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Remote monitoring systems for substructural health monitoring

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Remote Monitoring Systems For Substructural Health Monitoring

by

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Honorable mention must be paid to the iron-worker crews on the southbound pier of the St. Anthony Falls Bridge. Without their banter, working in sub-zero temperatures would not have been tolerable.

Finally, the people in my life who always believed I could do something great: My parents, Alice and David Collins, my grandparents, Warren and Dorothy Slack, and most of all, my wife, Abby.

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ABSTRACT

Remote Wireless Monitoring Systems have made a large impact in the area of Structural Health Monitoring (SHM). However in the specialized sub-field of Substructural Health Monitoring (SSHM), remote monitoring techniques have not made as much headway. First, monitoring systems are often retrofitted onto a structure. Therefore it is much harder to retrofit the substructure of a bridge or building. Second, many foundation elements such as driven piles or auger-cast piles are constructed in a way that makes installation difficult or can severely damage the sensing materials.

This thesis presents two case studies of Remote Monitoring Systems for Substructural Health Monitoring applications that were carried out by the Geotechnical Research Department of The University of South Florida. The first is a thermal monitoring system for a Voided Shaft study. The second is a thermal, construction load, and ongoing health monitoring system of the St. Anthony Falls Bridge in Minnesota.

Results show that the systems that were used provide adequate data collection, data storage, and data transmission. Furthermore, this data is easily analyzed and provided for public or private use on a dedicated website, which provides a fully automated and remote Substructural Health Monitoring System.

Chapter 1

Introduction

As a Civil Engineering application, remote monitoring has not yet made a great breakthrough into the field. However as a research and development tool, its benefits are finally coming to a realization. In all walks of life there is a push for our society to become wireless. Therefore it increasingly becomes a necessity for the Civil Engineering profession to lead the way in wireless. This will come in the area of remote structural health monitoring.

Remote monitoring, at its most basic, provides the user with a way to collect data from an event of particular interest, such as a foundation capacity test or ongoing thermal recording, and then transmit that collected data to another location, such as a database or spreadsheet file on a computer. This concept can be taken one step further by introducing limits on the data collector for alerting users or programming triggers on the data collector to initiate retroactive data collection and transmitting.

Remote monitoring can be used for many different Civil Engineering applications, from quality assurance in construction to ongoing health verification and much more. Remote monitoring can and will provide assurance to engineers and society as a whole that the infrastructure that we all rely on will carry us safely into the next generation. Furthermore, as new technology continues to upgrade daily, the cost and effectiveness benefits of remote monitoring continue to increase. Many times, yesterday's technology is more than sufficient for the needs of the project, providing us with exceptional technology at yesterday's prices. This is a large reason why the move to remote monitoring is making such progress.

1.1 Problem Statement

As a civil engineering tool, remote monitoring is a priceless benefit for health monitoring of structural members. As of now, the most common monitoring technique for inspection bridges is by visual inspection. By FDOT and FHWA standards, every bridge is required to undergo a visual inspection once every two years. While this method is satisfactory for structurally sufficient, non-critical structures, it does not provide a reliable way to determine the actual health of a structure. By providing a remote monitoring system, a bridge can be monitored in real-time at a remote location. This is a way to reduce man hours, as well as provide more accurate results and up-to-date data that can assess the structural integrity of a member, not just its visual appearance.

1.2 Research Scope

This research proposes the use of wireless communication and internet systems technologies as a means of providing remote monitoring capabilities for structural members or systems for agencies such as state DOTs and the FHWA. However, the use of these technologies as described herein would not be limited to the use as needed by these agencies. The original intent of the research was not to determine the best technology to carry out the project, but rather to provide examples of monitoring procedures and providing data from a variety of tests which were monitored using this system.

Another focus of this research was to provide a number of different monitoring techniques that could be applied to a structural member to be monitored throughout its life. This included sensors and devices to provide data related to temperature, load, and strain as well as video recordings. All of these parameters are considered vital for the determination of structural health of a member or system.

1.3 Thesis Organization

This thesis consists of 5 chapters. Chapter 2 is a summary of the state of the practice of Structural Health Monitoring in general, with an emphasis placed on Substructural Health Monitoring, and the ability to convert current wired systems into wireless. Chapter 3 is an in-depth look at a case study that was carried out on an innovative type of drilled shaft. This was where the original Remote Monitoring System was first implemented. It summarizes the successes and learning experiences gained from this project. Chapter 4 is a look at the culmination of all the work on this project. It reviews the short- and long-term monitoring procedures implemented on a bridge in Minnesota. This section will explain in detail the construction, setup and instrumentation, and monitoring procedure and results for a full-scale Remote Structural Health Monitoring System. Chapter 5 will summarize the main discoveries made throughout the project and will present conclusions and recommendations for future work in this area.

Chapter 2

State of the Practice

From an investigation into the state of the practice of structural health monitoring (SHM), it is seen that there are a number of different monitoring systems and techniques. All of them have their pros and cons, but each can be useful to a certain degree. At the moment, most of the advances in SHM have been made in the monitoring of the superstructure elements of bridges and other structures. However the importance of substructure health monitoring (SSHM) can not be underestimated.

Since a great deal of the modern technology of SHM is already widely used and documented as it pertains to superstructure monitoring, this review of the state of the practice will primarily focus on common technology and its practicality for use in a SSHM system.

2.1 General Monitoring Systems

Monitoring systems range widely in their functionality, cost, applied technology and monitoring approach. A system generally contains three components: a measuring device, a method of reading that device, and a method of storing the measurements taken. Depending on the complexity of the measurement being taken, the measuring device and readout component may be one and the same such as dial gages or pressure gages

(Figure 2-1). These devices convert a measurement parameter into mechanical gage movement. These devices can be considered the most basic of transducers as they transfer one physical aspect into another. Virtually all types of measurements have specialized devices to read that particular occurrence (i.e. time, displacement, velocity, acceleration, load, pressure, frequency, EMF, light intensity, strain, sound intensity, x-rays, voltage, inductance, capacitance, and more). For most measurement types, there are numerous ways to take that measurement which in turn dictate the capabilities and/or limitations of a monitoring system.

The most basic systems use fully manual devices and readouts (e.g. dial gages, proving rings, pressure gages, etc) coupled with manual record keeping. The limitations imposed on this method by requiring physical on-site personnel (recording/storage rate, man-hours, and travel) is in some ways offset by the unforeseen observations and the ability to react to and record unplanned secondary happenings. The most exotic systems use complex measurement devices requiring sophisticated readout units coupled with multifunction data acquisition systems capable of sending the recorded data via cellular or satellite communications. These systems are often enabled to accept remote configuration/scheme changes, are self-powered or self contained, and require little to no site visits. The most extreme cases of this type of system would likely be used by NASA for space exploration, as it is impossible to access the unit during use. Aside from the obvious cost, these systems are rarely adaptable to unforeseen occurrences. For SHM and SSHM applications, some mid-range systems can be selected to provide a balance between equipment cost and required on-site man-hours, which will allow most projects to be affordable.

2.2 Case Study

A study was done by Shannon & Wilson, Inc. along with the Federal Highway Administration (FHWA), the Washington State Department of Transportation (WSDOT), the City of Seattle, and the Bridge Design Team on the West Seattle Freeway Bridge (Shannon & Wilson, 1982). It presents a good example of SSHM for the structural elements of a bridge pier during construction of the bridge as well as data collection over time. The West Seattle Freeway Bridge was built between 1981 and 1984. The original bridge was struck by a freighter in 1978 and was deemed inoperable as a result of the incident. The goal was to advance the state-of-the-art of pile group design and analysis, and the information collected would be used in increasing pile group efficiency.

The City of Seattle authorized the use of instrumentation on Pier EA-31, which is a single column pier that supports the eastbound approach ramp from Spokane Street near the East Waterway and the Duwamish River (Figure 2-2). Shannon & Wilson, Inc. designed, specified and installed the instrumentation that was reviewed by the FHWA, the City of Seattle and the Bridge Design Team.

As stated above, the purpose of the project was to improve the state-of-the-art of pile group design and analysis. This would be done by collecting information regarding the load distribution amongst the pile group, the load transfer from the piles to the soil, the portion of the load transferred from the pier footing to the piles, and the settlement of the pier footing. Furthermore, the results gathered from this data were compared with theoretical predictions that would either validate the theoretical models or allow for the modification of those models.

In order to provide measurements for the above mentioned data collection criteria, measurements were selected as follows: First, pile tip load was measured, as wells as the load at six elevations along the pile, to determine the individual pile load distribution. A load cell placed at the pile tip permitted direct measurement of the load. Second, six telltale rods were installed on each pile to determine the pile tip displacement. The pile deformation as measured by the rods was converted to strain and used as a check. Third, strain gages were installed at the top of the piles which provided information of the load transferred from the pier footing to the individual piles. Fourth, settlement of the pier footing was measured by using a precise surveying measurement at the four corners of the footing. Fifth, soil settlement below the pier footing and within the pile group was measured to determine the soils reaction to the loading and the subsequent deformation of the piles.

In total, three of the 12 piles were instrumented with a load cell at the pile tip, six elevations of strain gage pairs, and a five position telltale extensometer (Figure 2-3). Data from the instrumentation was collected in the field using portable manual readout units and recorded on field sheets. During construction, the measurements were made at irregular intervals dependent on accessibility and other constraints due to the construction progress. The instruments were monitored as each significant phase of construction was completed as well to provide realistic data from the construction process. Instrumentation monitoring was conducted by Shannon & Wilson engineers throughout construction and continued through 1987, five years after start of construction. Data was again collected in 09/1988, 09/1999, and 10/1993. Two additional sets of data were taken in 1999 and 2002, which extended the period of monitoring to 20 years. The report by Shannon & Wilson presents a summary of the existing working gages, as well as the date at which failed gages were considered to be no longer working (Figure 2-4).

As reported, all pile tip load cells are functioning after 20 years of service, with the exception of one transducer from pile 7, which was damaged during pile driving. From the data collected in 2002, the average load for all three piles was 100 tons with a maximum deviation of approximately 11% (Figure 2-5). This suggests that all 12 piles in the pile group are carrying approximately the same load, which is assumed in typical pile group design.

During the instrumentation phase, pairs of strain gages were installed into the three monitored piles at six different levels along the pile. This provided 12 gages in each pile for a total of 36 strain gages. All of these gages were located beneath the groundwater level, and 17 of these gages were no longer functioning after 20 years of service. However, all the gages were reported to have worked at least until October of 1987, which provided 4 years of data collection. Since all of the gages were installed below the groundwater level, it is suggested that their failure was due to the water resistance of the system. The data from the strain gages that were still in commission were plotted over time (Figures 2-6 through 2-8). For piles 1 and 10, the average strain change in the pile was between -300 and -500 micro strain, with pile 1 being on the higher end of that range. However for pile 7 the average strain change in the pile was approximately -225 micro strain. This suggests that the piles farther away from the center of the pile cap, where the column is sitting, experienced more strain change, likely due to bending. The gages installed at the top of the other piles as well as the strain gages in the column were all still functioning after 20 years.

The conclusions of this study show that SSHM using wired gages is extremely useful. With the advances in the durability of data collection and monitoring systems, it is likely that this same system, if installed today, would not have the number of failed gages. While this study required a worker to be on-site to record the data, the usefulness of the instrumentation far outweighed the cost of the man-hours required. While the technology used in this study is somewhat outdated, the information gleaned from this study is highly useful in today's monitoring systems.

2.3 Wireless Sensors for Health Monitoring

Wireless systems use basically the same measurement devices (or transducers) as wired systems, but replace the lead wires with a transmitter and receiver system. Wire costs range between $0.4/ft to $1.0/ft per gage installed. Transmitters, like data logging equipment, are limited by their sampling and transmission rates, meaning higher reading rates come at higher costs with an upper rate limit in the range of 5-10k samples/sec/channel. Transmitter/receiver systems can cost thousands of dollars per channel depending on the required transmission/sampling rate. The cost comparison of wireless to wired systems is generally site specific, but leans towards wired systems. However, in the case of moveable structures or mechanical devices, slip rings or other features which allow the movements of the wires are required which tend to tip the scales in favor of wireless systems.

Wireless sensors for SHM systems are being used more frequently as the technology becomes more widely available. Since no wires are required between the gages and the data acquisition system, installation time and those costs associated are reduced as compared to traditional wired systems. Typically, wireless sensors are installed over an entire structure to get a full mapping of the desired measurement (i.e. stress, strain, displacement, temperature, velocity, etc.) across the entire structure. A wireless data acquisition system collects the data sent back from these sensors and either stores the collected data to a data logger or is sends it wirelessly using a modem to a remote site.

A study by Arms et al. introduced the idea of a SHM system in which even the data acquisition software could be reprogrammed remotely. The goal is that one should be able to alter the operating parameters of a monitoring system, such as sampling rate, triggering parameters, downloading intervals, etc., from a remote location and therefore never have to go back to the site after initial installation. This provides a fully remote monitoring system in which all the parameters of the data logging and collection can be altered from a separate location (Arms et al., 2004).

The wireless transmittable gages were installed on the existing structure at main points of interest. Wireless sensors received transmitted data and the data was uploaded to an on-site laptop (Figure 2-9). The laptop transmitted the data through a cellular uplink to the base station. From this base station, the software that was running on the laptop could be altered to change the data collection parameters. The software could also be altered with trigger parameters, so that the system could be sleeping, but would wake up when an event occurred, such as a train crossing the bridge, that increased the change in strain levels (Figure 2-10).

While this provides for a completely wireless system, its use as a SSHM system is not as probable. For installation in the deep foundation system, wireless sensors would have to be extremely powerful to transmit data wirelessly through surrounding soil, sometimes at depths upwards of 100 feet. Sensors capable of this would most likely be expensive enough to negate the cost savings from not dealing with wired sensors. Furthermore, sensors used for reinforced concrete structural elements can provide much better data when installed within the concrete member where the reinforcement is located. Once again, a typical wireless sensor would not have the capability to transmit signals through hardened concrete. However the wireless data acquisition system could still be used with no obstructions.

A second study by Susoy et al. researched the development of a standardized SHM system for the movable bridges in Florida. The assumption was that due to the multitude of elements, movable bridges are more prone to damage and deterioration and that the typical visual inspection as required by FHWA is not adequate. The study detailed the SHM system that was installed on the SR-401N Bascule Bridge over the Barge Canal in Port Canaveral (Figure 2-11). A detailed finite element analysis was run to determine the probable locations for stress concentrations on the bridge. Once this was complete, wireless transmitting strain gages were mounted on the bridge in these locations (Figure 2-12). The strain sensors transmitted their data wirelessly to the installed data acquisition system and the data was logged on a field computer also installed on-site (Susoy et al., 2006).

For this study, the wireless sensors were almost a necessity, due to the type of project. Installing wired sensors on a movable bridge could prove to be quite difficult and could cause damage to the wires. No mention was made concerning the accessibility of the data once it was collected, so it is assumed that the data was downloaded by a worker sent to the site. However once again, this study was based on the idea of wireless sensors for the monitoring system, and therefore would have the same difficulty translating to SSHM as the Arms 2004 study.

A final study by Watters et al. introduces the idea of a special design for a wireless sensor capable of detecting threshold levels. The sensor is coupled with radio-frequency identification (RFID) chip. The sensor is read by scanning the system with a radio-frequency (RF) transceiver. The RF transceiver awakens the RFID chip to power the sensor to collect data. Once the data is collected, the RFID chip transmits the data back to the transceiver to be read (Watters et al., 2001).

This study focuses on the use of the sensor to determine whether certain data may have crossed a threshold, namely chloride ingress into reinforced concrete structures. A particular threshold is set and then the system will read the data and determine if the threshold has been met. This system is extremely useful for data that does not need to be streamed. For chloride intrusion into reinforced concrete structures, the critical point at which the chloride concentration is reached could take years to be met. Therefore, a DAS capable of collecting and logging data at a high rate is not needed. In typical concrete inspection, a core sample of the concrete deck must be taken and then analyzed in a lab. With this technology, a sensor can be embedded into a structure and then routinely checked at a predetermined interval. Furthermore, the trends can be plotted over time to help owners and engineers predict when the chlorine intrusion will reach a critical level. The capability to send an alert when a certain threshold level is reached would be extremely useful in bridge monitoring. If an alert was programmed into the transducer that would react when a certain level is met, it would allow authorities to react and make a decision about keeping a bridge open or closing it down depending on the severity of the event, possibly saving lives.

While this is a useful system for data that need only be monitored over long intervals, from a strict SHM point of view, the system would not be beneficial for structures loaded with highly irregular or dynamic loading, such as a bridge. The sensors for a bridge SHM system would need to be read and have the data collected and stored at a relatively high rate in order for the owner or engineer to determine what is happening to the structure during its service life.

2.4 Fiber Optic Sensors for Health Monitoring

With the recent advances in the telecommunications field with fiber optics, the interest in fiber optic sensors (FOS) has increased and has made way for extremely powerful new sensors to be used for SHM. FOSs are used by sending light beams through the fiber optic cable at regular intervals and measuring the return time. When the cross-sectional area of the cable changes, the return time changes, and this change in return time can be related to engineering parameters (i.e., strain, displacement) of the structural member to which they are attached. They are considered to be beneficial because they are relatively immune to interference from radio frequencies, electric or magnetic fields, and even temperature differences.

A study by Udd et al. introduced the use of FOS in existing structures. The paper introduces the use of single axis fiber grating strain gages for the use of non-destructive evaluation of existing structures. The benefits of these are said to include a long service life and can be installed in long gage lengths, providing more accurate results. There was nothing in the study that related to remote or wireless monitoring. The study was instead focused on the sensitivity of the gages as well as the installation requirements of working on an existing structure.

In this case, the bridge required structural strengthening in order to accommodate increased loads on the structures that were not expected at the time of construction. The bridge was strengthened using FRP composites that would not alter the look of the bridge while still providing increased strength (Figure 2-13). The fiber grating strain gages were installed embedded into saw cuts in the bottom of the bridge girders, as well as on the outside of the adhered FRP coating (Udd et al., 1999) (Figures 2-14, 2-15).

This study, again, focused primarily on the monitoring of the bridge superstructure, but the FOS could have been installed just as easily to the pile foundation of the bridge. This would have provided data to show how the bridge foundation reacts to the same loads that are visible in the data from the superstructure. The sensors proved to be extremely sensitive. The gages were able to detect not only small cars crossing the bridge, but also, on one occasion, the effect of a single person running out to the center of the bridge, jumping up and down 5 times, and then walking back off the bridge (Figure 2-16). Furthermore, gages could easily be installed embedded within the structure as well as applied to the exterior of the structure with adequate results from each installation. FOS would be helpful in a SSHM system because of their relative immunity to temperature effects. Typically, bridge foundations are designed with mass concrete elements, such as drilled shafts or piles for the subsurface foundation, a shaft or pile cap, and large concrete columns. The temperature changes that can take place inside these mass concrete elements are quite large. Typical resistance type or vibrating wire gages can show large amounts of strain on a mass concrete element just due to temperature when the element is otherwise unloaded. Therefore, if a sensor were able to be unaffected by these temperature changes, it would greatly aid in the simplification of the conversion from strain to load.

A second study by Hemphill studies the marriage of wireless technology with Fiber-Optic sensors. It proposed and tested the idea of a fully integrated, continuous wireless SHM system for the East 12th Street Bridge in Des Moines, Iowa (Figure 2-17). Fiber Bragg Grating (FBG) strain sensors were installed at 40 different locations on the bridge. The data collector scans the FBG sensors, and then transmits the data wirelessly to a dedicated computer in a secure facility close to the site (Figure 2-18). The data was stored as a data file and automatically uploaded to an FTP site. When this site was accessed, the data file was downloaded and deleted from the FTP site to make room for the next data file. This data was compiled and processed and then posted to a website that allowed users to view real-time strain data along with real-time streaming video of the bridge (Hemphill, 2004).

