Final Report

In-Service Behavior of a Cast-in-Place Segmental Concrete Box Girder Bridge

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Project Objectives

The goal of this study is to further the state of knowledge with regard to the behavior of segmental concrete box girder bridges. The existing Sam Houston Ship Channel Bridge (SHSCB) (Figure 1) is in the process of being replaced by the Harris County Toll Road Authority (HCTRA) in Houston, Texas, after nearly 40 years of service due to traffic volume and ship clearance demands. SHSCB is a critical structure in the history of segmental bridges. When constructed, it was the longest of its kind in the United States [1]. A unique opportunity is presented now that the structure is being decommissioned. The project team is taking full advantage of this situation through a detailed field study that can identify the in-service behavior, primarily under thermal variations[1].

Figure 1: Sam Houston Ship Channel Bridge (SHSCB)

Approach

The approach to identifying the in-service behavior of SHSCB was through an in-depth field evaluation of the structure. The process began with a comprehensive review of the existing documentation. Then planning for the field study was performed. This included the development of instrumentation plans for a monitoring system, followed by implementation. The structure was monitored for over a year, and data was recorded. The next step was to perform data analysis to evaluate daily and seasonal behavior. A comparison was made to theoretical models. Finally, conclusions were drawn from the study.

Related Prior Research

Studies on the thermal behavior of segmental concrete box girder bridges have been conducted. Hedegaard et al. (2012) performed an extensive study on the I-35W St. Anthony Fall Bridge [2]. This study included field monitoring and comparison of the results to theoretical models [3]. Recommendations for the design of thermal gradients for segmental concrete box girder bridges were proposed.

Other related studies that have focused on the long-term behavior of segmental concrete bridges have been conducted for many decades. Tadros et al. (1979) recognized the

[1] The original scope for this study included a final phase for deconstruction monitoring to field determine the locked-in prestressing forces. This task was cut from the project due to continual delays in the construction timeline, which is outside the control of the research team.

importance and presented a computer method to predict stresses and deflections due to concrete creep and shrinkage along with steel relaxation [4]. Load testing to evaluate the behavior of these structures dates back over 30 years [5]. Even research into probabilistic prediction methods for long-term deflections and internal forces was carried out [6].

Despite these studies, and many not mentioned, there have been serious issues with the long-term deflections and stresses of segmental concrete bridges. Bazant et al. (2011) presented a "wake up call for creep" and indicated 66 long-span segmental concrete box girder bridges that had experienced excessive deflections [7]. Bazant et al. (2012) also studied the excessive deflection and eventual collapse of the Koror-Babeldaob Bridge in Palau, which was built at nearly the same time as the SHSCB [8, 9]. The results indicated that the 1971 ACI model (reapproved in 2008) [10], CEB [11], and JSCE [12] models severely underestimated long-term deflections as well as prestress losses. The GL model [13] provided better predictions but was still not sufficient. Many recent studies have further explored the long-term losses and further compared their results to different creep and shrinkage models [14]. Other studies have explored model-updating techniques [15].

The SHSCB presents a great opportunity to learn from a segmental concrete box girder bridge for 40 years. This structure has been regarded as a success for the segmental concrete industry and the bridge owner. The research aims to understand the in-service behavior of the structure.

Background on SHSCB

The SHSCB is a three-span cast-in-place variable depth segmental concrete box girder bridge. The bridge was built from 1980 to 1981 and is located on the east side of Houston, Texas. The bridge spans the Buffalo Bayou and runs nearly directly north-south. Figure 2 illustrates the location and orientation of the bridge.

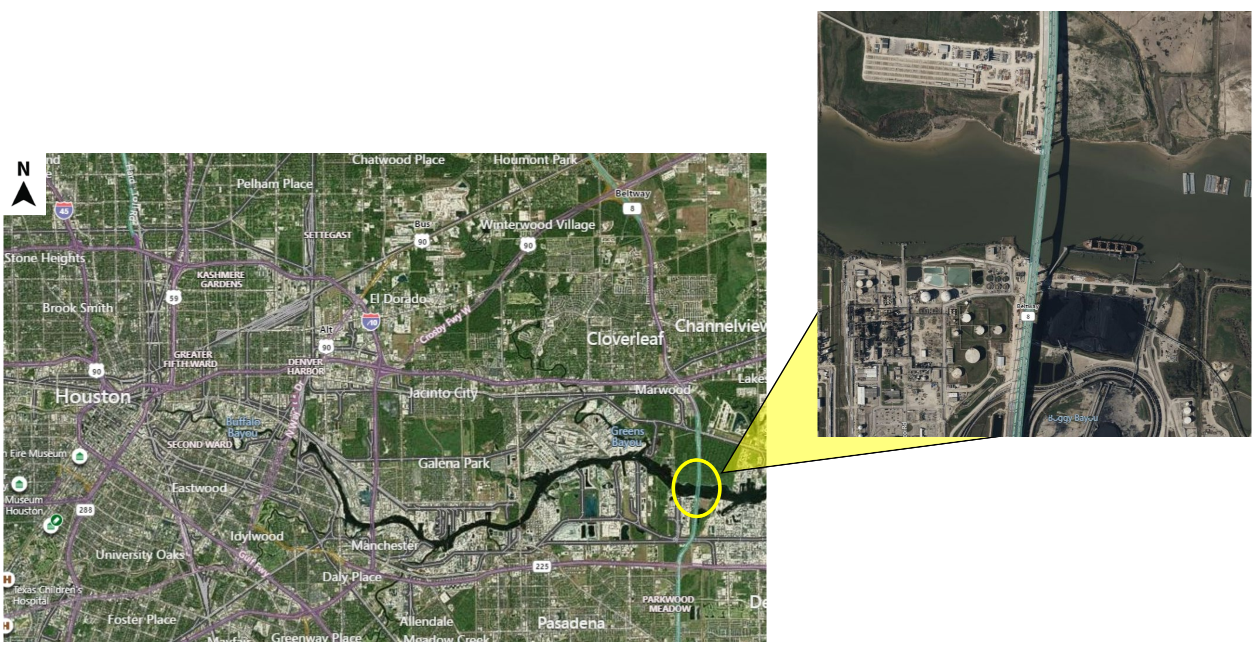

Figure 2: Ariel View of the SHSCB

The bridge carries four lanes of traffic (two in each direction) with a median barrier. The construction of the bridge utilized the balanced cantilever method. This produced the final span arrangement of 375-750-375 feet, as illustrated in Figure 3.

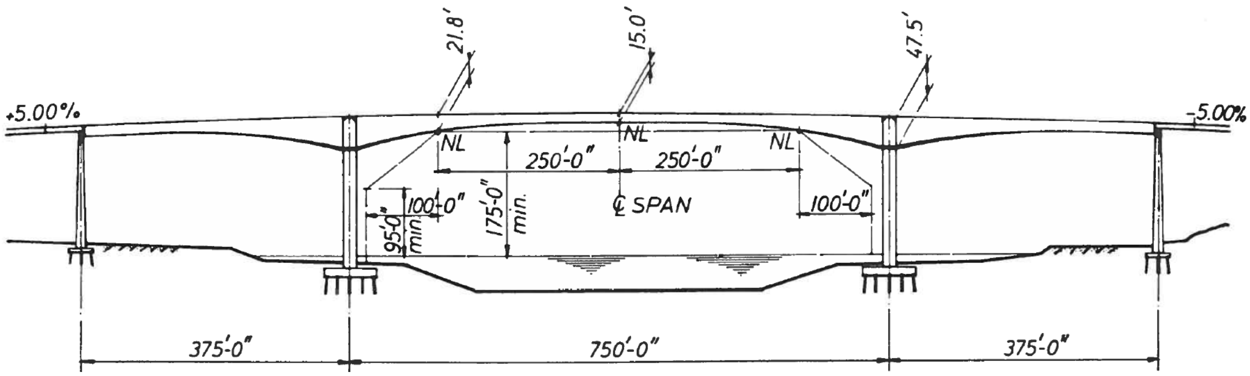

Figure 3: SHSCB Elevation View

The box girder has two cells with a depth varying from 15 feet to roughly 47.5 feet (see Figure 4). Post-tensioning of the box girder is included in the longitudinal, transverse, and vertical directions.

