)
3	5,330	4,110	9,110	4,920
7	5,380	3,980	9,730	5,350
14	6,250	5,060	12,250	5,640
28	7,170	5,070	13,400	5,830
56	7,960	5,410	13,540	6,390

TIME-DEPENDENT CONCRETE STRAINS OF PRESTRESSED CONCRETE BEAMS

Figs. 8 and 9 show the time-dependent concrete strains at midspan sections of NSC and HSC beams after prestress release, respectively. The bridge beams were monitored for more than 250 days. During this period of time, the bridge went through several construction stages including erection of beams, placement of precast concrete deck panels, and casting of cast-in-place deck. The measured data shown in Figs. 8 and 9 reflected the time-dependent strains changes and the instantaneous strain changes of concrete at those construction stages. It can be seen from Fig. 8 that the strains of NSC beam changed continuously with time due to creep and shrinkage of concrete. These strains became relatively constant in about 150 day. In contrast, the strains of HSC beam, as shown in Fig. 9, increased rapidly with time during the early-age of concrete and stabilized in only about 40 days, a much shorter period of time compared to the NSC beam. Similar results were observed from the strains recorded at other sections of NSC and HSC beams. It can also been seen from Figs. 8 and 9 that the magnitudes of strain changes in NSC beam were larger than that in HSC beam. These results were consistent with the test results of material properties of NSC and HSC. The creep and shrinkage of HSC normally develop more rapidly in the early age and reach a stable level in a shorter period of time. Also, the ultimate shrinkage strains and creep coefficients of HSC were smaller than those of NSC.[1] As expected, the strain gradient along the height of either the NSC or the HSC beam section varied almost linearly.