This system is useful because it can provide the end user with simple, easy to follow data viewing that can easily be monitored. With the addition of the real-time streaming video, a data monitor can simply look at the data and compare it with the live traffic on the bridge and make the needed correlations to the loading on the structure. The wireless transmitting of the data is also useful because it cuts down on the man-hours that are required to go to the site and download the data from the collection system, which can be time consuming and expensive. As stated above, this system is very efficient and has very few drawbacks, if any. The fiber optic strain gages could be installed in the substructure as well as on the superstructure, and there are really no limiting factors to the system.

2.5 Current and Future Possibilities for Health Monitoring

A report by Weyl studies the proposal for a full-scale Structural Health Monitoring system for the Indian River Inlet Bridge in Delaware. The design of the SHM system was fully integrated throughout the design phase of the project so that it would fit seamlessly with the construction phase. The following types of gages will be installed throughout the bridge: Vibrating wire strain gages, weldable foil strain gages, accelerometers, GPS sensors, load cells, linear potentiometers, corrosion monitors and more. This combines for a total of 240 sensors, 11 DASs, and 39 Data Loggers (Weyl, 2005).

The project will be carried out in three phases. Phase 1 will take place during construction to determine live construction loads. Phase 2 will take place immediately after bridge construction to determine the initial response of the bridge to traffic, thermal, and wind loading. Phase 3 will take place during the intended service life of the bridge to compare against the data collected during Phase 2 (Weyl, 2005).

Finally, a web-based user interface will be programmed to present data in an easy to read and understand format that will be accessible to Delaware DOT and those that worked on the project. At the time of this report, there is no data to report from this project. It is currently in the preliminary construction phase.

This project is a very good example of the future possibilities that Structural Health Monitoring holds for the sustainability of the nation’s infrastructure. While integrating the monitoring system fully in the design phase of the project, the construction is not held-up, nor is the monitoring system held back. The data that will be collected from this system can be archived as useful data for the history of the bridge and will most likely be very useful in the determination of any possible problems that might take place in the distant future.

This particular study involved a very high number of sensors, gages, and data acquisition systems for the full SHM system, but it is still very similar to the proposed monitoring for the I-35W St. Anthony Falls Bridge monitoring system that is studied in this report. The use of common everyday technology, such as the dedicated website that provides certain users with real-time data from the bridge, coupled with the advanced technology of resistance and vibrating wire strain gages will propel Structural Health Monitoring and Substructure Health Monitoring into the next phase.

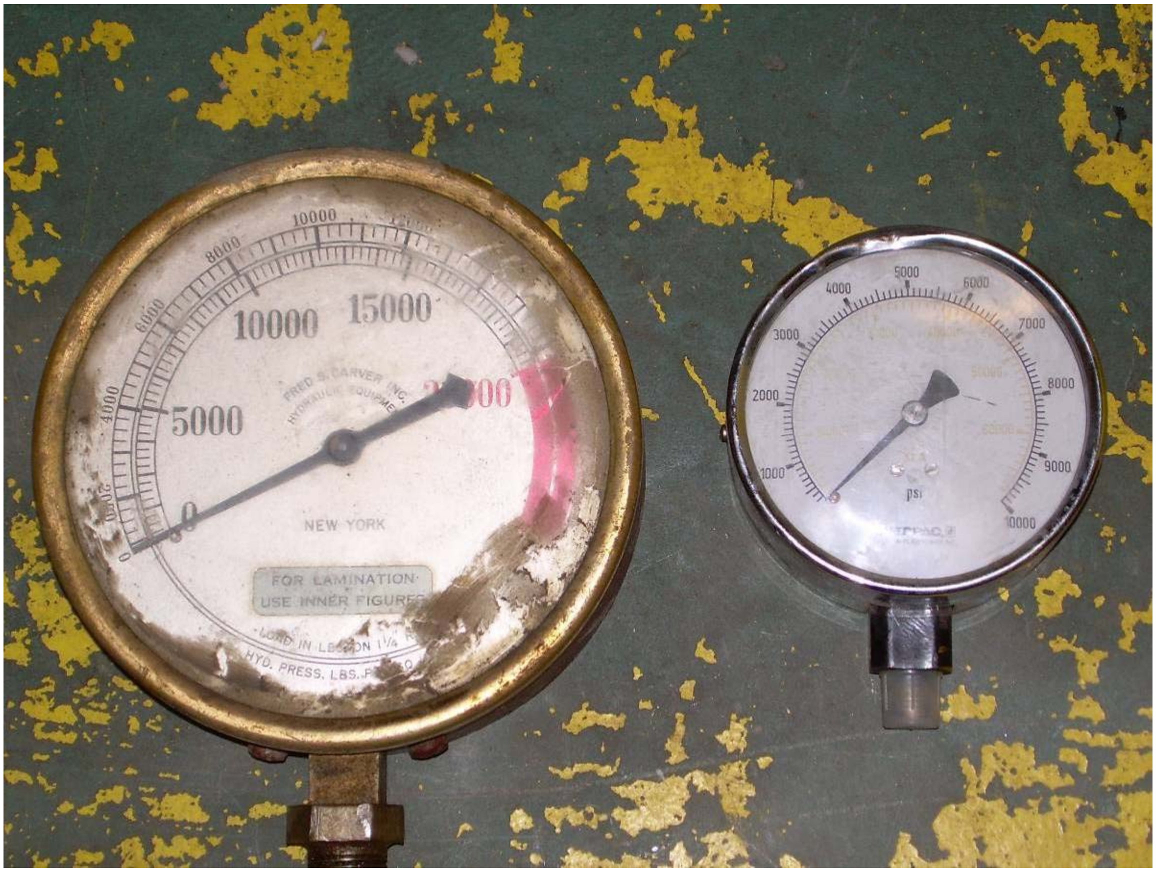

Figure 2-1: Standard Rotary Dial Gages.

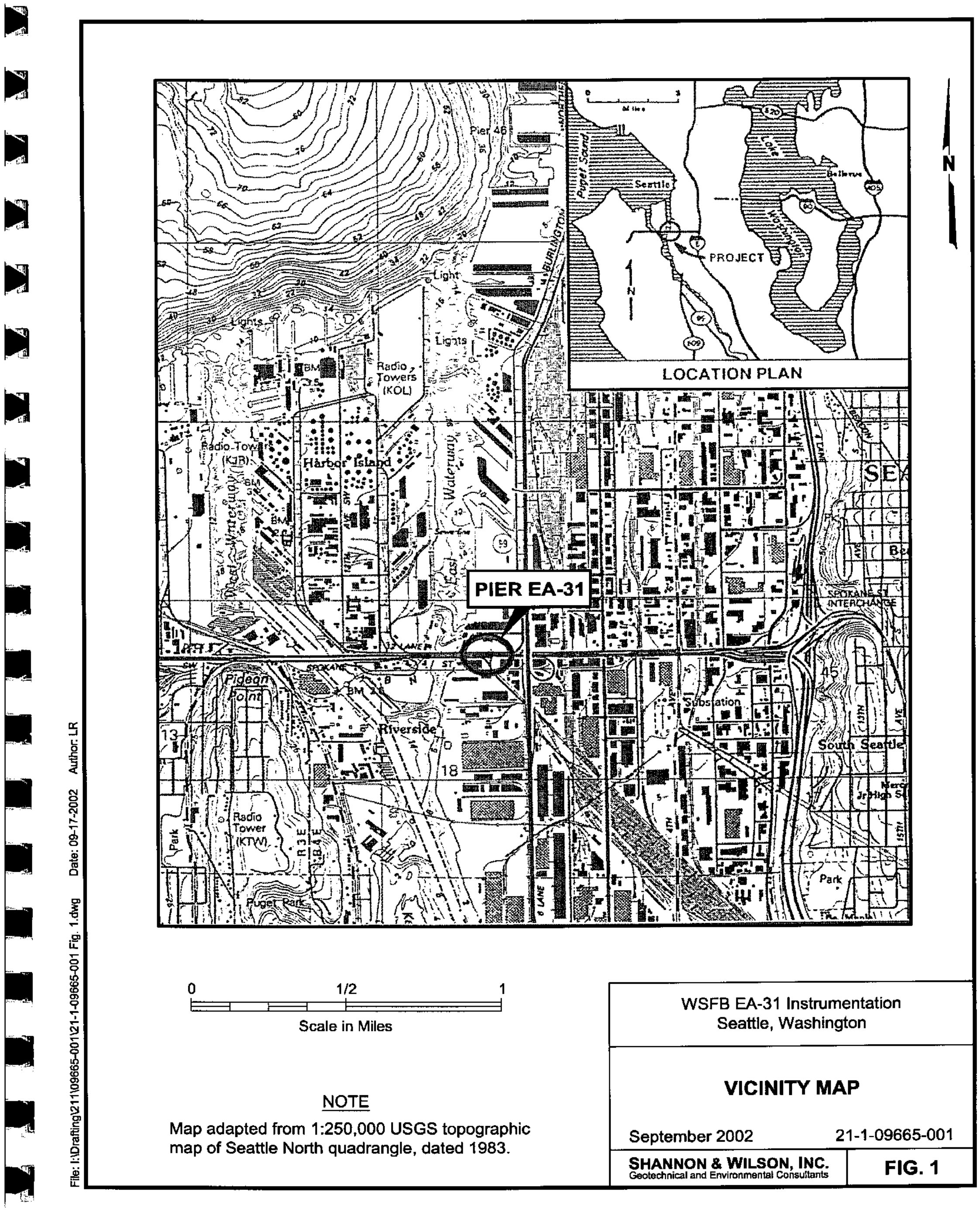

Figure 2-2: Pier EA-31 Site Map. (Shannon & Wilson 2002)

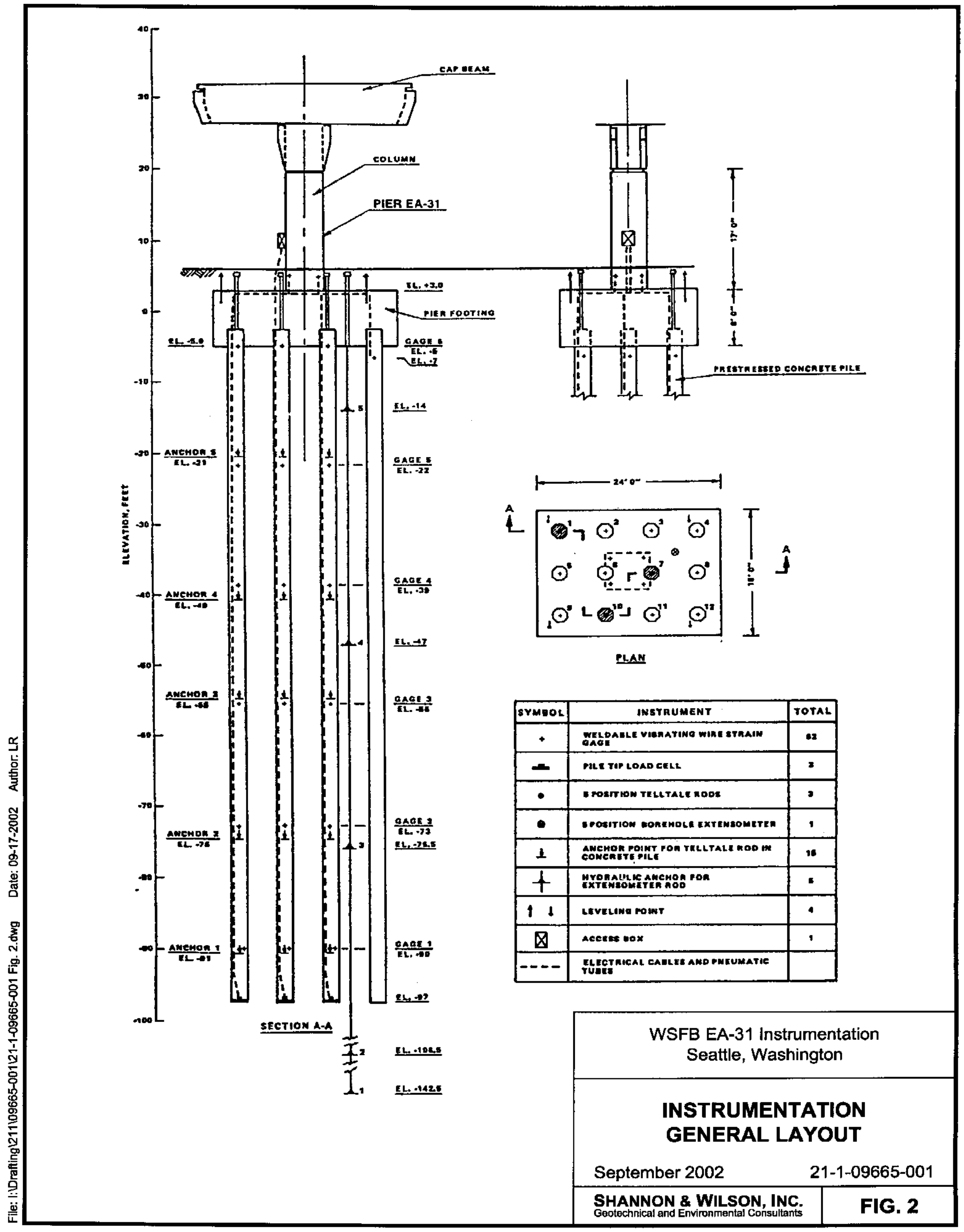

Figure 2-3: Pier EA-31 Pile Instrumentation Layout. (Shannon & Wilson, 2002)

GAGE	HISTORY
1 Quadrant of Pile 7, Pile Tip Load Cell Transducer 5194	Damaged during pile driving
Rod Extensometer Anchor (Pile 1, Anchor 1)	"Failure" during installation
Strain Gages	
1-1-A	Has not functioned since 7/99 **
1-1-B	Has not functioned since 9/88 *
1-3-B	Has not functioned since 7/99 **
1-4-A	Has not functioned since 7/99 **
1-4-B	Has not functioned since 7/99 **
1-5-B	Has not functioned since 7/99 **
1-6-A	Has not functioned since 3/02 ***
7-1-A	Has not functioned since 10/87*
7-1-B	Has not functioned since 7/99 **
7-5-A	Has not functioned since 12/84 *
7-6-A	Has not functioned since 7/99 **
10-1-A	Has not functioned since 7/87 *
10-1-B	Has not functioned since 10/87 *
10-4-A	Has not functioned since 7/99 **
10-4-B	Has not functioned since 10/93 *
10-6-A	Has not functioned since 7/99 **
10-6-B	Has not functioned since 7/99 **

Notes: 1. * 6 gages have not functioned since 12/84 – 10/93.
** 10 gages have not functioned since 7/99.
*** 1 gage has not functioned since 7/99 – 3/02
2. Total of 17 out of 62 gages (27%) not functioning as of 3/02 (20-year period).
3. Total of 17 out of 36 gages (47%) gages in piles, below water table, not functioning as of 3/02, (after 20-year period).

Figure 2-4: Pier EA-31 Gage Failure Summary.

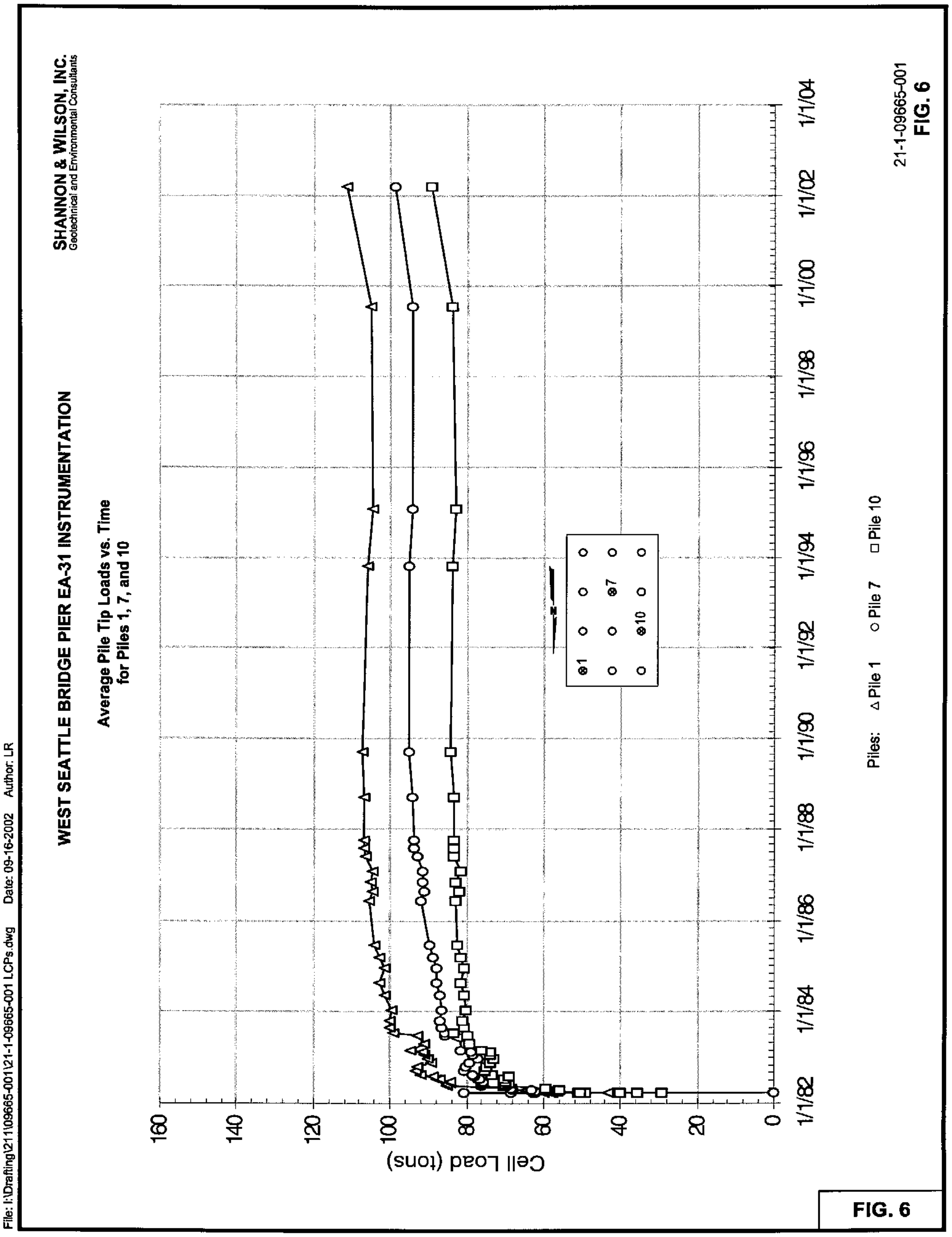

Figure 2-5: Pier EA-31 Average Pile Tip Load Piles 1,7,10. (Shannon & Wilson, 2002)

