The Short and Long-Term Effects of Temperature and Strain on a Concrete Bulb-Tree Girder Bridge

Ethan Pickett
Utah State University
THE SHORT AND LONG-TERM EFFECTS OF TEMPERATURE AND STRAIN
ON A CONCRETE BULB-TEE GIRDER BRIDGE

by

Ethan Pickett

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Approved:

__________________________________________  ________________________________________
Paul J. Barr                                  Marc Maguire
Major Professor                              Committee Member

__________________________________________  ________________________________________
Joe Caliendo                                  Mark R. McLellan
Committee Member                             Vice President for Research and
                                            Dean of the School of Graduate Studies

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ABSTRACT

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by

Ethan Pickett, Master of Science
Utah State University, 2017

Major Professor: Dr. Paul Barr
Department: Civil and Environmental Engineering

The Utah Transportation Center (UTC) as well as the Mountain Plains Consortium, sponsored a study to investigate the long-term performance of a deck bulb tee girder bridge. The bridge in question is located in Nibley, Utah and was erected in early 2016. Temperature and prestress losses were analyzed from embedded instrumentation placed within two of the bridge girders before casting. These two girders contained a total of 50 thermocouples and 16 vibrating wire strain gauges. These instruments were placed at the mid-span and end of an exterior girder and the mid-span, quarter-span, and end of a center girder in order to effectively monitor the bridge response in one quarter of the bridge superstructure.

The monitoring performed with the thermocouples included the temperature of the girders during curing, weekly maximum and minimum temperatures compared to methods for predicting the average bridge temperature, maximum and minimum thermal gradients at each of the five selected cross sections compared to Code thermal gradients, and thermal
camber by measured temperature compared to models to predict thermal gradients. The 16 strain gauges measured prestress losses at four girder cross-sections, which were compared to two predictive methods provided by AASHTO as well as a method by PCI. An additional comparison of the equations provided by AASHTO and a newly available equation used for determining the modulus of elasticity of concretes with a compressive strength of 6,000 – 12,000 psi was performed.

Additional exterior instrumentation were provided by Bridge Diagnostics Inc. (BDI) in order to monitor short-term changes within the bridge. A total of 8 strain gauges were attached to the exterior of the girders with 6 attached at the bottom face of 6 girders and 2 attached at the centroid of 2 girders. These sensors as well as the software and wireless data acquisition provided a method to measure the magnitude and frequency of the ranges of strain experienced by the Nibley Bridge.
PUBLIC ABSTRACT

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Ethan Pickett

The Utah Transportation Center (UTC) as well as the Mountain Plains Consortium, sponsored a study to investigate the long-term performance of a deck bulb tee girder bridge. The bridge in question is located in Nibley, Utah and was erected in early 2016. Temperature and prestress losses were analyzed from embedded instrumentation placed within two of the bridge girders before casting the concrete. These two girders contained a total of 50 thermocouples as well as 16 vibrating wire strain gauges.

The monitoring performed with the thermocouples included the temperature of the girders during curing, weekly maximum and minimum temperatures, maximum and minimum thermal gradients, and thermal camber. Each of these measurements were then used to determine the thermal response of the bridge in comparison to several methods used to predict these measurements. The 16 strain gauges were used to measure prestress losses, and these measurements were compared to two predictive methods for determining the prestress losses.

Additional exterior instrumentation were provided by Bridge Diagnostics Inc. (BDI) in order to monitor short-term changes within the bridge. A total of 8 strain gauges were attached to the exterior of the girders. These sensors as well as the software and wireless data acquisition provided a method to measure the magnitude and frequency of the ranges of strain experienced by the Nibley Bridge.
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CHAPTER 1
INTRODUCTION

1.1. Context

In order to more accurately quantify the behavior and degradation of bridges throughout their service life, the Federal Highway Administration launched the Long-Term Bridge Performance Program (LTBP). As part of this program a concrete bulb tee girder located in Nibley, Utah was instrumented with embedded thermocouples, vibrating wire strain gauges, and velocity transducers. Additional strain gauges and uniaxial accelerometers were attached to the exterior of the bridge in order to monitor the short-term performance of the bridge.

The Utah Transportation Center (UTC) as well as the Mountain Plains Consortium, sponsored a study to investigate the long-term performance of a deck bulb tee girder bridge. This bridge was instrumented using 50 thermocouples at five locations on the bridge, as well as 16 vibrating wire strain gauges located at four of the same five locations. The instrument locations were at mid-span, quarter-span, and the end of an interior girder. An exterior girder contained sensors corresponding to the mid-span and end of the girder. Of the 50 thermocouples, 45 were located in the top flange of two girders at five different locations. An additional location contained the remaining five thermocouples within the bottom two inches of the girder. These sensors were all located in the north-west corner of the bridge, thus one-quarter of the bridge contained sensors and it was assumed that the remaining portions of the bridge would produce similar results.

Additional strain gauges, a data acquisition system, and a software package were provided by Bridge Diagnostics Inc. (BDI). A total of 8 strain gauges were externally mounted at the mid-span of the bridge. Of the 8 gauges, 2 were mounted at the centroid of
two of the girders, while the remaining 6 gauges were attached to the bottom face of individual girders. These gauges provided close inspection of strain variances that occur as vehicles pass over the bridge.

1.2. Research Objectives

This research compared measured temperature, prestress, and strain differences of a bridge constructed with precast deck bulb tee girders to various established codes and predictive models. This research was intended to monitor the actual behavior of a deck bulb tee bridge over a period of 16 months as well as a period of only one week. The long-term data was then analyzed based on temperature differences, temperature gradients, girder camber due to temperature, and prestress losses. These differences were then quantified and recommendations were provided to more accurately design bridges of this type.

Additionally, the short-term variations in strain were studied and observations were determined. This research was also intended to monitor the strain experienced in the bridge and provide a report of the magnitude and frequency of differences in strain at the bottom face of the bridge girders.

1.3. Organization of Thesis

The gathered research data for this thesis is organized in the following manner:

- Chapter 2: presents a review of previous studies related to temperature variations, prestress losses, camber, and temperature gradients of similar concrete bridges
• Chapter 3: Provides details of the bridge parameters, instrumentation arrangement, maturity of girder concrete, temperature changes including gradients and cambers, as well as prestress losses of the girders

• Chapter 4: Provides details of the short-term instrumentation usage, test categories performed with short-term software analysis, and determines the results of short-term strain experienced by the bridge

• Chapter 5: Provides a summary of the research, several conclusions, and recommendations for future research
CHAPTER 2
LITERATURE REVIEWS

Bridge design and bridge monitoring has experienced many changes over the last several years. Numerous aspects are needed in the day to day design and maintenance of concrete bridges. One of these aspects that often lacks in attention are temperature loads. Temperature loads are not given much significance on the day to day design, but they can have considerable effects on the lifespan of the bridge structure. This chapter presents changes to and research performed for bridge design, as well as previous research in the area of temperature effects on concrete bridges. Additionally, this chapter presents previous research performed regarding prestress losses.

2.1. Topic 1

*Long-Term Performance of Prestressed, Pretensioned High Strength Concrete Bridge Girders*

This study was performed by Roller et al. (1995) in order to determine the integrity of high-strength concrete, bridge girders. It included the manufacture, long-term performance, and load testing to failure of two 70 ft (21.3 m) long, 54 in. (1372 mm) deep, pretensioned, prestressed high strength concrete bulb-tee girders. The first girder (Girder 3) was subject to a static load, approximating the in-service design dead load, for a duration of 18 months to evaluate time-dependent properties of the girder. The second girder (Girder 4) was subject to 5 million cycles of fatigue loading in order to evaluate the long-term performance of the girder. After the end of both tests, the girders were tested in flexure until failure.
The prestress losses were measured in Girder 3 over the testing period of 18 months. This girder had a load of 6800 lbs, applied using a four point loading scheme during the entire 18 month duration. This was done in order to monitor the behavior of the girder under full design dead load. The study concluded that the measured prestress losses and creep and shrinkage from cylinders were significantly less than those predicted by the AASHTO provisions. Thus, it was recommended that further research is needed in order to determine the prestress loss characteristics of high strength concrete. This research was performed in 1995, since then a more current process for determining the prestress loss was determined in the AASHTO 2010 code.

The prestress losses in Girder 4 were measured over a period of 18 months as the bridge was subject to 5 million cycles of fatigue loading. The girder was placed on supports to simulate a simply supported beam scenario. Two hydraulic actuators were each placed at 6 ft (1.83 m) from the center of the span. A digital data acquisition system was used throughout the test that measured results from load cells (actuator and support), internal strain gauges, external strain gauges, crack width gauges, and LVDTs. This girder performed well over the 5 million cycles of fatigue loading without any noteworthy increase in prestress losses and cracking in the girder concrete.

After the 18 month period that each of the girders were monitored, they were tested in flexure until failure. These two girders were both placed upon supports that would represent a simply supported beam situation. A load was applied to the top of the girders and increased in increments of 2000 lbs (8,896 kN) using hydraulic jacks. The measured flexural properties for both girders were adequate with respect to both design and
specification requirements. The researchers concluded that “Based on results from this investigation, high strength concrete bridge girders can be expected to perform adequately over the long term when designed and fabricated in accordance with current AASHTO provisions.”

*Camber and Prestress Losses in Alabama HPC Bridge Girders*

This article by Stallings et al. (2003) focuses on the measured camber and prestress losses for High Performance Concrete (HPC) Bridge Girders located on a bridge in Alabama. The bridge was constructed using seven separate spans of 114 feet each. Structural parameters were monitored on five of the girders located in a central span of the bridge. This study was performed due to the relatively long span of the girders and the discouragement that may arise from the overly conservative calculations of camber losses and prestress losses. The article focuses on describing the tests performed on the concrete girders and comparing these results with current standard calculations.

For this study, several structural parameters were monitored to determine the camber and prestress losses. Some of the structural parameters monitored include camber, which was measured at an average concrete age of 200 days on thirty-one girders, as well as strains from the time of prestress transfer to bridge completion. Other structural parameters monitored include the creep, shrinkage, and modulus of elasticity of the concrete based on cylinder samples. In addition to the recorded parameters, several calculated parameters were also determined. Some of the calculations came from an incremental time-step analysis and an approximate time step method used in order to
predict girder strains, camber losses, and prestress losses up to the time of deck construction.

The document describes in detail the construction of the girders, the curing process, and the structural parameters that were determined over time. Information recorded initiates with the properties of the concrete in each of the girders. The batch plant required eighteen production castings to complete all of the girders. Several 4 in. x 8 in. cylinders were also cast at the time of pouring and thermocouples were embedded in the girders in order to maintain the cylinder temperatures to within 5 °F of the girder temperature. Every girder was prestressed with forty-two 0.6 in., 270 ksi, low-relaxation steel strands. Twenty-eight of the strands in the bottom flange were straight strand, and ten of them were bonded for 48 in. at each end. Additionally, strain gauges were installed in all five girders of one of the interior spans, which is the span considered for all of the calculations and is considered a typical span. The data from each of the strain gauges was continuously monitored over time and compared to calculated values. Additionally, the camber was checked at an average concrete age of 200 days.

The results of all of the described measurements indicated that the Field measurements corresponded well to values calculated with measured material properties. The measured mid-span prestress losses were, on average, 14 percent greater than the predicted prestress losses using the measured concrete properties. Additionally the researchers concluded that “Current analytical techniques can result in accurate predictions of camber and prestress losses for HPC girders if the material properties used in the analysis are representative of the actual concrete used in girder production.”
2.2. Topic 2

Effects of Temperature Variations on Precast, Prestressed Concrete Bridge Girders

This journal article by Barr et al. (2005) focuses on the effects of changes in temperature on precast, prestressed concrete bridge girders. The authors provide a brief background on the design of precast, prestressed concrete girders. They continue to explain that the working limits for flexure typically exceeded in the bottom flange of the girders, however designers typically neglect the effects of temperature. The authors focus their research on explaining the observations of temperature variations on precast, prestressed concrete bridge girders.

The research results were based on a three span bridge in Washington. Vibrating-wire strain gauges were embedded in five bridge girders in two different spans as well as a test girder. The gauges were embedded at mid-span as well as 5 feet from the end of the girder. These sensors were used to measure strain and temperature effects.

Precast fabricators will usually heat the concrete during the curing process in order to achieve a faster release strength in the concrete. It was observed that high curing temperatures cause prestress losses and camber losses. These losses were observed due to three different processes, which include: heating the prestressing strand while its length is fixed between the abutments, having a coefficient of thermal expansion of the concrete which exceeds that of the prestressing steel, and a significant temperature distribution in the girders as the concrete hardens. The authors provided details and findings of these three implications.

The authors then concluded with an explanation of the consequences of elevated curing temperatures, variations in service temperatures, and of exceeding stress limits. The
findings are that heating the strand before a bond to the concrete is achieved results in an increase in bottom flange tension by up to 26%. The strand stress was also monitored to have dropped by as much as 12.1 ksi, which is a 40% decrease to the expected camber dimensions. The inclusion of temperature gradients in the AASHTO calculations will significantly increase the predicted service tensile stress in the bottom flange of concrete girders. The prestress needed in girders is increased through the inclusion of temperature gradients in order to satisfy the code working stress limits.

In conclusion, the girders that were studied in this analysis experienced a 3 to 7% decrease in calculated prestressing stress, a 26 to 40% reduction in initial camber, and a 12 to 27% increase in bottom flange tension stress.

\textit{The Effect of Temperature Variations on the camber of precast, prestressed concrete girders}

The prediction of girder camber at a particular time is difficult because it depends on temperature variations within the cross section of the girder, as well as concrete properties and prestress losses. Several individuals have previously monitored camber variations up to 0.63 in. for prestressed concrete girders due to temperature variations throughout a single day. This research article by Nguyen et al. (2015) discusses a study performed on two prestressed concrete girders that were instrumented to quantify daily temperature variations within the girders. This was necessary in order to develop an understanding of the effects of temperature variations on camber, and to calibrate a
theoretical model for predicting thermal camber. Internal temperatures were recorded at fifteen locations along the mid-span every minute.

Through principles of mechanics and an assumed value of the coefficient of thermal expansion, the thermal camber can be predicted. The curvature of a strain profile along the girders cross section can be determined based on several common assumptions as well. Through the use of these assumptions, this study was able to determine two methods to derive the thermally induced camber of prestressed concrete, I girders. The first method was derived based on temperature profiles throughout the day that the two girders were analyzed. The second model was derived based on the assumption of peak temperatures within a 24 hour period and that linear temperature variations existed throughout the girder. These two models were then compared to the actual data that was recorded and a calculation of an error using the root mean square approach was performed.

It was concluded that both the temperature history camber model and the peak temperature camber model had root mean square average camber errors over time of about 0.1 inch. With this minor error there would not be any significant affect to the installation of girders onsite. The camber prediction models also depend on the amount of exposure to sun at various points of the day. One of the girders in this study was exposed to the sun along the web and bottom flange, while the second girder only experienced sun exposure to the top flange of the girder. These two camber prediction models are more accurate when the girder is only exposed to sunlight across the top flange, thus an edge girder may need additional calculations and studies. The temperature profile in a concrete girder can be highly nonlinear during the afternoon, with most of the temperature variations occurring
within the top flange. Thus the model derived based on temperature profiles was found to be more accurate than that based on peak temperature differences.

*Behavior of prestressed concrete bridge girders due to time dependent and temperature effects*

Time dependent effects such as creep, shrinkage, and ambient temperature significantly affect the service behavior of prestressed concrete bridges. The long-term serviceability, durability, and stability of a bridge are reduced due to these time-dependent effects, which cause additional induced forces internally. This study by Debbarma et al. (2011) is based on field data that was collected on two prestressed concrete box girders. The development of strain in the girder due to creep, shrinkage, and atmospheric temperature was recorded and analyzed. Additionally, this study presents the effects caused due to thermal stresses from daily temperature changes in two prestressed concrete box girders. These structural effects and changes are then compared to two commonly used predictive methods presented in the, ACI209R-92 and CEB-FIP 90 specifications.