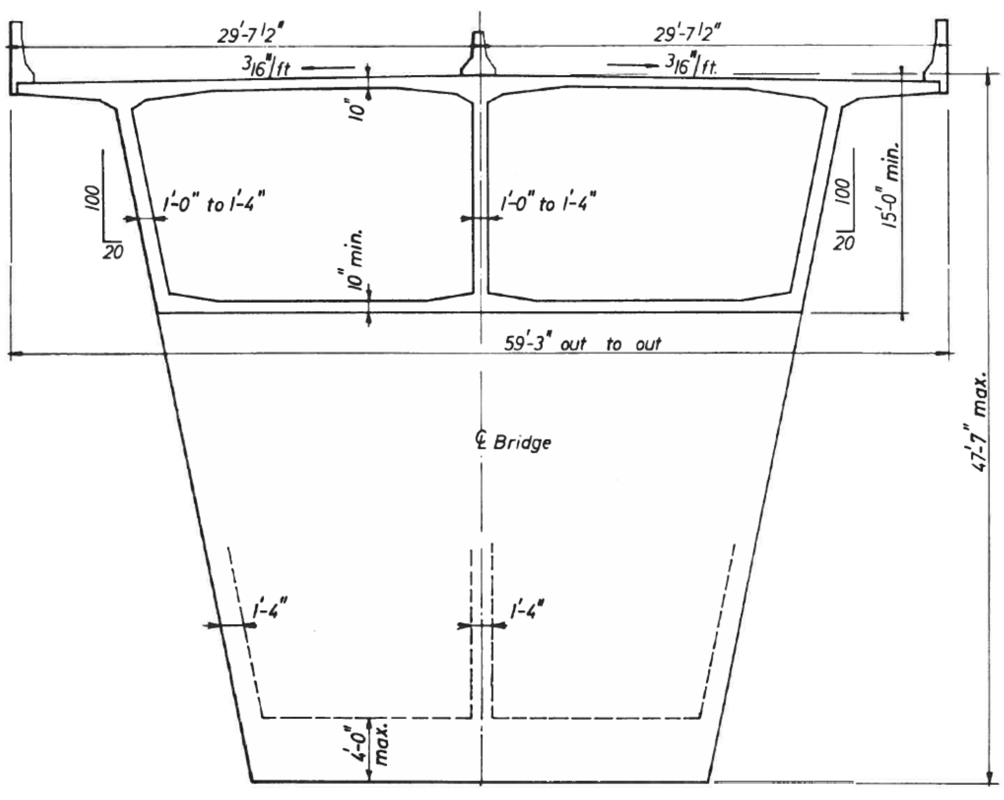

Figure 4: SHSCB Cross-Section View

The SHSCB was the longest box girder bridge in the United States at the time of construction. Overall, the performance (per the owner) has been excellent. The reason the structure is being replaced is primarily due to the traffic volume demands and not structural reasons.

Research Performed

This section explains the primary research tasks performed as part of this study. The preparations and field monitoring system installation are first discussed. Then the in-service monitoring is discussed, followed by the data analysis performed. The overall findings of the project are presented in the following section.

Preparations and Field Installation

Preparations

Prior to the field study, several preliminary tasks were performed. The first task was a comprehensive review of the existing documentation. Mr. Gowen Dishman, from the advisory team, supplied the researchers with the as-built drawings and hundreds of photographs from when the bridge was originally constructed. These photographs are labeled by the date taken; therefore, they are helpful in understanding the specific construction timeline. Several journal/magazine articles on the SHSCB were also reviewed, which provide further information on the construction process/loading (Figure 5) [16-18]. In addition, the textbook "Construction and Design of Prestressed Concrete Segmental Bridges" by Podolny and Muller (1982) utilizes the SHSCB for a full design example [19]. This provides great information on the intended structural behavior.

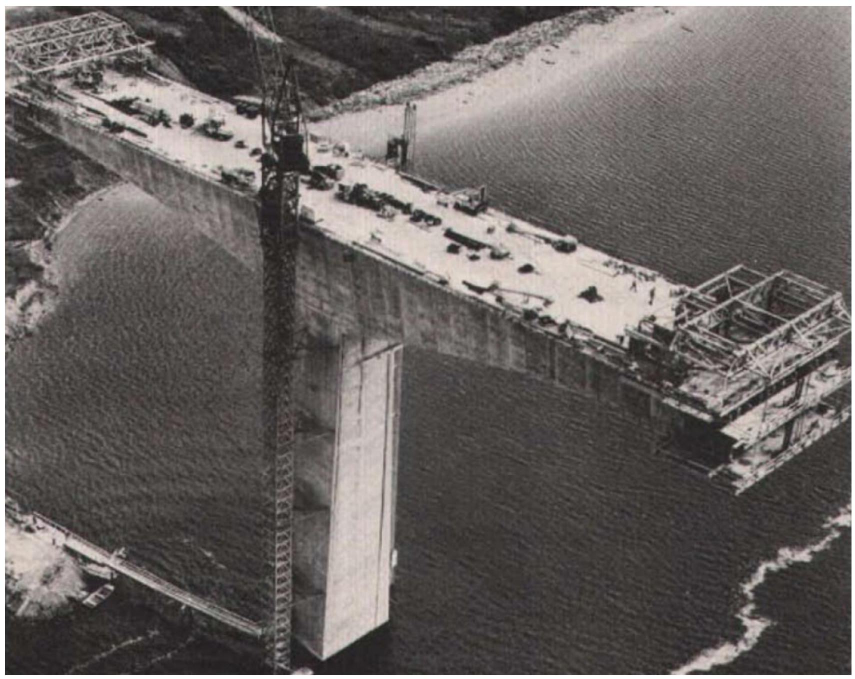

Figure 5: SHSCB Construction Photo from October 1981

The next task was the development of the instrumentation plan. This included three primary efforts: (1) measurement location selection, (2) sensor selection, and (3) data acquisition (DAQ) selection. Each is briefly discussed below.

(1) Measurement Locations

The measurement locations were to focus on the thermal behavior of the SHSCB. The approach was to select three bridge cross-sections to be instrumented with paired strain and temperature measurements. These three sections are illustrated in Figure 6.