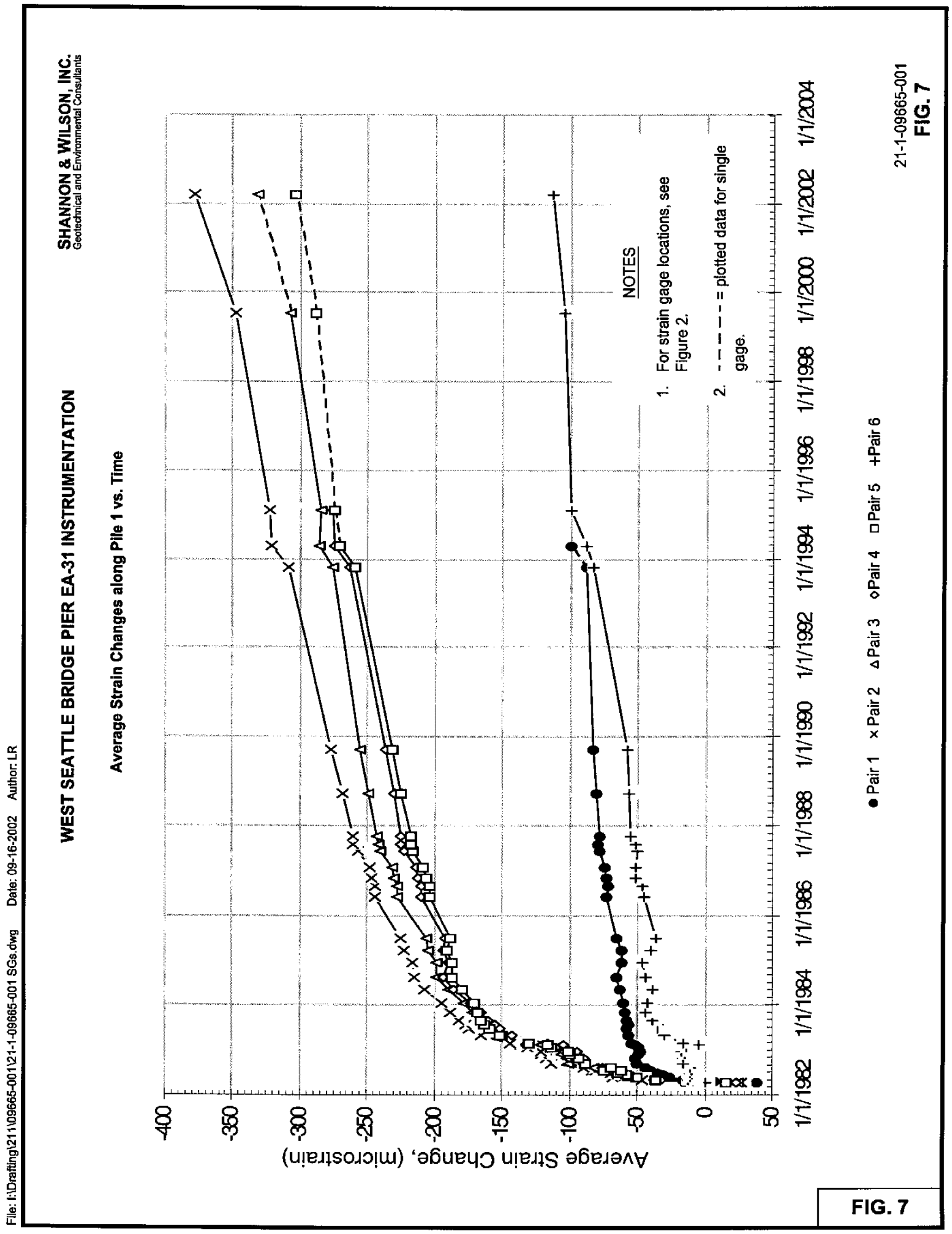

Figure 2-6: Pier EA-31 Average Strain Change Pile 1. (Shannon & Wilson, 2002)

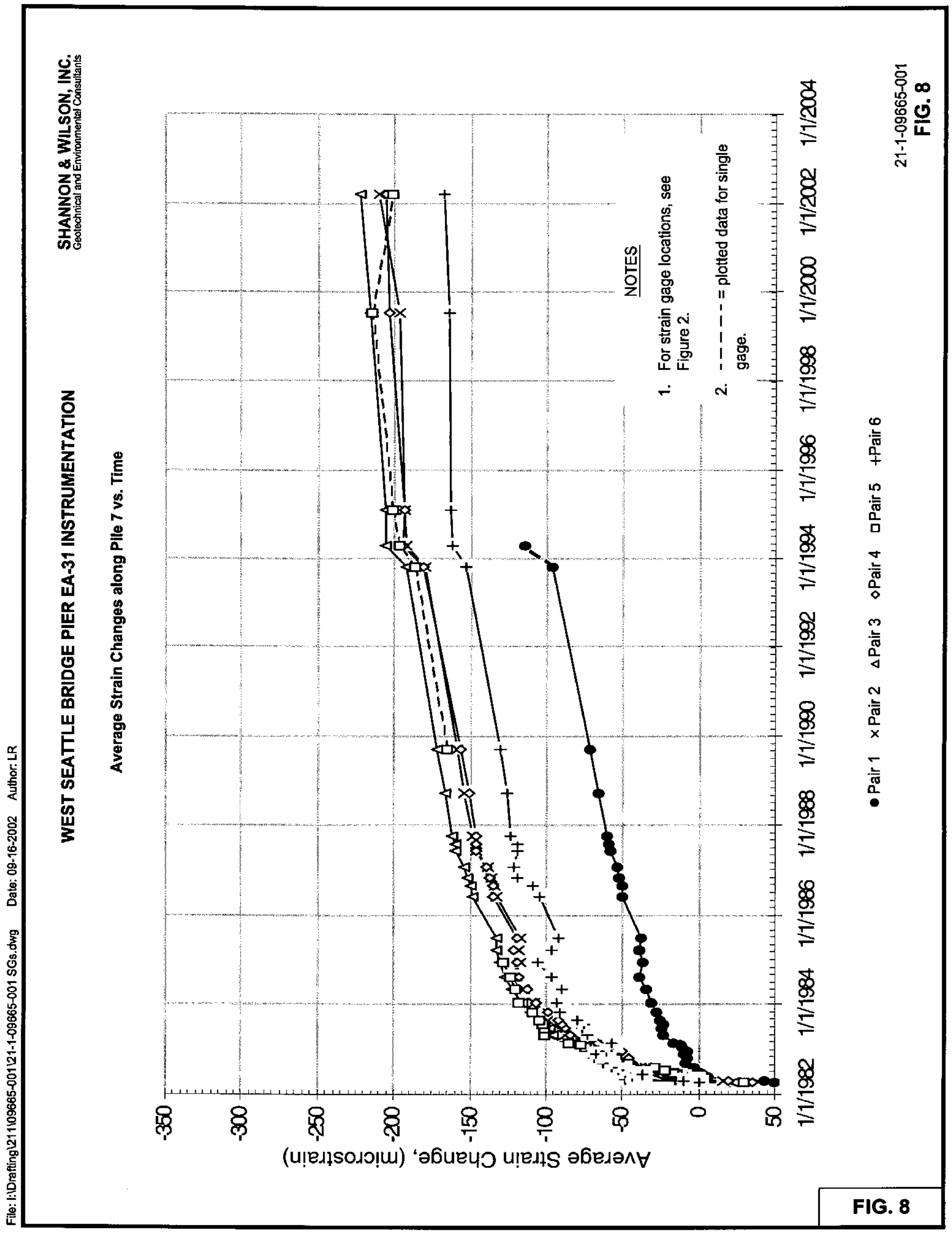

Figure 2-7: Pier EA-31 Average Strain Change Pile 7. (Shannon & Wilson, 2002)

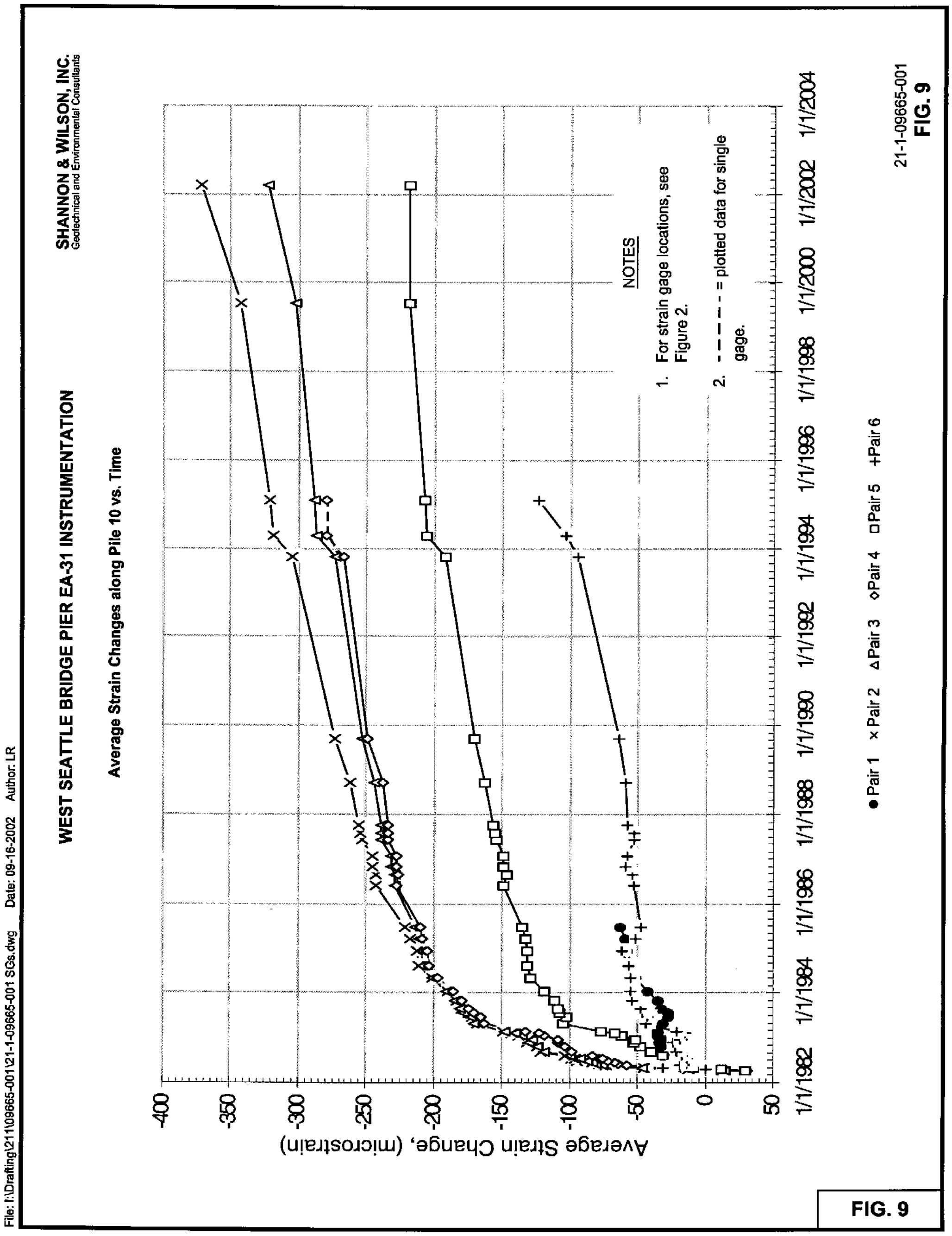

Figure 2-8: Pier EA-31 Average Strain Change Pile 10. (Shannon & Wilson, 2002)

Figure 2-9: Wireless Data Collection and Transmit Setup. (Arms et al., 2004)

Figure 2-10: Train Crossing Bridge Causes a Strain Event. (Arms et al., 2004)

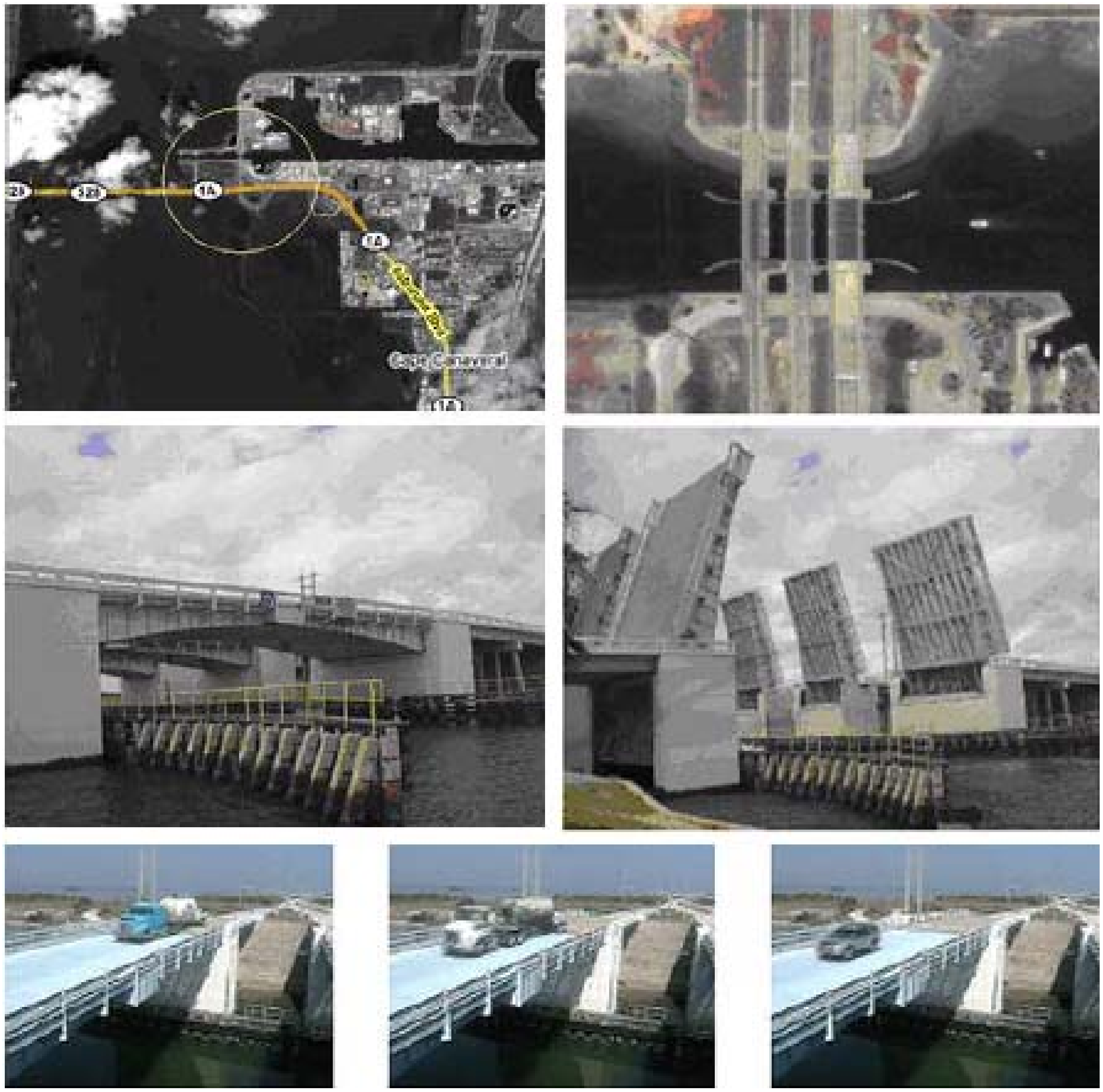

Figure 2-11: Bascule Bridge on SR-401N, Port Canaveral, FL. (Susoy et al., 2006)

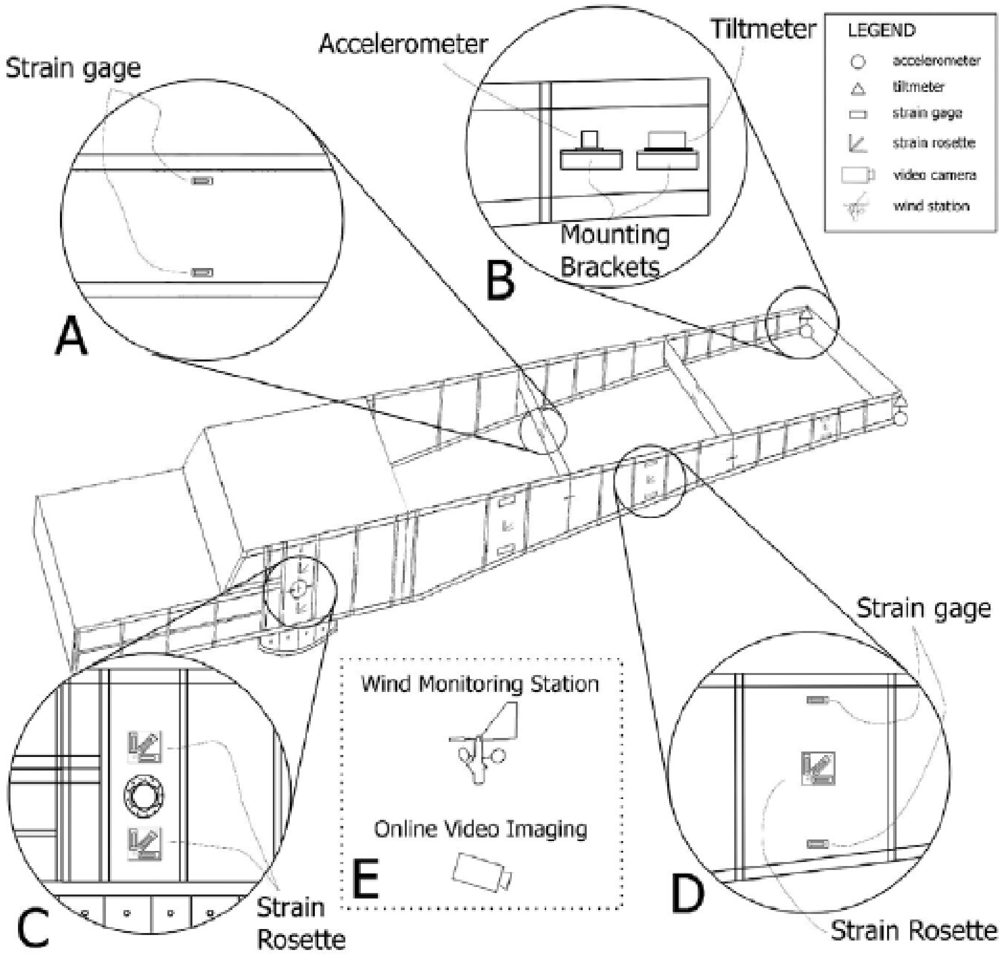

Figure 2-12: Locations and Types of Sensors on Bascule Bridge. (Susoy et al., 2006)

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Figure 2-14: FOS Installation Beneath FRP Layers. (Udd et al., 1999)


Figure 2-15: FOS Installation Above FRP Layers. (Udd et al., 1999)

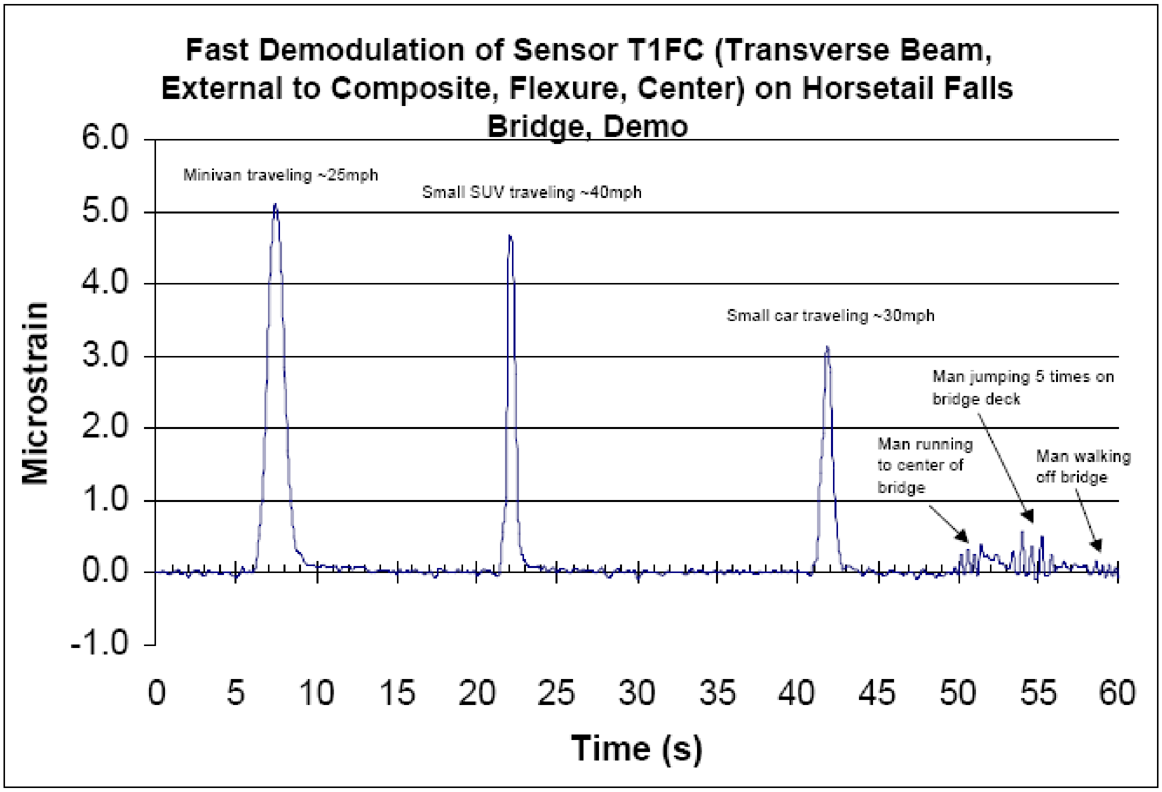

Figure 2-16: Measured Strain Induced on Bridge from Events. (Udd et al., 1999)


Figure 2-17: East 12th Street Bridge, Des Moines, Iowa. (Hemphill, 2004)


Figure 2-18: Dedicated Host Computer near Bridge Site. (Hemphill, 2004)

Chapter 3

Voided Shaft Thermal Monitoring

The first study conducted during the period of this research involved the thermal monitoring of a drilled shaft. Florida's bridge substructures have continually grown in size due to the high demand of larger and larger bridges to accommodate the growing population. Typically, drilled shafts were not considered to behave as a mass concrete element due to their smaller size (usually no greater than 4 ft. in diameter). However with the increase in size of today's bridges, drilled shafts are more and more acting as mass concrete elements (such as the 9ft. diameter shafts for the Ringling Causeway Bridge in Sarasota, FL), yet are slipping through the mass concrete specifications without special review. One of the problems associated with mass concrete elements is the extremely high temperatures that occur during the concrete curing process. Due to the high heat experienced due to hydration reactions, cracking can occur in the concrete shaft. This, in turn, could translate to a loss in the strength of the foundation, potentially creating a hazard to human life. Therefore, the Civil Engineering Research department at the University of South Florida proposed the idea of construction of drilled shaft with a full length centralized void to mitigate the mass concrete effects exhibited by the foundation element.