The amount of shrinkage and creep were predicted using the two previously mentioned predictive methods. The two methods demonstrate that these predicted values are maximum around 350 days after pouring the girders, and the value then remains at the maximum value after that point. These values were then compared to the measured values for creep and shrinkage along the soffit slab, web slab, and deck slab of the two box girders. After about 800 days, the analysis was terminated and it was concluded that the field values shows development of maximum strains around 350 days after pouring, and there after
strain rates are reduced before becoming almost constant. Additionally, after 390 days from the time of pouring the deflection due to creep and shrinkage for the girders was observed to produce a maximum deflection of 0.22 in. (5.6 mm).

Concrete has a poor thermal conductivity, and a nonlinear temperature distribution arises across the cross section of the girder due to daily temperature changes. The temperature and strains were measured for this study using vibrating wire strain gauges embedded at the mid span of the girder. It was observed that a temperature increase resulted in the development of compression in the soffit slab and tension in the deck slab. It was also observed that with an increase of only 5 °C in the ambient temperature during the day time, resulted in an increase in girder deflection of 0.374 in. (9.5 mm) at the mid span. An additional cantilever style box girder bridge was analyzed for deflections due to thermal stress. This particular bridge contains a cantilever section that is 192.5 ft (58.7 m) long and at the end experienced a max deflection of 0.96 in. (24.5 mm).

It was concluded that predicted as well as the measured time dependent effects such as creep, shrinkage, and ambient temperature fluctuations can dramatically alter the serviceability of concrete bridges. It was also determined that after only 15 years of service, the observed bridges will need to be strengthened with external prestressing. This study also concluded with the statement that additional research needs to be performed to develop smart concrete structures, such as replacing a percentage of conventional steel reinforcement with a superelastic alloy. This would require laboratory scale experiments using superplastic alloys to overcome the time dependent and atmospheric temperature effects in concrete girders to enhance the service life of concrete bridges.
CHAPTER 3
LONG-TERM DATA MONITORING

3.1. Bridge Description

Accelerated Bridge Construction (ABC) techniques are being more frequently implemented throughout the United States due to their short construction time. However, long-term performance of these bridges is scarce. This study focuses on a bridge located in Nibley, Utah, which is about 5 miles south of Logan, Utah. The bridge was constructed near 2600 south and State Highway 165. It crosses over the Blacksmith Fork River, which is a tributary for the Logan River and carries spring runoff from Blacksmith Fork Canyon. This canyon is located 6 miles to the south of the bridge location. The purpose of the bridge is to manage the traffic flow from state Highway 165 to the newly constructed Ridgeline High School. The specific coordinates of the bridge are 41°41.8’41”N Latitude and 111°49.56’42”W Longitude. Fig. 3-1 shows an image of the bridge with the Ridgeline High School in the background. This photo was taken on June 30, 2016.

![Fig. 3-1. A view of the north side of the bridge](image-url)
The Nibley Bridge contains two lanes of traffic, one lane of traffic for each direction of flow. The Average Daily Traffic (ADT) is expected to be low because the majority of daily traffic will consist of school buses and passenger vehicles for students and staff of the high school. Fig. 3-2 shows a plan view of the bridge with all ten girders depicted. The length of the girders is 89.5 ft. (27.28 m) measured from end to end. The girders have bearings as supports and are 88 ft. (26.82 m) apart. The abutment faces measure 91 ft. (27.74 m) to the outside face (out to out) and 85 ft. (25.91 m) to the inside face (in to in). The overall width as measured from the outside of each barrier is 60 ft. (18.3 m) and as measured to the inside of each barrier is 57 ft. 2 in. (17.4 m). The bridge was designed as a single span, and the superstructure consists of deck bulb tee girders with fixed in place abutments at each end. There is no skew or super elevation in the overall dimensions of this bridge.

Fig. 3-2. Plan view of the Nibley bridge
Each girder is 89.5 ft (27.3 m) long with a height of 3 ft 6 in (1.1 m). The girder dimensions are shown in Fig. 3-3 below. The bridge contains 10 girders total (G1 through G10 in Fig. 3-2), which were all precast during December 2015 in Salt Lake City, Utah. The girders cured in the casting yard for approximately six weeks prior to shipping to the jobsite. The prestressing strand for each girder was designed using harp points located at 0.4L, 35 feet 10 in. (10.9 m), from each end of the girders. A total of 34 strands were used in each girder, 10 of which were harped and the remaining 24 strands remained straight throughout the bottom flange of the girders. The centroid for all strands at the mid-span is 3-3/4 in. (9.52 cm) from the bottom. At the girder ends, the centroid of the strands measured from the bottom is 3 feet (0.9 m) for the harped strands and 2-7/8 in. (7.3 cm) for the straight strands. The combined centroid of the harped strands and the straight strands at the girders end is 12.6 in. (32.05 cm) from the bottom of the girder. The prestressing strands are 0.6 in. (1.5 cm) diameter low relaxation cables, which have a specific yield of 270 ksi (1862 MPa). The final jack prestressing force was 1260.1 kips (5.61 MN), which is a stress of 202.5 ksi (1397 Mpa).

The bridge deck was cast integral with the girders. It is comprised of 8 in. (20.3 cm) of reinforced concrete with a future 3 in. (7.62 cm) of asphalt overlay. Grade 60 (GR60) mild reinforcing steel was used in all elements of the structure, and is reinforced with size 5 steel bars. The integral abutments span the width of the bridge and are 2 feet 3 in. (0.7 m) thick and 9 feet (2.7 m) tall. Piles were driven at eight locations of each abutment to transfer the load from the girders. These piles are constructed out of concrete and are 12-3/4 in. (0.3 m) in diameter.
The barriers run parallel to the bridge and have a height of 3.5 feet (1.1 m). The barriers are reinforced with size 4 steel bars with specified yield of 60 ksi (413.7 MPa). Additionally, wing walls were cast adjacent to both ends of the abutments that run parallel to the length of the bridge. Fig. 3-4 shows a cross section of the Nibley Bridge.

Fig. 3-3. Cross section of bridge girders

Fig. 3-4. Nibley bridge cross section
3.2. Embedded Instrumentation

The Nibley Bridge was embedded with permanent sensors to monitor the long-term changes in temperature and strain at discrete locations throughout the superstructure. Sensors were also attached to the exterior of the girders in various places in order to monitor short-term changes (refer to Chapter 4). The embedded sensors were all placed within two of the girders (G1 and G5 from Fig. 3-2) at the mid-span as well as near the west end. It was assumed that bridge symmetry could be used for placement of the sensors and any data collected could be reasonably used to estimate global conditions. These instrumentation locations are able to provide temperature and strain changes for locations in an interior and an exterior girder. Fig. 3-5 below displays the instrument locations. Girder 1 contains instruments at Site A and Site B. Girder 5 contains instruments at Site C, Site D, and Site E. Site A and C are located 9 in. (0.23 m) from the west abutment. Site B and D are located at the respective girder mid-span, which is 44 feet 9 in. (13.64 m) from the end of the girder. Site E is located at approximately the quarter point, which is 22 feet 4.5 in (6.82 m) from the west end of the girder.

One focus of this research is on changes in temperature and prestress measured in the girders of the Nibley Bridge. In order to determine these differences, the bridge was instrumented with a total of 50 thermocouples, 16 Geokon model 4200 vibrating wire strain gauges with internal thermistors, as well as 4 velocity transducers. The velocity transducers were not be analyzed in this research, but will be reported at a later date. All of the 16 strain gauges and 50 thermocouples were embedded into the girders before casting the concrete. Fig. 3-6 displays the location of the vibrating wire strain gauges at Sites A, B, C, and D. At these instrumentation cross sections, two strain gauges were placed in the girder web
(LW and UW) and two strain gauges were placed in the girder flanges (BR and BL). These are noted in Fig. 3-6 as Upper Web (UW), Lower Web (LW), Bottom Left (BL), and Bottom Right (BR). At Sites B and D, the strain gauges denoted as BR and BL are located at the centroid of the prestressing steel. At the end of the girder, the congestion of steel prohibited the BR and BL gauges to be placed at the centroid of the prestressing steel. Therefore, they were placed at the same elevation as gauges BR and BL at Sites B and D. The strain gauges denoted as UW are located at 24 in. (61 cm) from the bottom of the girder and gauges LW are located at 17-5/16 in. (44.05 cm) from the bottom of the girder. All of the BL and BR gauges are located at 3-3/4 in. (10 cm) from the bottom of the girder and at 7 in. (18 cm) from the centerline.
Thermocouples were embedded in the girder concrete in order to measure the thermal gradient, maximum and minimum temperatures, and predict thermal camber. The thermocouples were more densely placed towards the top of the deck and the spacing gradually increased towards the bottom of the girders. Fig. 3-7 shows an image that depicts the short piece of 1 in diameter PVC pipe that was used to house the thermocouples into a cluster and keep them at the correct vertical distances at each instrumentation site. The thermocouples were spaced from the top of the girders as follows: 0.25 in., 0.5 in., 0.75 in., 1 in., 1.5 in., 2 in., 3 in., 4 in., 5.5 in., and 7 in. (0.64 cm, 1.27 cm, 1.91 cm, 2.54 cm, 3.81 cm, 5.08 cm, 7.62 cm, 10.16 cm, 13.97 cm, and 17.78 cm). Holes were drilled into the PVC pipes and the thermocouples were placed within the pipe so that 1/4 in. of wire extended through the outside of the small holes. The PVC pipe was sub sequentially filled with an epoxy compound and an epoxy glue was placed over the exposed thermocouple in order to
protect them from corrosion. At Site E, two thermocouple clusters were embedded into the girder, one near the deck surface and another near the bottom of the girder. These clusters consisted of five thermocouples, spaced the same as the top 5 thermocouples of the typical thermocouple clusters.

Fig. 3-8 below displays an image of a typical site with thermocouples as well as vibrating wire strain gauges (Site A, B, C, and D). The sensors and data-loggers were provided by Campbell Scientific. A CR1000 datalogger was initially used at each girder in order to collect the necessary data. In the spring of 2017, the CR1000 dataloggers were changed to a single CR6 datalogger in order to analyze the accelerometers more accurately.
The measurements were recorded every minute during casting, curing, and for at least two weeks after the girders were shipped to the jobsite. After this point, the measurements were recorded every fifteen minutes for several weeks, after which the measurements were changed to record every hour and on the hour.

### 3.3. Concrete Maturity

Forced temperature can be utilized to alter how mass concrete cures and influence the rate of its strength development. The concept of concrete maturity was developed in the 1950’s and continues to evolve, however the method is not without limitations. The limitations are outlined in ASTM C 1074-74 (ASTM 1987) and they include the following:
• Concrete must be maintained in a condition that permits cement hydration,

• The method does not take into account the effects of early-age concrete temperature on the long-term strength, and

• The method needs to be supplemented by other indicators of the potential strength of the concrete mixture.

Kim et al. (2008) researched the second limitation listed above and determined that “the analysis of strength development based on actual age demonstrated a clear crossover effect.” This means that if a concrete has a higher curing temperature during an early age, it would produce a lower strength at a later age when compared to the concrete strength subjected to a lower early age curing temperature. However, Kim et al. (2008) analyzed the strength development of several samples of concrete at the same curing age and determined that there were inconsistent strength estimates at later ages.

The Nibley bridge girders followed a curing program of increasing the temperature of the early age concrete with the use of steam. Both Girder 1 and Girder 5 were cured with steam application, but only when required to match the pre-determined curing temperatures. The girders for the Nibley Bridge were required to have specific concrete compressive strength at release. It was desired that the compressive strength of all girders at release be 6,000 psi (41.37 MPa) and have an ultimate strength of at least 8,500 psi (58.61 MPa). Table 3-1 displays the actual strength of the girders. Information regarding the de-tensioning procedure from the casting yard indicated that the day after casting a release break verification was performed and indicated that Girder 1 presented a compressive strength of 6,910 psi. Girder 1 was then released 5 days after casting, but the
Table 3-1. Concrete strength of Nibley bridge girders

<table>
<thead>
<tr>
<th>Girder</th>
<th>Date Poured</th>
<th>Date Released</th>
<th>Release Strength</th>
<th>7-Day Strength</th>
<th>28-Day Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12/9/2015</td>
<td>12/14/2015</td>
<td>7,890 psi (54.40 MPa)</td>
<td>8,830 psi (60.88 MPa)</td>
<td>10,940 psi (75.45 MPa)</td>
</tr>
<tr>
<td>5</td>
<td>12/7/2015</td>
<td>12/8/2015</td>
<td>6,720 psi (46.33 MPa)</td>
<td>9,280 psi (63.98 MPa)</td>
<td>11,770 psi (81.15 MPa)</td>
</tr>
</tbody>
</table>

Compressive strength was only checked 7 days after casting. Thus, a weighted average was performed and presented that the compressive strength of Girder 1 was 7,890 psi at the day of release. Additionally, the 28-day compressive strength of the concrete is assumed as a close estimate to the ultimate strength of the concrete. It is interesting to note that the ultimate strength of Girder 1 is 28.7% greater than designed and 38.4% for Girder 5.

The concrete temperature data was measured at one minute intervals during the curing process for Girder 1. All of the strain gauge thermistors in Girder 1 (Site A and Site B in Fig. 3-2) did not record the temperature due to an error in the datalogger program. Thus Fig. 3-9 displays only the thermocouple temperature data recorded from Site A and Site B for Girder 1. Individual thermocouples are not easily distinguishable in the figure, as a consequence this figure is best used to present the range of temperatures within Girder 1 during curing. The concrete for Girder 1 was poured during the early morning hours between 6:00 am and 9:00 am of December 9, 2015. During curing, the maximum temperature was 151.86 °F (66.59 °C) and occurred at thermocouple B-10 on December 10, 2015 at 5:23 am. The minimum temperature during curing was 47.32 °F (8.51 °C) and
Fig. 3-9. Thermocouple curing temperature data for Girder 1 during curing occurred at thermocouple B-1 on December 10, 2015 at 9:57 pm. The cover over the girder was removed at approximately 9:00 am on December 14, 2015, and this is when the controlled temperature cure was considered finished.

The temperature data was measured at fifteen minute intervals during the curing process for Girder 5. The upper and lower strain gauges in Girder 5 (Site C and Site D in Fig. 3-2) did not record the temperature due to an error in the datalogger program. Thus Fig. 3-10 displays all of the thermocouple and strain gauge temperature data recorded from Site C and Site D for Girder 5. The concrete for Girder 5 was poured during the morning of December 7, 2015. Unfortunately, the datalogger was not set to record during the time that the concrete was poured. During curing, the maximum temperature recorded was 142.16 °F (61.2 °C) and occurred at thermocouple D-10 on December 8, 2015 at 6:00 am.
The additional ten thermocouples at Site E of Girder 5 are displayed in Fig. 3-11. These were plotted separately from Sites C and D (Fig. 3-10) because the plot began to be overcrowded with data. Additionally, Fig. 3-11 clearly displays the difference in the recorded curing temperatures from the top five thermocouples (Site E) against the bottom five thermocouples (Site F). The peaks that occur near mid-afternoon in the figure occur in the top five thermocouples in the girder. The bottom five thermocouples do not peak in temperature to the same degree. This is to be expected since the top five are closer to direct sunlight in mid-afternoon. The bottom thermocouples experience a small change in temperature from the stream below as well. During curing, the maximum temperature recorded was 142.93 °F (61.63 °C) and occurred at thermocouple F-5 on December 8, 2015 at 6:00 am.
3.4. **Long-Term Temperature Changes**

3.4.1. **Measured Data**

Campbell Scientific in Logan, Utah provided the dataloggers and sensors necessary to record temperature and strain data over time. The recordings began on December 8, 2015 for both Girder 1 and Girder 5. The measured temperature using the embedded sensors was continuously measured starting on December 8, 2015. Yet there were a few instances that data was not recorded due to errors in the dataloggers or other complications. The temperature data recorded over time in Girder 1 is shown in Fig. 3-12 and Fig. 3-13. These figures display the temperature data recorded from the 10 thermocouples as well as the four strain gauges at each site.
Fig. 3-12. Recorded temperature data of Site A, Girder 1

Fig. 3-13. Recorded temperature data of Site B, Girder 1
The temperature data recorded for the sensors embedded in Girder 5 are displayed in Fig. 3-14, Fig. 3-15, and Fig. 3-16. The figures for Site C and Site D display the recorded data from the 10 thermocouples as well as the temperature data from the four strain gauges. Fig. 3-16 displays the temperature data from the five upper and the five lower thermocouples at Site E. On June 30, 2016 the asphalt was cast on the bridge in order to complete its construction. This date shows a large spike in the recorded temperature data for Girder 5. Additionally, due to errors in the dataloggers and other complication, there are several days that temperature data is missing from Girder 5 during the month of April 2016. Additional data is missing during the months of February and March 2017 because the dataloggers were switched in order to collect information regarding the embedded accelerometers. This information will be published at a future date.