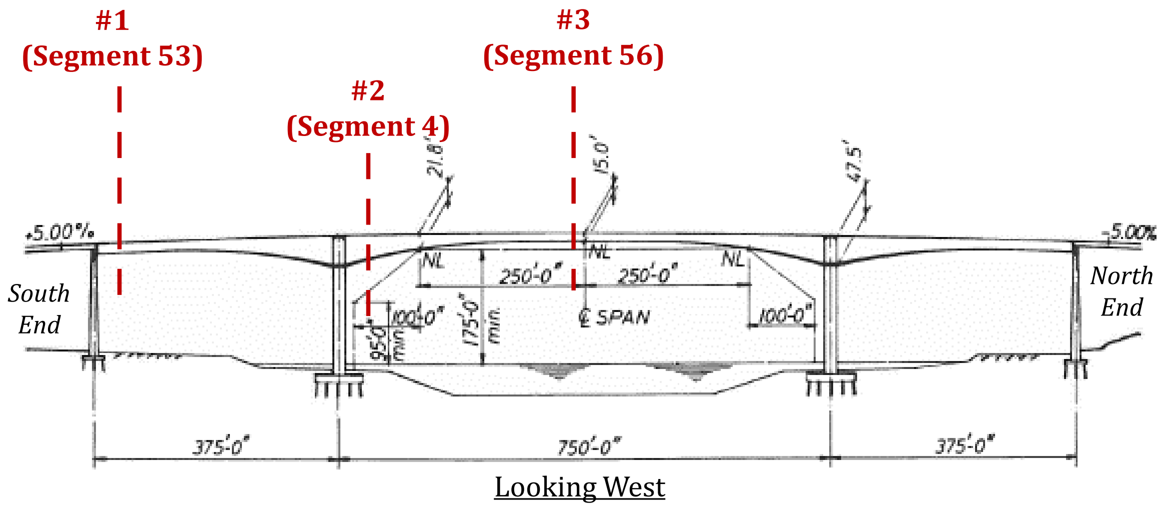

Figure 6: Global Measurement Sections and Segment Numbers

Section 1 (Segment 53) and Section 3 (Segment 56) were selected to help distinguish between the primary and secondary thermal stresses [20]. The primary thermal stresses are those that result from temperature change without internal boundary conditions (two interior piers in this case). The secondary thermal stresses result from continuous structures where the curvature from the primary thermal response is partially restrained. As a result, Section 1 shall predominantly undergo primary thermal stresses, where Section 3 will include a combination.

Figure 7 shows the specific measurement positions for Sections 1 and 3. The intent was to obtain a measurement spread throughout the cross-section. This should allow for in-plane and out-of-plane bending of the section. It also provides a reasonably comprehensive temperature distribution inside the structure.

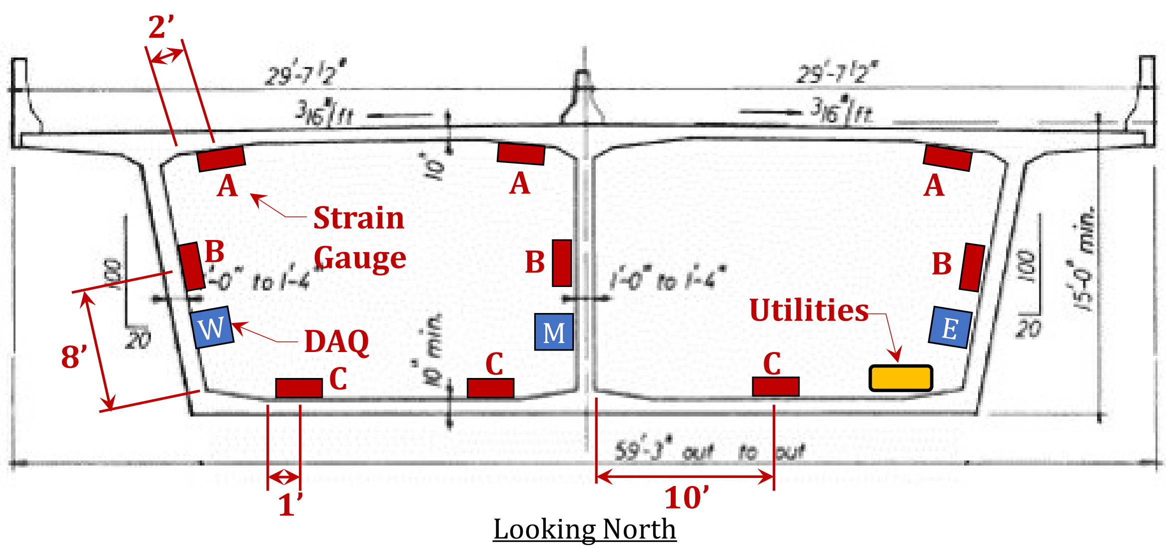

Figure 7: Local Measurement Positions in Sections 1 and 3

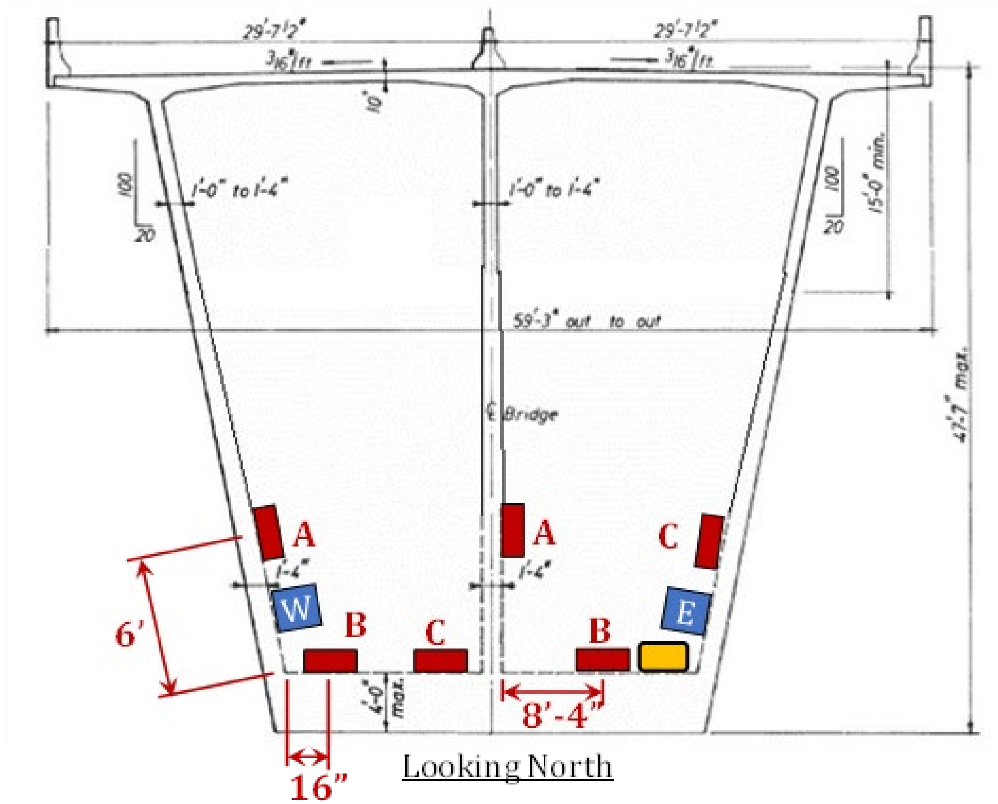

Figure 8: Local Measurement Positions in Section 2

Section 2 (Segment 4) was added for additional information near a support and at a location where the cross-section is deeper. Figure 8 shows the specific measurement positions for Section 2. Note that due to field logistics (sloped surface), only the bottom section of the box was targeted for measurements.

(2) <u>Sensor Selection</u>

To achieve the measurements desired, a long-term stable sensor is needed. For this study, the literature (and the experience of the research team) indicated that vibrating wire (VW) or fiber optic sensors are the best selections [21]. Due to the cost and familiarity of the research team, VW sensors were chosen.