This section of the report will focus on the remote thermal monitoring procedure that was used for the research done on the USF Voided Shaft Research project. Of particular interest will be the installation and instrumentation of the drilled shaft, the thermal monitoring procedure and a review of its efficacy, and the results from the remote thermal monitoring system and its individual parts. More emphasis will be placed on the actual monitoring procedure than the results from the voided shaft; however these thermal results will be presented in a summary.

3.1 Test Specimen Instrumentation

The testing site for the thermal monitoring of the voided shaft will take place at R.W. Harris, Inc. in Clearwater, FL (Figure 3-1). Prior to the construction of the drilled shaft, the instrumentation for the thermal monitoring was put into place. The first step was the instrumentation of the rebar cage which would be installed in the shaft. The reinforcement cage was built using 36 longitudinal bars with 26 - #5 stirrups at 12 inches on center. The cage was equipped with 9 - 26 ft long, 2 inch Schedule 80 PVC pipe for thermal testing (Figure 3-2). On three of these tubes, at 120 degree spacing from each other, thermocouples (TCs) were placed at the top, middle, and bottom of the tubes to provide readings from all around the shaft. The inner steel casing (needed to provide the central void in the shaft) was outfitted with 3 cross-bar supports welded to the interior of the casing which allowed for a central tube to be run through the center of the void for thermal integrity testing (Figure 3-3). TCs were also placed at the top, middle, and bottom of each side of the inner casing, spaced 120 degrees away on the cross-bars, as well as attached to the top, middle, and bottom of the central tube (Figure 3-4). More TCs were placed at the top, middle, and bottom of the outside of the inner casing (Figure 3-5). In the surrounding soil, ground monitoring tubes were installed at ¼ Shaft Diameters (Ds), ½ D, 1D and 2Ds away from the edge of the shaft (Figure 3-6). TCs were also installed with the tubes at these locations.

3.2 Test Specimen Construction

The voided shaft was constructed at the R.W. Harris test site on September 25, 2007. The entire process was broadcast via webcam from the USF geotechnical webpage for those who were unable to visit the construction site. Records of the construction sequence, thermal testing, and long-term thermal monitoring were posted and updated every 15 minutes to http://geotech.eng.usf.edu/voided.html. A 9ft diameter drilled shaft with a 4 ft diameter central void was constructed. The first step was the excavation. An oversized surface casing of 10 ft in diameter and 8 ft in length was embedded 7 ft into the soil. Excavation was carried out in the dry condition with a 9 ft diameter auger for the first several feet. After that, polymer slurry was introduced into the excavation for stabilization. The excavation proceeded without issue do a depth of 25 ft (Figure 3-7). A clean out bucket was used to scrape the bottom of the excavation of debris immediately after the auger and then again after a 30 minute wait period.

The reinforcement cage was picked at two locations to avoid excess bending (Figure 3-8). Locking wheel cage spacers were placed at the top and bottom of the reinforcement cage to maintain 6 inches of clear cover (Figure 3-9). The reinforcement cage was hung in-place during the pour so that the finished concrete would be level with the top of the cage (Figure 3-10).

The central casing to create the full length void was actually 46 inch outer diameter steel casing that was 30.5 ft long. It was set into the center of the excavation with a crane (Figures 3-11 and 3-12). The self-weight of the steel casing penetrated the soil to about 3 to 6 inches. This prevented the concrete from entering the void area. To prevent the top of the inner casing from shifting during the initial concrete pour, a back-hoe bucket was used to hold the top of the casing steady (Figure 3-13). A double tremie system was used to place the concrete on opposite sides of the excavation (Figure 3-14). Concrete specifications were a standard 4000 psi, 8 inch slump, #57 stone mix design. During the concrete placement, concrete level at three points around the shaft was measured to ensure concrete was flowing around the void and through the reinforcement cage. The temporary surface casing was removed after final concrete placement (Figures 3-15 and 3-16).

3.3 Monitoring System Instrumentation and Procedure

Once the construction of the voided shaft was complete, all the thermocouple (TC) wires were accesses through the tubes so they could be attached to the data collection system. The entire remote monitoring system is made of a number of parts: A Campbell Scientific CR1000 data logger, an AM25T 25-channel multiplexer, a Raven100 CDMA AirLink Cellular Modem, PS100 12V power supply and 7Ahr rechargeable battery, a 12W solar cell panel from Unidata, and a large environmental enclosure to protect all the materials from the elements (Figures 3-17 through 3-21). The total cost of the system, including all equipment and ongoing services was approximately $4,500. The TC wires were connected to the multiplexer as there were not enough channels on the

CR1000 to read all of the TCs. The multiplexer was then connected to the CR1000 (Figure 3-22). Loggernet, the data collection software, was pre-installed into the CR1000 and setup to monitor the system. The data collection system was equipped with the solar panel to help sustain the battery voltage (Figure 3-23). The system was programmed to wake up every 15 minutes, take a temperature reading and record it, and then go back to sleep. The Raven modem was programmed to wake up once every 60 minutes and transmit the collected data back to the host computer, which was stationed in the Geotechnical Research Department at USF, where the data could be processed. Due to the high number of TCs that were being monitored, two TCs were not attached to the remote monitoring system. The TCs that were located at 1D and 2Ds away from the shaft were attached to an OMEGA OM-220 data logger that collected data at the same rate as the CR1000, however the data was simply stored and a site visit was required to collect that data. The battery voltage was also monitored and sent to the host computer along with the thermal data so that the power consumption could be tracked.

3.4 Results and Conclusions

Overall the system worked extremely well. At one point during the monitoring period, there was a cellular timeout and the modem stopped transmitting the data to the host computer. This was fixed by a site visit to reset the modem and the problem did not occur again. However the main problem that was encountered was an issue with power usage. At the beginning of the monitoring procedure, the Raven modem was left on and would send back its data every hour. However this used an extremely large amount of power and the system lost power after just a few hours (Figure 3-24). The monitoring procedure was revised so that the modem would go to sleep and only wake up once every hour to transmit the collected data. Even with this alteration, the battery was still losing an ongoing battle with the power consumption of the Raven modem. Once the battery voltage dropped below 11.6V, the data collection system has approximately 8 hours of life before it quits. Due to this large amount of power usage, three site visits were required to get to the system and recharge the battery. These three visits can clearly be seen in the plot of the battery voltage over time (Figure 3-25). In order to provide a completely remote unit, a larger solar cell is recommended/required as the 12W did not gain enough power to make the system fully remote.

Originally, the data collection period was supposed to last until it was seen that the temperatures in the shaft had reached somewhat equilibrium. However, in reviewing the data, the temperatures recorded from the soil surrounding the shaft were increasing while the temperatures within the shaft had reached equilibrium (Figure 3-26). Therefore data collection was continued as a result. The data was collected for another period of time until it was determined that the temperatures both in the shaft and in the surrounding soil had reached equilibrium. From the final temperature plot, it can be seen that the temperature in the soil at 1D away from the shaft was the last to eventually reach equilibrium. It can also be seen that the temperature in the soil at 2Ds away from the shaft was affected only slightly by the immense heat coming from the shaft (Figure 3-27).

Figure 3-1: Map of Voided Shaft Testing Site.

Figure 3-2: Voided Shaft Reinforcement Cage Instrumentation.

Figure 3-3: Voided Shaft Central Casing Center Tube Supports.

Figure 3-4: Voided Shaft Thermocouples Installed in Central Casing.

Figure 3-5: Voided Shaft Thermocouples on Outside of Central Casing.

Figure 3-6: Voided Shaft Ground Monitoring Tube Installation.

Figure 3-7: Excavation for Voided Shaft.

Figure 3-8: Picking of Reinforcement Cage for Voided Shaft.

Figure 3-9: Placement of Reinforcement Cage for Voided Shaft.

Figure 3-10: Hanging of Reinforcement Cage for Voided Shaft.

Figure 3-11: Picking of Central Casing for Voided Shaft.

Figure 3-12: Placement of Central Casing for Voided Shaft.

Figure 3-13: Voided Shaft Central Casing Stabilization.

Figure 3-14: Double Tremie Concrete Placement.

Figure 3-15: Outer Steel Casing Removal.

Figure 3-16: Final Voided Shaft at Ground Level.

Figure 3-17: Campbell Scientific CR1000 Data Logger.

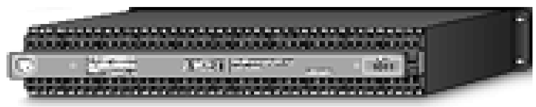

Figure 3-18: AM25T 25-Channel Multiplexer.

Figure 3-19: Raven100 CDMA AirLink Cellular Modem.

Figure 3-20: PS100 12V Power Supply with Rechargeable Battery.

Figure 3-21: ENC12x14 Environmental Enclosure.

Figure 3-22: Thermocouple Wire Connection to AM25T to CR1000.

Figure 3-23: Remote Thermal Monitoring System for Voided Shaft.

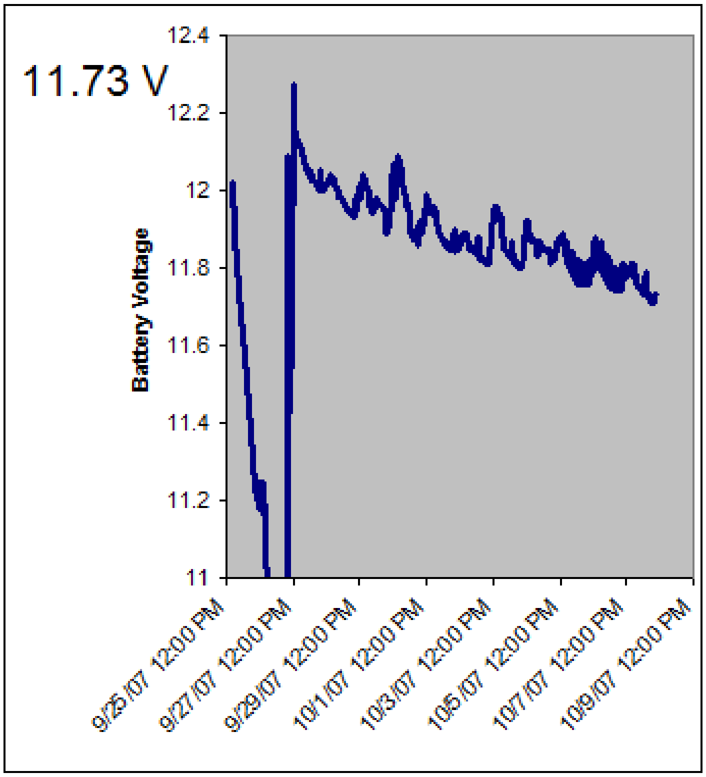

Figure 3-24: Battery Voltage as of 10/8/07.

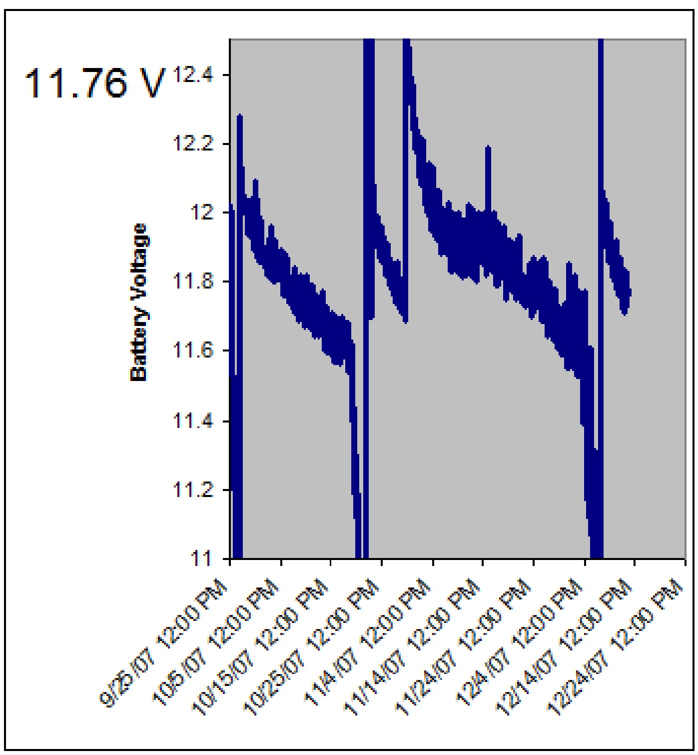

Figure 3-25: Battery Voltage as of 12/14/07.

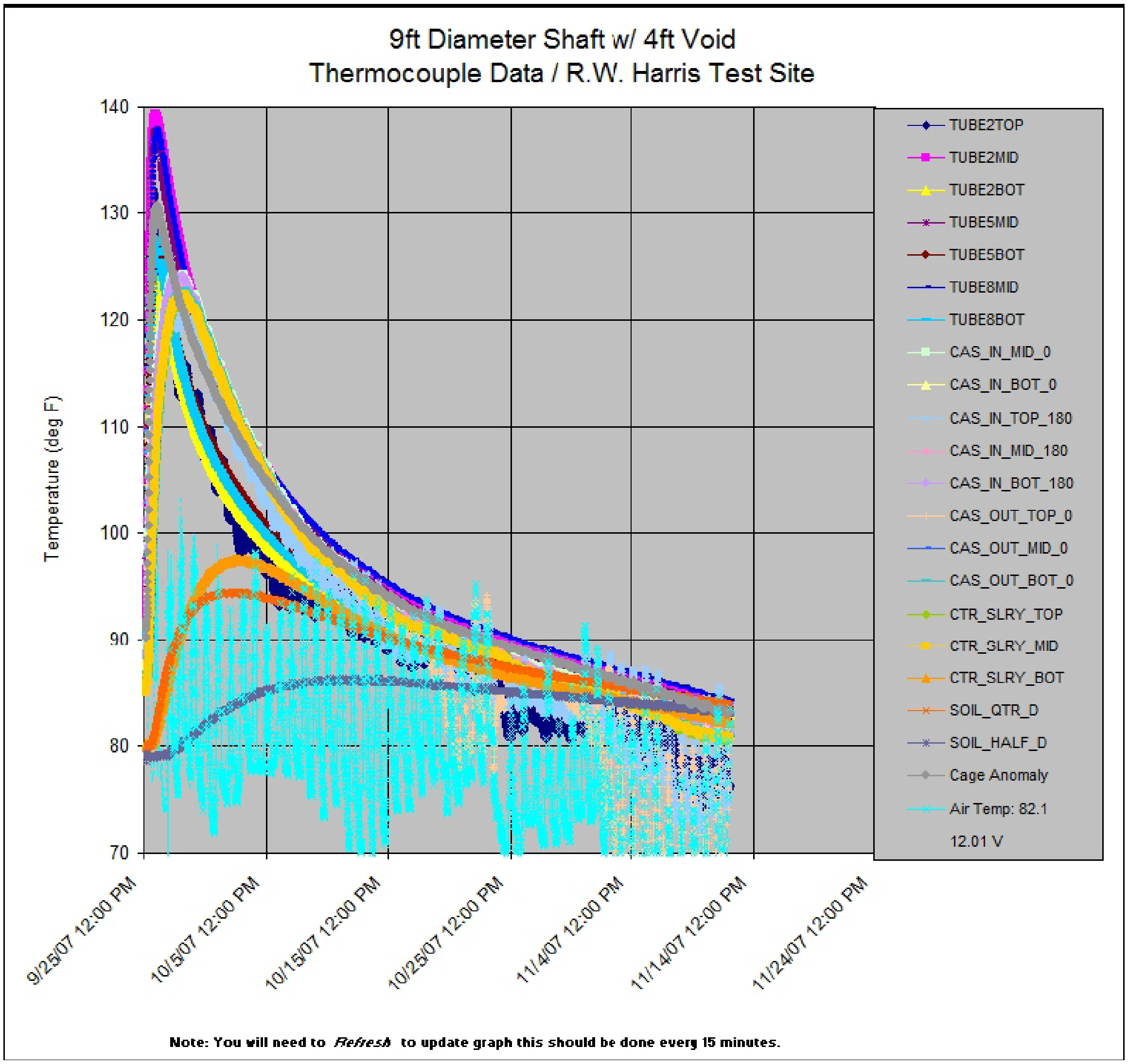

Figure 3-26: Thermocouple Data as of 11/12/07.