Fig. 3-14. Recorded temperature data of Site C, Girder 5
Fig. 3-15. Recorded temperature data of Site D, Girder 5

Fig. 3-16. Recorded temperature data of Site E, Girder 5
3.4.2. Average Bridge Temperature Range by Week

The American Association of State Highway and Transportation Officials (AASHTO 2010) defines two methods,Procedure A and Procedure B, to determine the design uniform temperature ranges of bridges. Procedure A is a historic method that can be used for bridge design based on the number of freezing days per year at the bridge site. Procedure B is believed to be more accurate and provides the maximum and minimum design temperatures for concrete girder bridges with concrete decks and steel girder bridges with concrete decks. Each type of bridge contains contour maps which provide the design temperatures. In using these contour maps for concrete girders with concrete decks, the following design temperatures were determined for the Nibley Bridge: \( T_{MaxDesign} = 104 \) °F (40 °C) and \( T_{MinDesign} = -10 \) °F (-23.33 °C).

A heat flow analysis of both steel and concrete bridges in a wide range of climates was performed by Roeder (2002) based on analytical methods performed by others. Roeder further investigated and focused calculations based on extreme temperature events. They provided an equation to calculate the average temperature, Equation 3-1, which is based on equilibrium principles integrated over the girder cross section.

\[
T_{avg} = \frac{\sum A_i T_i}{\sum T_i}
\]

Equation 3-1

Where:

\( A_i \) = Cross sectional area of the ith segment

\( T_i \) = Temperature of the ith segment of the cross section
The Nibley Bridge is oriented in a mostly east-west orientation. This orientation results in different exposure to sunlight in the early morning and late evening across varying cross sections of the bridge. The bridge sensors at the different locations also confirm that the bridge experiences different temperature variation at different points of the day. Because the sensors at Site A and Site C (Fig. 3-2) are near the abutment they do not experience varying thermal effects due to the ambient shade temperature from the underside of the bridge. Additionally, there exists a sidewalk and barrier over the top of Girder 1, Whereas Girder 5 is completely exposed to the thermal conditions existing directly at the deck. Thus it is necessary to calculate the Average Bridge Temperature (ABT) at each sensor site location using Equation 3-1.

The bridge temperatures were recorded and monitored by each sensor for a period of 16 months for this study. Fig. 3-17 and Fig. 3-18 display the graphical results of the maximum and minimum ABT of Girder 1. An error occurred in the dataloggers and the data was not recorded for the end of April as well as the beginning of May. Additionally, the data acquisition system was altered in the spring 2017 and more data was lost. This however, does not affect the weekly calculations of maximum and minimum ABT. For Girder 1 the maximum ABT occurred at Site A during the week of July 17, 2016 with a magnitude of 91.79 °F (33.22 °C) and the minimum ABT occurred at Site A during the week of Jan 1, 2017 with a magnitude of 1.26 °F (-17.08 °C). Thus the $T_{MaxDesign}$ is conservative by 12.21 °F (6.78 °C) and $T_{MinDesign}$ is conservative by 11.26 °F (6.25 °C).

Fig. 3-19, Fig. 3-20, and Fig. 3-21 display the graphical results of the maximum and minimum ABT of Girder 5. The dataloggers experienced an error during the week of
Fig. 3-17. Site A maximum and minimum ABT

Fig. 3-18. Site B maximum and minimum ABT
April 10, 2016, and thus no data was recorded during this week. Additionally, the data acquisition system was altered in the spring 2017 and more data was lost. The minimum ABT at each site on Girder 5 is mostly consistent, yet the maximum ABT of Girder 5 is the highest at Site E and lowest at Site C. For Girder 5 the maximum ABT occurred at Site E during the week of July 17, 2016 with a magnitude of 103.84 °F (39.91 °C) and the minimum ABT occurred at Site C during the week of Jan 1, 2017 with a magnitude of -5.54 °F (-20.85 °C). Thus, $T_{\text{MaxDesign}}$ is conservative by only 0.16 °F (0.09 °C) and $T_{\text{MinDesign}}$ is conservative by 4.46 °F (2.48 °C).

$T_{\text{MaxDesign}}$ was not exceeded at any of the sites, but was conservative by a very small amount at Site E. The hottest temperature recorded was during the week of June 26, 2016. The uppermost thermocouple at Site E recorded a temperature of 124.90 °F (51.61°C) on June 29, 2016 at 4:00 pm. However the remaining thermocouples in the cross section measured much lower temperatures, and this time resulted in an ABT of 102.54 °F (39.19 °C), still 1.4 °F (0.81 °C) under the $T_{\text{MaxDesign}}$ limit. The reason why the ABT is lower is because the five thermocouples in the bottom of the girder recorded temperatures within the range of 84 °F (29 °C). This is likely due in part to the shade, but also the cold river water near the bottom of the bridge. Thus the ABT value is below the design temperature, but must be questioned if the bridge will experience exceeding temperatures in the future. The site of the bridge also has historically experienced mornings much colder in the winter than those recorded. It must also be asked if the bridge will experience these cold temperature in the future.
Fig. 3-19. Site C maximum and minimum ABT

Fig. 3-20. Site D maximum and minimum ABT
3.4.3. Prediction of Average Bridge Temperature

In order to ensure that the bridge will not exceed the $T_{MaxDesign}$ and $T_{MinDesign}$ as defined previously, predictive methods to quantify the maximum ABT and minimum ABT were developed. In 2002, Charles W. Roeder (2002) described two methods [the Kuppa Method (Kuppa et al.1991) and the Emerson Method (Emerson 1976)] that can be used to estimate the average temperature. The accuracy of these methods were compared to the measured Average Bridge Temperature (ABT) for the Nibley Bridge. The Kuppa Method uses one equation for the maximum ABT for steel bridges and one for the minimum ABT for all concrete bridges. The Emerson Method uses a single equation to determine the minimum ABT and a related constant that is then used to calculate the maximum ABT as a function of the minimum.
Roeder (2002) provided calculations to derive Equation 3-1, which included all thermal properties that a bridge may experience including conduction, convection, and radiation heat transfer. Kuppa et al. provided by Roeder (2002) had access to many US sites that contained complete weather data, and their calculations considered actual air temperature, cloud cover, precipitation, and wind velocity. These calculations suggest that the maximum ABT depends on the four day averages of the high air temperatures and likewise for the minimum ABT subject to the low air temperatures. That is, the extreme maximum ABT, Equation 3-2, depends upon the average high air temperatures for four consecutive days in the hottest part of the summer. The extreme minimum ABT, Equation 3-3, depends on the average of the low air temperature for four consecutive days in the coldest part of the winter. For concrete bridges with concrete decks the Kuppa Method (Roeder 2002) can predict bridge temperatures by the use of Equation 3-2 and Equation 3-3, which are based on temperatures in °F.

\[
T_{AvgMax} = \frac{T_{MaxAir1} + T_{MaxAir2} + T_{MaxAir3} + T_{MaxAir4}}{4} 0.953 + 4.6 \tag{Equation 3-2}
\]

\[
T_{AvgMin} = \frac{T_{MinAir1} + T_{MinAir2} + T_{MinAir3} + T_{MinAir4}}{4} 1.186 + 17.24 \tag{Equation 3-3}
\]

Where:

\(T_{AvgMax}\) = Maximum average bridge temperature

\(T_{MaxAir}\) = Maximum air temperature of the hottest days

\(T_{AvgMin}\) = Minimum average bridge temperature

\(T_{MinAir}\) = Minimum air temperature of the coldest days
The Emerson Method (Roeder 2002) was based on a correlation between the measured daily minimum ABT and the mean of the measured low temperatures from the early morning and the previous day’s high temperature in the shade, for a two day period. $T_{AvgMin}$ was then associated with the two day average of the early morning low and the previous day’s high temperature from the shade through an empirical equation given as Equation 3-4, which is based on temperatures in °F.

$$T_{AvgMin} = \frac{T_{MaxAir1} + T_{MaxAir2} + T_{MinAir1} + T_{MinAir2}}{4} + 1.14 + 10.96$$  \hspace{1cm} \text{Equation 3-4}

The minimum ABT occurs in the early morning because the bridge is approaching a state of thermal equilibrium. Emerson provided by Roeder (2002) estimated that the maximum ABT could be obtained by adding a temperature range to the minimum value for that day. The maximum ABT by the Emerson Method (Roeder 2002) is dependent upon the type of bridge, the season of the year, and the cloud cover. Table 3-2 displays the maximum daily temperature ranges for concrete bridges based on the Emerson Method. This method is based upon temperature in the shade rather than weather data at a weather station. The air in the shade, such as under a bridge, is sheltered and has less extreme variations than normal air temperatures. Thus, the Emerson Method will always overestimate the magnitude of bridge movements if normal air temperatures are used.

An additional third method was established to predict the maximum ABT and the minimum ABT. This method comes from research performed at Utah State University (USU) and is titled the ERL Method by Edyson Rojas Lopez (Rojas 2014). Rojas performed a study based on thermal prediction of a bridge in Perry, Utah and a bridge near
Table 3-2. Maximum daily temperature ranges by the Emerson Method

<table>
<thead>
<tr>
<th>Season</th>
<th>Clear and Sunny</th>
<th>Cloudy, but not overcast</th>
<th>Overcast / rain, snow</th>
</tr>
</thead>
<tbody>
<tr>
<td>Winter</td>
<td>5.4 (3)</td>
<td>1.8 (1)</td>
<td>0 (0)</td>
</tr>
<tr>
<td>Spring/Autumn</td>
<td>10.8 (6)</td>
<td>5.4 (3)</td>
<td>1.8 (1)</td>
</tr>
<tr>
<td>Summer</td>
<td>10.8 (6)</td>
<td>7.2 (4)</td>
<td>3.6 (2)</td>
</tr>
</tbody>
</table>

Sacramento, California. The Perry Bridge was a precast, prestress concrete I girder bridge, The California Bridge was a cast in place, post-tensioned, box girder bridge. Through his research he determined that the Kuppa and Emerson Methods predict well the maximum ABT. However, the two methods consistently underestimate the minimum ABT by at least 15.45 °F (8.58 °C) and up to 30.31 °F (16.84 °C) based on the two bridges.

The ERL Method is an empirical equation based on the maximum air temperature of the hottest day and the previous day, as well as the average of the minimum air temperature from that day and the previous day to calculate the maximum ABT. The minimum ABT is based on the minimum air temperature of the coldest day and the previous day as well as the average of the maximum air temperature from that day and the previous day. As Rojas (2014) calibrated this equation he realized that the maximum ABT for both bridges was the same. However, the minimum ABT for the California and Utah bridges was different. Thus he determined separate factors for I girder and box girder bridges. The ERL maximum ABT is presented as Equation 3-5 and the minimum ABT based on the ERL method for concrete girder bridges is presented as Equation 3-6.
\[ T_{\text{AvgMax}} = \frac{T_{\text{MaxAir}_1} + T_{\text{MaxAir}_2}}{2} 1.32 - \frac{T_{\text{MinAirMax}_1} + T_{\text{MinAirMax}_2}}{2} 0.20 - 12.68 \quad (°F) \]

Equation 3-5

\[ T_{\text{AvgMin}} = \frac{T_{\text{MaxAirMin}_1} + T_{\text{MaxAirMin}_2}}{2} 0.44 + \frac{T_{\text{MinAir}_1} + T_{\text{MinAir}_2}}{2} 0.67 - 8.64 \quad (°F) \]

Equation 3-6

Where:
- \( T_{\text{AvgMax}} \) = Maximum average bridge temperature
- \( T_{\text{AvgMin}} \) = Minimum average bridge temperature
- \( T_{\text{MaxAir}_1} \) = Maximum air temperature of the hottest day
- \( T_{\text{MaxAir}_2} \) = Maximum air temperature of the day before the hottest day
- \( T_{\text{MinAirMax}_1} \) = Minimum air temperature of the hottest day
- \( T_{\text{MinAirMax}_2} \) = Minimum air temperature of the day before the hottest day
- \( T_{\text{MaxAirMin}_1} \) = Maximum air temperature of the coldest day
- \( T_{\text{MaxAirMin}_2} \) = Maximum air temperature of the day before the coldest day
- \( T_{\text{MinAir}_1} \) = Minimum air temperature of the coldest day
- \( T_{\text{MinAir}_2} \) = Minimum air temperature of the day before the coldest day

3.4.4. Yearly Prediction of Average Bridge Temperatures

The ambient temperature database from the National Oceanic and Atmospheric Administration (NOAA 1970-2017) was accessed in order to obtain historic temperature data for modeling temperature patterns near the bridge site. The nearest location with
significant recorded data was from the USU experimental farm, which is 3 miles to the west of the bridge site. This location contained daily maximum and minimum temperature data starting at the year 1970. Using this data and the three predictive temperature models derived previously, a model of the behavior of the ABT could be reasonably determined. The results of the yearly maximum and minimum ABT as determined by the three predictive methods are compared to the AASHTO $T_{\text{Max Design}} = 104^\circ\text{F} (40\ ^\circ\text{C})$ and $T_{\text{Min Design}} = -10^\circ\text{F} (-23.33\ ^\circ\text{C})$ for the Nibley bridge.

Fig. 3-22 compares the yearly maximum recorded temperature, $T_{\text{Max Design}}$, and the predicted maximum ABT from the three predictive models described. $T_{\text{Max Design}}$ was never exceeded by the recorded temperature, but was exceeded several times using the Emerson Method and the ERL Method. The Kuppa Method never exceeded the AASHTO limit and is consistently within a few degrees of the recorded maximum temperature. It is also interesting to note that the values for the Kuppa Method are consistently slightly lower than the recorded temperature. In order for this small difference in temperature, the Kuppa Method could be calibrated and the constant in Equation 3-2 could be recalibrated to better fit the data. Additionally, the maximum ABT according to the Emerson Method is drastically different than the other two models. This demonstrates the high variability of the Emerson Method in calculating the yearly Maximum ABT.

Fig. 3-23 compares the minimum recorded temperature, $T_{\text{Min Design}}$, and the predicted minimum ABT from the three predictive models described. $T_{\text{Min Design}}$ was exceeded by the recorded temperature as well as all three of the predictive models. The values for the Kuppa Method are consistently slightly higher than the recorded temperature.
Meanwhile, the ERL Method is consistently within a few degrees of the recorded minimum temperature. In order to reduce this small difference in temperature, the ERL Method could be calibrated and the constant in Equation 3-6 could be recalibrated to better fit the data. Additionally, the measured values are extreme in this comparison because they are ambient air temperature. The results of an ABT during these years would likely be much less excessive if in bridge temperatures were available.