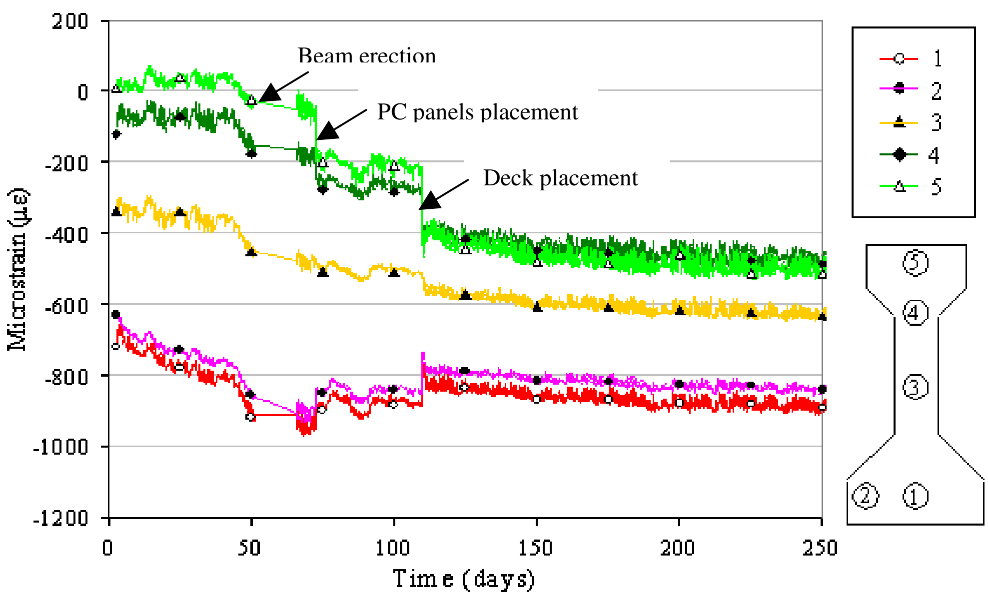


Fig. 8 Measured Time-Dependent Concrete Strains at Midspan Section of NSC

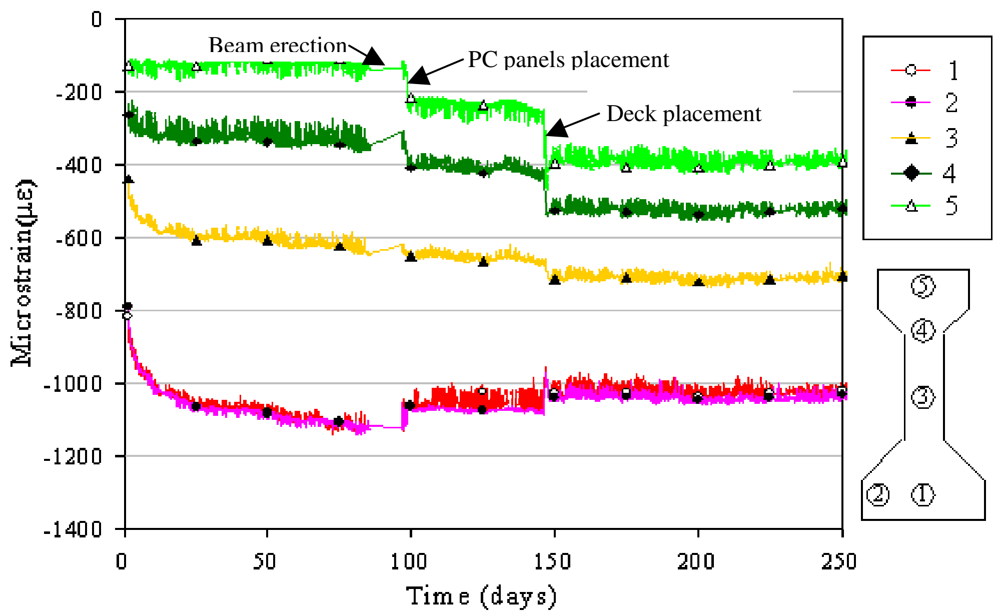


Fig. 9 Measured Time-Dependent Concrete Strains at Midspan Section of HSC

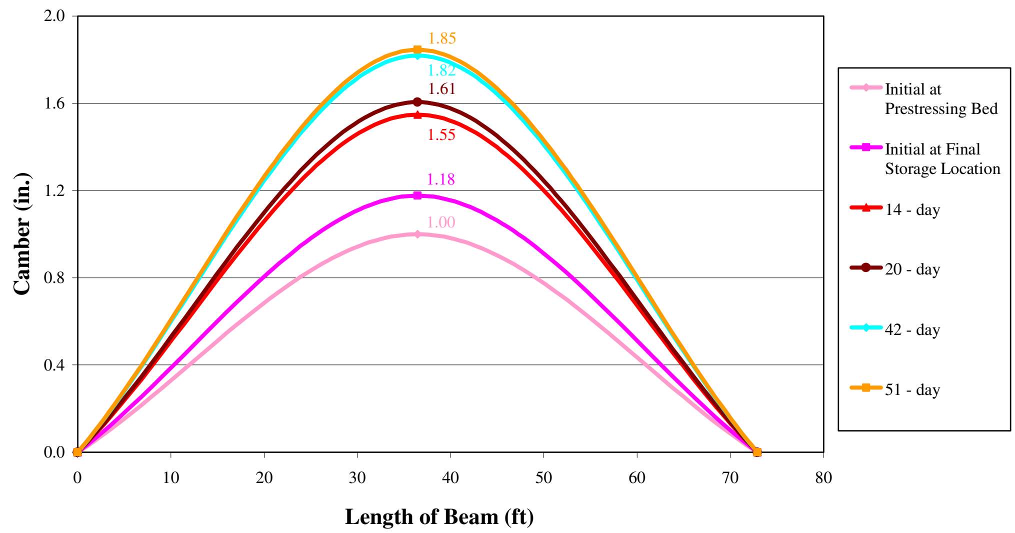
CAMBERS OF PRESTRESSED NSC AND HSC BEAMS

The cambers were measured immediately after prestressing strands were released, and measurements were continuously taken in a two-week or three-week interval until the prestressed concrete beams were transported to the bridge site. The first measurement of beam camber was taken on prestressing bed using measuring tape after prestress release. After the beams were moved to the storage location camber was measured again using autolevel survey equipment.