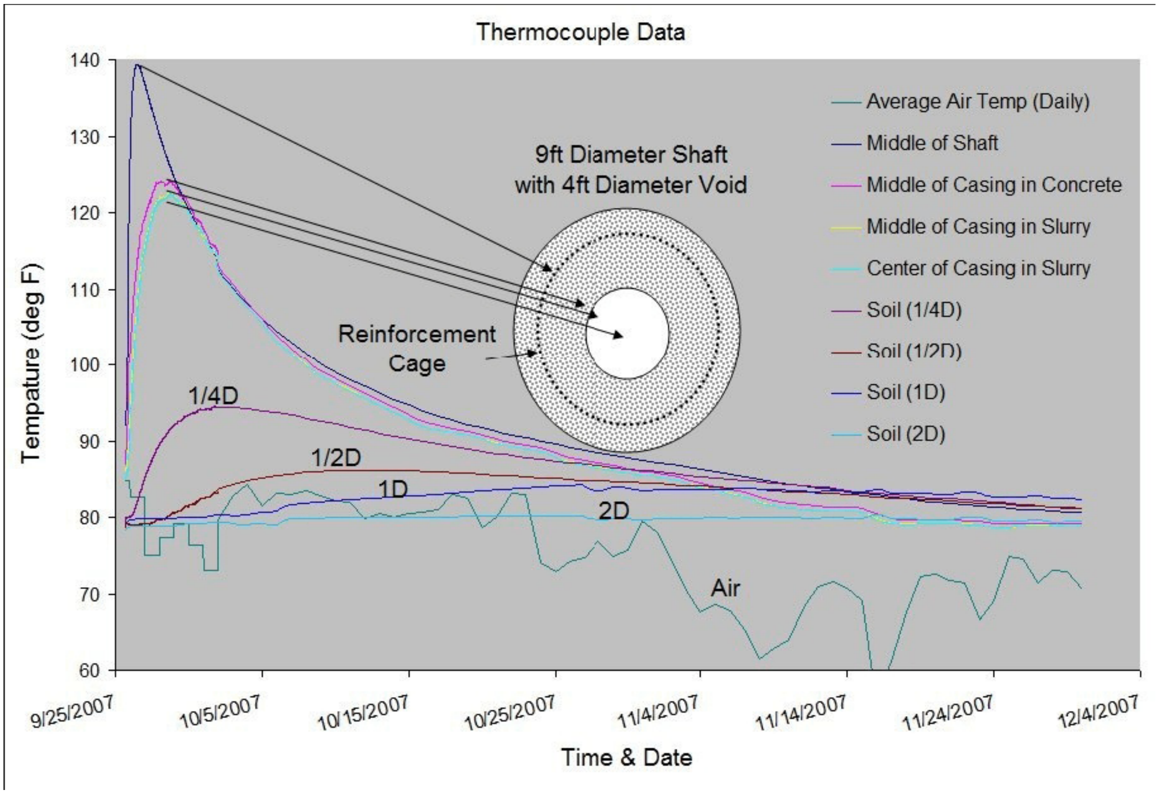

Figure 3-27: Final Average Thermocouple Data for All Locations.

Chapter 4

St. Anthony Falls Bridge Foundation Monitoring

On August 1st, 2007, a portion of the Interstate 35 Westbound Bridge over the Mississippi River collapsed in the middle of rush hour. The collapse killed 13 people and opened the eyes of engineers across the country to America's failing infrastructure. Part of this research is proposing that a catastrophe such as this could be prevented through the use of remote monitoring systems with the capability to alert users when certain structural members reach a predetermined level of stress. In order to fully understand the forced induced into a structure such as a bridge, the Minnesota Department of Transportation (MnDOT), the Federal Highway Administration (FHWA) and USF Geotechnical Research Department are working together on a remote monitoring system that will provide such much needed information. As the MnDOT is re-building I-35 Westbound, USF will be instrumenting a number of structural members to provide real-time information about the stresses being felt by the bridge (Figure 4-1).

This study was broken into three phases. The first phase was during the construction of the concrete drilled shafts or caissons and the pier footing that ties the drilled shafts together. Thermocouples were placed in the re-bar cages of the shafts as well as throughout the pier footing and used to determine the core temperatures of the mass concrete elements. This part of the study was similar to the Voided Shaft study that was discussed in Chapter 3.

The second phase of the study slightly overlapped the first phase in that it involved the drilled shafts, but it also branched upwards to the columns. Two different types of strain gages were placed in the re-bar cages of the shafts and at the center height of the columns. These were used to more accurately determine the load induced in the shafts by the pier footing, columns and superstructure, and the loads induced in the columns by the bridge superstructure during the bridge construction. Furthermore, as each new section of the concrete box-girder superstructure is added to the columns, the added weights of the sections can be correlated to the strain in the columns measured by the installed gages. This will provide more accurate calibrations to be used in the ongoing health monitoring of the bridge, which is phase three of the project.

At the time of completion of this report, the final phase of the study has not yet started. It will use the same strain gages that are embedded in the shafts and columns, as well as strain gages that will be installed in the superstructure components of the bridge by the University of Minnesota. The final phase of the project will monitor the loads on the bridge throughout its service life, which can be used to determine the Structural Health of the bridge, and can provide MnDOT and FHWA with real-time strain and load data from the bridge (Figure 4-2).

4.1 Phase I – Thermal Monitoring

As stated above, the first phase of this project was to monitor the internal temperatures of the mass concrete elements (drilled shafts and pier footing). While the overall procedure of the thermal monitoring was very similar to the R.W. Harris Voided Shaft study, there were some major differences. First, the shafts are solid, not voided. Voided shafts have not been tested for their strength capabilities, so therefore would not be used in a bridge's substructure. Secondly, the ambient temperature at the site is much different. As seen from Figures 3-26 and 3-27, in the Tampa Bay area during the monitoring period, the air temperature ranges from approximately 100˚ F down to 65˚ F. In Minnesota during the construction and thermal monitoring period, the temperature ranges from approximately 35˚ F down to -10˚ F. This should be expected to have a significant effect on the temperatures reached by the mass concrete elements.

4.1.1 Construction and Instrumentation

Prior to construction and installation of the drilled shafts, the instrumentation for the thermal monitoring was put into place. The first step was the instrumentation of the reinforcement cage for the drilled shafts. The reinforcement cage was built using high strength longitudinal steel and mild stirrup steel. The cage has 20-63mm threaded longitudinal bars with #6 bar circular ties at 5 inches on center. Locking wheel cage spacers were placed along the reinforcement cage to maintain 6 inches of clear cover (Figure 4-3).

After the reinforcement cages were assembled, they were instrumented with thermocouples (TCs) and strain gages. The strain gages will be discussed in the section on Phase II. The TCs were installed in pairs at 4 levels along the shafts, later named GL1, GL2, GL3 and GL4, for a total of 10 TCs per shaft. GL4 was located at the bottom of the shaft, GL3 at the level of competent rock, GL2 at the bottom of the permanent casing, and GL1 at the top of the shaft (Figure 4-4). The wires from the TCs were bundled with the wires from the strain gages and run to the top of the shafts in two groups (Figure 4-5).

After the cages were fully instrumented, the excavations for the shafts were made. The shafts were drilled with two distinct sections. The top section is 7’-0” in diameter with a ½” thick permanent steel casing surrounding the shaft (Figure 4-6). This section is surrounded by dirt all around and needs the casing to keep the excavation clear. The casing runs down approximately 3’-0” below the level of bedrock. The lower section is 6’-6” in diameter with no steel casing. This section is placed in a bedrock socket and therefore has no need for a casing. GL2, GL3, and GL4 are all in this lower section of the shaft. After the excavation was made, the reinforcement cages were lifted and lowered into the excavation (Figure 4-7). After reinforcement cage placement, the concrete for the shafts were poured with a single tremie. Upon removal of the tremie after concrete placement, a rebar instrumented with two more TCs was inserted down the center of the shaft. The wires from all the TCs and strain gages were run out through a 1½” diameter schedule 40 PVC conduit that was placed running out through the top of the shaft, underneath the future pier footing that would be constructed, and out to the temporary Data Acquisition Systems (DASs) that were installed on site (Figure 4-8).

Two of the eight shafts were instrumented, (these can be seen in Figure 4-8) and when all eight shafts were finished, time was allowed for the concrete to cure, as well as the formwork and reinforcement for the pier footing. The pier footing is a large mass of concrete that sits above the drilled shafts. It supports the columns as well as ties the tops of the drilled shafts together. The pier footing for this project is 81-2” long by 34’-0” wide by 14’-0” tall (Figure 4-9). It is reinforced with 3 layers of #18 bars at the bottom of the footing and 3 layers of #18 bars at the top. Along the top, steel W-Shapes were used to support the reinforcing bars to prevent excess bending. TCs were installed at the base of the footing, the center of the footing, and the top of the footing. These TC wires were run out through a 1½" diameter schedule 40 PVC conduit down and out of the footing out to the DAS boxes alongside the conduits from the shafts. The MnDOT also ran PVC cooling tubes that were cast into the footing. Water was run through the tubes to help mitigate the mass concrete effects (Figure 4-10).

4.1.2 Monitoring Setup and Procedure

For this first phase of the study, the data collection was actually split into two sub phases. The first was the thermal monitoring of the shaft, and the second was the thermal monitoring of the pier footing. The two phases were done similarly, however, and the setup for the thermal monitoring system was very similar to the setup used in the Voided Shaft study discussed in Chapter 3. The system was made up of the following pieces: A Campbell Scientific CR1000 data logger, an AM25T 25-channel multiplexer, a Raven100 CDMA AirLink Cellular Modem, PS100 12V power supply and 7Ahr rechargeable battery, and a large environmental enclosure to protect all the materials from the elements (Figure 4-11). From the Voided Shaft study, it was learned that a larger solar panel would be needed to provide power to the system, and so a 35W solar cell panel was utilized (Figure 4-12).

The Thermal Monitoring procedure was identical to that of the Voided Shaft study. A thermal data sample was taken every 15 minutes and stored to the data logger at the same interval. Every 60 minutes, the Raven modem sent the collected data to the host computer at USF for data analysis. Once this data was received, it was reviewed and plotted for use on the USF Geotechnical Research website. This thermal data from the shafts was collected from 1/9/08 until 1/21/08. At this time, the TC wires from the shaft were disconnected, however the vibrating wire strain gages (discussed in Phase II) come with a thermistor. This thermistor was used to continue the thermal data from the shafts. The thermal data from the pier footing was collected from 2/6/08 until 2/25/08. No strain gages were installed in the pier footing, so the only thermal data collected was stopped after this date. As with the Voided Shaft study, the battery voltage for the data logger was also monitored, so that the logger would not lose power.

Along with the thermal monitoring setup, a CC640 camera was set up to take hourly photographs of the construction site (Figure 4-13 and 4-14). It was powered by the same solar panel as the thermal monitoring system. The photos taken by the camera were sent back with the data collected from the TCs by the CR1000. The camera was useful for the thermal monitoring phase, but it was really installed as an aid in the construction load monitoring phase, which will be discussed later.

4.1.3 System Results and Conclusions

The thermal monitoring procedure fared extremely well. From the information gathered from the Voided Shaft study about the power consumption, the 35 watt solar cell panel worked much better and the battery voltage never dipped below 12 volts (Figure 4-15). Twice during the thermal monitoring phase, the system lost and then regained cellular communication with the host server. These occurrences seemed to correspond to the use of a large electric power plant directly adjacent the system's cellular modem. This type of EMF is known to adversely affect such systems and is therefore a reasonable explanation. Other than these interferences, the thermal monitoring system worked as planned.

The concrete mix that was used was self consolidating concrete that was designed to have a lower heat of hydration (Figure 4-16). Therefore, the temperature traces were expected to be lower than that of the Voided Shaft study. The thermal data from Shaft 1 shows that the general average heat attained in the concrete was approximately 90˚ F, however there are two TCs that record a higher temperature of approximately 126˚ F, a 36˚ difference (Figure 4-17). Similarly in Shaft 2, the most of the TCs recorded a temperature of approximately 85˚ F, however there are two TCs that record a higher temperature of approximately 110˚ F, a 25˚ difference (Figure 4-18). This was not the result of a bad TC level. In reality, the shafts were poured one level at a time, alternating between shafts. When the trucks were complete, the shafts were not fully concreted. Therefore, extra trucks of concrete were required. It is assumed that the extra trucks did not use the same concrete mix as the first sections, and therefore this second batch had a higher heat of hydration, causing an elevated temperature reading in the top of the shafts.

As discussed in the monitoring procedure, the TC wires from the shafts were cut on 1/21/08 and the thermal data was no longer collected. Upon connection of the vibrating wire gages from the shafts the thermistors again started collection thermal data. This thermal data was analyzed and compiled with the data from the TCs and the continuation of the thermal curves were plotted (Figures 4-19 and 4-20).

As stated above, the thermal data from the pier footing was collected from 2/6/08 until 2/25/08 (Figure 4-21). As seen on the plot of the temperature over time, the TC in the extreme center of the footing recorded a maximum temperature of approximately 140˚ F, while the TC at the center bottom of the footing only reached a temperature of approximately 90˚ F. The same concrete mix was used throughout the pier footing, so it should all be roughly the same temperature, however the ambient temperature, which ranged from 40˚ F down to -10˚ F, caused the temperatures to drop drastically closer to the outside edges of the footing.

4.2 Phase II – Construction Load Monitoring

This phase of the study expands above and beyond what was done in the Voided Shaft study. In Phase II, the loads placed on the shafts by the pier footing, columns, and superstructure, and the loads placed on the columns by just the superstructure will be monitored. As shown in Figure 4-2, this phase actually begins at the start of the footing construction, but no real data was expected until the footing concrete was poured.

For the section on construction and instrumentation, there is obviously an overlap with the construction sequence. Therefore, this section of the report will not go into the details of the construction of the drilled shafts nor of the pier footing. However more emphasis will be placed on the strain gages that were installed in the drilled shafts. For the pier columns however, the construction will be explained as well as the instrumentation. Furthermore, focus will be paid to the phases of the construction of the column and how it affected the construction loads placed on the drilled shafts.

4.2.1 Construction and Instrumentation

For the instrumentation of the drilled shafts, some information will overlap, but it is necessary to explain the strain gages and their placement within the shaft. The strain gages used in this study were provided by Geokon, Inc. They are Model 4911 "Sister Bars" and are specifically made for ease of installation (Figure 4-22). They come with the strain gage pre-installed on a 54.25" length of #4 bar. This bar is then tied to the existing reinforcement in the shaft or column. Since the gage is on a #4 bar, it does not provide enough extra steel area that the cross-section of the element would be altered (providing the element is quite large) and therefore does not affect the calculations of converting strain to load. The strain gages in the shafts were installed at the same four levels as the TCs: GL1, GL2, GL3, and GL4 (Figure 4-4). However, two types of strain gages were used. At each level, 4 vibrating wire (VW) strain gages and 2 resistance (RT) strain gages were installed, which makes for a total of 16 VW gages and 8 RT gages. The VW gages were installed at 90˚ separation (Figure 4-23), with the RT gages at 180˚ separation, coupled with the VW gages. The VW gages, as explained in Phase I, come equipped with a thermistor. These gages are not capable of recording strains at extremely high rates, which is why RT gages were also installed.

At each main pier, two reinforced concrete columns sit on top of the shaft cap to support the superstructure for one direction of traffic. The columns were constructed with a varying cross-section (Figure 4-1). The critical cross- section is at the mid-height of the columns. This is where the strain gages were placed. The columns were cast in three separate pours to get the full length of the columns.

First, the longitudinal bars running up through the columns were spliced to the longitudinal bars embedded in the pier footing (Figure 4-25). Then the formwork for the lower half of the column was set in place. The first pour was a small 200 yd^3 pour to get the column started. After that, the horizontal reinforcement was set in place inside the formwork up to the mid-height of the column. After the horizontal steel was in place, the next level of longitudinal steel was spliced to the first level so that the bottom of the bars would be embedded in the lower half of the column. After the reinforcement up to mid height was installed, the second pour took place. This finished the concrete up to mid-height of the column (Figure 4-26). The next phase of construction was to place the formwork for the top half of the column, and then install the horizontal steel in the column. This was when the gage installation took place. The critical section of the column is 8'-0" by 16'-0" with reinforcement that consists of 44 #18 bars with 8 on the short sides and 16 on the long sides (Figure 4-27).

The total instrumentation for each column consists of 4 vibrating wire strain gages and 4 resistance type strain gages. The same "coupled" gages that were installed in the shafts were used in the columns (one VW gage and one RT gage). One installation unit was installed at each corner of the column in the critical section (Figure 4-28). By placing the gages in the corners of the cross-section, the strain at the extreme fiber of the column could be measured. Once the gage installation units were tied and secured in place (Figure 4-29), the wires were run out of the top of the column formwork so that the cables could be bundled together. Then the wires were brought back down to the mid-section of the column and were run out through the 1 ½" Sch. 40 PVC conduit that reached up through to the mid-height of the columns (Figure 4-30). The wires ran through the conduit down through the column and the shaft cap and then out to the temporary DAS that was installed on site. In addition to these strain gages, the University of Minnesota Civil Engineering department also placed 5 strain gages in each column. These strain gages were installed in the same locations as those done by USF, but with an additional gage located in the center of the column. The wires for these gages were bundled with the wires from the USF gages and drawn out to the DAS at the same time. These cables are grey in color (as opposed to the blue and green used by USF) and can be seen clearly in Figure 4-30.

4.2.2 Monitoring Setup and Procedure

For this second phase of the study, the data collection was actually split into two sub phases. The first was the load monitoring of the shaft, and the second was the load monitoring of the columns. The reason for this is that a large amount of dead load on the shaft comes from the construction of the pier footing and the columns. Furthermore, if the loads on the shafts are monitored first, then checking that the measured loads are correct is much easier as the load is simply the dead load of the footing and columns. Each phase of monitoring was carried out in the same way. The monitoring setup and procedure will be explained by discussing the three different systems that were installed and used during this phase of the study.

System 1 was the same thermal monitoring system that was used in the first phase of the study as well as the Voided Shaft study discussed in Chapter 3. It was re-used during this phase of the study as the monitoring and transmission system for the CC640 field camera. The camera was set up to take a picture every hour and then transmit that picture back to the host computer via the cellular modem. During the thermal monitoring phase of the study, System 1 was powered by the installed solar cell panel with a back-up deep cycle battery. During the 2^{nd} phase of the study the system was moved to A/C power, but with a deep cycle battery still in reserve should the A/C power be disrupted. This A/C power was provided by the Army Corps of Engineers who had an A/C power source at the site.

The second and third systems were installed at almost the same time but have different capabilities/assignments. System 2 was designated to collect data from the vibrating wire gages installed in shafts 1 and 2 as well as those in the interior and exterior columns. This system also recorded the gage temperatures via changes in thermistor resistance. In all, 50 vibrating wire gages and 50 thermistors were connected to this logger via two AVW-200 two channel spectrum analyzers. Each channel of the AVW200 units is connected to a MUX 16/32B multiplexer (four in all). MUX 1 was connected to shaft 2 (16 gages), MUX 2 was connected to shaft 1 (16 gages), MUX 3 was connected to the interior column (10 gages), and MUX 4 was connected to the exterior column (10 gages) (Figure 4-31). The true value of the AVW200 data is at present unclear as many pieces of data quality is recorded along with the raw strain and temperature values of interest. These additional measures of data quality (e.g. signal to noise ratio, etc.; four in all) are intended to provide insight into the health of the gage, but triple the required storage space and therein significantly reduce the overall duration of monitoring without remote intervention. The system monitoring the VW gages (System 2) used a Campbell Scientific CR1000 data logger, while the system monitoring the RT gages (System 3) used a Campbell Scientific CR9000 data logger. System 2 worked similarly to the thermal monitoring system. A sample was taken by and stored to the data logger every 15 minutes. Every hour, this stored information was sent back to the host computer at USF to be compiled and analyzed. System 3, using the CR9000, took a sample at a rate of 100 Hz (100 samples per second). However, all of this data was not stored. Rather, the mean, maximum, and standard deviation of these samples was stored every 15 minutes. Then, every hour, the stored data points were sent back to the host computer along with the data from System 2. This provided the user with a better idea of the strain in the system because of the high sampling rate; however it used a large amount of power, which will be discussed in an upcoming section. The monitoring system sampling and storage rates and other information are given in Table 4-1. Between the two systems were a shared Raven100 CDMA AirLink Cellular Modem, PS100 12V power supply and 7Ahr rechargeable battery, and three large environmental enclosures to protect all the materials from the elements (Figure 4-32).

