# FIELD STUDY OF A TWIN-BRIDGE WITH HIGH-STRENGTH AND NORMAL-STRENGTH CONCRETE BEAMS IN BLOUNT COUNTY, TENNESSEE

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## ABSTRACT

*This paper presents a current study on a bridge research project 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 specified compressive strength of 69 MPa and the ones in the other lane are of normal-strength concrete (NSC) with a specified compressive strength of 42 MPa. The primary objective of the research is to experimentally study the time-dependent behavior of HSC and NSC bridge beams and cast-in-place concrete deck during various stages of construction and service. The selected bridge was instrumented for the measurements of concrete temperature, strains of prestressed beams and cast-in-place decks and diaphragms, and cambers of prestressed beams during curing and other stages. Test results presented in the paper include material properties of concretes, temperature of concrete during curing, time-dependent strains of HSC and NSC beams and cast-in-place concrete decks, and cambers of prestressed concrete beams. The test results obtained from HSC and NSC beams are compared and the differences between HSC and NSC beams are discussed. The recommendation for design and construction of HSC bridge beams is also presented in the paper.*

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

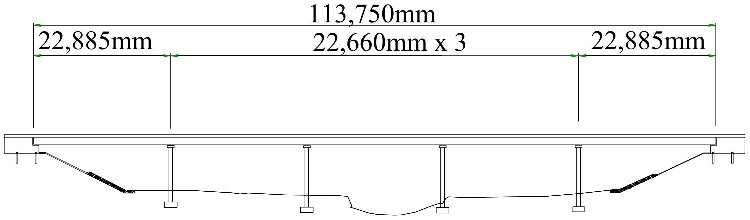
## INTRODUCTION

The use of high-performance concrete (HPC) in highway bridges has grown significantly in recent years. With enhanced durability and strength parameters HPC often possesses qualities such as high strength, low permeability, good workability, and excellent long-term durability. Recent researches have shown that the material properties of high strength concrete (HSC) are different with those of normal strength concrete (NSC), especially the time-dependent properties.[1] Because of the different material properties, the time-dependent behavior of HSC and NSC members are different too. It is very important for bridge engineers to have a better understanding of the behavior of HSC bridge members as well as the differences between the behavior of HSC and NSC members so that they can use proper procedures to design HPC bridges more economically and safely. With more knowledge of HSC bridge members, the engineers in the State Department of Transportation would be able to specify more proper provisions on the production and erection of HSC prestressed beams and construction of HPC bridges. As a result, the HPC Bridge will have the best performance in service.[2] Field study of actual bridge along with laboratory experiments is an essential method to obtain the information of the behavior of HPC bridges.[3,4,5,6]

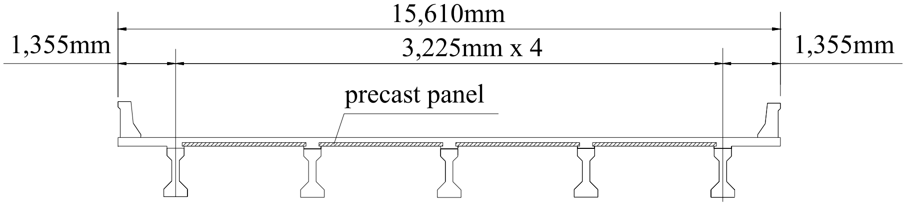
Tennessee Department of Transportation (TDOT) has sponsored a research project to experimentally study the short-term and long-term behavior of prestressed HSC and NSC bridges. The selected bridge is a twin-bridge with HSC beams in one bridge lane and NSC beams in the other lane. The primary objective of the study is to investigate the time-dependent behaviors of HSC and NSC bridge beams and cast-in-place concrete deck during various stages of construction and service.

## 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 bridge and a westbound left lane bridge. The beams in right lane of the bridge are HSC beams with specified compressive strengths of 38 MPa at release and 69 MPa at service, and the beams in the left lane are NSC beams with specified compressive strengths of 38 MPa at release and 42 MPa at service The bridge has five spans and five lines of prestressed concrete beams. The total length of the bridge is 113.75 m and the width of the bridge is 15.61 m. Prestressed concrete beams are AASHTO Type III beams, and are placed in a spacing of 3.23 m center to center. The thickness of cast-in-place concrete deck is 222 mm. In each prestressed beam, there are thirty 13-mm diameter low-relaxation prestressing strands located in the bottom flange of the beam. The strands are straight along the beam and 12 of the strands are debonded at both ends. The elevation and the typical cross-section of the bridge are shown in Fig. 1.