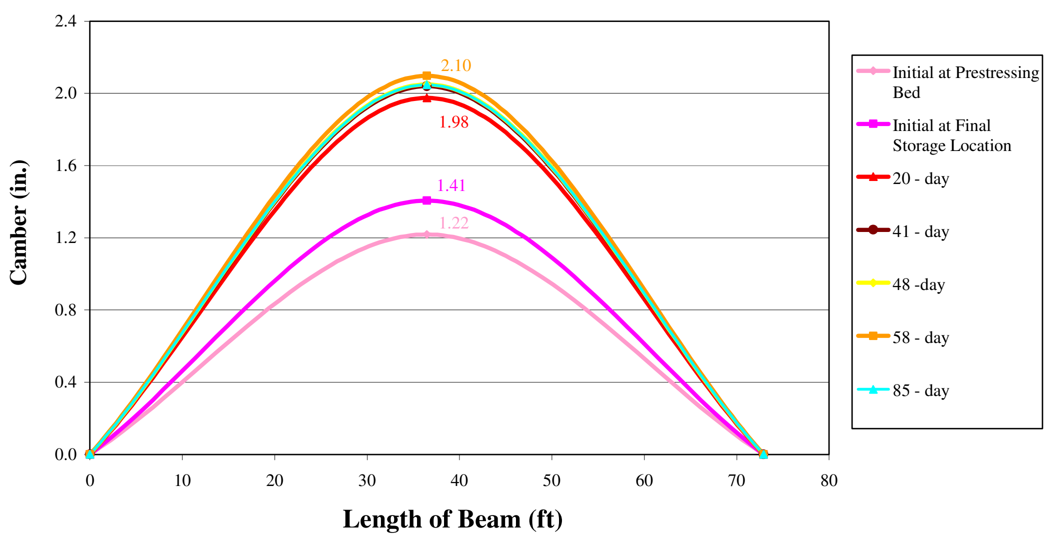
The average initial camber of NSC beams was 1 in. when measured on prestressing bed. The average camber of the same beams immediately after beams were placed at a storage location was 1.18 in., which was 17.6 percent higher than the initial camber on prestressing bed. The increase in camber was mainly due to the removal of possible restraints on beams that was imposed by prestressing bed. Once the beam was lifted up from prestressing bed, the member was free to deform and more camber was observed at the new location. Fig. 10 (a) shows the average measured camber of NSC beams at different time. The cambers of NSC beams increased continuously with time until the last measurement at 7-week. The increase of camber was primarily due to the creep and shrinkage of concrete.

The average initial camber of the prestressed HSC beams was 1.22 in. when the beams were on prestressing bed. The camber was increased to 1.41 in. when measured immediately after the beams were placed at storage location. The percentage increase in the second initial cambers of HSC beams was 15.3 percent over the first initial readings. Fig. 10(b) shows the average camber of HSC beams measured from the time of prestress release to the time of erection. During the first three weeks the average camber of HSC beams had increased to

1.97 in. The camber increase during these three weeks was almost 50% of the initial camber. During the next three weeks the measured camber increased very little to 2.05 in. with only 5 percent increase to the previous camber measurement. Almost no change of cambers could be observed for HSC beams after six weeks. The test results indicated that a large percentage of the potential camber happened during the first three to four weeks of concrete age. Similar observations have been found from the tests on material properties of HSC. Majority of the creep and shrinkage of HSC would happen during the early age of concrete. This phenomenon is also considered as a feature of HSC members.[1]



(a) NSC Beams



(b) HSC Beams

Fig. 10 Measured Cambers of Prestressed Concrete Beams

TIME-DEPENDENT STRAINS OF CAST-IN-PLACE CONCRETE DECKS

The cast-in-place concrete deck on NSC beams was cast on the109th day of NSC beam age. The actual compressive strength of concrete was 5,240 psi that was greater than the required strength of 4,000 psi. Fig. 11 shows the measured time-dependent strains of cast-in-place concrete deck over NSC beams. For the deck in the NSC bridge, the compressive strains of deck concrete increased continuously from the 109th day to the 200th day of concrete age. Although there existed a differential shrinkage between old beam concrete and new deck concrete, the beams and the deck still acted together and developed the time-dependent deformation like a composite section. Because the creep and shrinkage of NSC beams were still in the developing process as observed in concrete strains of NSC beams, the interaction due to differential shrinkage between deck and beam concretes introduced compressive strains in the precast beam. These additional compressive strains caused beam to have more time-dependent deformation. After about 200 days of beam age or 90 days of deck age, the time-dependent deformation of deck became stable and maintained at around 120 microstrains in compression. The minor ripples in concrete strains were mainly due to the thermal effect from daily temperature changes.