It can be concluded that based on the historical temperature data, the minimum ambient temperature was exceeded on multiple occasions since 1970. The historical data of 1990 measured the lowest temperature on record of -44 °F (-42.3 °C). At such a drastically low temperature it can be concluded that the $T_{\text{MinDesign}}$ would likely be exceeded and that the AASHTO LRFD Bridge design specification should be revised.
3.4.5. Weekly Prediction of Average Bridge Temperatures

It was decided to use weekly maximum and minimum ambient temperatures from the NOAA database and compare to the ABT measured from the bridge sensors. The results of this comparison are presented in the following plots, which display the three predictive methods compared to each girder separately. Fig. 3-24 displays a comparison of $T_{AvgMax}$ of the three temperature predictive methods and the maximum measured ABT of Girder 1. The minimum measured ABT of Girder 1 and the three values for $T_{AvgMin}$ of each week are displayed in Fig. 3-25. A comparison of $T_{AvgMax}$ from the three temperature predictive methods and the maximum measured ABT of Girder 5 is displayed in Fig. 3-26. The minimum measured ABT of Girder 5 and the three values for $T_{AvgMin}$ of each week are displayed in Fig. 3-27.
Fig. 3-24. Predictive temperature methods and maximum measured ABT for Girder 1

Fig. 3-25. Predictive temperature methods and minimum measured ABT for Girder 1
Fig. 3-26. Predictive temperature methods and maximum measured ABT for Girder 5

Fig. 3-27. Predictive temperature methods and minimum measured ABT for Girder 5
For both Girder 1 and Girder 5 it is not easily noticeable which predictive method best matches the measured temperatures. It is apparent however, that the Emerson method continuously provides a warmer measurement than the Kuppa and ERL methods for the minimum ABT. In order to provide a more accurate summary of the method that best matches the data, a statistical analysis needs to be performed. The quantified evidence of the measured and predicted data is obtained by means of the Mean Squared Error (MSE) and the Coefficient of Determination, also known as the correlation coefficient. The correlation coefficient formula is presented below as Equation 3-7 and the MSE is presented as Equation 3-8.

\[
\begin{align*}
    r &= \frac{\sum_{i=1}^{n}((x_i - \bar{x})(y_i - \bar{y}))}{\sqrt{\sum_{i=1}^{n}(x_i - \bar{x})^2(y_i - \bar{y})^2}} \quad \text{Equation 3-7} \\
    MSE &= \frac{\sum_{i=1}^{n}(x_i - y_i)^2}{n} \quad \text{Equation 3-8}
\end{align*}
\]

Where

\[x_i\] = Measured ABT for a particular week
\[y_i\] = Predicted ABT for a particular week
\[n\] = Total number of weeks
\[\bar{x} = \frac{\sum_{i=1}^{n}x_i}{n}\]
\[\bar{y} = \frac{\sum_{i=1}^{n}y_i}{n}\]

The correlation coefficient and the MSE were calculated for each predictive method with respect to the measured average girder temperatures. Table 3-3 displays this comparison for Girder 1 and Table 3-4 displays this data for Girder 5. The correlation
 coefficient is shown as a percentage. From the calculations of the MSE, it can be concluded that the Kuppa method contains the least amount of error for the maximum ABT for Girder 1, and the ERL method contains the best match for the minimum ABT for Girder 1. The values of the correlation coefficient for Girder 1 indicate that all of the method would fit a regression line well within the data. Additionally, it can be concluded that the Kuppa method contains the least amount of error for the maximum ABT, and the ERL method contains the least amount of error for the minimum ABT for Girder 5. It is interesting to note that the correlation coefficient values for both girders contain a similar trend for each category for ABT. Thus, it can be concluded that the MSE is the method to use when comparing the ABT predictive models.

Table 3-3. Correlation coefficient and MSE for Girder 1

<table>
<thead>
<tr>
<th>Method</th>
<th>Maximum ABT</th>
<th></th>
<th>Minimum ABT</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>r (%)</td>
<td>MSE</td>
<td>r (%)</td>
<td>MSE</td>
</tr>
<tr>
<td>Kuppa</td>
<td>96.76</td>
<td>21.17</td>
<td>96.01</td>
<td>39.33</td>
</tr>
<tr>
<td>Emerson</td>
<td>97.12</td>
<td>45.72</td>
<td>96.01</td>
<td>86.05</td>
</tr>
<tr>
<td>ERL</td>
<td>97.10</td>
<td>34.69</td>
<td>96.20</td>
<td>31.30</td>
</tr>
</tbody>
</table>

Table 3-4. Correlation coefficient and MSE for Girder 5

<table>
<thead>
<tr>
<th>Method</th>
<th>Maximum ABT</th>
<th></th>
<th>Minimum ABT</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>r (%)</td>
<td>MSE</td>
<td>r (%)</td>
<td>MSE</td>
</tr>
<tr>
<td>Kuppa</td>
<td>96.59</td>
<td>23.91</td>
<td>95.85</td>
<td>45.19</td>
</tr>
<tr>
<td>Emerson</td>
<td>96.68</td>
<td>32.78</td>
<td>95.83</td>
<td>93.55</td>
</tr>
<tr>
<td>ERL</td>
<td>96.63</td>
<td>30.76</td>
<td>95.99</td>
<td>28.09</td>
</tr>
</tbody>
</table>
3.5. **Temperature Gradients**

Changes in ambient temperatures around a bridge cause nonlinear temperature gradients to develop throughout a bridge resulting in flexural deformation, axial and lateral stresses, and moments. The LRFD Bridge Design Specification (2010) of the American Association of State Highway and Transportation (AASHTO 2010) has defined a method to determine the design temperature gradient to be applied to a bridge. The AASHTO method divides the United States into four regions with a value of $T_1$ and $T_2$ at each region, which defines the design positive temperature gradient. The Nibley Bridge is located in region 1 of the AASHTO LRFD Bridge Design Specification (2010), which would define $T_1$ as 54 °F ($30 \, ^\circ$C) and $T_2$ as 14 °F ($7.78 \, ^\circ$C). The negative values of $T_1$ and $T_2$ for the design negative gradient are obtained by multiplying the values for the positive temperature gradient by -0.30 for plain concrete decks, and -0.20 for decks with asphalt overlay.

The temperature gradient as defined by the LRFD specification of AASHTO is displayed in Fig. 3-28. For concrete bridges $T_1$ is applied at the top of the cross-section and decreases linearly to $T_2$ over the first 4 in. (10.16 cm). The temperature gradient decreases linearly from $T_2$ to zero over a distance defined as $A$. This value is different for steel and concrete sections. For concrete structures $A$ is a function of the depth, where $A$ is 12 in. (30.48 cm) for superstructures that are 16 in. (40.64 cm) or deeper. The superstructure of the Nibley Bridge is greater than 16 in. (40.64 cm), thus the value for $A$ is 12 in. (30.48 cm). The LRFD Bridge Design Specifications (2010) denotes that $T_3$ should be 0 °F (0 °C), unless a site-specific study determines a more appropriate value, yet the value should never exceed 5 °F (2.78 °C).
An additional temperature gradient that has been proposed for bridge design was developed by Priestley (1978), displayed in Fig. 3-29. Roeder (2002) illustrated that the design gradient proposed by Priestley (1978) is a fifth-order curve decreasing from a maximum gradient temperature $T_0$ at the top of the deck to zero at a depth of 47.2 in. (1,200 mm). Additionally the temperature gradient was proposed to contain a linear increase in temperature over the bottom 7.9 in. (200 mm). However, the Nibley bridge girders only reach an overall depth of 42 in. (1,066.8 mm). Thus, Priestley proposed that the two components of the temperature gradient be superimposed in such a situation (Priestley 1978). As such, a comparison of the overall depth was used instead of the full depth of 1,200 mm recommended by Priestley. The proposed equation is presented as Equation 3-9, where $y$ is the depth of the section (the original equation would use a depth of 1,200 mm).

The standard value for $T_0$ is 57.6 °F (30 °C) for a bridge without an asphalt overlay and 30.17 °F (16.76 °C) when there is a 3 in. (76.2 mm) overlay present. At the bottom of the
Fig. 3-29. Positive design gradient proposed by Priestley (1978)

section a linear gradient is applied over a height of 8 in. (200 mm), which changes from
2.7 °F (1.5 °C) and terminates at zero. Fig. 3-29 displays the Priestley gradient with and
without a 3 in. (76.2 mm) asphalt overlay.

\[ T(y) = T_0 \left( \frac{y}{\text{depth}} \right)^5 \]  

Equation 3-9

3.5.1. Measured Temperature Gradients

The embedded sensors through the cross-section of the Nibley Bridge were used to
form temperature profiles from which the temperature gradients were extracted. The
orientation of the thermocouples and the strain gauges are as defined in Section 3.2. The
positive temperature gradient was defined as the measured sensor temperature at the top of
the section minus the minimum measurement in the web. The negative temperature
gradient was similarly defined as the minimum temperature reading at the top of the cross section minus the maximum measured temperature within the web.

It was observed that Girder 5 experienced several particularly large gradients in the month of May. This is an interesting observation rather than having the largest gradients occurring in July, which is the hottest month of the year. Fig. 3-30 displays the maximum positive temperature gradient measured within the bridge. This gradient occurred on May 13, 2016 at 2:00 pm. It was concluded that the bridge experienced these particularly large gradients during May because the top asphalt layer had not yet been installed over the bridge. Additionally, Fig. 3-31 displays the measured temperature history during the large gradient of May 13 at bridge site D (refer to Fig. 3-2). From this figure, it can easily be seen that the difference in the uppermost and second thermocouple is much larger than the differences in successive thermocouples. This difference in temperature is because the top

![Fig. 3-30. Girder 5 maximum positive temperature gradient without asphalt](image_url)
Fig. 3-31. Recorded temperature values at Site D during time of largest gradient

thermocouple is at the very top of the girder, and was experiencing direct sunlight before the application of the asphalt.

An additional source to measure the temperature gradients are the measurements found at Site E. Site E contains five thermocouples in the uppermost portion of the girders and five thermocouples in the lowest portion of the girders. Fig. 3-32 displays the maximum positive temperature gradient of Girder 5 with the addition of data from Site E. The temperature data in the five uppermost thermocouple was average between Sites C, D, and E. The temperature data for the web was averaged between the thermocouples and the strain gauge thermistors at Sites C and D. Note that the overall gradient using this approach is about 11.9 °F (6.6 °C) less than the overall temperature gradient considering only Site D on the afternoon of May 13, 2016.
The largest temperature gradient of Girder 5 following the application of the asphalt occurred on July 14, 2016 at 5:00 pm. This temperature gradient compared to both the AASHTO gradient and the Priestley gradient with 3” asphalt overlay is displayed in Fig. 3-33. The timing of this gradient is to be expected since July is typically the hottest month of the year based on the historical data from NOAA. Note the difference in comparing the gradient measurements as well as the code gradients with and without a 3 in. (7.62 cm) asphalt overlay. The sun and warmer temperatures cause a more drastic effect on the concrete girders before the asphalt is applied.

The temperature gradients of Girder 1 are not affected by an application of asphalt because this is an edge girder and is covered by a sidewalk (refer to Fig. 3-4). The largest positive temperature gradient of Girder 1 occurred at Site C on March 10, 2016 at 2:15 pm.
Fig. 3-33. Girder 5 maximum positive temperature gradient with asphalt

Fig. 3-34 displays the maximum positive temperature gradient of Girder 1. Note that the uppermost thermocouple and the second thermocouple recorded a temperature difference of nearly 10.8 °F (6 °C). This is an interesting observation, and could possibly be due to direct sunlight on the deck of the bridge.

The Priestley Method cannot be used to design for a negative temperature gradient, the AASHTO LRFD Bridge Design Specification (2010) must be used for negative temperature gradient design. Fig. 3-35 displays the maximum measured negative temperature gradient as well as the AASHTO Design Code gradient. The maximum negative gradient was measured on January 9, 2017 at 9:00 am at a temperature difference of -13.28 °F (-7.38 °C). On February 6, 2016 the abutments of the bridge were poured and caused the bridge to experience a negative gradient of 32.9 °F (18.3 °C) (Powelson 2017).
Fig. 3-34. Girder 1 maximum positive temperature gradient

Fig. 3-35. Maximum measured negative temperature gradient
The maximum and minimum measured temperature gradients were considered for each of the five sites of the bridge. Table 3-5 displays the measured temperature gradients before the asphalt was laid on the deck. Table 3-6 displays the measured temperature gradients after the asphalt was laid on the deck.

Table 3-5. Measured temperature gradients before the application of asphalt

<table>
<thead>
<tr>
<th>Site</th>
<th>Max Gradient, °F (°C)</th>
<th>Date Measured</th>
<th>Min Gradient, °F (°C)</th>
<th>Date Measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>34.06 18.92</td>
<td>3/10/2016 14:15</td>
<td>-13.28 -7.38</td>
<td>1/9/2017 9:00</td>
</tr>
<tr>
<td>B</td>
<td>46.64 25.91</td>
<td>3/10/2016 14:15</td>
<td>-12.29 -6.83</td>
<td>1/9/2017 8:00</td>
</tr>
<tr>
<td>D</td>
<td>55.85 31.03</td>
<td>5/13/2016 14:00</td>
<td>-15.04 -8.35</td>
<td>3/28/2016 2:30</td>
</tr>
<tr>
<td>E</td>
<td>48.53 26.96</td>
<td>6/29/2016 16:00</td>
<td>-12.51 -6.95</td>
<td>12/27/2015 8:00</td>
</tr>
</tbody>
</table>

Table 3-6. Measured temperature gradients after the application of asphalt

<table>
<thead>
<tr>
<th>Site</th>
<th>Max Gradient, °F (°C)</th>
<th>Date Measured</th>
<th>Min Gradient, °F (°C)</th>
<th>Date Measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>34.06 18.92</td>
<td>3/10/2016 14:15</td>
<td>-13.28 -7.38</td>
<td>1/9/2017 9:00</td>
</tr>
<tr>
<td>B</td>
<td>46.64 25.91</td>
<td>3/10/2016 14:15</td>
<td>-12.29 -6.83</td>
<td>1/9/2017 8:00</td>
</tr>
<tr>
<td>C</td>
<td>28.2 15.66</td>
<td>7/13/2016 18:00</td>
<td>-6.07 -3.37</td>
<td>12/19/2016 10:00</td>
</tr>
<tr>
<td>D</td>
<td>38.38 21.32</td>
<td>7/14/2016 17:00</td>
<td>-9.63 -5.35</td>
<td>1/6/2017 9:00</td>
</tr>
<tr>
<td>E</td>
<td>40.84 22.69</td>
<td>7/14/2016 17:00</td>
<td>-12.11 -6.73</td>
<td>1/6/2017 12:00</td>
</tr>
</tbody>
</table>
It is interesting to note that Girder 5 experienced a considerably smaller temperature gradient after the asphalt was laid, and thus it can be said that the asphalt serves to shadow the thermocouples. Additionally, a negative gradient of -15.04 °F (-8.35 °C) was experienced by Girder 5 on March 28, 2016. However, this gradient was not considered as part of the maximum gradients due to construction performed on the bridge at this date.

3.5.2. Temperature Gradient Analysis by use of Finite Elements

Computer models were used in order to obtain an additional analysis compared to measured thermal stresses and cambers. Initially, a model of a simple girder of the bridge was analyzed using SAP 2000 version 18. In addition, a finite-element model of the entire Nibley Bridge was analyzed using CSiBridge 2015 version 17.1.1. A model of a single Girder was necessary from SAP 2000 in order to verify correct results in CSiBridge. The model created in SAP 2000 utilized Area, Solid, Extrude, and Replicate tools to produce the girder. The analysis of the entire bridge was created using Layout line, Offset, and pre-defined bridge element tools in CSiBridge. The deflections of these two analyses are also compared to numerical models, described in Section 3.6.1 and 3.6.3. These models are used to determine the thermal camber of a structural element based on measured girder temperatures as well as historical ambient temperature.

It was determined that a particular type of model was necessary in order to assign the full range of temperature gradients to the computer analyses. Both SAP 2000 and CSiBridge contain different tool sets that allow the user to apply a predefined temperature gradient that can be assigned to the girder’s cross section, however the temperature gradient is limited to the AASHTO gradient. Thus, it was necessary to use a more detailed analysis
approach to model the cross section with the entire temperature gradient as shown in Fig. 3-36. The cross section of a girder was created using incremental rectangles and polygon tools to create an Area element within SAP 2000. This Area element was then extruded to the defined girder length in order to create a group of Solids. A temperature load was then assigned to the Solids according to the maximum temperature gradient from May 13, 2016 (defined in Section 3.5.1). The stresses at the mid-span of Girder 5 due to the maximum thermal gradient from May 13 is also displayed in Fig. 3-36. This group of Solids with varying temperature assignment was then replicated along the length of the beam in order to create the model at 88 feet long. For this model it was decided to increment the solids every foot of the length of the beam. An image of the model analyzed in SAP is shown in Fig. 3-37.