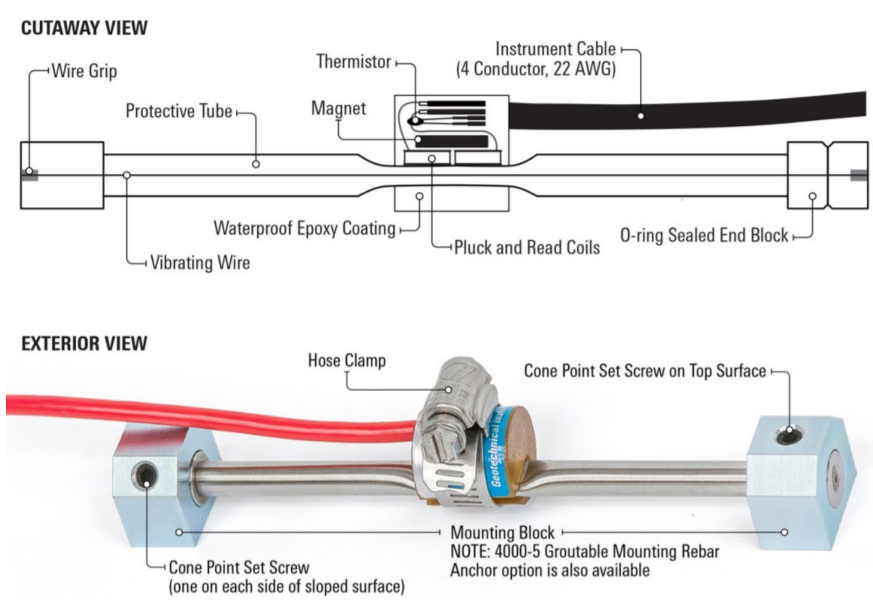

Figure 9: Geokon VW Strain Gauges Selected for the Study

The specific type of sensor is the Geokon Model 4,000 6-inch VW strain gauges with built-in thermistors. Figure 9 illustrates the primary components and inner workings of the sensor. These sensors are highly rugged and epoxy anchored into concrete.

(3) Data Acquisition Selection
The data acquisition (DAQ) system for this study needed to reliably read VW sensors. Another area of importance is to have a low-power system since there is no on-site power inside the bridge and no access outside for solar power. Also, the equipment needs to be rugged to withstand the harsh environment.

Figure 10: Campbell Scientific CRVW3 Datalogger

Campbell Scientific was the logical choice for the DAQ system due to its proven capabilities in these areas. The CRVW3 datalogger was selected (Figure 10). This system has the capability to sample three paired VW sensors with thermistors at slow speeds. It is powered by a 12-volt battery that can run for over six months, depending on the sampling rate. The entire DAQ system comes housed in a rugged weatherproof enclosure. There is an option for an RFI radio, but that was not selected for this study due to the additional power drain and cost. Each datalogger runs autonomously. The download of the data was conducted through a direct connection.

Field Installation

The field installation was done in stages due to the significant logistical challenges of accessing the inside of the box girder. The manhole to access the inside is within an active traffic lane. Therefore, the shutdown of a lane is required. In addition, the manhole cover is welded shut, requiring the Toll Authority personnel to burn off the weld (and weld it back later). The heavy traffic volume in the Houston area also necessitates limited lane shutdown durations to avoid rush hour. The research team accessed the inside of the structure four times as part of this study. Each of these ventures is discussed below.

(1) July 2, 2019 - Reconnaissance
The first field visit was a reconnaissance to plan for future installations. This was allowed by the bridge owner because the annual bridge inspection was occurring. Therefore, a traffic lane was already closed. The field visit was tremendously helpful in fully understanding the complexity of working inside the box girder. For example, no lighting is available inside, and challenges associated with this were realized. The future

installations included handheld lighting and headlamps. In addition, the scale of the two diaphragms at each pier was better appreciated. All work in the center span required transporting equipment over two sets of ladders. Figure 11 shows photos from inside the box girder looking at the end diaphragm (a) and the pier diaphragm (b).

Figure 11: Inside SHSCB Showing the (a) End Diaphragm and (b) Pier Diaphragm

Other helpful information included the dimensions of access holes. The manhole into the box girder and the hole between the two cells were relatively small. Only narrow ladders and equipment could be used (see Figure 12).

Figure 12: SHSCB (a) Manhole and (b) Ladder Access Inside the Segmental Box Girder

(2) November 9, 2019 – Installation #1
This installation was conducted on a Saturday to allow for a longer time window of the lane shutdown. The work conducted was the installation of the monitoring systems at Sections 1 and 3, shown in Figure 6 and Figure 7. The research team members included five individuals from Texas A&M University. The primary tasks at each location were the installation of the VW strain gauges (epoxy anchored), mounting the DAQ, and then connecting the cables from the gauges to the DAQ securely.

The installation procedure for each strain gauge was as follows.

1. Identify the location with the cross-section
2. Mark the holes to be drilled
3. Drill pilot holes
4. Drill ½" diameter holes 1.0" deep (see Figure 13)
5. Clean out the holes of dust and debris
6. Verify the sensor with a dry fit
7. Inject adhesive into the holes and install the gauge (see Figure 14)
8. After sufficient cure time, attach the plucking coil assembly

Figure 13: Installation of Strain Gauge in Section 1

A similar procedure was utilized to mount the DAQ system. The primary difference was that wedge anchors were utilized to increase the speed of installation. Figure 15 (a) shows a field-installed DAQ system.

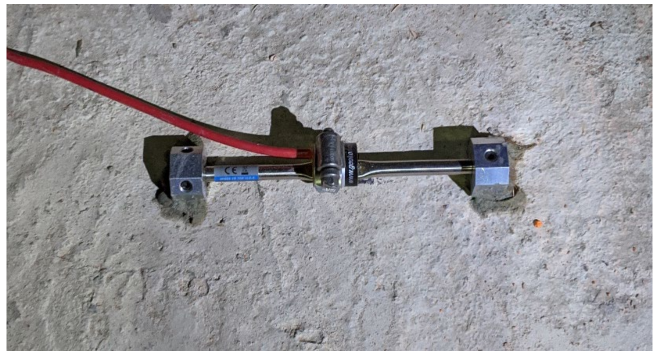

Figure 14: Epoxy Anchored Strain Gauge

The biggest field installation challenge was mobilizing at Section 3. All the equipment (including the ladder) had to be carried roughly 750 feet. This included transferring everything over both internal diaphragms at the south pier. The height of the opening in the diaphragm is 16 feet. Fortunately, there are fixed ladders at this location. However, this process took a good portion of the installation time.