The cast-in-place concrete deck on HSC beams was cast on the 146th day of HSC beam age. The actual compressive strength of concrete was 5,820 psi that also met the strength requirement of 4,000 psi. Fig. 12 shows the measured time-dependent concrete strains of cast-in-place concrete deck over HSC beams. Quite opposite to what were observed in NSC bridge deck, the strains in deck concrete on HSC beams showed very minor changes. Most of the measured strains in deck were in tension and of very small values. It was observed from the HSC beam strains that majority of the shrinkage and creep of HSC beams had happened and concrete strains of the beams had become very stable. After deck slab placement, the newly placed deck concrete would experience a larger shrinkage deformation. However, HSC beams had no new time-dependent strain developed and they acted as restrains to the shrinkage deformation of deck concrete. As a result, the differential shrinkage between old beam concrete and new deck concrete caused tension in concrete deck. The tensile strains in the top of the deck were smaller than those in the bottom of the deck because the top portion of the deck concrete had less restrains than the bottom portion[8]. Although these tensile strains were small, they could cause tensile stress and even cracking in the deck concrete. Therefore it is important to limit this type of tensile strains in deck concrete. The strain changes after 340 days were primarily due to the changes in weather temperature.

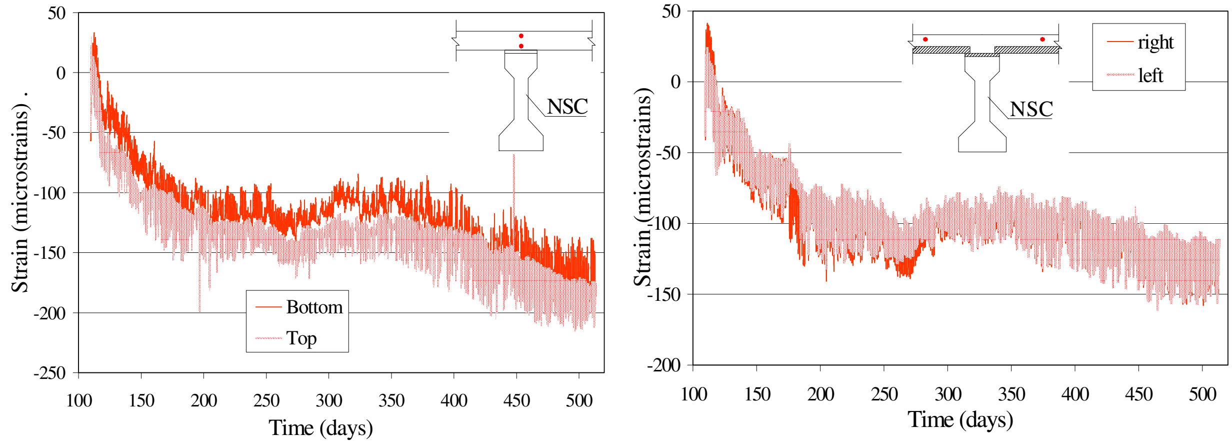


Fig. 11 Measured Time-Dependent Concrete Strains in Deck (0.4L Section) of NSC Bridge