CSiBridge contains several preloaded structural elements for an easier analysis. The Nibley Bridge contains ten girders that were modeled as simply supported beams adjacent to one another. The first step was to create a Layout line and then offset a distance for two lanes of the bridge. Following this step, the material properties and the girder dimensions were specified. An advantage to using CSiBridge versus SAP 2000 was that the exact dimensions of the girders can be specified. Following this step the various components of the bridge were then assigned including the tendons, abutments, diaphragms, temperature loads, vehicle loads, etc. An image of the model from CSiBridge is displayed in Fig. 3-38. One disadvantage to modeling the temperature effects within CSiBridge is that the temperature gradient is limited to six discrete temperature values on the cross section of the girder, a similar analysis of the AASHTO gradient from Fig. 3-28.
Fig. 3-36. Cross section of the Girder examined in SAP 2000 with thermal stresses

Fig. 3-37. Display of Girder is SAP 2000 model
The greatest thermal gradient from May 13, 2016 was assigned to the computer models. Sap 2000 reported a maximum camber of 0.8259 in. at mid-span, while CSiBridge reported a maximum camber of 0.7531 in. at mid-span. The difference in camber is only an error of 5%, which could simply be due to the fact that SAP 2000 had to be modeled with dimensions that did not match the exact dimensions of the girders. CSiBridge provided a convenient analysis and was capable of modeling the exact dimensions of the bridge. Thus, it was decided that CSiBridge provided accurate results to model the thermal camber of the Nibley Bridge. See Section 3.6 for a full discussion of other models that can be used to determine the thermal camber of the bridge.

Fig. 3-38. Display of the Nibley bridge from CSiBridge analysis
3.6. **Camber Models**

Through principles of mechanics and an assumed value of the coefficient of thermal expansion, the thermal camber of a girder can be predicted. The total strain at any point of the girder will consist of components from temperature and stress as is displayed in Fig. 3-39. The curvature of a strain profile along the girders cross section can be determined based on several common assumptions, which include:

- Plane sections remain plane.
- The free thermal strain is constant across the width at any elevation.
- The concrete and steel are within their linear elastic ranges, and the concrete is uncracked.
- The changes in axial force and bending moment due to thermal effects are both zero for the simply supported girder.

Through the use of these assumptions, Nguyen et al. (2015) were able to determine two methods to derive the thermally induced camber of prestressed concrete, I girders. The first method, named the temperature history camber model, was derived based on temperature profiles throughout the day. The second model, named the peak temperature camber model, was derived based on the assumption of peak temperatures within a 24 hour period and that linear temperature variations existed throughout the girder.

The Nibley Bridge did not have a temperature station at the site. Thus historical temperature data was obtained from the National Oceanic and Atmospheric Administration (NOAA 1970-2017) at a site 3 miles to the west of the bridge. This temperature data only includes the daily maximum and minimum temperature data, thus only the peak temperature camber model could be used in this analysis.
3.6.1. Thermal Camber by Measured Temperature

The dataloggers provided by Campbell Scientific were set to record temperature and strain values every hour. Using this data, a derivation can be completed in order to determine the measured thermal camber every hour throughout a given day. Nguyen et al. (2015) provide a detailed derivation to produce the following equations.

\[
\begin{bmatrix}
\int E dA & \int EydA \\
\int EydA & \int Ey^2dA
\end{bmatrix}
\begin{bmatrix}
\varepsilon_0 \\
\phi
\end{bmatrix}
= \begin{bmatrix}
\int E\varepsilon_e dA \\
\int Ey\varepsilon_e dA
\end{bmatrix}
\]

Equation 3-10

Where:

\[\varepsilon_0 = \text{total strain at the origin}\]
\[\phi = \text{curvature}\]

Additionally, E is the modulus of elasticity, A is the gross area of the nth cross section, and y is the distance from the centroid of the nth section to the bottom of the girder. Equation 3-10 can be simplified by setting the term \(\int AydA\) to zero and the result is Equation 3-11 and Equation 3-12.
\[
\int EdA = EA_{tr} \quad \text{Equation 3-11}
\]
\[
\int Ey^2dA = EI_{tr} \quad \text{Equation 3-12}
\]

Where:
\[
A_{tr} = \text{area of the transformed section}
\]
\[
I_{tr} = \text{moment of inertia of the transformed section}
\]

The matrix equations then can be decoupled to produce Equation 3-13 and Equation 3-14.

\[
\varepsilon_0 EA_{tr} = \int E\varepsilon_e b(y)dy \quad \text{Equation 3-13}
\]
\[
\phi EI_{tr} = \int Ey\varepsilon_e b(y)dy \quad \text{Equation 3-14}
\]

In determining the camber of a concrete member it is often most convenient to use Equation 3-13 and Equation 3-14 because the girder width b(y) cannot be defined with a single equation. The equations can be further simplified by dividing through by the modulus of elasticity, E, if it is constant through the girder cross section. Also, if the environmental strain,\(\varepsilon_0\), gradient is linear, the right side of Equation 3-14 simplifies to \(-EI_{tr}\alpha \Delta T/h\), where \(\alpha\) is the coefficient of thermal expansion and \(\Delta T\) is the temperature difference over the height of the girder. Then Equation 3-14 reduces down to produce Equation 3-15.
\[ \phi = -\frac{\alpha \Delta T}{h} \]  

Equation 3-15

The curvature must generally be computed at several locations along the girder and then integrated in order to give the value for camber. Yet, if the girder is prismatic and the temperature is constant along the length of the girder then the camber may be calculated at a single location. The mid-span deflection, for a simply supported girder, is obtained by integrating the curvature twice and if the girder is simply supported then Equation 3-16 is used to determine the mid-span camber.

\[ \Delta_{camber} = -\frac{\Phi L^2}{8} \]  

Equation 3-16

Where:

\[ L \] = Length of the girder

The negative sign in the equation is used because the calculated deflection is negative, but for the purposes here the camber is taken as positive upwards. The coefficient of thermal expansion used in these calculations was based on the value determined by Nguyen et al. (2015) as 5.5 x 10^{-6} °F^{-1} (9.9 x 10^{-6} °C^{-1}).

### 3.6.2. Peak Temperature Camber Model

Nguyen et al. (2015) provide a detailed description to obtain a thermal camber using hourly ambient temperature data. Unfortunately, the ambient temperature data obtained only records the maximum and minimum temperature throughout the day. Thus, the Temperature History Camber Model will not be used here. A second model developed by
Nguyen et al. (2015) does not require a detailed daily temperature history and can be derived using peak ambient temperatures in a single day. This model is based on an assumption that the effective temperature difference between the girder top and bottom $\Delta T_{eff}$ is related to the daytime high and nighttime low temperatures for a 24-hour period. This effective temperature is approximated from the maximum and minimum temperatures over a single day using a cosine interpolation as a function of time as shown in Equation 3-17.

$$\Delta T_{eff}(t) = A_1 (T_{max} - T_{min}) \left\{ 1 - \cos \left[ \frac{(t - t_0)}{24} (2\pi) \right] \right\}$$  
Equation 3-17

Where:

$A_1$ = Calibration factor

$t_0$ = reference time for counting the thermal camber during that day

$T_{max}$ = maximum air temperature during the 24-hour period

$T_{min}$ = minimum air temperature during the 24-hour period

The reference time $t_0$ represents the time at which the ambient temperature is at a minimum as well as the lag time between the concrete and air temperatures. This causes $\Delta T_{eff}$ to have a zero value when $t$ equals $t_0$ (as well as at $t_0 + 24$ hours) and reaches its maximum at $t_0 + 12$ hours. Then the affective thermal strain is calculated in Equation 3-18.

$$\varepsilon_{TH,eff} = \alpha A_1 (T_{max} - T_{min}) \left\{ 1 - \cos \left[ \frac{(t - t_0)}{24} (2\pi) \right] \right\}$$  
Equation 3-18
The thermal camber at mid-span for a simply supported girder may then be determined by combining Equation 3-16 and Equation 3-18. The result is displayed as Equation 3-19.

\[
\Delta_{\text{camber}} = \left(\frac{\alpha A_1}{h}\right) \left(\frac{(T_{\text{max}} - T_{\text{min}})}{2}\right) \left(1 - \cos \left[\frac{(t - t_0)}{24} (2\pi)\right]\right) \left(\frac{L^2}{8}\right)
\]

Equation 3-19

In order for this model to be accurate it needed to be calibrated by optimizing predictions of the cambers. Nguyen et al. (2015) compared this model against the cambers from several other tests performed by other researchers. The model was then compared against the data from all girders at once. This comparison determined that the values of \( A_1 \) at 1.28 and \( t_0 \) at 4.53 were the most optimal values with a root mean square camber error of 0.009 in. (2.3 mm). By the use of the peak temperature model with \( t_0 \) at 4.53, the result is that the minimum camber will always occur at 4:32 am and the peak camber will always occur at 4:32 pm.

The Nibley Bridge experienced several instances of large positive and negative temperature gradients. The largest gradient, as discussed in Section 3.5.1, occurred on May 13, 2016 with an overall difference in temperature of 58.7 °F (32.6 °C). The embedded thermocouples provided the temperature data to determine the thermal camber based on the greatest non-linear temperature gradient. The thermal camber of May 13, 2016 is displayed in Fig. 3-40. This figure shows a comparison of the cambers according to the Peak Temperature Camber Model, the recorded temperatures, and from the CSiBridge
finite element model. The thermocouples recorded every hour and the maximum camber was found to occur at 4:00 pm. The peak temperature predictive model displays a peak camber at 4:32 pm. If the thermocouples recorded more frequently it is possible that the peak temperature model would prove to be more accurate.

3.6.3. Comparison of Computer Analyses and Mathematical Models

The greatest maximum and minimum gradients experienced in the Nibley Bridge occurred on May 13, 2016 and January 9, 2017, as described in Section 3.5.1. Using these two temperature gradients, a camber and stress distribution was determined using both computer and numerical models. The computer models used were from SAP 2000 and CSiBridge. The first numerical model was performed using the measured temperature data.
from sensors embedded into the girders. The second model, labeled the Peak Temperature Camber Model, is determined by use of maximum and minimum daily temperature data obtained from the NOAA. Table 3-7 below displays the results of these analyses by the temperature assignments of May 13, 2016 at 2:00 pm.

The results from Table 3-7 indicate that the CSiBridge model contains a close representation of the Nibley Bridge. The SAP 2000 model overestimated the greatest positive temperature gradient, however it did not overestimate the greatest negative temperature gradient. Additionally, the Peak Temperature Camber model underestimated the measured camber for both the greatest maximum and minimum temperature gradient.

3.7. Prestress Losses

Precast girders facilitate rapid construction of a bridge because the girders are fabricated off-site and then shipped to and erected at the job site. This eliminates the need for formwork and lengthy curing time at the bridge site. Precast bridge girders are prestressed in order to overcome the concrete’s natural weakness in tension that occurs during service

<table>
<thead>
<tr>
<th>Model</th>
<th>Peak camber due to max. positive gradient (in.)</th>
<th>Peak camber due to max. negative gradient (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SAP 2000</td>
<td>0.8259</td>
<td>-0.2619 Estimated</td>
</tr>
<tr>
<td>CSiBridge</td>
<td>0.7531</td>
<td>-0.1465</td>
</tr>
<tr>
<td>Measured temperature data</td>
<td>0.8002</td>
<td>-0.2804</td>
</tr>
<tr>
<td>Peak Temperature</td>
<td>0.6499</td>
<td>-0.2166</td>
</tr>
</tbody>
</table>
cross the bottom flange. To achieve the required prestressing force, concrete is cast around high strength steel tendons, cables, or bars that have been stressed in tension. The concrete will bond to the strands as it cures. The strands are then cut to release the tension and this force is transferred into the girders as compression due to the concrete and tension bond. However, not all of the stress that initially existed as tension in the strands will transfer as compressive stress. At release, a significant amount of prestress loss occurs due to elastic shortening. Over time, additional loss occurs due to creep and shrinkage of the concrete, and relaxation of the prestressing strands. The AASHTO LRFD Bridge Design Specifications (AASHTO 2010) provide two methods to determine the magnitude of prestress loss that occurs. The two methods are titled the Approximate and the Refined Method. The losses are categorized into two groups for both methods. The same procedure is used to calculate the instantaneous losses for both methods, but have different procedures to calculate the time-dependent losses.

The Precast Concrete Institute Design Handbook (PCI 2010) also has a method that may be used to predict the prestress losses due to elastic shortening as well as long-term losses. This section presents information regarding the measured prestress losses compared to the predictive methods from The AASHTO LRFD Bridge Design Specifications (AASHTO 2010), and from the PCI Design Handbook (PCI 2010).

3.7.1. Approximate Method

The procedure for determining prestress losses, according to the approximate method, provides a single value of the total loss that is expected to occur in a concrete member. This method is only for use with girders that are constructed with normal and
lightweight concrete, have specified strengths up to 15.0 ksi, and prestressed in a single
stage rather than segmental construction. The general equation comes from AASHTO
5.9.5.1-1 and is listed below as Equation 3-20.

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$$

Equation 3-20

Where:

- $\Delta f_{pT} = \text{total prestress loss (ksi)}$
- $\Delta f_{pES} = \text{sum of all losses or gains due to elastic shortening or extension at the time of application of prestress and/or external loads (ksi)}$
- $\Delta f_{pLT} = \text{losses due to long-term shrinkage and creep of concrete, and relaxation of the steel (ksi)}$

The two variables provided in Equation 3-20 are the values of elastic shortening and creep and shrinkage respectively. These values are determined through the application of the AASHTO C5.9.5.2.3a-1 listed as Equation 3-21 below, and through AASHTO 5.9.5.3-1 listed as Equation 3-24 below. The values used for Equation 3-21 are listed in Table 3-8, where the highlighted values are based on measured properties of the Nibley Bridge.

$$\Delta f_{pES} = \frac{A_p s f_{pbt} (I_g + e_m^2 A_g) - e_m M_g A_g}{A_p s (I_g + e_m^2 A_g) + \frac{A_g I_g E_{ci}}{E_p}}$$

Equation 3-21

In which:

$$E_c = 33 k_1 w_1^{1.5} \sqrt{f_c}$$

Equation 3-22

Where:
It has been suggested that the AASHTO Code equations may overestimate $E_c$ for concretes with a compressive strength in the range of 6,000 to 12,000 psi. Thus, Equation 3-23 is recommended as a renewed equation by Darwin et al. (2016). The variables and units are described above. The prestress losses were determined using Equation 3-22 and Equation 3-23 and these comparisons will be labeled as E1 and E2 respectively.

$$E_c = \left( 40,000\sqrt{f_c'} + 1,000,000 \right) \left( \frac{w_c}{145} \right)^{1.5}$$

Equation 3-23
Table 3-8. Values used in the calculations for Equation 3-21

<table>
<thead>
<tr>
<th></th>
<th>Girder 1</th>
<th>Girder 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_{ps}$ (in²)</td>
<td>7.378</td>
<td>7.378</td>
</tr>
<tr>
<td>$A_g$ (in)</td>
<td>1157.3</td>
<td>1157.3</td>
</tr>
<tr>
<td>$E_c$ (E1) (ksi)</td>
<td>4746.77</td>
<td>4457.18</td>
</tr>
<tr>
<td>$E_c$ (E2) (ksi)</td>
<td>4222.74</td>
<td>4037.89</td>
</tr>
<tr>
<td>$k_1$</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>$w_c$ (kcf)</td>
<td>0.1379</td>
<td>0.1395</td>
</tr>
<tr>
<td>$f'_c$ (ksi)</td>
<td>7.89</td>
<td>6.72</td>
</tr>
<tr>
<td>$e_m$ (in.)</td>
<td>22.04</td>
<td>22.04</td>
</tr>
<tr>
<td>$E_p$ (ksi)</td>
<td>28500</td>
<td>28500</td>
</tr>
<tr>
<td>$f_{pbt}$ (ksi)</td>
<td>202.5</td>
<td>202.5</td>
</tr>
<tr>
<td>$I_g$ (in⁴)</td>
<td>259976.52</td>
<td>259976.52</td>
</tr>
<tr>
<td>$M_g$ (kip-in.)</td>
<td>12873.73</td>
<td>13023.10</td>
</tr>
</tbody>
</table>

Equation 3-24 is an estimate of the combined time dependent losses for standard precast, pretensioned members subject to normal loading and environmental conditions. The values used for Equation 3-24 are based on the properties of the Nibley Bridge and listed in Table 3-9. This formula includes the combined long-term prestress losses due to creep of concrete, shrinkage of concrete, and relaxation of steel, where:

$$
\Delta f_{pLT} = 10.0 \frac{f_{plt}A_{ps}}{A_g} \gamma_n \gamma_{st} + 12.0 \gamma_n \gamma_{st} + \Delta f_{pR}
$$

Equation 3-24

In which:
\[ \gamma_h = 1.7 - 0.01H \quad \text{Equation 3-25} \]
\[ \gamma_{st} = \frac{5}{(1 + f'_{ci})} \quad \text{Equation 3-26} \]