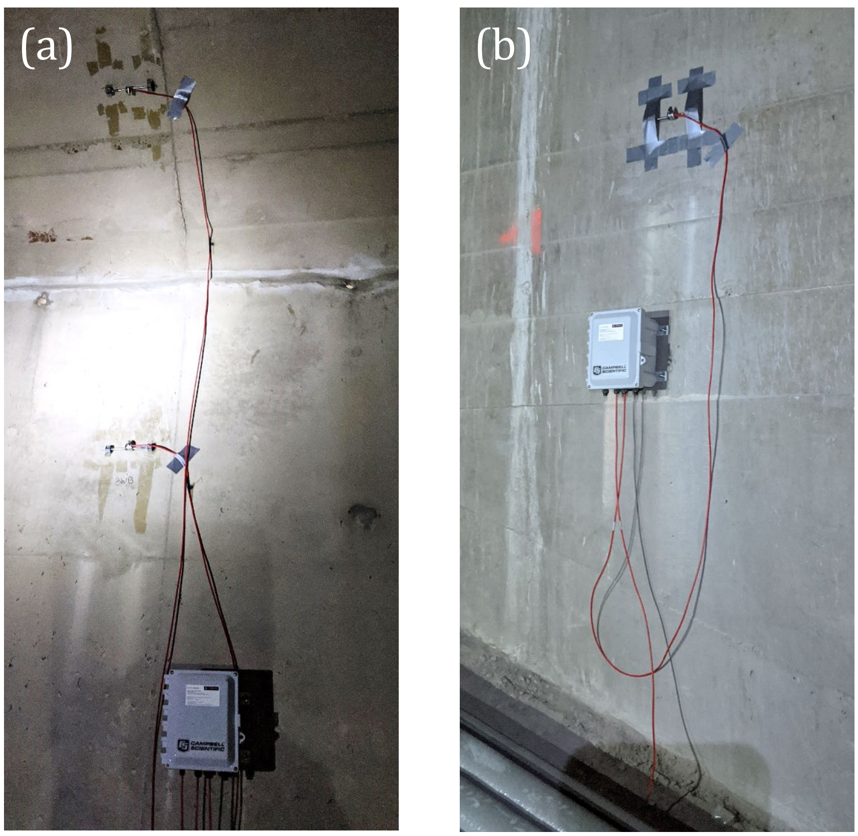

Figure 15: (a) Section 3 DAQ and Strain Gauges A and B and (b) Section 2 DAQ and Strain Gauge C (Note the tape is temporary)

(3) <u>July 25, 2020 – Installation #2</u>
This installation focused on the maintenance of Sections 1 and 3, along with the installation of Sections 2. The maintenance at Sections 1 and 3 was simply securing the cables, replacing the batteries, and downloading the data. The work in Section 2 was essentially the same as Installation #1, except the specific sensor locations were different

within the cross-section. Figure 15 (b) shows an example photo of the setup in Section 2. Only the west side of Section 2 was installed due to time constraints.

(4) March 3, 2021 – Recording
This fieldwork recorded data at all three cross-sections. The batteries were also replaced at each DAQ, and the clocks were reset. In addition, data quality checks were performed.

In-Service Monitoring

The following section summarizes the in-service monitoring along the SHSCB. The monitoring results are separated below into two categories. The first is long-term seasonal behavior, and the second is daily behavior. The following section goes further into data analysis through a comparison of the measured response with theoretical models.

Seasonal Behavior

The in-service behavior was monitored for roughly 16 months at Section 1 (Segment 53 – south end) and Section 3 (Segment 56 - midspan). In addition, approximately 8 months of behavior was captured for Section 2 (Segment 4). The measured response was primarily due to temperature variations. Overall, the data quality was good. One sensor (strain gauge A, west side of Section 3) did not record data. This is likely due to a cable connection issue.

The 16 months of in-service data from Sections 1 and 3 covered all seasons. This provided a good range of thermal variations. The temperature range inside the box girder was from 43°F to 97°F. Of course, the temperature magnitudes increased from the winter to the summer. The spread of temperatures inside the box girder also increased due to the increase in thermal gradients from increased solar radiation.

The recorded data has been converted from strains to stresses and plotted for the different sections. Figure 16 and Figure 17 illustrate the variations in stress for Section 1 and Section 3, respectively. As anticipated, the variation is within 2 ksi. This is relatively low compared to the locked-in stresses. However, these stresses are greater than those induced by live loading.

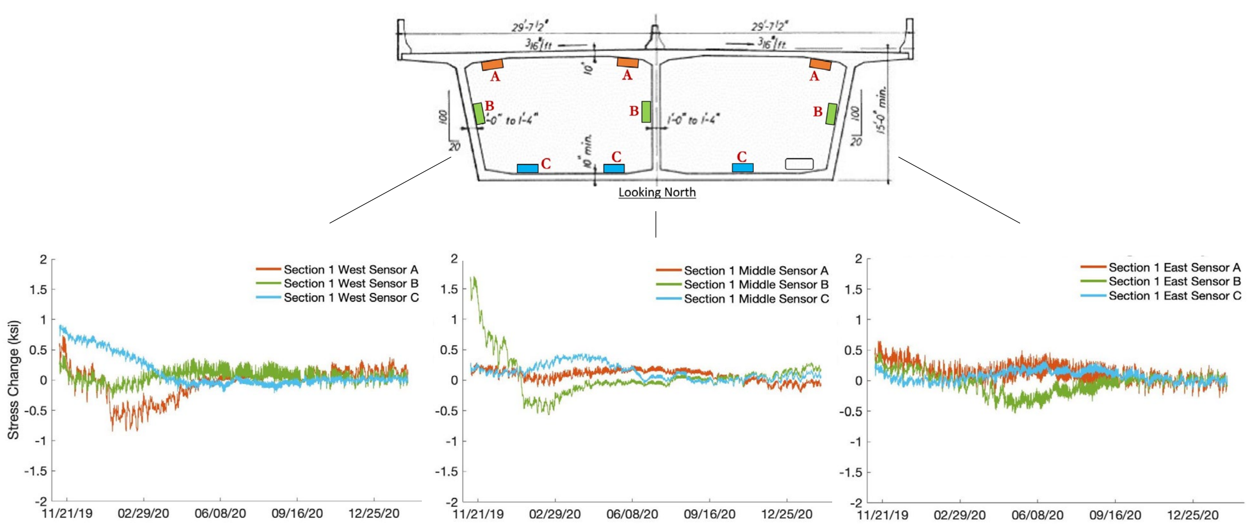

Figure 16: Section 1 (Segment 53) Stress Changes over 16 months of Monitoring

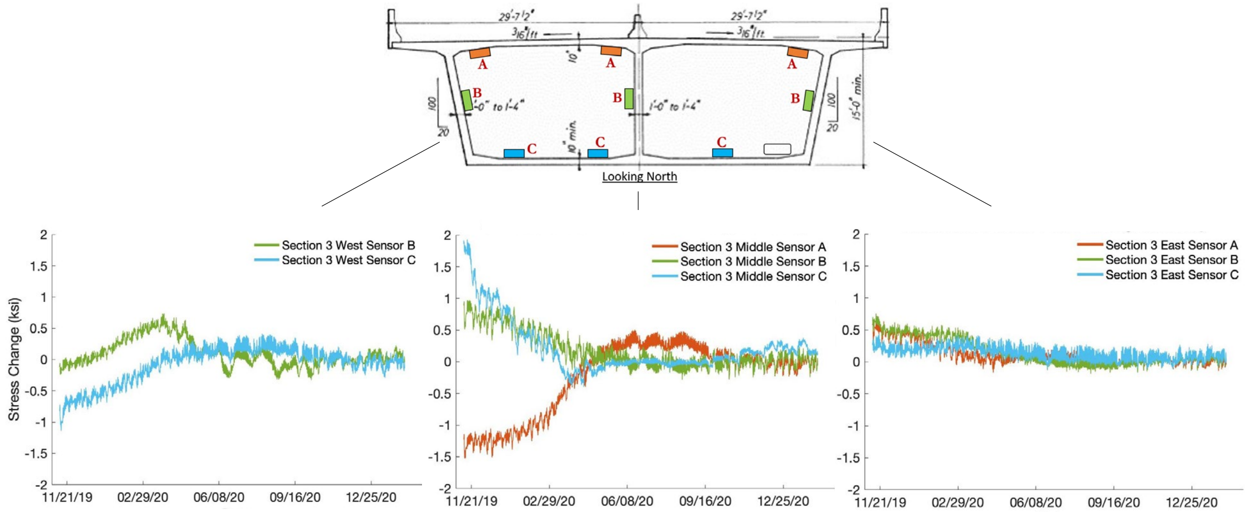

Figure 17: Section 3 (Segment 56) Stress Changes over 16 months of Monitoring

The general takeaways from seasonal behavior start with the stability of the performance. In all cases, the overall trend of the response is flat, indicating no declining performance of the system. In addition, the magnitude of the seasonal response is relatively minimal, which shows the continual release of forces due to boundary restraints. These mechanisms are still functioning adequately after more than forty years.