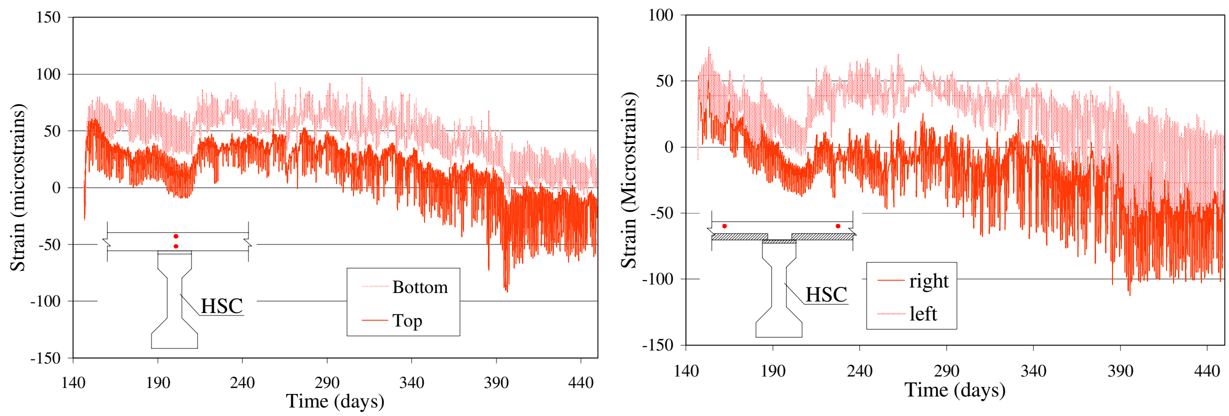


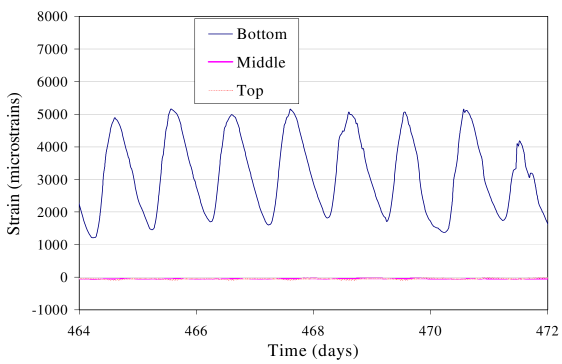
Fig. 12 Measured Time-Dependent Concrete Strains in Deck (0.4L Section) of HSC Bridge

TIME-DEPENDENT STRAINS OF CAST-IN-PLACE CONCRETE DIAPHRAGMS

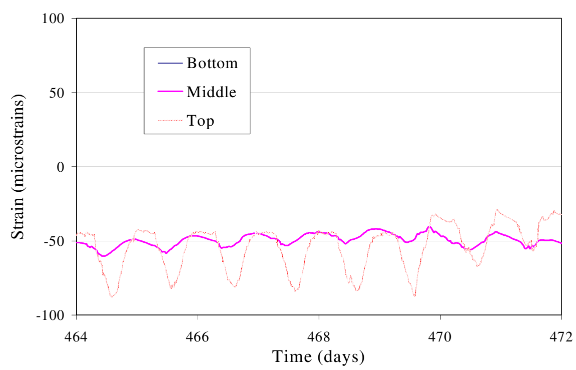
The Pistol Creek Bridge is a jointless bridge with integral abutments. The integral abutments and continuous structure made the diaphragms that connected precast beams very sensitive to temperature changes and time-dependent positive moment.

In the NSC bridge, small tensile strains were measured in the top portion of the diaphragm and small compressive strains were measured in the middle of the diaphragm during the early age after deck casting. The measured concrete strains in the top and middle of the diaphragm had small changes with time and small fluctuation due to daily temperature. In contrast, the strain in the bottom of the diaphragm changed dramatically, not only with daily temperature but also with time. The measured strain in the bottom portion of the diaphragm varied in a small magnitude with daily temperature during the early age. As time increased, the measured strains at the bottom of the diaphragm increased and a relatively larger variation due to daily temperature changes were observed. It is believed that the concrete at the bottom of the diaphragm had cracked because the measured tensile strains were much larger than concrete maximum tensile strain. Once the section cracked, the section properties became smaller and the deformation caused by external forces became larger. As was observed, the strain in the bottom of diaphragm became larger and larger and reached 8000 microstrains at the end of the monitoring. Fig. 13 shows the measured strain changes in the diaphragm during the later monitoring period in two scales.

The temperatures in the deck and the top flange of the beams were much higher than those in the bottom flange of the beams. The differential thermal expansion in beams, larger at the top and smaller at the bottom, introduced a camber of beams as well as tensile strains in the bottom portion of the diaphragm and compressive strains in the top portion of the diaphragm. In addition to thermal effects, time-dependent deformation also affected the strain development in the diaphragm. After the deck casting, the creep and shrinkage of concrete in the NSC beam developed continuously with time. The additional camber due to the time-dependent effects introduced a time-dependent positive moment in the diaphragm. This positive moment caused additional tensile strains at the bottom of the diaphragm. Both thermal forces and time-dependent deformation caused the strain change in the diaphragm. The strains in the top part of the diaphragm changed noticeably from tensile to compressive during the first 70 days after deck casting, while the strains in the bottom of diaphragm remained as tensile strains and increased enormously.