Elevation



Typical cross-section

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. Datalogger, multiplexers, and laptop computer were the main equipment in the data acquisition system. Fig. 2 shows the plan view of bridge instrumentation, and Fig. 3 shows the positions of vibrating wire strain gauges in cross-sections. 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 bridge deck are shown in Fig. 4. The diaphragm that connected two instrumented beams was also instrumented, as shown in Fig. 5.

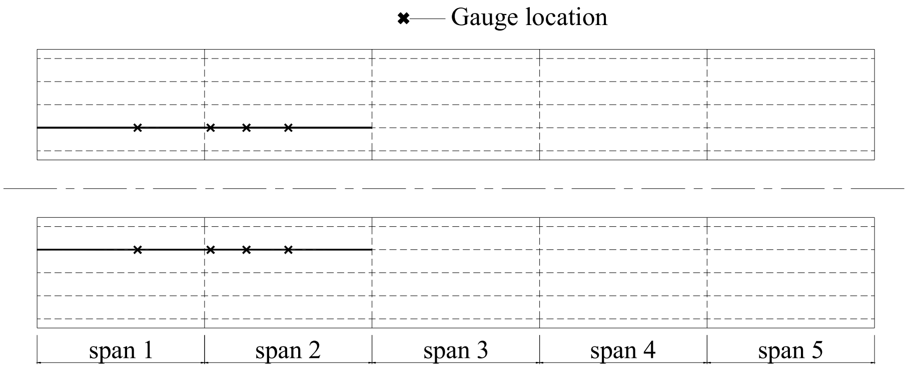
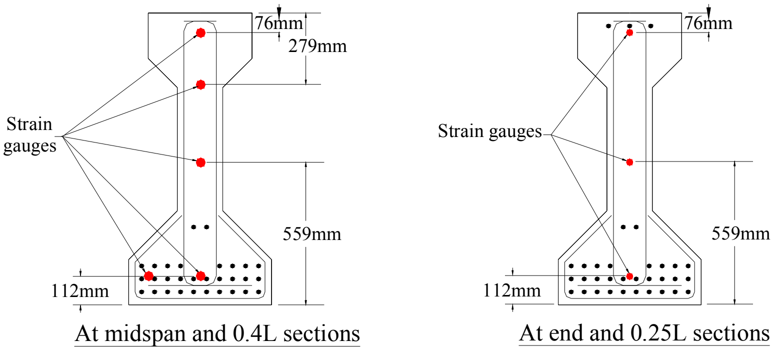