Daily Behavior

To evaluate the daily behavior, the data was also plotted over short (two-day) windows to better observe the thermal effects. Data presented herein are representative datasets observed in the winter and summer seasons. The stress variations are plotted along with the temperature variations at the sensor locations.

Figure 18 presents two-day datasets from the winter season for Sections 1 and Section 3. The data was zeroed at the start of this dataset, where the temperature was relatively uniform. It should be noted that the temperature is at the location of the sensors on the inside surfaces of the box girder. Concrete has a relatively slow thermal inertia. As a result, the temperature shown lagged from ambient temperature trends.

Section 1 results (top row of Figure 18) exhibit general behavior trends. The first is the expected vertical gradient effects which induce curvature and resulting flexural stresses. For example, looking at the Section 1 West data, the temperature drops at different rates, producing a vertical gradient with the higher temperature at the top of the box girder (clearly seen around 5 PM the first day). As a result, the bridge wants to deform in a concave down shape. The restoring forces to prevent this induced curvature counteract the deformation, inducing a positive bending moment. The positive bending moment produces compressive stresses at the top and tensile stresses at the bottom. As the temperatures continue to decline throughout the evening, it is mostly a uniform change in temperature. The near-uniform decrease in temperature causes the structure to want to contract, but the restraints provided by the supports counteract this. Therefore, tensile stresses are induced, peaking around 5 PM the second day.

Section 1 East data have similar trends. Some of the differences between the East and West datasets are due to the transverse temperature gradients. A north-south oriented bridge is more complex in this aspect. The sun rising on the east side of the bridge heats this side first. Of course, the reverse happens at the end of the day. This is supported by the Middle data, which shows lower variations in temperature and stresses.

The data from Section 3 (bottom row of Figure 18) is even more complex than Section 1. As mentioned earlier, this location of the bridge experiences primary and secondary stresses from thermal variations. This can change the behavior (bottom row of Figure 18). However, the impact is not substantial, with the magnitudes of the response still within a similar range.

The Section 2 data is not shown since it is an incomplete picture, only having the West side data. The general trends are consistent with the other sections. However, the magnitude of temperature variation and stresses is lower. This is due to the limited height of the sensor placement at Section 2, along with the substantial depth of the box girder at this location.

Figure 19 presents two-day datasets from the summer season for Sections 1 and Section 3. Again the data was zeroed at the start of the dataset. However, in this case, there was more of a gradient at this timeframe compared to the winter dataset illustration above. This should be taken into account when reviewing the data presented in Figure 19.

Section 1 results (top row of Figure 19) and Section 3 results (bottom row of Figure 19) illustrate typical behavior. Relatively similar trends are observed compared to the winter data. The consistent summer sunny days continually heat the top of the box girder. The lagged temperature effect that was previously described is more pronounced in the summer. The box girder continues to increase in temperature until midnight.

The daily summer behavior did produce higher tensile stresses compared to the winter. For example, Section 1 East data shows a drop in temperature that produces a restoring negative bending moment where the tensile stresses at the top surface are over 0.3 ksi. This occurs over both days.

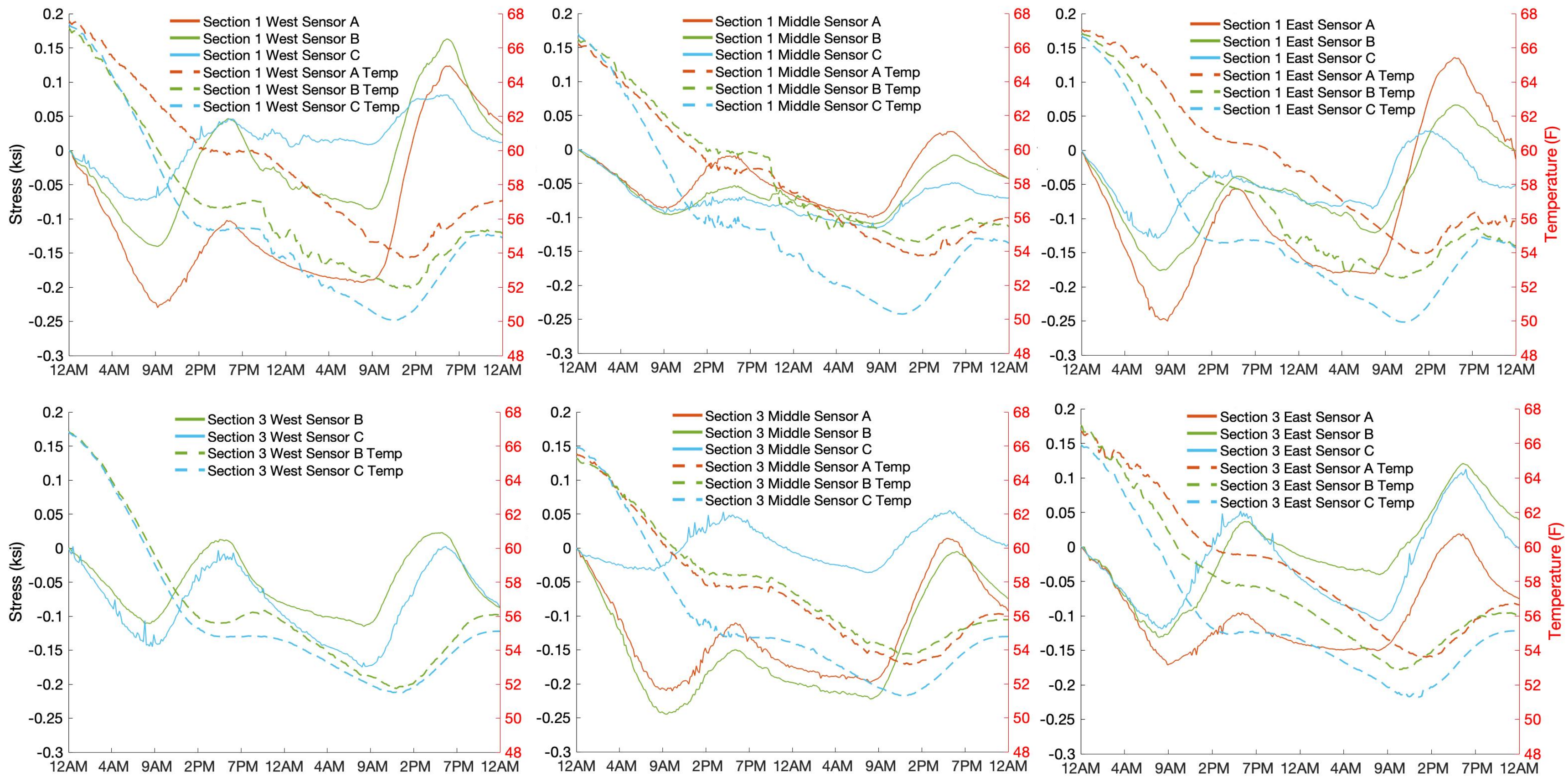

Figure 18: Section 1 (top row) and Section 3 (bottom row) Stress Changes over December 24th and 25th of 2020