Where:

\[ f_{pi} = \] prestressing steel stress immediately prior to transfer (ksi)

\[ H = \] The average annual ambient relative humidity (%), determined by use of Figure 5.4.2.3-1 in AASHTO

\[ \gamma_h = \] correction factor for specified concrete strength at time of prestress transfer to the concrete member

\[ \gamma_{st} = \] correction factor for specified concrete strength at time of prestress transfer to the concrete member

\[ \Delta f_{pR} = \] an estimate of relaxation loss taken as 2.4 ksi for low relaxation strand, 10.0 ksi for stress relieved strand, and in accordance with manufacturers recommendation for other types of strand (ksi)

**Table 3-9. Additional values used for the Approximate method**

<table>
<thead>
<tr>
<th></th>
<th>Girder 1</th>
<th>Girder 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_{pi} ) (ksi)</td>
<td>202.5</td>
<td>202.5</td>
</tr>
<tr>
<td>( H ) (%)</td>
<td>57</td>
<td>57</td>
</tr>
<tr>
<td>( \gamma_h )</td>
<td>1.13</td>
<td>1.13</td>
</tr>
<tr>
<td>( \gamma_{st} )</td>
<td>0.56</td>
<td>0.65</td>
</tr>
<tr>
<td>( \Delta f_{pR} ) (ksi)</td>
<td>2.4</td>
<td>2.4</td>
</tr>
</tbody>
</table>
3.7.2. Refined Method

The Refined Method produces a measure of the prestress loss at any specific time after transfer, whereas the Approximate Method only provides a single value for elastic shortening and long-term losses. This method is applicable with girders that are constructed using normal and lightweight concrete, with specified strengths up to 15.0 ksi, and prestressed in a single stage rather than segmental construction. This method is also considered to be more accurate (less conservative) than the approximate method for determining values of creep, shrinkage, and relaxation related losses. The instantaneous losses of elastic shortening are calculated using Equation 3-21. However, the long-term losses are calculated differently and each component of loss is calculated independently. The required equations for this method come from the AASHTO Specifications in section 5.9.5.4 and 5.4.2.3.2, and are listed in the following equations. The prestress loss due to shrinkage of girder concrete is determined by Equation 3-27.

\[ \Delta f_{pSR} = \varepsilon_{bid} E_p K_{id} \]  

Equation 3-27

In which:

\[ K_{id} = \frac{1}{1 + \frac{E_p A_{ps}}{E_{ci} A_g} \left( 1 + \frac{A_g e_{pg}^2}{I_g} \right) \left[ 1 + 0.7 \Psi_b(t_f, t_i) \right]} \]  

Equation 3-28

Where:

\[ \varepsilon_{bid} = \] concrete shrinkage strain of girder between time of transfer and deck placement per AASHTO Eq. 5.4.2.3-1
\[ K_{id} = \text{transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in the section being considered for time period between transfer and deck placement} \]

\[ e_{pg} = \text{eccentricity of prestressing force with respect to centroid of girder (in.); positive in common construction where it is below girder centroid} \]

\[ \Psi_b(t_f, t_i) = \text{girder creep coefficient at final time to loading introduced at transfer per AASHTO Eq. 5.4.2.3.2-1} \]

\[ t_f = \text{final age (days)} \]

\[ t_i = \text{age at transfer (days)} \]

The various values for the loss components change over time and are dependent upon the value of \( t_f \) used in Equation 3-28. The value of \( t_i \) remains constant as 1 day for Girder 5 and 5 days for Girder 5. The prestress loss due to creep of girder concrete is determined by Equation 3-29.

\[ \Delta f_{pCR} = \Delta f_{pES} \Psi_b(t_f, t_i) K_{id} \quad \text{Equation 3-29} \]

After the values for Equation 3-27, Equation 3-28, and Equation 3-29 are determined, then Equation 3-20 and Equation 3-24 are used to determine the total prestress loss. Fig. 3-41 displays the long-term prestress losses of girders 1 and 5 compared to the Refined method using E1. Fig. 3-42 displays the long-term prestress losses of girders 1 and 5 compared to the Refined Method using E2.
Fig. 3-41. Measured and calculated prestress losses with E1 for the Refined method

Fig. 3-42. Measured and calculated prestress losses with E2 for the Refined method
Several differences are apparent in a comparison of the Refined Method using E1 and E2. Refer to section 3.7.4 and 3.7.5 for a discussion on the differences in measured and predicted prestress losses.

### 3.7.3. PCI Method

The Precast Concrete Institute (PCI) design manual contains an additional method of determining prestress losses (PCI 2010). It is computationally a simpler method in comparison to the Refined Method (Section 3.7.2), and is similar in complexity to the Approximate Method (Section 3.7.1). Equation 3-20 still applies in this method in order to determine the total prestress losses. The intermediate steps for elastic shortening, creep, shrinkage, and relaxation differ. The losses due to Elastic Shortening are determined by use of Equation 3-30.

\[
ES = K_{es} E_{ps} \frac{f_{c\sigma}}{E_{c\sigma}}
\]

Equation 3-30

Where:

\( K_{es} = \) 1.0 for pretensioned components
\( E_{ps} = \) Modulus of elasticity of prestressing tendons \((28.5 \times 10^6 \text{ psi})\)
\( E_{c\sigma} = \) Modulus of elasticity of concrete at time of prestress is applied \((\text{psi})\)
\( f_{c\sigma} = \) Net compressive stress in concrete at center of gravity of prestressing force immediately after prestress has been applied to the concrete \((\text{psi})\)

\[
f_{c\sigma} = K_{c\sigma} \left( \frac{P_{l}}{A_{g}} + \frac{P_{le}^2}{I_{g}} \right) - \frac{M_{g} e}{I_{g}}
\]

Equation 3-31
Where:

\[ K_{cir} = 0.9 \text{ for pretensioned components} \]

\[ P_i = \text{Initial prestress force (lbs)} \]

\[ e = \text{Eccentricity of center of gravity of tendons with respect to center of gravity of concrete at the cross section considered (in.)} \]

\[ A_g = \text{Area of gross concrete section at the cross section considered (in}^2) \]

\[ I_g = \text{Moment of inertia of gross concrete section at the cross section considered (in}^4) \]

\[ M_g = \text{Bending moment due to dead weight of prestressed component and any other permanent loads in place at time of prestressing (lb-in)} \]

The PCI manual makes the assumption that the \( f_{cir} \) in Equation 3-31 is 90% of the initial prestress force, which is determined by \( K_{cir} \) in Equation 3-31. \( f_{cir} \) is the concrete stress at the center of gravity of the prestressing force immediately after release. The value of \( K_{cir} \) may be removed and then \( f_{cir} \) becomes dependent on \( P_i \) and \( P_i \) is also dependent of \( f_{cir} \). Thus these values can be determined through iteration.

The losses due to creep of the concrete are calculated by Equation 3-32

\[ CR = K_{cr} \left( \frac{E_{ps}}{E_c} \right) (f_{cir} - f_{cds}) \]

Where:

\[ K_{cr} = 2.0 \text{ for normalweight concrete, 1.6 for sand-lightweight concrete} \]
\[ f_{cds} = \text{Stress in concrete at center of gravity of prestressing force due to all superimposed, permanent dead loads that are applied to the member after it has been prestressed (psi)} \]

\[ E_c = \text{Modulus of elasticity of concrete at 28 days (psi)} \]

\[ f_{cds} = \frac{M_{sd}e}{I_g} \]  

\text{Equation 3-33}

Where:

\[ M_{sd} = \text{Moment due to all superimposed, permanent dead load and sustained load applied after prestressing (lb-in)} \]

The losses due to the shrinkage of concrete are calculated by Equation 3-34. The average relative humidity was determined as 57% from AASHTO at the bridge site.

\[ SH = (8.2 \times 10^{-6})K_{sh}E_{ps}(1 - 0.06 V/S)(100 - RH) \]  

\text{Equation 3-34}

Where:

\[ K_{sh} = 1.0 \text{ for pretensioned components} \]

\[ V/S = \text{Volume-to-surface ratio} \]

\[ RH = \text{Average ambient relative humidity (%)} \]

The losses due to relaxation of steel are calculated by Equation 3-35

\[ RE = f_{pi} \left( \frac{\log t_2 - \log t_1}{45} \right) \left( \frac{f_{pi}}{f_{py}} - 0.55 \right) \]  

\text{Equation 3-35}

Where:

\[ f_{pi} = \text{Initial jacking stress in prestressing steel (psi)} \]
3.7.4. Measured Elastic Shortening Losses

The tendons of a prestressed concrete member experience two main types of prestress losses, which are instantaneous losses and time-dependent losses. An instantaneous loss, which occurs at the time of tension release, is referred to as elastic shortening. The Precast Concrete Institute (PCI 2010) design handbook describes elastic shortening as the phenomenon when the concrete around the tendons shortens as the prestressing force is applied to it (PCI 2010). Time-dependent losses include creep of the concrete, shrinkage of the concrete, relaxation of the steel, and temperature losses. The three main losses that will be considered here are elastic shortening, creep, and shrinkage.

As was described in Section 3.3, Girder 1 and Girder 5 had varying maturity processes. The concrete for Girder 5 was poured on December 7, 2015 and the tendons were released the following day. Whereas, the concrete for Girder 1 was poured on December 9, 2015, yet the tendons were not released until December 14, 2015. Thus Girder 5 experienced a time of one day to transfer and Girder 1 experienced a time of five days until transfer. Refer to Table 3-1 for the compressive strength of the girders at release.

Fig. 3-43 displays the measured prestress losses during the destressing process of both girders. Since Girder 1 was not released until December 14, 2015, the data in Fig. 3-43 was shifted in order to clearly display the elastic shortening at the time of release for each girder. The overall elastic shortening for each girder was determined as the four
vertical lines from each girder shown in Fig. 3-43. Girder 5 contains an overall measured elastic shortening of 24.3 ksi (169.5 Mpa) and Girder 1 contains an overall measured elastic shortening of about 22.8 ksi (157.2 Mpa).

The calculations presented previously were compared to the measured prestress losses. Fig. 3-44 displays the values of elastic shortening from the AASHTO method with both E1 and E2, the PCI method, as well as the measured values for each girder. Table 3-10 displays the percent errors of the calculations versus the measured values of the elastic shortening for both Girder 1 and Girder 5. It is interesting to note that the AASHTO E1 and PCI methods both determine a very similar value for the elastic shortening in both girders. Table 3-11 displays the amount of prestress loss due to elastic shortening in terms of percent from the initial jacking stress of 202.5 ksi. Thus to equate the percent loss, \((\Delta f_{ps} / 202.5) \times 100 \%\) is used.

![Diagram](image)

**Fig. 3-43. Elastic shortening losses**
Fig. 3-44. Elastic shortening by the AASHTO method, PCI method, and measured

Table 3-10. Percent error for the elastic shortening losses

<table>
<thead>
<tr>
<th>Method</th>
<th>Girder 1</th>
<th>Girder 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO E1 Method</td>
<td>29.83</td>
<td>30.47</td>
</tr>
<tr>
<td>AASHTO E2 Method</td>
<td>22.16</td>
<td>24.15</td>
</tr>
<tr>
<td>PCI Method</td>
<td>29.89</td>
<td>30.53</td>
</tr>
</tbody>
</table>

Table 3-11. Percent of prestress loss due to elastic shortening

<table>
<thead>
<tr>
<th>Method</th>
<th>Girder 1</th>
<th>Girder 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO E1 Method</td>
<td>7.92</td>
<td>8.34</td>
</tr>
<tr>
<td>AASHTO E2 Method</td>
<td>8.78</td>
<td>9.10</td>
</tr>
<tr>
<td>PCI Method</td>
<td>7.91</td>
<td>8.33</td>
</tr>
<tr>
<td>Measured</td>
<td>11.28</td>
<td>11.99</td>
</tr>
</tbody>
</table>
3.7.5. Measured Creep and Shrinkage Losses

Concrete is a material that is time-dependent and gains strength overtime, typically a 28 day strength is specified as the ultimate compressive strength for design purposes. Creep is also a time-dependent effect on concrete which is defined by Nawy as a flow or deformation in a material over time when constant load or stress exists (Nawy 2006). A small amount of creep was observed in the girders between the various release stages, refer to Fig. 3-43. These moments of creep are observed as the gradual rises in prestress losses directly after a moment of elastic shortening experienced by cutting a portion of the strands.

It was determined that the overall instantaneous prestress losses of Girder 5 were 31.7 ksi (218.6 Mpa) and 26.5 ksi (182.7 Mpa) for Girder 1. Thus if the instantaneous losses due to elastic shortening were as determined from Section 3.7.4, then the instantaneous losses due to creep would be 7.4 ksi (51.0 Mpa) for Girder 5 and 3.7 ksi (25.5 Mpa) for Girder 1.

Prestress losses due to shrinkage are defined as a decrease in volume over time. Nawy described that shrinkage of concrete is affected by several factors, which include: mixture proportions, type of aggregate, type of cement, curing time, time between the end of external curing and the application of prestressing, size of the member, and environmental conditions (Nawy 2006). It is difficult to differentiate the overall time-dependent losses as either creep or shrinkage losses. Thus the overall losses following elastic shortening will be lumped together as both creep and shrinkage losses.

The calculations presented previously were compared to the measured prestress losses. Fig. 3-45 displays the values for creep and shrinkage from the approximate, refined, and PCI method, along with the measured values. Table 3-12 displays the percent errors of the calculations and measured values of the creep and shrinkage losses for both Girder 1
and Girder 5. Note that the refined method best determines the long term prestress losses for Girder 5, but the approximate method is best for Girder 1. Additionally, Table 3-13 displays the amount of prestress loss due to creep and shrinkage in terms of percent from the initial jacking stress of 202.5 ksi. Thus to equate the percent loss, \( \left( \frac{\Delta f_{p,c}}{202.5} \right) \times 100\% \) is used. It can be noted that the three methods estimate an average of about 11% of the initial jacking stress is lost over time.