Fig.2 Plane View of Bridge Instrumentation



At midspan and 0.4L sections

At end and 0.25L sections

Fig. 3 Gauge Positions in the Beam Cross-Sections

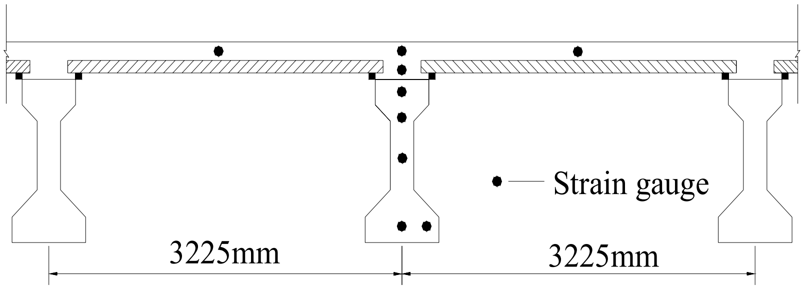


Fig. 4 Gauge Positions in Bridge Deck

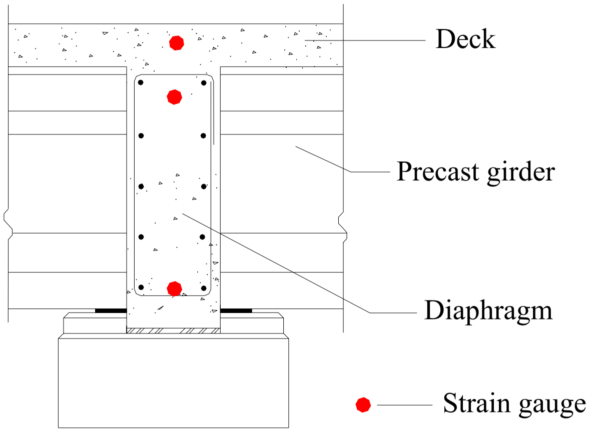


Fig. 5 Gauge Positions in Diaphragm

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 Prestress 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 Proportion

| Material | NSC | HSC |
|---|---|---|
| Cement, (kg/m$^3$) | 446 | 475 |
| Silica fume, (kg/m$^3$) | - | 30 |
| Fine aggregate, (kg/m$^3$) | 850 | 812 |
| Coarse Aggregate, (kg/m$^3$) | 1080 | 1056 |
| Water, (kg/m$^3$) | 163 | 163 |
| WRDA/Retarder, (kg/m$^3$) | 8.2 | - |
| RB-1000, (g/m$^3$) | 37.1 | - |
| RB 3000 FC, (g/m$^3$) | - | 31.5 |
| W/C | 0.37 | 0.32 |

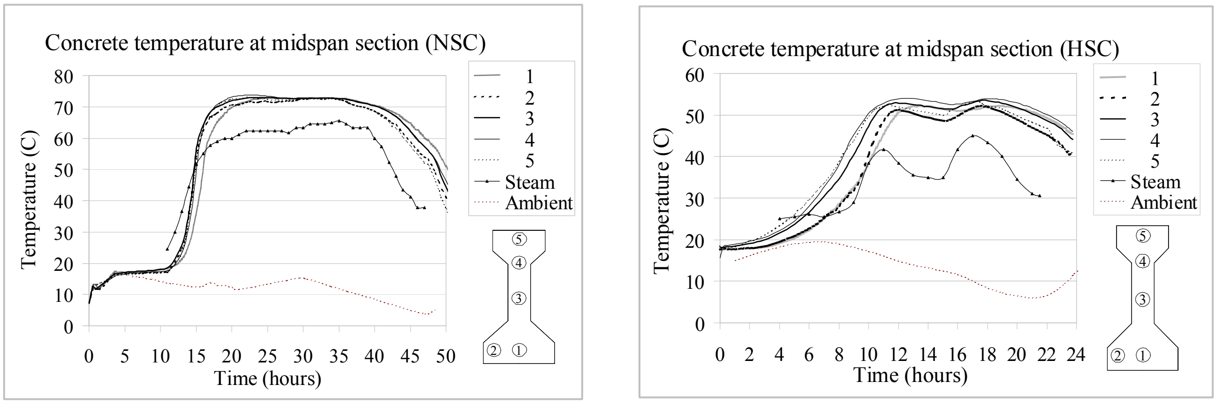
The instrumentation of strain gauges was completed prior to beam concrete casting. Gauges were tied to the pre-made mounting frames and were placed at the locations as designed.

Fig. 6 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. Concrete cylinders were made during beam casting and were placed near beams during steam curing. A few cylinders were tested after initial curing to determine the concrete strength for releasing of prestressing steel. Other cylinders were brought back to Tennessee Technological University for extensive experiments of other concrete material properties.

(a) Gauges in midspan section (b) Gauges in end section

Fig. 6 Instrumented Prestressed Concrete Beams

Fig 7 shows the concrete temperatures across midspan section, steam temperature, and ambient temperature during curing. For NSC beams the steam curing temperature was 63$^{o}$C and the curing duration was 40 hours. The temperatures at different points of the section were very close to one another and were higher than the steam curing temperature because of the hydration heat of concrete. The average temperature of NSC was 71$^{o}$C. The steam curing temperature for HSC beams was around 42$^{o}$C, 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 the section of HSC beams were very close. The average temperature of HSC was 50$^{o}$C. 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.[3,6,7] .