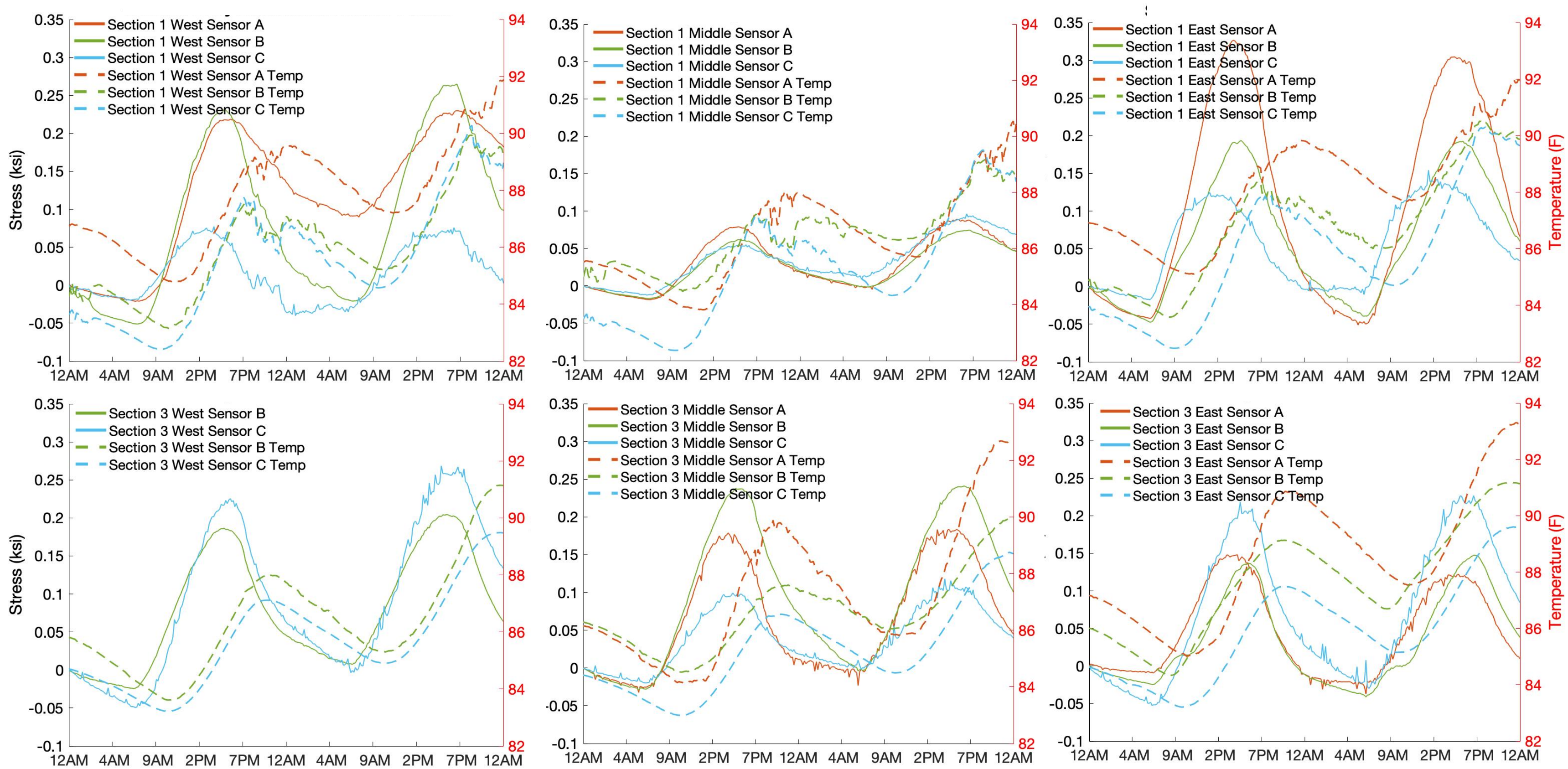

Figure 19: Section 1 (top row) and Section 3 (bottom row) Stress Changes over August 2nd and 3rd of 2020

Data Analysis

The primary method for data analysis was to compare the measured results from this study to well-established theoretical models. The first model selected is from the AASHTO LRFD Bridge Design Specification [22]. Figure 20 (a) illustrates the AASHTO design positive gradient for solar radiation Zone 2. The plotted gradient temperatures are given as the temperature difference of the cross-section from the temperature in the webs. The design negative gradient for structures with plain concrete decks and no asphalt overlay is found by multiplying the design positive gradient temperatures by –0.3. The second theoretical model selected is from the New Zealand Code. A fifth-order curve is used to define the temperature gradient, as illustrated in Figure 20 (b). Detailed information can be found in Priestly (1978) [23].

Overall, the theoretical models are intended for the design of new bridges. There is significant uncertainty regarding thermal gradient demands. Parameters such as the time of day, season, the orientation of the bridge (north-south vs. east-west), cross-sectional geometry, and so on will impact the thermal behavior of the structure.

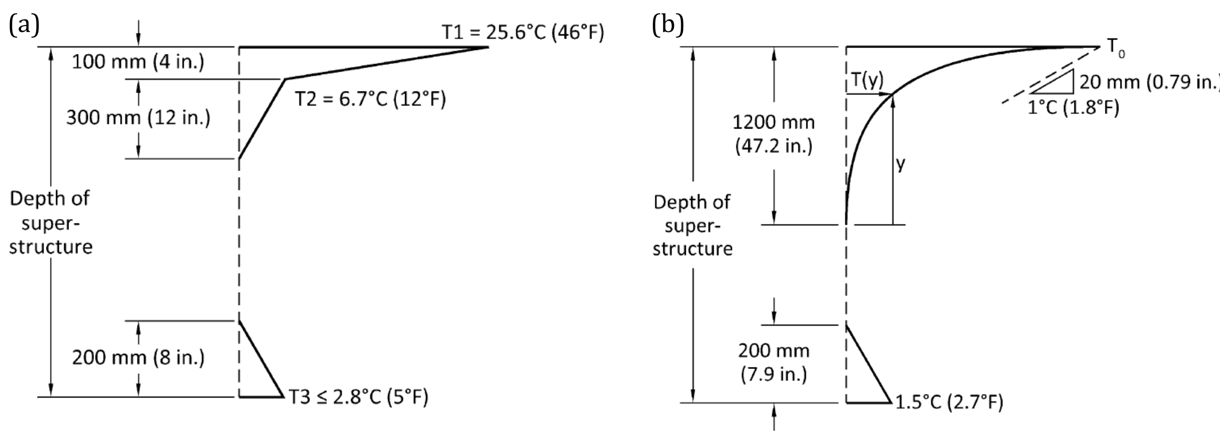

Figure 20: Positive Thermal Gradient Model from (a) AASHTO LRFD Bridge Design Specification [22] and (b) New Zealand Code as Presented by Priestly [23]

The approach for comparison of the measured response to the theoretical models was to compare five days of peak stress within two seasons (winter and summer). Figure 21 provides a comparison in Section 1. Overall, the measured stresses were relatively low in magnitude. The peak stresses are at the outer fibers, which logistically could not be measured. However, the locations that could be compared were within a reasonable range. The winter results (Figure 21 (a)) showed larger variability in the data compared to the summer results (Figure 21 (b)). The peak summer stresses were tensile but relatively low in magnitude. In summary, neither theoretical model showed favor over the other in this case. Overall, both models are reasonable for general design criteria.