Table 4-1 Overview of Monitoring Systems for I-35W Bridge Study

	System 1	System 2	System 3
Gage Type	Thermocouples	VW Strain Gages Thermistors	RT Strain Gages
Data Logger	CR1000	CR1000	CR9000
Sampling Rate	15 Min	15 Min	100 Hz
Storage Rate	15 Min	15 Min	15 Min (Sample Mean, Sample Max, Std. Deviation)
Transmit Rate	1 Hr	1 Hr	1 Hr

Both System 1 and System 2 are powered by the A/C power source as provided by the Army Corps of Engineers, however each are connected to a deep-cycle battery and a solar cell panel as a backup. For System 3, it was known that a large amount of power would be consumed by the monitoring system. Therefore it was necessary to provide the system with enough back-up power so there would be no power problems as there were with the Voided Shaft study. The CR1000 could only be run from either the A/C source on site or the solar panel, not both. Therefore, if the A/C were ever to cut out, there would not be the back-up of the solar cell panel to run the system. Therefore a battery manager was set in place to bypass this limitation. The CR1000 was hooked up to run off the A/C source. The battery manager was hooked up to the solar cell panel and the back-up Deep Cell battery as well as the external battery pin on the CR1000. The battery manager allowed the Deep Cell battery to be charged by the solar cell panel, and kept the A/C power from back feeding the solar cell panel when it received little power (such as at night).

As with the thermal data from the shafts, once this data was received and reviewed it was plotted for use on the USF Geotechnical Research website. The strain in the shafts at the four different levels was monitored beginning on 2/6/08 with the pier footing concrete placement. Back in the Geotechnical Research department at USF, the strain data from the shafts was computed into construction loads and an annotated graph was updated to the USF Geotechnical Research website (Figure 4-33). Along with this graph, using the CC640 field camera, pictures from these events were captured and they can be related to the points of interest on the graph. This aids in verifying the loading event, as well as verifying the amount of load that is calculated in the shaft (Figures 4-34 through 4-37). At the time of the report construction is still in progress, therefore the construction loads on the shafts are still being monitored.

As seen in Figure 4-37, the column foundation quickly became too large to be seen in its entirety by the close-up camera location. Therefore the CC640 field camera was moved to a new location atop the University of Minnesota "Bob Main" building on the south west bank of the river. This new position affords oversight of the entire project from end bent to end bent and can now be used to dovetail recorded strains to construction events (Figure 4-38).

4.2.3 System Results and Conclusions

The three monitoring systems used during the construction load monitoring phase fared very well. System 1 twice lost and then regained communication with the host server. These occurrences seemed to correspond to the use of a large electric power plant directly adjacent the system's cellular modem. This type of EMF is known to adversely affect such systems and is therefore a reasonable explanation. As stated in the monitoring procedure, the system as repositioned in early March. This system worked without issues from Mar 5th to March 19th when communication between the camera and logger failed. Review of the system revealed the camera was still recording images to its internal compact flash card, but images were not transferred to the logger for scheduled collection. Cellular communication with the logger has not faltered since its repositioning atop of the University of Minnesota BOBMAIN building.

As stated in the monitoring procedure, power consumption was a large concern for this phase of monitoring. The power of System 1 was stable throughout this phase. As stated earlier, the System was originally powered by completely via solar energy while a deep cycle battery was used as a back-up. In early March, the power source was switched to constant A/C and provided the system with more stabile voltage (Figure 4-39). At no time did the voltage approach the critical logger shut down voltage. Results of both the close-up pictures and the overview pictures are shown in the previously documented figures.

The results of System 2 were a little less desirable. The cellular communication with this system became somewhat of a concern with regards to reliability. This system, which was similar to System 1, logged data that was collected without issue from Feb 5th until March 26th. For a short period following this time frame, no collections were possible. It was unclear whether the system was still powered and logging although up until the last collection the power cycles were regular (Figure 4-40). Since the critical threshold voltage of 11.2 volts was not approached at any time, it is unlikely that power interruption was the cause of the communication errors. The concern with the intermittent communication was resolved, but the data collected from one of the four multiplexing units responsible for monitoring nine of the vibrating wire gages was unintelligible. This is thought to be a total failure due to a cut wire or a bad connection between that device and the data logger.

The results of System 3 were much better than those of System 2. To date, System 3 communication has not faltered. The primary difference between this system and the other two is the logger type, CR9000 vs. CR1000. The latter of which has not been consistent. The battery voltage of System 3 varies much less than the battery voltage of System 2, yet neither system seems to have had a power interruption (Figure 4-41).

Along with the results from the gages, the website hosted by the University of South Florida host computer was drastically changed. The main page now has hover points associated with pathways to videos or data locations (Figure 4-42). The link to the South Camera Perspective takes the user to a page that shows a video made of time lapse photos taken by the CC640 Field Camera in its altered position atop the University of Minnesota Building. The link to the West Camera takes the user to a page that shows a video made of time lapse photos taken from the web camera set up by the Minnesota DOT. The Pier 2 Close-up Camera link takes the user to a page that shows a video made of time lapse photos taken by the CC640 Field Camera in its original close-up position. All of these videos provide a quick look at the construction progress of the bridge from different vantage points and were used to relate the strain data to specific construction events. The FHWA Substructure Health Monitoring Site link takes the user to a separate page with a close-up view of the site with more hover points (Figure 4-43). Each of the texts is a link that will take the user to a plot of the strain of that subject over time (Figures 4-44 through 4-47). These graphs were broken down into daily increments as shown by the dotted lines running vertical on the graphs. The space between these dotted lines is a link that takes the user to the pages with the web cameras showing the construction progress up to that date. This way the strain data can be more accurately related to construction events. The graphs shown show the strain data up to 5/2/08. These graphs are provided just to show the capabilities of the updated website. Figures 4-48 through 4-51 provide the latest strain data from the shafts and columns as of 6/25/08. In these graphs it is easier to see the tremendous increase in load that the columns and shafts are experiencing due to the construction of the superstructure.

4.3 Phase III – Long Term Health Monitoring

The third and final phase of the St. Anthony Falls Bridge Monitoring Project is the long term health monitoring of the substructure as well as the superstructure. In Phase

III the loads induced on the entire bridge by the ongoing daily use of the bridge will be monitored. As shown in Figure 4-2, this phase will actually begin once the bridge is fully constructed and open to the public. At the time of this report, the bridge is still being constructed and is set to be completed near the end of December, 2008. Therefore this phase of monitoring will not be started until that time.

For the section on instrumentation, focus will be placed on the planned added instrumentation that will be required for this phase of monitoring, but there will obviously be no final information that can be provided. For the monitoring setup and procedure, once again focus will be placed on the planned monitoring procedure. This phase will utilize the information gleaned from the first two phases of the study to provide more accurate data collection and analysis, as well as the steps necessary to guarantee the full automation of the monitoring systems.

4.3.1 Instrumentation

Upon completion of construction of the bridge superstructure, the entire group of 64 wires from the substructure strain gages will be disconnected from the temporary DAS boxes at the base of the pier foundation. These wires will require hermetically sealed splices, and then will be run back up through the pier footing, columns, and into the bridge superstructure. Here the wires will be re-connected, along with the wires from the superstructure gages, to the permanent DAS boxes. The type of box and location is yet to be determined, but similarly to the temporary DAS boxes, they should be generally resistant to rodent-induced damage, environmental damage, and vandalism. There will be no added gages from the substructure used in the long term health monitoring phase.

The University of Minnesota will be responsible for the determination and location of the gages that will be installed on the bridge superstructure. At the time of this report, the planned gages for the superstructure include the following: vibrating wire strain gages with temperature gages (thermistors), chloride penetration sensors, accelerometers, and acoustic monitors. Figure 4-48 through 4-50 show the planned locations for the superstructure gages. These gage locations have not been finalized and are subject to change if the proposed locations are not accessible.

4.3.2 Monitoring Setup and Procedure

The substructure gages are planned to be continually monitored with the same monitoring procedure that was used during the construction load monitoring phase of the study. It is expected that the resistance type gages will be very useful during this phase because the sampling rate of resistance type gages is basically only limited by the sampling rate of the DAS. However it is important for the vibrating wire gages because the resistance gages are more susceptible to signal decay from long lead wires than the vibrating wire gages. The monitoring procedure for the superstructure gages will be determined by the University of Minnesota research team, and has not been determined at the time of this report. It is expected that the CC640 field camera will stay installed atop the University of Minnesota BOBMAIN building and it will continue to take pictures every hour. This will be useful in relating strain events to actual traffic conditions that are visible on the bridge.

4.3.3 System Results and Conclusions

As stated previously, at the time of this report the long term health monitoring phase of this project is not underway and is not expected to begin until December 2008. Therefore at this time there are no results or conclusions to be made based on the long term health monitoring procedure. However it is expected that the substructure monitoring procedure will be completely finalized and that therefore once the gages are permanently installed to the DAS boxes, no work will need to be done on them.

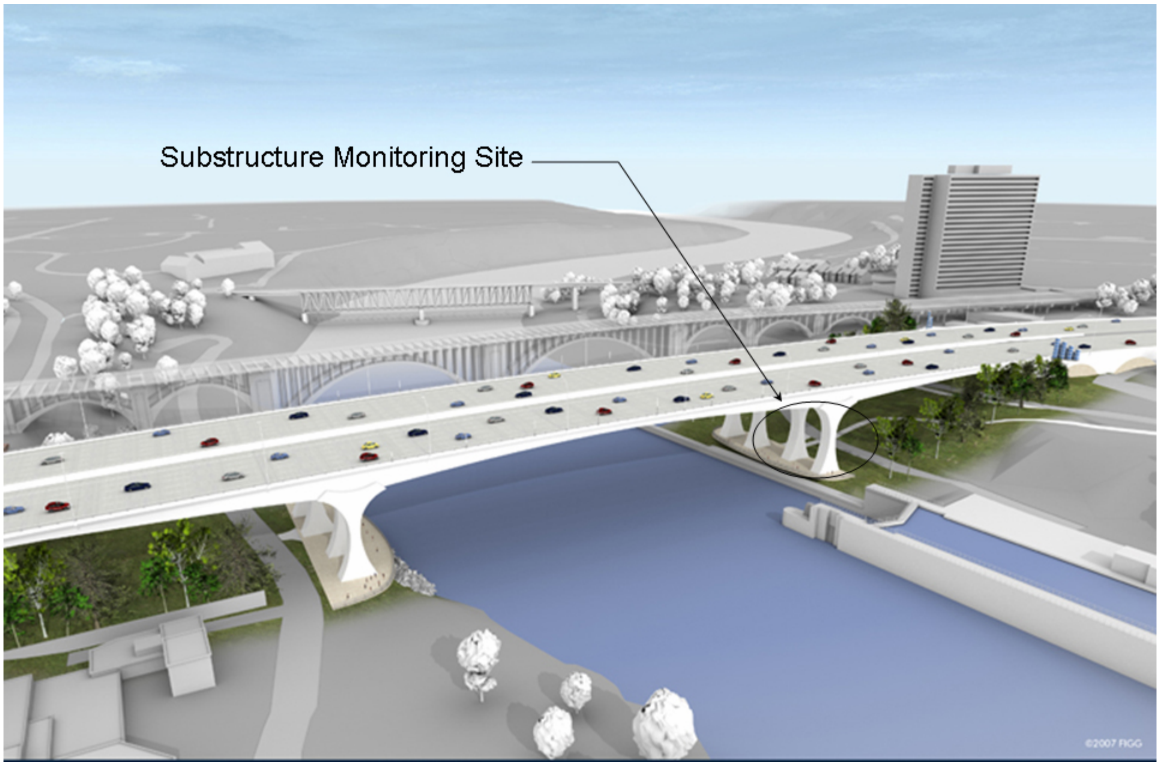

Figure 4-1: Artist's Rendering of I-35W Bridge over Mississippi River.

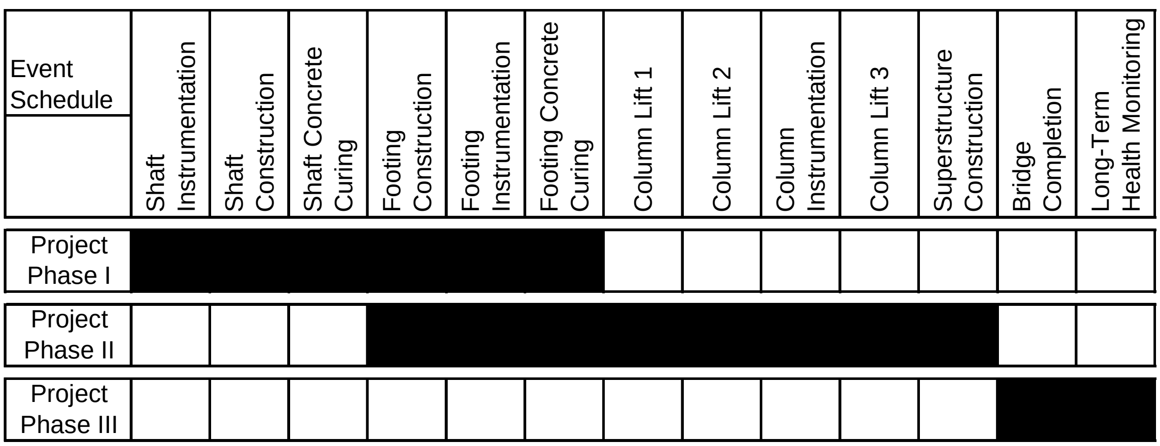

Figure 4-2: Event Schedule and Overlap of I-35W Bridge Project Phases.

Figure 4-3: I-35W Bridge Shaft Reinforcement Cage Construction.

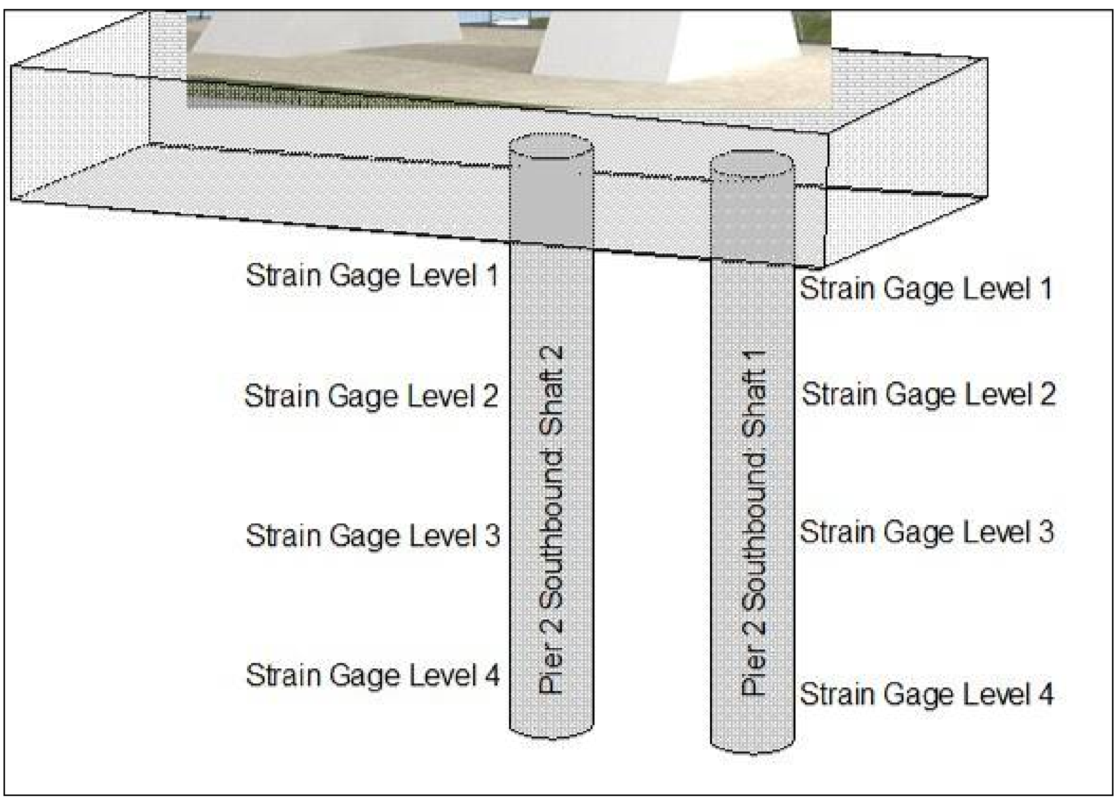

Figure 4-4: I-35W Bridge Gage Levels on Drilled Shafts.

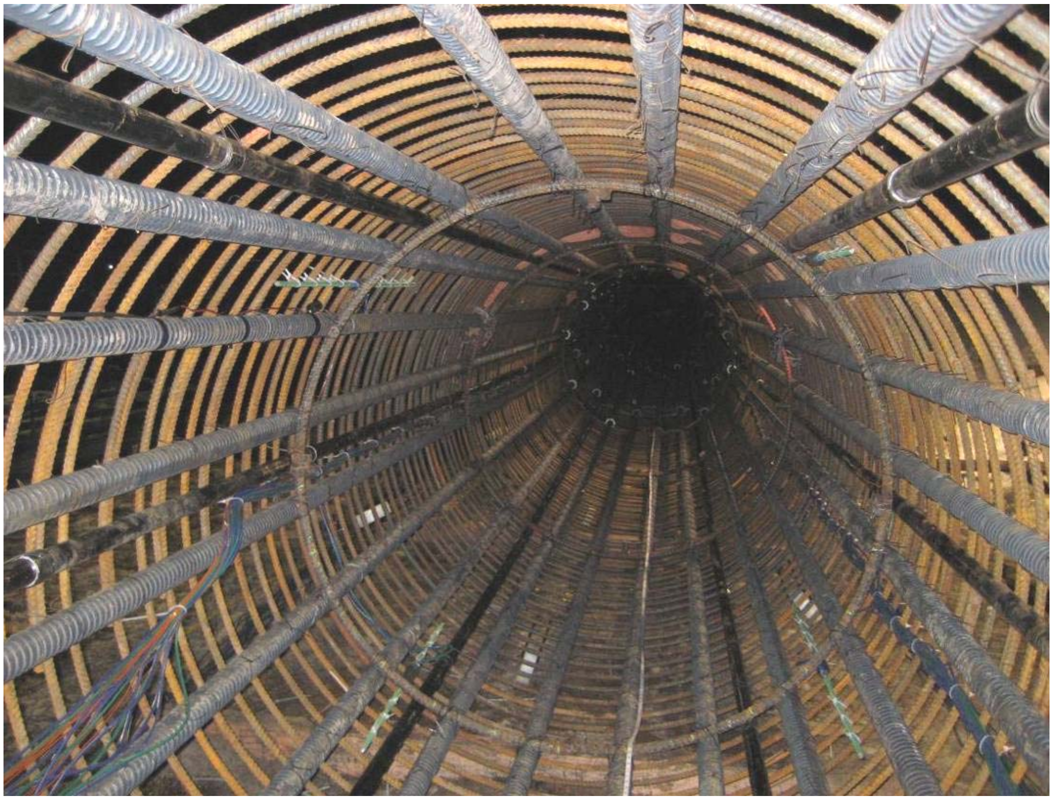

Figure 4-5: Cable Bundles in Reinforcement Cage for I-35W Bridge.