a. NSC beam b. HSC beam

Fig. 7 Concrete Temperature during Steam Curing

## 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 27 MPa. As described in the instrumentation plan, the cast-in-place concrete deck in each bridge lane were instrumented with six vibrating wire strain gauges, four at the 0.4 span-length and midspan sections over the instrumented beams and two at the 0.4 span-length between beams over precast prestressed concrete panels. Fig. 8 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. 9 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. 8 Instrumented Cast-In-Place Deck

Fig. 9 Instrumented Diaphragm

## 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 150mm diameter cylinders. The cylinders were cured and tested following the ASTM standards. The compressive strength of HSC at 28 days was 92 MPa that was much greater than the required strength of 69 MPa, while the compressive strength of NSC at 28 days was 49 MPa that was greater than the specified strength of 42 MPa 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 when the moduli of elasticity of NSC and HSC at different ages are compared. In this study, 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. This fact could be observed in other observed results. 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 (MPa) | Modulus of Elasticity (GPa) | Compressive Strength (MPa) | Modulus of Elasticity (GPa) |
| 3 | 37 | 28 | 63 | 34 |
| 7 | 37 | 27 | 67 | 37 |
| 14 | 43 | 35 | 85 | 39 |
| 28 | 49 | 35 | 92 | 40 |
| 56 | 55 | 37 | 93 | 44 |

## TIME-DEPENDENT CONCRETE STRAINS OF PRESTRESSED CONCRETE BEAMS

The time-dependent behavior of prestressed concrete beams was monitored with embedded vibrating wire strain gauges. The strain and temperature of concrete were recorded in a 30-minute interval during the experiment. The monitoring was paused twice because of beam transportation. The first disconnection of strain gauges was due to the moving of the beams from prestressing bed to storage location in producer's yard and the second disconnection was due to the shipping of beams from precast producer's plant to the bridge site. The data monitoring was continued after the gauges were reconnected to the data acquisition systems. The obtained strain measurement presented the actual variation of concrete strain with time, while the temperature measurement was mainly used for temperature correction of the strain measurement.

Figs. 10 and 11 show the measured time-dependent concrete strains at midspan section of NSC and HSC beams after prestress release. The bridge beams had been monitored for more than 250 days. During this period, 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 data shown in the Figs 10 and 11 included the time-dependent strains changes as well as the instantaneous strain changes of concrete at those construction stages. It can be seen from Fig. 10 that the strains of the NSC beam changed continuously with time due to creep and shrinkage of concrete. The strains from NSC beam became relatively constant in about 150 day. In contrast, the strains of the HSC beam, as shown in Fig. 11, varied with time during the early-age of concrete and stabilized in only about 30-40 days, a much shorter period of time comparing to the observed results from the NSC beam. Similar results were observed from the strains recorded at other sections of NSC and HSC beams. The initial strains of the HSC beam were higher than that of NSC beams possibly due to the lower initial modulus of elasticity of HSC at prestress release. It can also been seen from Figs. 10 and 11 that the magnitudes of strain changes for NSC beam were higher than that of HSC beam. These results were consistent with the test results of material properties of NSC and HSC. Normally, the creep and shrinkage of HSC develop more rapidly in the early age and reach a stable level in a shorter period of time than that of NSC. Also, the ultimate shrinkage strains and creep coefficients of HSC were smaller than those of NSC.[1] The minor ripples seen in the Figs. 10 and 11 were caused by daily temperature changes, and larger values were observed in the top portion of beams. As expected, the strain gradient along the cross-section of either the NSC or the HSC beam section varied almost linearly on the height of beam.