![Graph](image)

Fig. 3-45. Creep and shrinkage losses determined by all three methods, and measured

<table>
<thead>
<tr>
<th>Table 3-12. Percent error for the creep and shrinkage losses</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Girder 1</strong></td>
</tr>
<tr>
<td>Approximate Method</td>
</tr>
<tr>
<td>Refined Method</td>
</tr>
<tr>
<td>PCI Method</td>
</tr>
</tbody>
</table>
Table 3-13. Percent of prestress loss due to creep and shrinkage

<table>
<thead>
<tr>
<th>Method</th>
<th>Girder 1</th>
<th>Girder 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Approximate</td>
<td>9.01</td>
<td>10.19</td>
</tr>
<tr>
<td>Refined</td>
<td>9.62</td>
<td>12.40</td>
</tr>
<tr>
<td>PCI</td>
<td>10.32</td>
<td>13.87</td>
</tr>
<tr>
<td>Measured</td>
<td>7.26</td>
<td>12.95</td>
</tr>
</tbody>
</table>

3.7.6. Total Measured Losses

The bar graph in Fig. 3-46 displays the total prestress losses determined from the approximate E1 and E2, refined E1 and E2, and PCI methods, as well as the total measured prestress losses. Table 3-14 displays the percent errors of the calculations and measured values of the total prestress losses for both Girder 1 and Girder 5. Based on the total percent errors it can be determined that the PCI method represents the best method for determining the prestress losses for both girders of the Nibley bridge. It is interesting to note that the total prestress loss calculations by AASHTO match well to the measured losses in Girder 1. However, the measured and calculated values in Girder 1 result in an error by about 25%, as is shown in Table 3-14. There are several reasons why this might be. One reason that Girder 1 and Girder 5 present different calculated prestress losses could be because Girder 1 contained a longer cure time before release than did Girder 5. Additionally, it can be noted that the use of E2 from Equation 3-23 present more accurate results for the AASHTO Methods than does E1.
Fig. 3-46. Total losses determined by all three methods, and measured

Table 3-14. Percent error for the total prestress losses

<table>
<thead>
<tr>
<th></th>
<th>Girder 1</th>
<th>Girder 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Approximate E1 Method</td>
<td>6.65</td>
<td>24.48</td>
</tr>
<tr>
<td>Approximate E2 Method</td>
<td>1.88</td>
<td>21.39</td>
</tr>
<tr>
<td>Refined E1 Method</td>
<td>3.14</td>
<td>15.30</td>
</tr>
<tr>
<td>Refined E2</td>
<td>4.14</td>
<td>10.14</td>
</tr>
<tr>
<td>PCI Method</td>
<td>0.57</td>
<td>9.50</td>
</tr>
</tbody>
</table>

Table 3-15 displays the amount of total prestress loss in terms of percent from the initial jacking stress of 202.5 ksi, thus to equate the percent loss, \((\Delta f_{pe}/202.5) \times 100\%\) is used. It is interesting to note that a total loss of 25% was measured in Girder 5, yet Girder 1 had a total loss of 18%. This is likely due to curing time before release (See Section 3.3).
Table 3-15. Percent of total prestress loss

<table>
<thead>
<tr>
<th>Method</th>
<th>Girder 1</th>
<th>Girder 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Approximate E1 Method</td>
<td>16.92</td>
<td>18.53</td>
</tr>
<tr>
<td>Approximate E2 Method</td>
<td>17.79</td>
<td>19.29</td>
</tr>
<tr>
<td>Refined E1 Method</td>
<td>17.56</td>
<td>20.78</td>
</tr>
<tr>
<td>Refined E2 Method</td>
<td>18.88</td>
<td>22.05</td>
</tr>
<tr>
<td>PCI Method</td>
<td>18.23</td>
<td>22.20</td>
</tr>
<tr>
<td>Measured</td>
<td>18.13</td>
<td>24.53</td>
</tr>
</tbody>
</table>

3.7.7. Relaxation

Tendons suffer a loss in the prestressing force due to constant elongation with time. Nawy described that the magnitude of the decrease in the prestress depends not only on the duration of the sustained prestressing force, but also on the ratio of the initial prestress to the yield strength (Nawy 2006). In the case of the Nibley bridge, the amount of relaxation was not measured because the strain gauges were not attached to the tendons. Fig. 3-8 displays that the strain gauges were placed within the concrete between the tendons, thus the gauges are unable to measure the actual relaxation losses in the steel. The Geokon model 4200 vibrating wire strain gauges are shaped somewhat like a barbell, and are therefore able to remain within the concrete and measure the stress changes within the concrete. Additionally, the gauges were placed at the height of the neutral axis of the tendons, and thus able to measure the stress losses at the location of the tendons neutral axis.
CHAPTER 4
SHORT-TERM DATA MONITORING

4.1. Exterior Instrumentation

An additional data collection system was used to monitor strain and acceleration of the bridge during a shorter duration of time. This system was developed by Bridge Diagnostics Inc. (BDI), and is monitored wirelessly with cell towers. The BDI system contains a data acquisition box that was mounted under the bridge, and was used to house the data collection computer, the wireless modem, and all of the connections necessary to make the wireless system function and easy to work with. Additionally, two nodes are mounted under the bridge at locations near the mid-span. Then these nodes were connected to the data acquisition system. From each of these two nodes, four sensors were connected and then attached at predetermined locations on the bottom of select bridge girders.

A total of eight ST350 strain transducers and eight A1521 Uniaxial Accelerometers were attached to the bridge. These sensors were attached to the exterior of the bridge using a Loctite 410 adhesive and small brackets. An example of this attachment is shown in Fig. 4-1. Six of the strain gauges were attached to the bottom of the girders at the mid-span on Girders 1 thru 6. The other two strain gauges were attached at mid-span to the web of Girder 2 and Girder 3 at a height that correlates to the centroid of the girders. Fig. 4-2 displays a cross section of the mid-span of the bridge where each of the sensors were attached. Strain Gauges (SG in Fig. 4-2) 1 thru 8 indicate the locations of each of the eight strain gauges at the mid-span of the bridge.

The accelerometers were placed at mid-span as well as quarter points of various
girders. Long-term accelerometers are also embedded in the bridge at locations that correlated with the quarter point, thus it provided the opportunity to correlate the data at mid-span as well as quarter points. The location of the accelerometers was somewhat limited based on the length of the wire attached to each sensor. Each of these sensors were required to connect into one of two available nodes. The nodes could only allow four sensors to connect to them at one time, and thus the required arrangement of accelerometers is displayed in Fig. 4-3. The strain gauges and accelerometers were also provided by BDI. An example of the accelerometers is provided in Fig. 4-4 and the strain gauges are depicted in Fig. 4-1.
Fig. 4-3. Plan view of bridge depicting the location of the BDI accelerometers

Fig. 4-4. Rendering of BDI A1521 uniaxial accelerometer
4.2. **STS Software Description**

The equipment provided by BDI requires the use of a particular software titled STS. The software is found on the desktop of a laptop in the USU Long Term Bridge Performance (LTBP) program. The software can be operated as a continuous data acquisition system and the software required is STS Monitor. The system can also serve as a data acquisition for live load tests and the STS Live software would be required. For the purposes of this report, only STS Monitor will be described in detail.

The first step to operate the software is to engage the core computer by beginning a remote connection. The Core computer is referring to part of the data acquisition system located under the Nibley Bridge. To do this, open the start menu on the laptop and search for Remote Connection and select Remote Desktop Connection from the start menu. The Remote Desktop Connection window will appear and the credentials for the computer will already be displayed as shown in Fig. 4-5. Select Connect and allow the laptop to open the core computer mainframe. Once within the core computer, select BDI CORE to open STS Core Monitor. Opening BDI CORE will also engage the core computer and allow the laptop to connect to the data acquisition system.

Now minimize the core and select the most recent version of STS Monitor from the desktop. This will open the STS Base window as is shown in Fig. 4-6. From the STS Base window, select remote in the dropdown screen and then select Connect to Core. After a few seconds an icon at the bottom of the screen will appear green and indicate “connected.” After STS Base is connected to the core computer, select STS UIF to open the STS Monitor User Interface (UIF). The first screen of the STS Monitor User Interface (UIF) is shown in Fig. 4-7. The STS Monitor UIF contains four primary tabs at the top left of the screen.
These four tabs are the first four icons listed in Table 4-1. When beginning a new recording, start at the left tab and move to the right in order. Thus, the order of edits will be to start with the Settings Tab, then move on to the System Hardware Tab, Data Display Tab, and the Test Attributes Tab consecutively.

The settings tab contains the information about the particular recording that will be performed. Before making any adjustment at this screen, make sure that an icon resembling the pulse of a wave, similar to the icon of the test attributes tab, is displayed. Otherwise the system is set in live recording mode and must be changed to a continuous recording. Fill in any particular information that is necessary to distinguish the recording from any other past recordings.

The system hardware tab displays all of the nodes and sensors currently connected to the BDI system. It is important to verify that each sensor is displayed with its correct label, calibration, and that no errors are displayed. This is also the tab where any changes
Fig. 4-6. STS Base software connection screen

Fig. 4-7. STS Monitor user interface startup screen
<table>
<thead>
<tr>
<th>Icon</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image" alt="Settings Tab" /></td>
<td>Allows the user to define specific parameters for each test</td>
</tr>
<tr>
<td><img src="image" alt="System Hardware Tab" /></td>
<td>Displays information on nodes, sensors, and test status</td>
</tr>
<tr>
<td><img src="image" alt="Data Display Tab" /></td>
<td>Allows the user to view plots of test data</td>
</tr>
<tr>
<td><img src="image" alt="Test Attributes Tab" /></td>
<td>Allows the user to define the specifications of the tests to be performed</td>
</tr>
<tr>
<td><img src="image" alt="Edit Button" /></td>
<td>Allows the user to edit a selected group of sensors, or to make edits to test specifications</td>
</tr>
<tr>
<td><img src="image" alt="Add Group Button" /></td>
<td>Allows the user to create a group of sensors</td>
</tr>
<tr>
<td><img src="image" alt="Delete Group Button" /></td>
<td>Allows the user to selectively remove sensors from a group</td>
</tr>
<tr>
<td><img src="image" alt="Save Changes Button" /></td>
<td>Allows the user to save changes made to a sensor, group of sensors, or to test specifications</td>
</tr>
<tr>
<td><img src="image" alt="Add Sensor to Group Button" /></td>
<td>Used to assign sensors to a group</td>
</tr>
<tr>
<td><img src="image" alt="Confirm Button" /></td>
<td>Used to accept changes and close the current edit window</td>
</tr>
<tr>
<td><img src="image" alt="Abort Button" /></td>
<td>Used to decline changes and close the current edit window</td>
</tr>
<tr>
<td><img src="image" alt="Record Button" /></td>
<td>Used to start a data acquisition test</td>
</tr>
<tr>
<td><img src="image" alt="Stop Button" /></td>
<td>Used to stop a currently running test</td>
</tr>
</tbody>
</table>
to sensor settings may be made. It is advisable to understand what the calibration of each sensor should be in order to ensure that correct measurements are being recorded.

The data display tab is where the user can view plots of current recordings to achieve real time information regarding tests. This is also the location that tests can be started and stopped using the large START and STOP icons at the bottom left of the screen. These icons are shown in Table 4-1, listed as the Record Button and Stop Button.

The test attributes tab of the STS Monitor UIF is where the test specifications are programmed. The next four icons listed in Table 4-1 are the primary icons that can be used to assign nodes to various recordings. To start a new test specification, the user will select the edit button which opens the STS Edit Test Specification (ETS) window. In this window, the user can view the current test specification as well as make changes to the current test specification. To create a new test category, for example: decimation, rain flow, and event recording; select the add group button. Then individually select the test categories desired for a particular specification. While still in the STS ETS window, select the edit button at the bottom right of the window. This will open the Detailed Test Editor window, where the aspects of each test category can be created. Then highlight a test category and select the edit button. This will allow a name to be assigned to the test category that was highlighted. Then highlight the name that was previously created and select the edit button. A window to configure and simulate each test category will then appear. It is important to remember to select the add sensor group button, refer to Table 4-1, to assign the sensors to the group. Make all of the necessary adjustments for each test category and select the confirm button to populate the information in the test editor. The last step will be to highlight each sensor
assigned to a test category, select the edit button, and make any last assignments for the sensors assigned to each group.

After all sensor assignments are made, the user may select the save changes button, shown in Table 4-1, to create a .csv file for the test specification. Multiple test specifications may be made if the sensors are changed for particular test scenarios. Now that all assignments are saved and no errors are displayed, the recordings can start by selecting the green start button at the bottom left of the screen. The user then may switch to the data display tab to view real time measurements of the sensors. Select the red stop button at the bottom left of the screen to end a recording. To access the files, open the core computer again, select computer from the start screen, and open the C drive. Then copy the files and place in another file anywhere other than the core computer. Each of the files are saved in a folder named according to the month and day that the recordings were made. Additionally, the files are saved as a .tdms format, which can be opened in Microsoft Excel using a TDM-Excel add in feature.

For additional help with the STS software, refer to the help guide included with the STS software. This is accessed by selecting the icon that resembles a question mark in the top right of the screen. Additional video support may also be accessed by searching for STS Monitor at www.youtube.com.

4.3. **STS Test Specification**

The STS software provided the ability to perform a collection of tests. When the strain gauges were attached to the nodes, three different categories of tests were performed. These categories included: decimation, event recording, and rain flow. The test
specification to use with the accelerometers would include normal recording and sliding Fast Fourier Transforms (FFT). However, the accelerometers will be analyzed and reported at a later date.

4.3.1. Decimation

In order to make verifications with other STS test categories, it was important to keep a record of the test decimation. Decimation is simply a constant recording of real data gathered from the strain gauges. The amount of strain measured in each sensor was recorded at a rate of 1 Hz (one record every second) in order to keep the file from becoming too large to analyze. A decimation file was created every 8 hours, thus three decimation files were created per day. Fig. 4-8 displays a plot of one file from an 8 hour decimation period. This plot shows a recording on March 22, 2017 from 8:00 am until 4:00 pm. The

![Graph of STS decimation during an 8 Hour period](image)

Fig. 4-8. STS decimation during an 8 Hour period
sudden vertical increases in strain shown on the plot is from each instance that a vehicle passes over the bridge. Since a majority of the sensors are placed on the north half of the bridge, most of the sudden strain increases are from vehicles passing over the bridge from the east to the west. The two lowest recordings shown in the plot are from SG 3 and SG 4 (refer to Fig. 4-2). The fact that the strain values of these two sensors remains near zero is good since these gauges are located at the centroid of the girders. The remaining recordings shown are from the remaining sensors located at the bottom of the girders.

4.3.2. **Event Recording**

It was desired to gather the changes in strain each time a vehicle passed over the bridge. However, this could not be achieved using a decimation file because the amount of data would be too large in each file to analyze. Thus, it was necessary to use an event recording category in the STS test specification. Event recording, also known as triggering, provides the ability to create a single file of data each time a vehicle passes over the bridge. It was decided that each event file would need to record at 50 Hz for a 6 second period. For an event file to record, the event recording settings needed to be adjusted by the user. For the Nibley bridge, it was determined that an event will happen each time a sensor experiences an increase of 2 με (micro strain) or more, and remains over 2 με for a period of at least 0.5 seconds. Fig. 4-9 displays an example of a typical trigger event that occurred on March 21, 2017 at 5:56:40.8 am. This particular trigger experienced an increase of 4 με, which was likely due to an average weight sedan. Additionally, the record shown as the peak was from SG 6, which is the location of the majority of the maximum strain values
It was also observed that the sensors at SG 2 and SG 1 would often increase in strain shortly after the vehicle has passed over the bridge. This observation is also shown in Fig. 4-9.

4.3.3. Rain Flow

With the decimation and event recording, it was desired to gather information regarding the strain variances experienced in the Nibley Bridge. Thus a rain flow algorithm was selected to perform the count analysis. Rain flow counting is a standard practice of cycle counting for fatigue analysis by the American Society for Testing and Materials (ASTM 2011). A rain flow algorithm will analyze variances in a data set (such as that from changing strain recordings on a bridge) and place a cycle difference into a bin for the range

Fig. 4-9. Example of event recording
of value difference. The particular range, means, and threshold of the rain flow is
determined by the user. For the Nibley Bridge, 20 range bins were created to gather the
difference of $1 \mu \varepsilon$ per range. Means refer to the average value between each range
difference, this was set from -8 up to +8. The threshold refers to the value or range
difference that needs to be exceeded in order to start a rain flow count. The threshold for
the Nibley Bridge was set at $0.5 \mu \varepsilon$ in order to keep the program from counting noise within
the data.

An example of data that can be analyzed using a rain flow algorithm is displayed
in Fig. 4-10. When a reversal in value occurs in a data set, the rain flow algorithm can
create a half count and place the half count into a bin depending on the range of the data.
A rain flow considers the range differences of three points of a data set individually. Thus,
the differences may be from two valleys and a peak or two peaks and a valley (refer to Fig.
4-10). Consider a range of data that commences with a valley-peak-valley range difference.
The peak-valley range difference must be greater than the valley-peak difference in order
for the half count to occur on the range of valley-peak. Once that occurs the new starting
point is at the next peak and the analysis starts over with the next three points. In the
instance where the second difference in range is not greater than the first difference, the
counting will move to the next three points. The algorithm will then continue moving by
increments of three points until a larger second range difference is found. Once this occurs,
the counting will return back to the points that were skipped and consider them. Essentially,
a range difference is rarely forgotten because the system is able to always remember points
that were skipped over and return to them at the end of the data set.
The data from SG 6 on March 22, 2017 from 8:00 am until 4:00 pm, portrayed in Section 4.3.1., is displayed as a rain flow counting in Table 4-2. The means displayed in the table is referring to the average absolute value between range differences. Thus, it is the absolute value of the peak minus the valley or the absolute value of the valley minus the peak for each count. The column on the right of Table 4-2 displays a summation of each range count, which essentially disregards the means. A total summation of each count provided a total count of 1,035. This indicates that 1,035 range variances occurred during the 8 hours, not that 1,035 vehicles passed over the bridge.

The rain flow data from the right column of Table 4-2 is plotted as Fig. 4-11. By observation, the majority of strain counts occurred in the range of 3 – 4 με. There are very few counts in the ranges exceeding 7 με. This is to be expected since passenger vehicles compose a majority of the vehicles causing an increase in strain on the bridge.