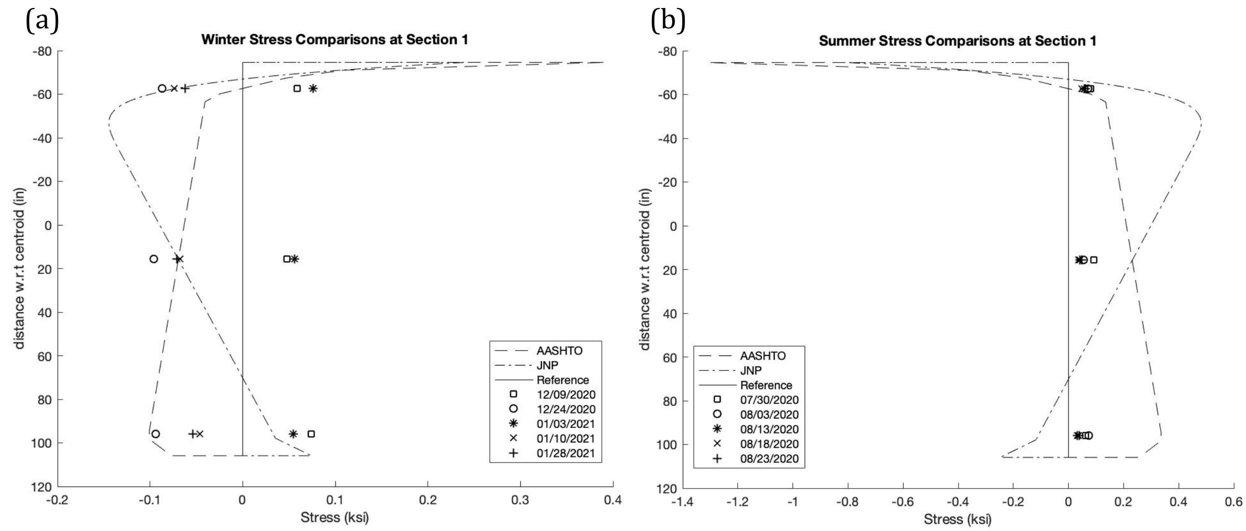

Figure 21: Comparison of the Measured Data and the Theoretical Models in Section 1 in the (a) Winter and (b) Summer

Figure 22 provides a comparison of the measured response in Section 3 with the theoretical models. The general findings were similar to those stated above in Section 1. Neither model showed a better comparison, but both produced reasonable magnitude stresses for the general design.

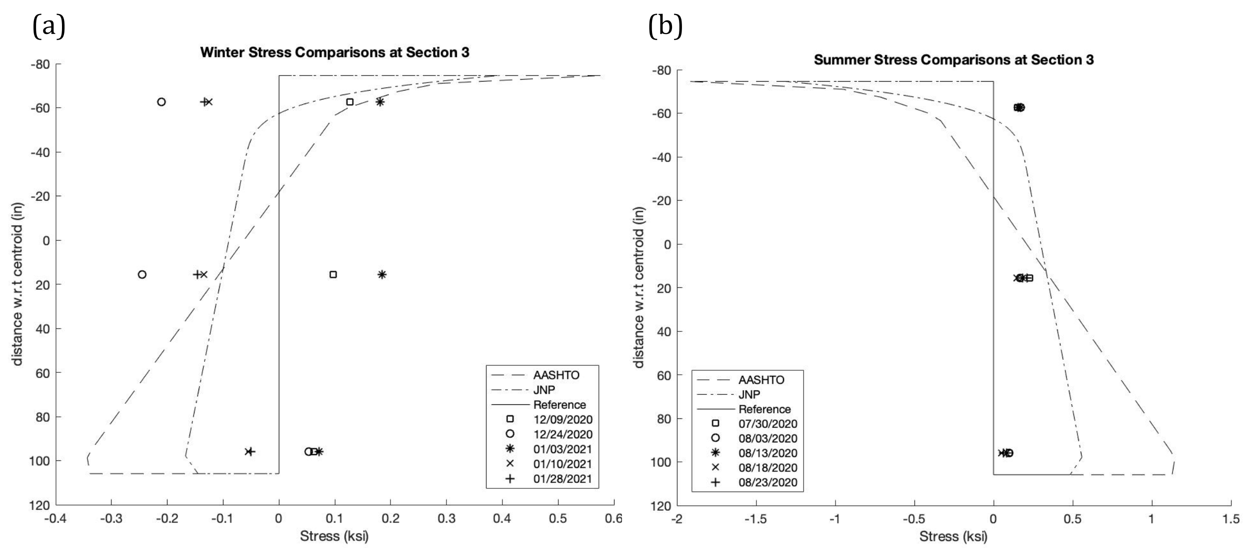

Figure 22: Comparison of the Measured Data and the Theoretical Models in Section 3 in the (a) Winter and (b) Summer

Overall Findings

In summary, this study provided further insight into the in-service behavior of segmental concrete box girder bridges. The approach to identifying this in-service behavior was through an in-depth field evaluation of the SHSCB. The process began with a comprehensive review of the existing documentation. Then planning and execution of a field monitoring system. SHSCB was monitored for over a year, and data was recorded. Data analysis was performed for seasonal and daily behavior. This included a comparison of the results with theoretical models.

The primary findings include the following.

- Seasonal behavior: The long-term behavior was stable, as indicated by the lack of slope in the data. In addition, the magnitude of seasonal response was relatively low, which shows the continual release of forces due to boundary restraints. These mechanisms are still functioning adequately after more than forty years.
- Daily behavior: Vertical temperature gradients were clearly captured, which induced flexural stresses. The differences between the East and West datasets indicate transverse temperature gradients are also present, which is due to the north-south orientation of the bridge. The gradient behavior overall was more complex in Section 3 due to the primary and secondary stresses. It is also worth noting that the daily summer behavior did produce higher tensile stresses compared to the winter.
- Comparison with Theoretical Models: The AASHTO and New Zealand models illustrated similar accuracies to the measured responses. Both models showed to be reasonable for general design criteria.

Acknowledgments

Student Researchers

Several students at Texas A&M University supported this research study. The numerical work was initially conducted by **Dachina Gunasekaran**, who worked on her M.S. degree under Dr. John Mander. This work was then continued by **John Nickson** as part of his Honors undergraduate degree requirements. Additional graduate students helped with the planning and execution of the field instrumentation efforts. These students included **Eric Stoddard**, **Yang Shen**, **Shengyi Shi**, **Claire Gasser**, **Sheyenne Davis**, and **Taylor Newcomb**.

Advisory Team

Two primary individuals form the advisory team. The first is **Dr. Michael Brown, FACI,** who is the U.S. bridge asset management leader at WSP. Dr. Brown has over 15 years of experience in bridge condition evaluation, corrosion evaluation and mitigation, bridge maintenance, preservation, and rehabilitation, and five additional years of experience in the evaluation and repair of buildings and parking structures. The research team conducted Zoom meetings with Dr. Brown for review and feedback on the research. The second advisor is **Mr. Gowen Dishman**, who is a Resident Engineer at HNTB. Mr. Dishman is representing the owner of the SHSCB and is responsible for the construction of the new bridge along with the deconstruction of the existing structure. He has roughly 30 years of bridge engineering experience. In addition, he was formerly employed by Figg Engineering and had extensive expertise with segmental concrete bridges. Mr. Dishman also serves as the industry liaison for the project. The research team has met with Mr. Dishman during each of the field visits. As mentioned above, he has supplied the research team with significant documentation on the existing bridge.

Additional Support

The project would not have been possible without the support of HCTRA, which owns and maintains SHSCB. The research team specifically thanks the help of **Mike Perez** and **John Vogel**.

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