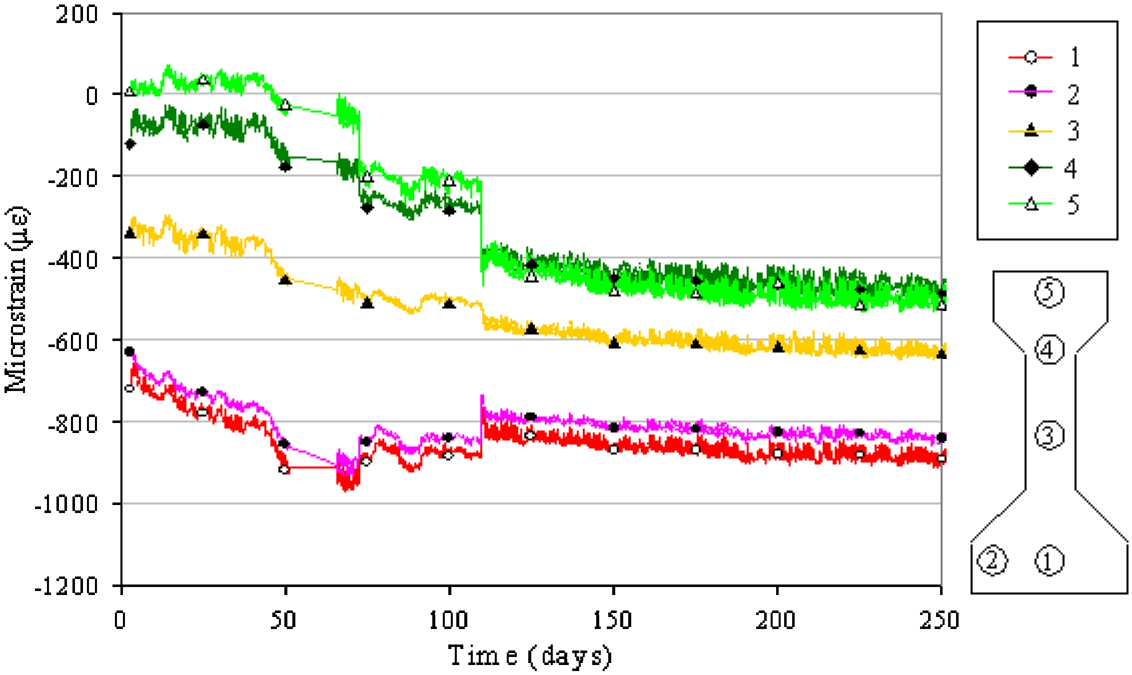


Fig. 10 Measured Time-Dependent Concrete Strains at Midspan Section of NSC

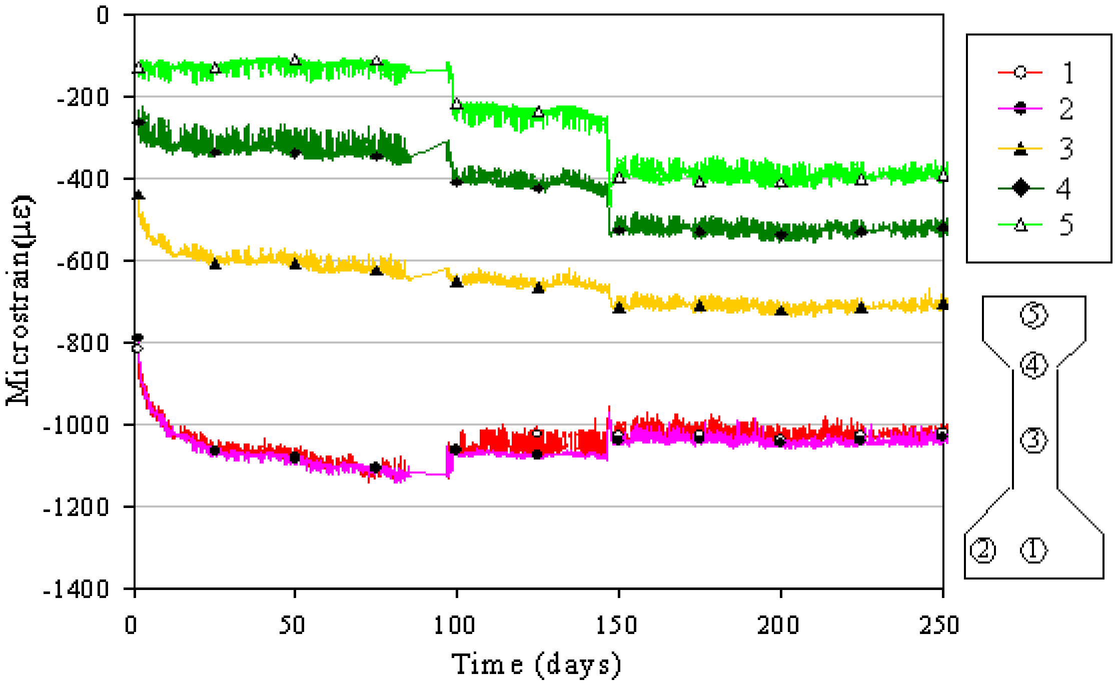


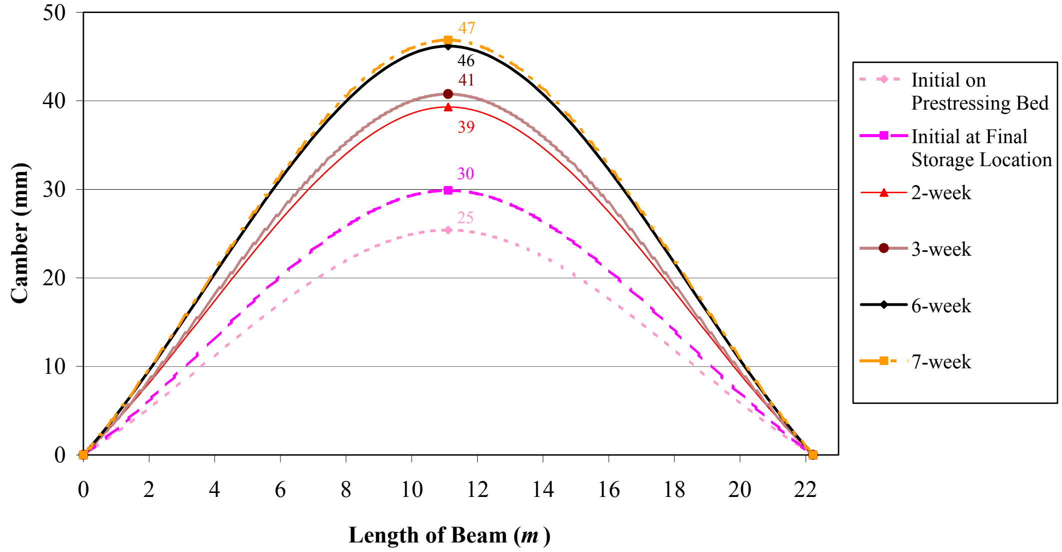
Fig. 11 Measured Time-Dependent Concrete Strains at Midspan Section of HSC

## CAMBERS OF PRESTRESSED NSC AND HSC BEAMS

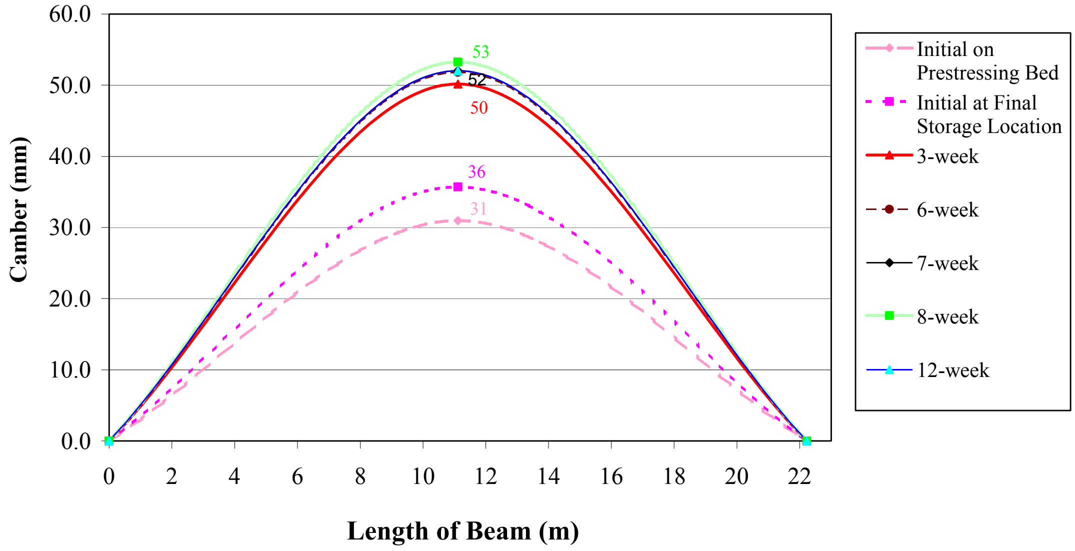
The cambers were measured immediately after prestressing strands were released, and measurements were continuously taken until the prestressed concrete beams were transported to the bridge site. The first measurement of beam camber was taken on prestressing bed using measuring tape after prestress release.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