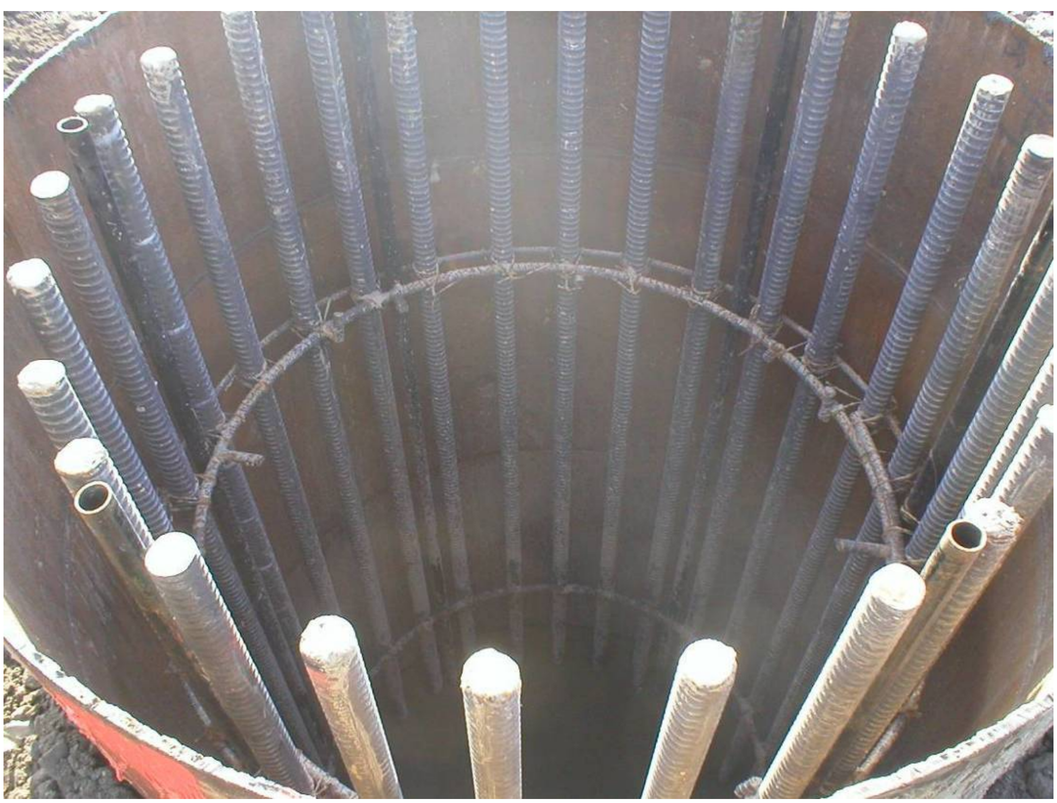

Figure 4-6: Top Section of Drilled Shaft for I-35W Bridge.

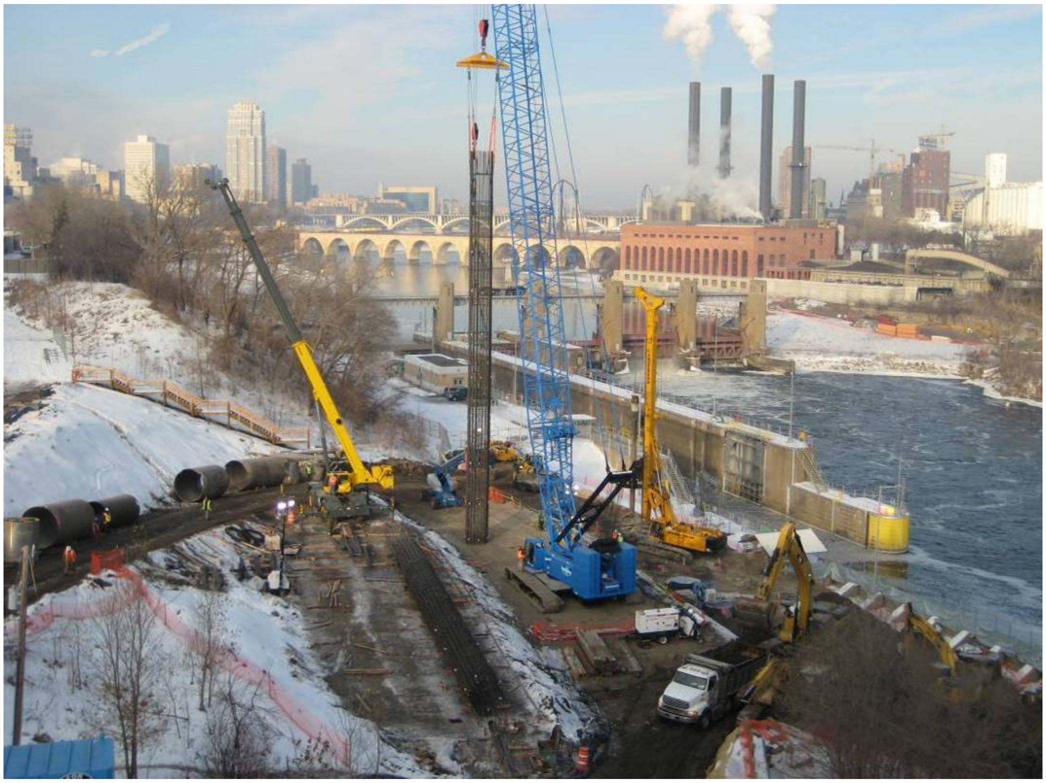

Figure 4-7: Placement of Reinforcement Cage for I-35W Bridge Shaft.

Figure 4-8: Conduits Running from Shafts to DAS Boxes.

Figure 4-9: Lower Layer of Pier Footing Reinforcement for I-35W Bridge.

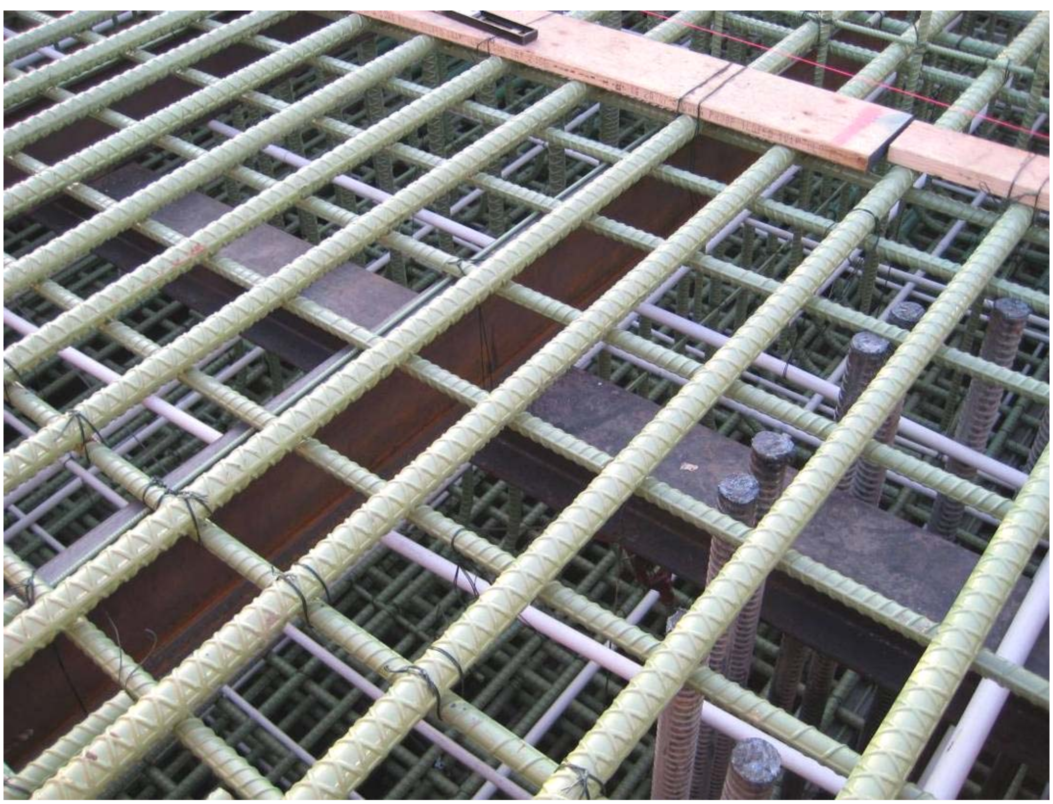

Figure 4-10: Upper Layer of Pier Footing Reinforcement for I-35W Bridge.

Figure 4-11: Thermal Monitoring DAS for I-35W Bridge Shafts.

Figure 4-12: 35 Watt Solar Cell Panel for I-35W Bridge Monitoring System.

Figure 4-13: CC640 Jobsite Camera with Perspective Outlines.

Figure 4-14: Sample Camera Shot from Close-Up Cam on I-35W Bridge.

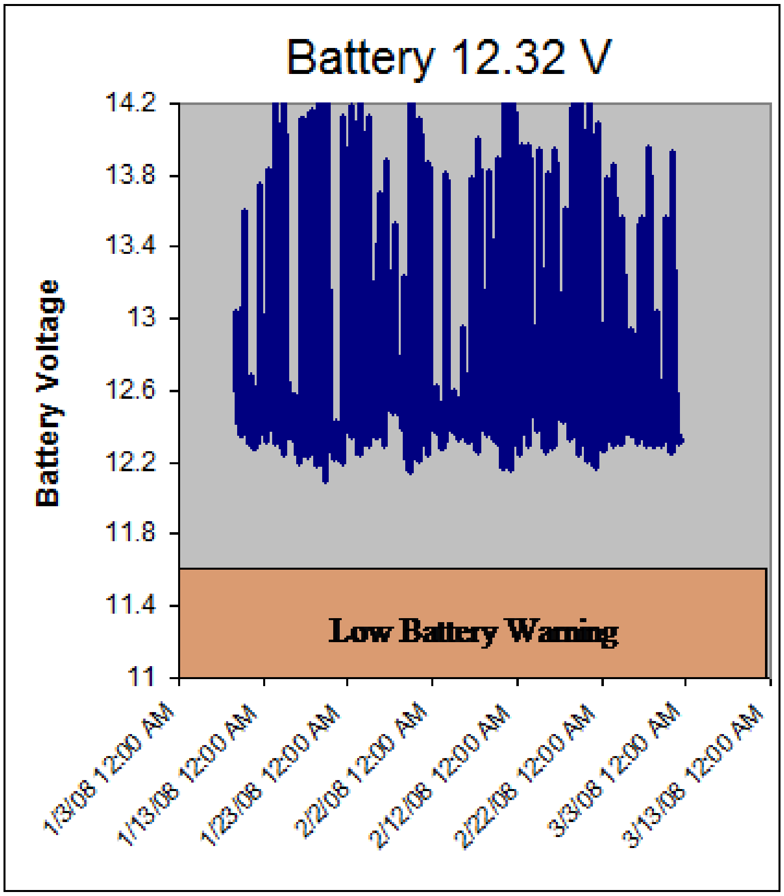

Figure 4-15: Data Logger Battery Voltage from I-35W Monitoring System.

District Metro FLATIRON MANSON		St. Anthony Falls (35W) Bridge Design-Build Project INSPECTION & TESTING PLAN	
Doc. No.: CQP413F	Rev. 0	10.08.07	Page CQP413F - 1 of 1

Subject: REQUEST FOR CONCRETE MIX DESIGN APPROVAL

Requested By:	Kevin Heindel	Phone	651-686-4233
Firm Name:	Cemstone Products Co.		
Agency Engineer/Inspector	Kevin Western (MnDOT)	SP #	(I-35W Bridge)

Proposed Aggregate Sources

	CA #1	CA #2	CA #3	CA #4	Sand
Pit Number	82001	73006			82001
Pit Name	Grey Cloud	Martin Marietta			Grey Cloud
Nearest Town	Newport	St. Cloud			Newport
Size	3/8" (CA-80)	¾" (CA-50)			Sand
Sp.G. [1]	2.66	2.72			2.62
Absorption [1]	0.013	0.004			0.010

[1]Provided by Mn/DOT

Proposed Cementitious Sources

	Cement	Fly Ash	Slag
Manufacturer/Distributor	Lehigh	Headwaters	Holcim
Mill/Power Plant	Mason City, IA	Coal Creek, ND	Chicago, IL
Type/Class	Type I	Class F	Grade 100
Specific Gravity	3.15	2.55	2.89

Proposed Mix Designs

Type of Work	Drilled Shafts		
Mix Number	ITF5035C		
Water (lbs/C.Y.)	270		
Cement (lbs/C.Y.)	242		
Fly Ash (lbs/C.Y.)	108		
Slag (lbs/C.Y.)	359		
W/CM Ratio	0.38		
Sand (Oven Dry, lbs/C.Y.)	1350		
CA #1 (Oven Dry, lbs/C.Y.)	410		
CA #2 (Oven Dry, lbs/C.Y.)	1280		
CA # 3 (Oven Dry, lbs/C.Y.)			
CA # 4 (Oven Dry, lbs/C.Y.)			
%Air Content	2.0		
Maximum Spread (3" Range)	20" to 23"		
VMA (oz/100 #CM) BASF-358	6.0		
HRWRA (oz/100 #CM) 7500	5.0		
AEA (oz/100 #CM) Daravair			

The above mixes are approved for use, contingent upon satisfactory site performance and continuous acceptability of all materials sources, by:

Requested By		Date
Mn/DOT Reviewer		Date
Reviewed by: Mn/DOT Concrete Office		Date

Comments: Mix ITF5035C is for information purposes only and has been created by adjusting the aggregate proportions of mix ITF5035B so that the JMF may be met. No new JMF for mix ITF5035C has been created.

Written by: FMM	Revised by:	Approved by:
Date: 10.08.07	Date:	Date:

Figure 4-16: Concrete Mix Design for Drilled Shafts on I-35W Bridge.

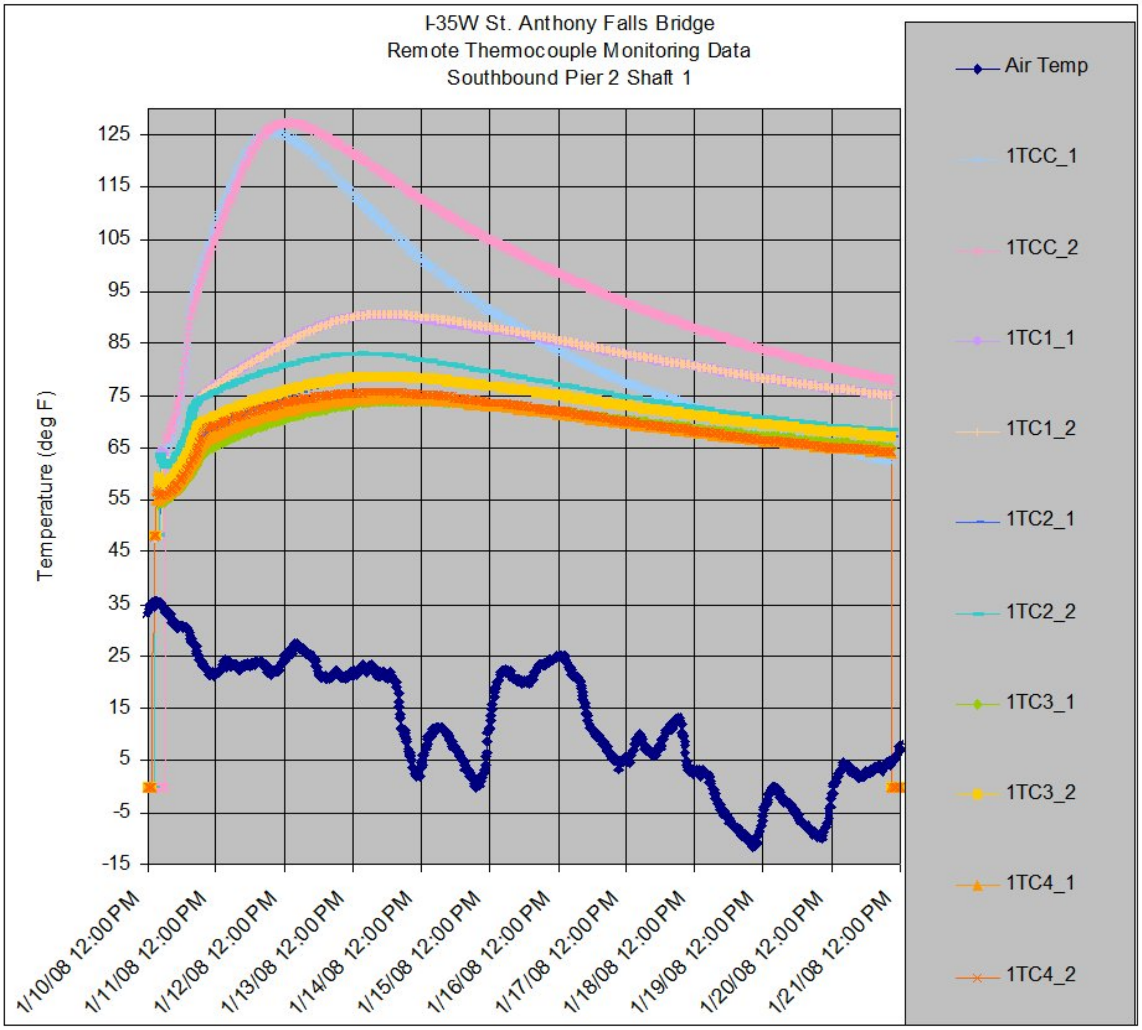

Figure 4-17: I-35W Bridge Southbound Pier 2 Shaft 1 Thermal Data.

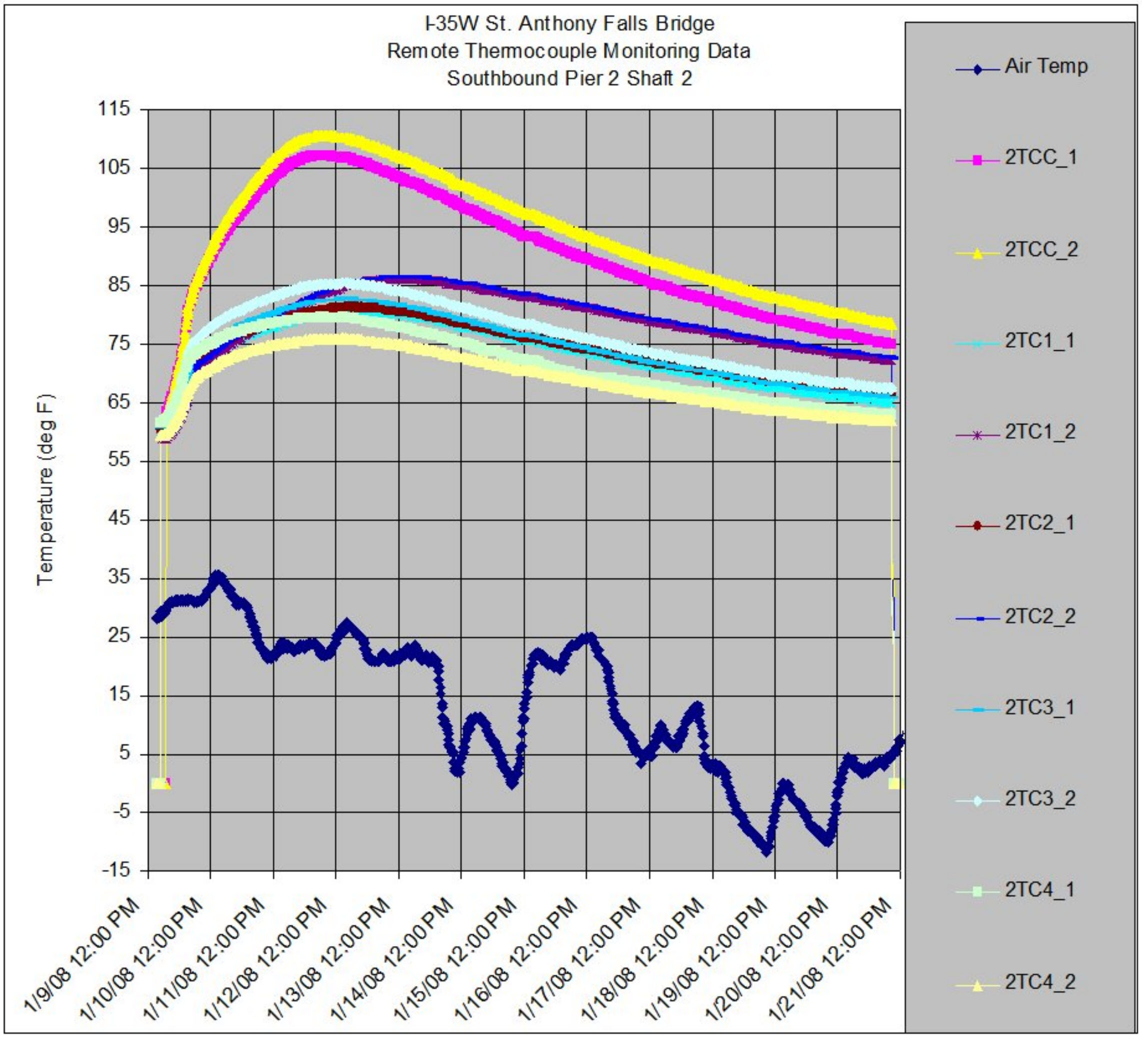

Figure 4-18: I-35W Bridge Southbound Pier 2 Shaft 2 Thermal Data.