(a) Large scale of microstrains



(b) Small scale of microstrains

Fig. 13 Measured Strains Variation in the Diaphragm (NSC Bridge)

For the HSC bridge, there were a small tensile strain of about 80 microstrains measured in the top of the diaphragm and a small compressive strain of about 100 microstrains measured in the middle of the diaphragm after deck curing. Again, the concrete strains in the bottom gauge varied with the temperature changes due to the camber increased. Unlike in the NSC bridge, the measured strains in the diaphragm showed very small effects of time-dependent deformation. Because the majority of creep and shrinkage of HSC beams had already occurred before deck casting, very small time-dependent positive moment was developed in the diaphragm of the HSC bridge. The strains in the top of diaphragm changed gradually from small tensile to almost zero and no strain changes were observed in the middle of the diaphragm. The strains in the bottom of diaphragm remained in tension and varied with daily temperature. For the HSC beam bridge, the thermal effect is the major cause to the strain changes in diaphragm. Fig. 14 shows the tensile strain measured in the bottom of the diaphragm fluctuated with daily temperature. Although the tensile strain in the bottom of the HSC bridge diaphragm was much smaller than that in the NSC bridge diaphragm, it also exceeded the tensile strain limit of concrete.

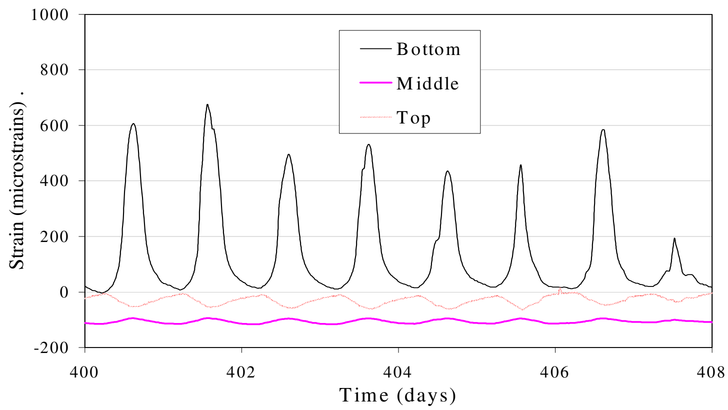


Fig. 14 Measured Strains Variation in the Diaphragm (HSC Bridge)

CONCLUSIONS AND RECOMMENDATIONS

Based on the test results of this research, following conclusions have been made: (1) Use of lower steam curing temperature in curing of HSC member was a successful practice in this research; (2) The time-dependent concrete strains measured in NSC beams increased with time continuously and became stable in about 150 days of concrete age whereas the time-dependent concrete strains in HSC beams became stable in about 40 days of concrete age, a much shorter period of time; (3) The measured camber of NSC beams increased with time during the first seven weeks of concrete age. In contrast, the measured cambers from HSC beams increased rapidly during the first four weeks and remained constant afterward; (4) Although the same deck concrete was used in NSC and HSC bridges the concrete strains in two decks were quite different because of the different time-dependent strain statues of the NSC and HSC precast beams at the time of deck placement. Smaller differential shrinkage between new deck concrete and old precast beam concrete would lessen the possibility of having tension strain in deck; and (5) Both ambient temperature changes and time-dependent positive moment developed in diaphragm affected the concrete strains in diaphragm that connected precast beam.

The test results obtained from the instrumented bridge beams in this study showed obvious differences between the behaviors of NSC and HSC bridge members. Following recommendations can be made based on the preliminary study: (1) the deformation of HSC beams developed rapidly in the early age and stabilized in about 35 - 45 days. Therefore, the HSC beams could be erected about 40 days after beam casting. The erection schedule recommended was based on the results of this research and on using Tennessee mixture design and local materials; (2) Lower steam curing temperature can be used in HSC beams production to obtain expected quality of HSC; and (3) A fogging system should be used in the construction procedures of concrete placement for bridge deck slab with HSC beams to prevent rapid evaporation of moisture, reduce earl-age shrinkage in the deck concrete, and eventually reduce the differential shrinkage between deck and beam concretes.

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