The average initial camber of NSC prestressed beams was 25 mm when measured on prestressing bed. The average initial camber of the same beams immediately after beams were placed at a storage location was 30 mm, 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. 12 (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. According to the obtained results, the cambers of NSC beams could increase more with time. The increase of camber was primarily due to the creep and shrinkage of concrete.

The average initial camber of the prestressed HSC beams was 31 mm when the beams were on prestressing bed. The camber was increased to 36 mm 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 reading. Fig. 12(b) shows the average measured camber of HSC beams from the time of prestress release to the time of erection. During the first three weeks the average camber of HSC beams increased to 50 mm. The camber increase during these three weeks was almost 50% of the initial camber. During the next three weeks the measured camber increased to 52 mm with only 5 percent increase over the initial camber, which is very small compared to the camber change in the first three weeks. Almost no camber changes were observed in 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 could be also considered as a feature of HSC members.[1]



(a) NSC Beams



(b) HSC Beams

Fig. 12 Measured Cambers of Prestressed Concrete Beams

## TIME-DEPENDENT CONCRETE 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 36 MPa that was greater than the required strength of 28 MPa. Fig. 13 shows the measured time-dependent concrete strains of cast-in-place deck over NSC beams. It can be seen from Fig. 13 that the compressive strains of deck concrete increased continuously from the 109th day to the 200th day of concrete age. Although the differential shrinkage existed between old beam concrete and new deck concrete, the beams and the deck acted together and developed the time-dependent deformation as a new concrete would. Normally, newly placed deck concrete would have larger shrinkage deformation than the old precast concrete. The interaction due to differential shrinkage could introduce compression in beams or tension in deck or both depending on how much the time-dependent deformation developed in old concrete.[8] Because the creep and shrinkage of NSC beams were still in the developing process as observed in concrete strains of NSC beams, the interaction between deck and beam concretes introduced compressive strains in the precast beam. These additional compressive strains caused beam to have more time-dependent deformation such as creep. 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 40 MPa that also met the strength requirement of 28 MPa. Fig. 14 shows the measured time-dependent concrete strains of cast-in-place deck over HSC beams. Quite opposite to the observed results in NSC bridge deck, the strains in deck concrete over HSC beams showed very minor changes and were almost constant. Most of the measured strains in deck were in tension and very small. As described in previous paragraph, differential shrinkage would introduce interactions between old and new concretes and additional strains in deck and beams. Because majority of the shrinkage and creep of HSC beams had happened in the first 30-40 days of concrete age the time-dependent strains of beams had become very stable after 40 days. As a result, the interactions due to differential shrinkage between beam concrete and deck concrete caused tension in concrete deck instead of compression as in the NSC bridge deck. In this case HSC beams acted as restrains to the shrinkage deformation of deck concrete and brought in tensile strains in the 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. These tensile strains in the deck concrete over HSC beams should be limited because a small tensile strain could cause tensile stress and cracking in the deck. The strain changes after 340 days were primarily due to the changes in weather temperature.