The total rain flow data for the week of March 18, 2017 thru March 25, 2017 is displayed in Table 4-3. Several observations may be made by the use of this table. Firstly,
it is apparent that something odd happened at SG 1. It may be that the gauge no longer registered the strain values correctly, or was reading rain flow values at a very low threshold, thus detecting a lot of noise. Additionally, SG 5, SG 7, and SG 8 registered the most counts in the range of 1 - 2 με. Most of these high counts at a low range are likely due to passenger vehicles passing towards the west. SG 6 contains a large amount of counts in the range of 4 – 5 με. These counts in a higher range are also likely from vehicles passing
Fig. 4-11. Sum of the STS rain flow of SG 6 during an 8 hour period

towards the west, but SG 6 registers a higher range count because the sensor is directly below the vehicles lane. Additional low range counts could be due to large vehicles passing over the bridge and each axle of the large vehicle registers several counts in low ranges. Section 4.4 discusses this phenomenon in greater detail.

The range counts of SG 6 for the week of March 18, 2017 thru March 25, 2017 is displayed in Fig. 4-12. A vast majority of the microstrain counts occur in the lower microstrain values. There are however, several counts again in the range of 13 – 15 με. The Microstrain that is registered by the gauge at this range was from large vehicles passing over the bridge. This figure displays that nearly all of the strain ranges in the bridge occur in a small range from passenger vehicles.
Table 4-3. Rain flow data of the Nibley bridge during a one-week period

<table>
<thead>
<tr>
<th>Range</th>
<th>SG 1</th>
<th>SG 2</th>
<th>SG 3</th>
<th>SG 4</th>
<th>SG 5</th>
<th>SG 6</th>
<th>SG 7</th>
<th>SG 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>8</td>
<td>4.5</td>
<td>11</td>
<td>8.5</td>
<td>4.5</td>
<td>3.5</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>1</td>
<td>26974</td>
<td>7803</td>
<td>77</td>
<td>284</td>
<td>5330</td>
<td>2430</td>
<td>7709</td>
<td>10408</td>
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<td>4363</td>
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<td>7</td>
<td>5.5</td>
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<td>11</td>
<td>-</td>
<td>0.5</td>
<td>-</td>
<td>-</td>
<td>15</td>
<td>11.5</td>
<td>3.5</td>
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<tr>
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<td>-</td>
<td>6.5</td>
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<td>-</td>
<td>-</td>
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<tr>
<td>&gt;20</td>
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<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>11</td>
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<td>327</td>
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<td>11278</td>
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</table>
Fig. 4-12. Rain flow data for SG 6 during a one-week period

4.4. STS Data during a 10 Minute Period

A rain flow algorithm does not necessarily gather information about the total strain that is caused due to a single vehicle passing over the bridge. An example of this is observed with Fig. 4-13, which depicts the strain on the bridge likely due to a school bus. For SG 6, a total strain difference in the range of 10 με can be assigned to this event due to observation. However, the rain flow algorithm would likely create half counts in the following order for SG 6 of Fig. 4-13. First half count in the range of 8 με, second half count in the range of 2 με, third half count in the range of 3 or 4 με, and the final half count in the range of 10 με. The intermediate alterations in strain would be skipped because the threshold is set to 0.5 με. Since these counts are inconclusive with a total strain caused by a vehicle, more analysis by hand was necessary.
In order to get a clearer idea of the counts being performed by the rain flow, it was decided to make in person observations at the Nibley Bridge. On March 25, 2017 a test was performed on site, the test was started at 5:44 pm and stopped at 5:55 pm. Table 4-4 displays the vehicle count that was performed at the time of this test as well as the direction of travel, which affected the magnitude of microstrain at each girder. When a vehicle passes the bridge towards the east, the sensors on the north part of the bridge don’t register very much strain. However, when a vehicle passes towards the west, all of the sensors (besides SG 3 and SG 4) experience an increase in strain. This is due to the fact that a majority of the sensors are placed on the north half of the bridge. SG 8 detects most of the strain as vehicles pass over the bridge towards the east.

Fig. 4-13. STS trigger with multiple peaks
Table 4-4. Vehicle count for in-person test

<table>
<thead>
<tr>
<th>Vehicle Number</th>
<th>Time From Test Start</th>
<th>Vehicle Type</th>
<th>Direction Of Travel</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1:06</td>
<td>Pickup Truck</td>
<td>East</td>
</tr>
<tr>
<td>2</td>
<td>1:40</td>
<td>Compact car</td>
<td>East</td>
</tr>
<tr>
<td>3</td>
<td>2:12</td>
<td>Minivan</td>
<td>West</td>
</tr>
<tr>
<td>4</td>
<td>2:33</td>
<td>Light truck</td>
<td>East</td>
</tr>
<tr>
<td>5</td>
<td>3:02</td>
<td>SUV</td>
<td>West</td>
</tr>
<tr>
<td>6</td>
<td>3:45</td>
<td>Light truck</td>
<td>West</td>
</tr>
<tr>
<td>7</td>
<td>4:26</td>
<td>Light truck</td>
<td>East</td>
</tr>
<tr>
<td>8</td>
<td>4:55</td>
<td>Light Truck</td>
<td>West</td>
</tr>
<tr>
<td>9</td>
<td>5:20</td>
<td>Light Truck</td>
<td>East</td>
</tr>
<tr>
<td>10</td>
<td>6:20</td>
<td>Light Truck</td>
<td>West</td>
</tr>
<tr>
<td>11</td>
<td>7:22</td>
<td>Mustang</td>
<td>East</td>
</tr>
<tr>
<td>12</td>
<td>10:29</td>
<td>SUV</td>
<td>West</td>
</tr>
</tbody>
</table>

Fig. 4-14 displays a plot of the decimation of SG 5 thru SG 8 during this test period. Each of the four sensors started recording at 0 \( \mu \varepsilon \), but the data of each sensor was shifted horizontally by 1 \( \mu \varepsilon \) in order to visually distinguish the data points. Additionally, Table 4-5 displays the rain flow data for this test. From this information it is shown that a rain flow algorithm serves as a close estimate for the total microstrain range and frequency for typical passenger vehicles. However, when a heavier weight vehicle passes over the bridge, the phenomenon displayed in Fig. 4-13 may occur. A rain flow would not produce the exact results of maximum microstrain range and frequency in such a situation.
Fig. 4-14. Decimation of SG 5 thru SG 8 during in-person test

Table 4-5. Rain flow data for in-person test

<table>
<thead>
<tr>
<th></th>
<th>SG 1</th>
<th>SG 2</th>
<th>SG 3</th>
<th>SG 4</th>
<th>SG 5</th>
<th>SG 6</th>
<th>SG 7</th>
<th>SG 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1</td>
<td>-</td>
<td>1</td>
<td>1</td>
<td>-</td>
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<tr>
<td>1</td>
<td>-</td>
<td>5.5</td>
<td>-</td>
<td>-</td>
<td>3.5</td>
<td>0.5</td>
<td>6</td>
<td>9</td>
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<tr>
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<td>2</td>
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<td>1.5</td>
<td>2</td>
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<tr>
<td>3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>6</td>
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<td>0.5</td>
</tr>
<tr>
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<td>1</td>
<td>1</td>
<td>5.5</td>
<td>6.5</td>
<td>7.5</td>
<td>11.5</td>
</tr>
</tbody>
</table>

4.5. **STS Data for March 21, 2017**

An additional comparison with the rain flow data was performed for a single day.

The rain flow table for SG 6 of March 21, 2017 is displayed in Table 4-6. The rain flow
from this day indicates that a total of 1,925 strain differences occurred on SG 6 during a 24 hour period. Additionally, it can be seen that a majority of these strain differences occurred in the ranges from $1 - 4 \mu e$. A majority of the vehicles that pass over the bridge are passenger vehicles which weigh an average of 4,000 lbs (1,814 kg). By observation, this type of vehicle would typically cause a strain of $3 - 4 \mu e$ to SG 6 when the vehicle passes from the east to the west. However, when a vehicle passes over the bridge from west to east, it is possible to develop up to $1 \mu e$ at SG 6. Thus the counts of 446.5 in the range of $1 \mu e$ shown in Table 4-6 is likely due vehicles passing the bridge from west to east. This value may also be from light vehicles passing from east to west such as a motorcycle.

On this day, the STS software recorded a total of 248 event triggers. All of these triggers were then analyzed individually and the absolute maximum value of micro strain was recorded. A counting of the maximum strain records from the 248 triggers for each of the strain gauges attached to the Nibley Bridge is shown in Table 4-7. Through observation of this table, sites SG 3 and SG 4 contain all of the counts at a range of $0 - 1 \mu e$. This is expected, and sought for since these gauges are located at the centroid of each girder.

A plot of range in micro strain and count for SG 6 due to triggered events on March 21, 2017 is displayed in Fig. 4-15. It can also be concluded that the 41 counts of micro strain in the range of $0 - 1 \mu e$ likely resulted from heavier than passenger vehicles passing the bridge from west to east. This conclusion can be made since every strain gauge is set to record a trigger event if that gauge exceeds a value of $2 \mu e$ for 0.5 seconds or more. Thus, if a vehicle passing from west to east triggers SG 8, then SG 6 will also record its values of micro strain and will likely fall in the range of $0 - 1 \mu e$. 
4.5.1. Large Recording of Strain

On the day of March 21, 2017, a series of successively large counts in micro strain were detected. Three of these large triggers are shown in Fig. 4-16. This micro strain difference exceeds the typical micro strain differences from vehicles by about 31 με at SG 6. It is expected that the range of micro strain caused by passenger vehicles will cause little damage to the bridge, even with numerous cycles. However, SG 6 experienced more than
Table 4-7. Trigger range counting from March 21, 2017

<table>
<thead>
<tr>
<th>Range</th>
<th>SG 1</th>
<th>SG 2</th>
<th>SG 3</th>
<th>SG 4</th>
<th>SG 5</th>
<th>SG 6</th>
<th>SG 7</th>
<th>SG 8</th>
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<td>130</td>
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<td>248</td>
<td>55</td>
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<td>248</td>
<td>248</td>
<td>248</td>
<td>248</td>
<td>247</td>
</tr>
</tbody>
</table>

35 με at its greatest value. This is an excessive amount of micro strain and can cause damage to a bridge after numerous cycles of load at such a magnitude. Additionally, it is again noticed that SG 2 increases in micro strain shortly after many of the remaining gauges detect the increase. This observation could indicate that vehicles cause much less damage to outer girders, and that these are not affected until after the center girders experience a peak in micro strain. Due to such large micro strain recordings without a sure knowledge of the vehicles that caused this, it is recommended to perform live load tests at the bridge
Fig. 4-15. Peak trigger range values for SG 6 on March 21, 2017

Fig. 4-16. Large readings in micro strain detected on March 21, 2017
CHAPTER 5
SUMMARY AND CONCLUSIONS

5.1. Summary

The Nibley Bridge is located near the intersection of Highway 165 and 2600 South on the border of Nibley, Utah and Millville, Utah. It serves the purpose of connecting Highway 165 with the newly constructed Ridgeline High School. It was constructed using deck bulb-tee girders rather than having a deck cast over typical girders. The Girders were fabricated in early December 2015 and delivered to the site in late January 2016. The bridge and its approaching road of 2600 South was completed and open to traffic in August 2016. The overall span of the bridge is 85 ft (25.91 m) from the face of each abutment and 89.5 ft (27.28 m) to the end of each girder. The bridge is a total of 60 ft (18.29 m) wide and serves two lanes of traffic.

The Utah Transportation Center (UTC) as well as the Mountain Plains Consortium, sponsored a study to investigate the long-term performance of the Nibley Bridge. In order to perform the study, the Nibley Bridge was instrumented using 50 thermocouples at five locations on the bridge. As well, 16 vibrating wire strain gauges were attached at four of the same five locations. The instrument locations were at mid-span, quarter-span, and the end of an interior girder. An exterior girder contained sensors corresponding to the mid-span and end of the girder.

Through close inspection of the data recorded from these sensors, strain and temperature fluctuations within the bridge were examined. Bridge monitoring with temperature gauges provided confirmation that the temperature fluctuations and temperature gradients throughout the depth of the bridge remained within a specified
threshold. Additionally, the strain gauges provided confirmation that the amount of prestress loss of the girders remained under a desired amount of stress.

Short-term bridge monitoring is an effective method to analyze changes that occur quickly within a bridge. Additional strain gauges, a datalogger system, and a software package were provided by Bridge Diagnostics Inc. (BDI). A total of 8 strain gauges were externally mounted at the mid-span of the Nibley Bridge. 2 of the gauges were mounted at the centroid of two of the girders, while the remaining 6 gauges were attached to the bottom face of individual girders. These gauges provided close inspection of strain variances that occur as vehicles pass over the bridge. Through the use of these gauges as well as its provided software, an analysis of its data was used to verify that the bridge was performing as desired.

5.2. Conclusions

Measured temperature changes from the installed thermocouples was analyzed and compared to various codes as well as predictive methods. The average bridge temperature was compared to the AASHTO LRFD Bridge Design Specifications (2010). The Average Bridge Temperature (ABT) was analyzed and compared to several methods to predict ABT. The temperature gradients were examined and compared to a gradient established by AASHTO as well as a design specification established by Priestley (1978). The calculated camber of the bridge was compared to various established predictive models and computer models. The measured prestress losses from the embedded strain gauges was studied and matched against two methods provided in AASHTO (2010) as well as a method provided by the Precast Concrete Institute (2010).
Additional analyses were performed using strain gauges and software provided by Bridge Diagnostics Incorporated (BDI). The results from the externally mounted strain gauges were studied and various observations were determined. Based on the findings of this research, several conclusions were formed.

1. The maximum average bridge temperature occurred during July 2016 with a magnitude of 103.84 °F (39.91 °C) short of the AASHTO LRFD Bridge Design Specifications (2010) of only 0.16 °F (0.09 °C). The minimum average bridge temperature occurred during January 2017 with a magnitude of -5.54 °F (-20.85 °C) short of the AASHTO LRFD Bridge Design Specifications (2010) by 4.46 °F (2.48 °C). The Kuppa method (Kuppa 1991) predicted the closest average bridge temperature for Girder 1. While the ERL method (Rojaz 2014) predicted the closest average bridge temperature for Girder 5.

2. The maximum positive temperature gradient exceeded the values established in the AASHTO LRFD Bridge Design Specifications (2010) when no asphalt overlay was present at over the bridge. After the asphalt overlay was applied, the maximum and minimum temperature gradients were below the values established by AASHTO as well as Priestley (1978). Which would suggest that the asphalt serves as an insulator to the temperature gradient.

3. The Peak Temperature Camber model served as a close representative of measured thermal camber by use of the equations presented by Nguyen et al. (2015). CSiBridge is also an effective tool to model a single span bridge for
determining the thermal camber due to large temperature gradients present in a concrete superstructure bridge.

4. The measured elastic shortening losses for both Girder 1 and Girder 5 exceeded the predictive methods described. The approximate, refined, and PCI methods overestimated the creep and shrinkage of Girder 1, while only the PCI method overestimated the creep and shrinkage of Girder 5. The predictive methods proved to be very accurate for Girder 1, yet predicted lower values for Girder 5. Additionally, the values for the two AASHTO LRFD Bridge Design Specification (2010) methods by use of E2 predicted an average of 3.75% more accurate results compared to the use of E1.

5. The event recording and rain flow features of the STS software provides accurate results of strain developed at the bottom surface of a bridge. The system proved to be an effective tool to develop a magnitude and frequency chart of strain that a bridge experiences due to applied loads. Passenger vehicles produce the majority of strain counts at the Nibley Bridge, yet heavy trucks also produce counts of high magnitude microstrain.

5.3. **Recommendations for Additional Research**

Several recommendations for further research regarding the concepts presented herein include the following:

- Use the Kuppa, Emerson, and ERL methods to develop a more accurate method for predicting the yearly average bridge temperature (ABT)
• Measure ambient temperature near the bridge site every hour in order to obtain more accurate thermal camber predictions

• Determine the tension and compression stresses of the Nibley Bridge using the obtained temperature data

• Perform a rain flow test with the STS software set at a minimum of $6 \mu\varepsilon$ in order to develop a magnitude and frequency history of large strains occurring at the Nibley Bridge.

• Develop a program with STS to record a maximum and minimum strain variance for triggered events

• Perform a live load test at the Nibley Bridge and record the strain data using the STS Live software
REFERENCES