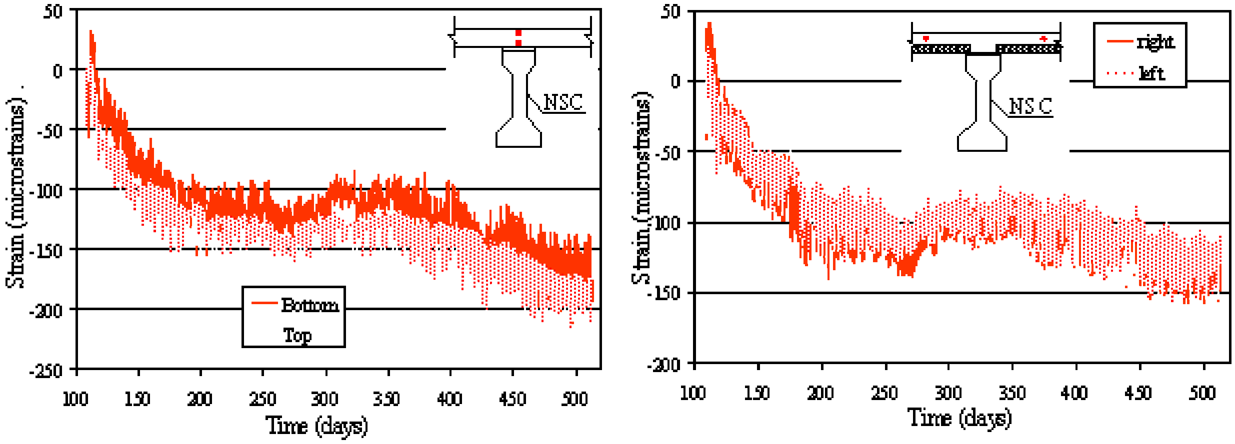


Fig. 13 Measured Time-Dependent Concrete Strains in Deck (0.4L Section) of NSC Bridge

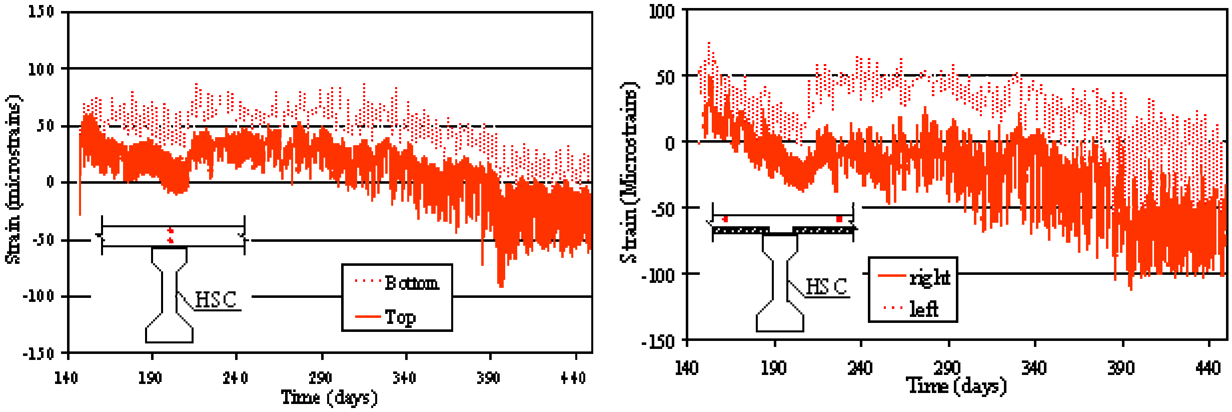


Fig. 14 Measured Time-Dependent Concrete Strains in Deck (0.4L Section) of HSC Bridge

## CONCLUSIONS AND RECOMMENDATION

Based on the preliminary study of the test results obtained from this research, following conclusions have been made: (a) Proper steam curing temperature would be helpful to ensure the compressive strength of HSC. Use of lower steam curing temperature in curing of HSC member was a successful practice in this research; (b) 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; (c) The measured camber of NSC beams increased with time during the 7-weeks when beams were placed in storage yard and additional camber increase could be projected based on obtained results. In contrast, the measured cambers from HSC beams increased rapidly during first four week and remained constant afterward; and (d) The behavior of cast-in-place concrete deck depended on the strain development of supporting precast beams. 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.

The test results obtained from the actual bridge beams in this study showed noticeable differences between the behaviors of NSC and HSC bridge members. Because of the rapid development of time-dependent deformation in the early-age of concrete, the HSC precast beams can be erected earlier. The earlier erection can reduce the storage time of precast HSC beams, speed up bridge construction, and, most important, will enhance the behavior of cast-in-place deck over HSC beams. One recommendation that can be made from this study is that for HSC beams a shorter period of storage time could be required before beam erection.

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