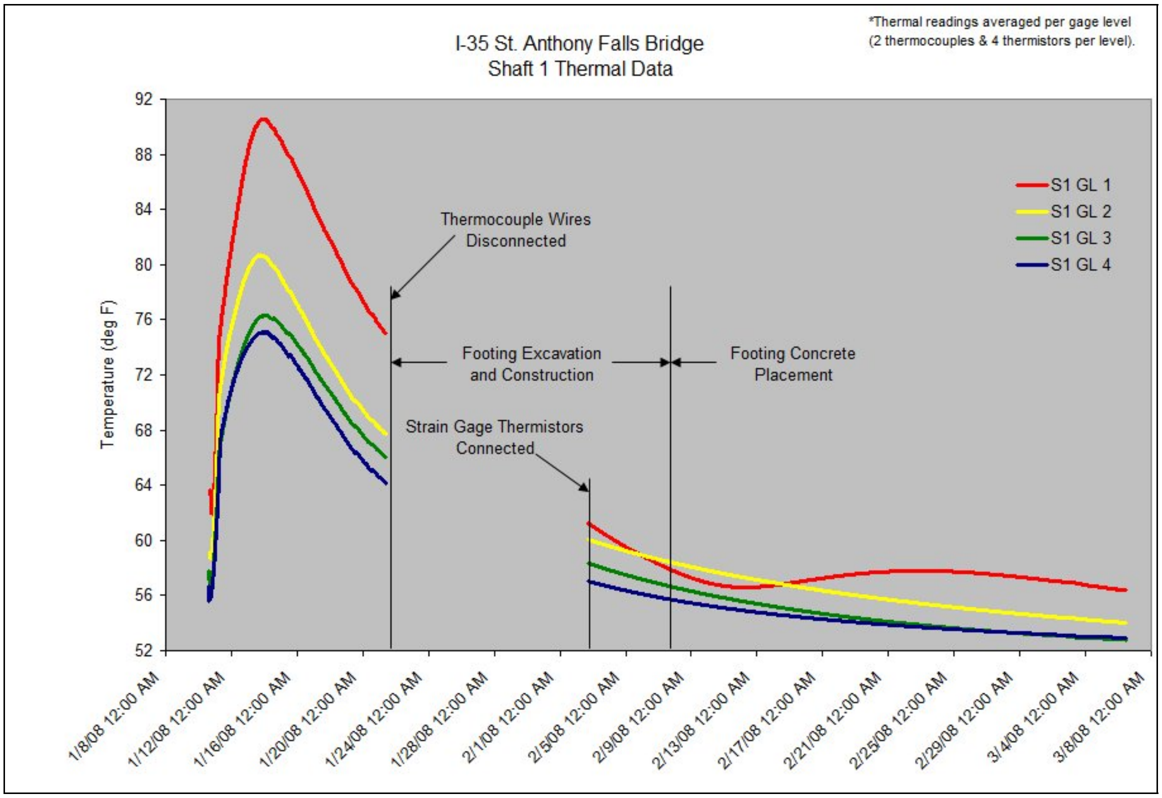

Figure 4-19: I-35W Bridge Shaft 1 Thermal Data from TCs and Thermistors.

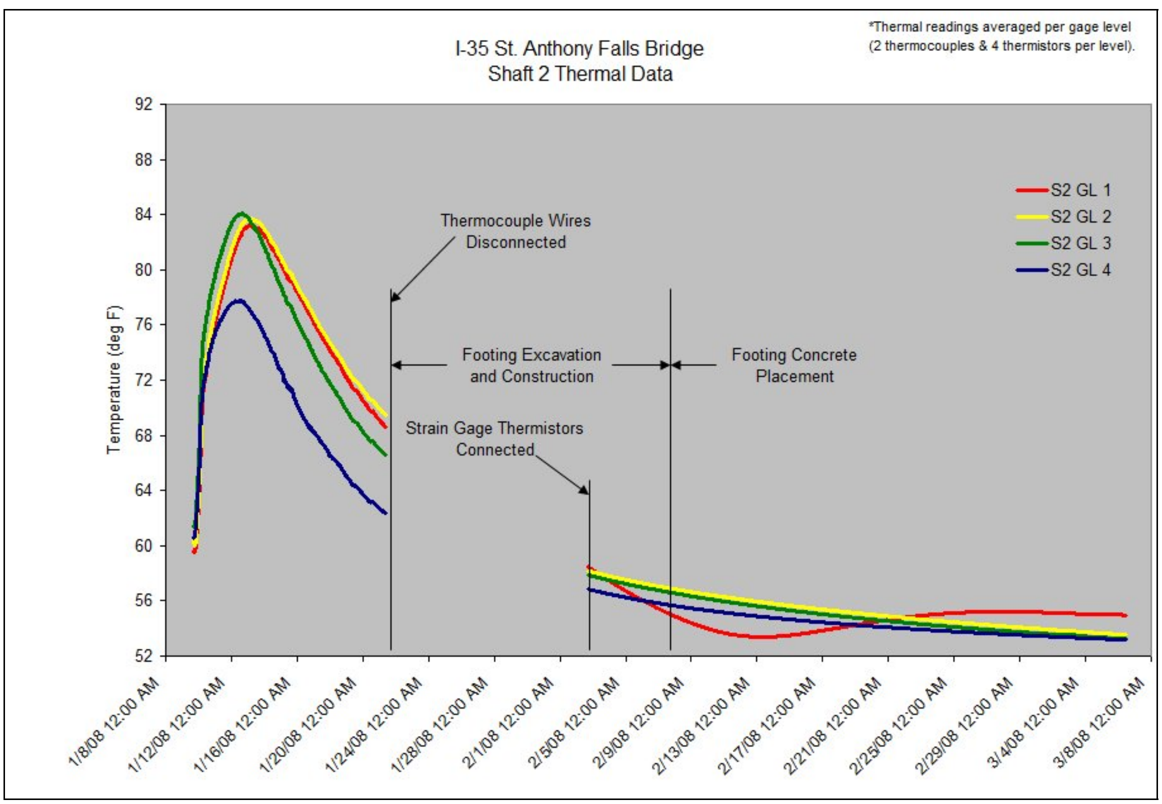

Figure 4-20: I-35W Bridge Shaft 2 Thermal Data from TCs and Thermistors.

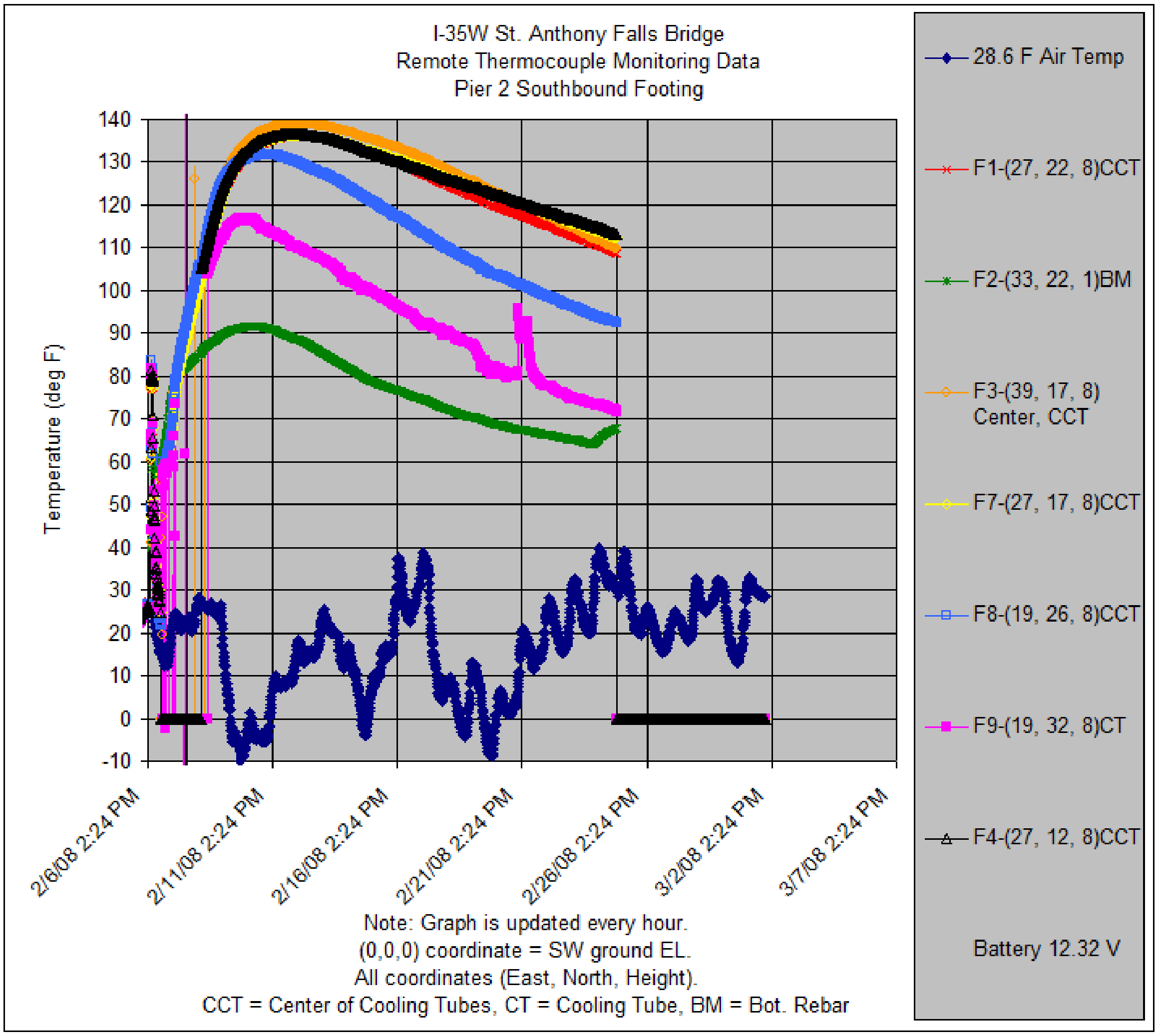

Figure 4-21: Pier 2 Southbound Footing Thermal Data from Thermocouples.

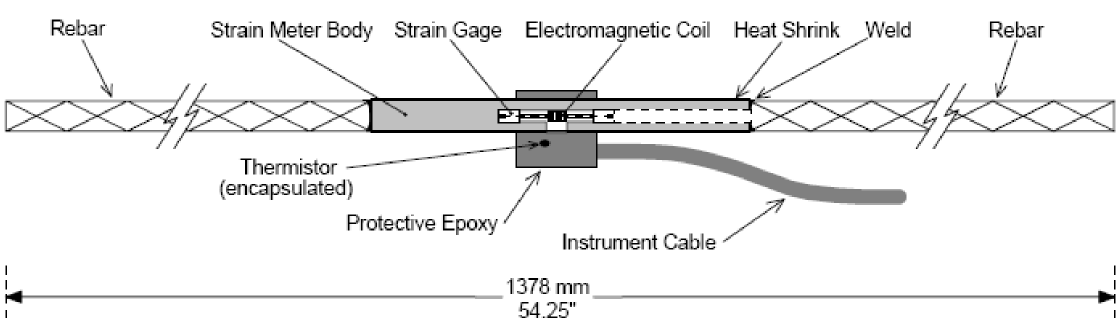

Figure 4-22: Detail of Geokon 4911 "Sister Bar" Strain Gage.

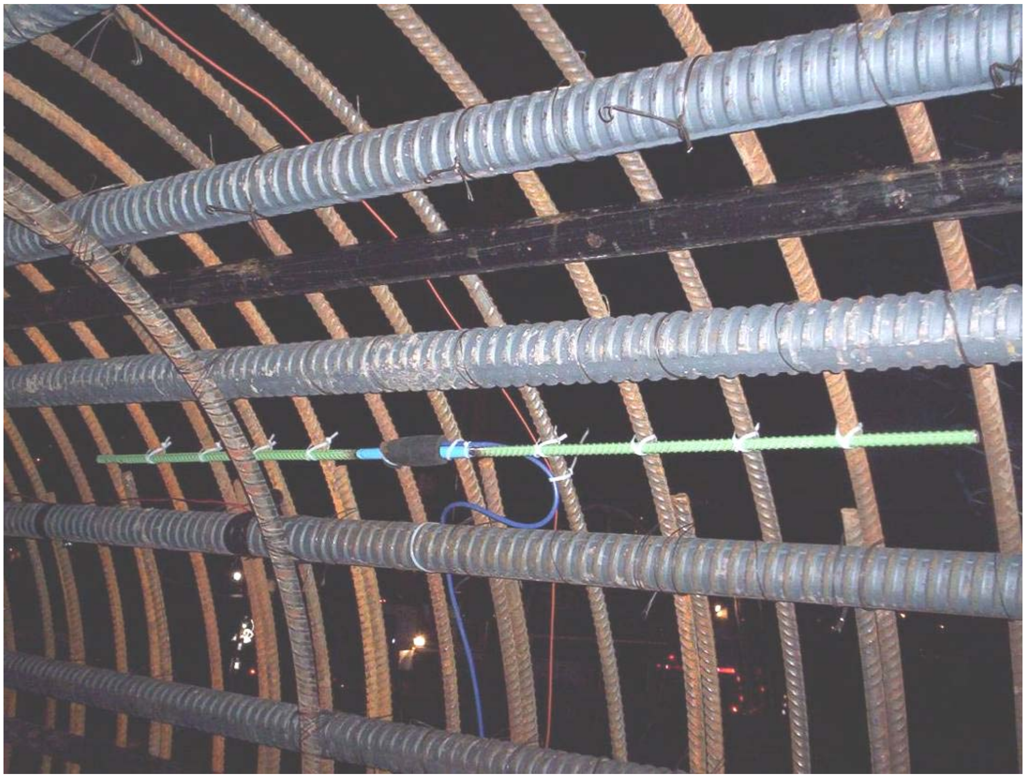

Figure 4-23: VW Gage Installed in Shaft Reinforcement Cage.

Figure 4-24: Coupled VW and RT Gages. (VW – Blue Cable, RT – Green Cable)

Figure 4-25: Reinforcement for 1st Column Pour for I-35W Bridge Columns.

Figure 4-26: Reinforcement at Mid-Section of Columns for I-35W Bridge.

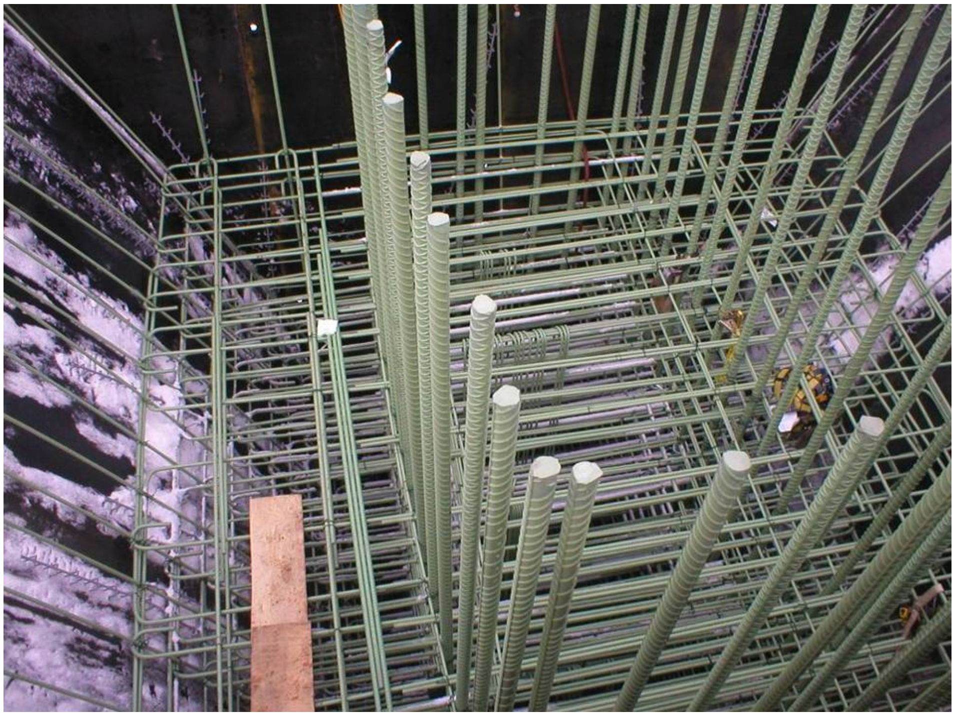

Figure 4-27: Longitudinal and Horizontal Column Reinforcement.

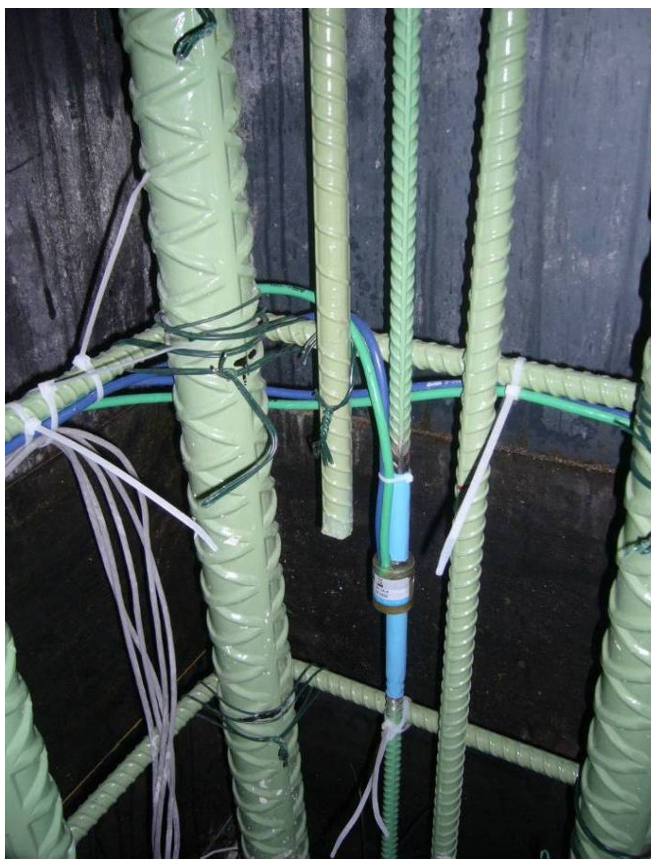

Figure 4-28: Coupled Gage Installed in Corner of Column.

Figure 4-29: Gage Wires Tied and Secured.

Figure 4-30: Gage Wires Exiting Through Conduit.

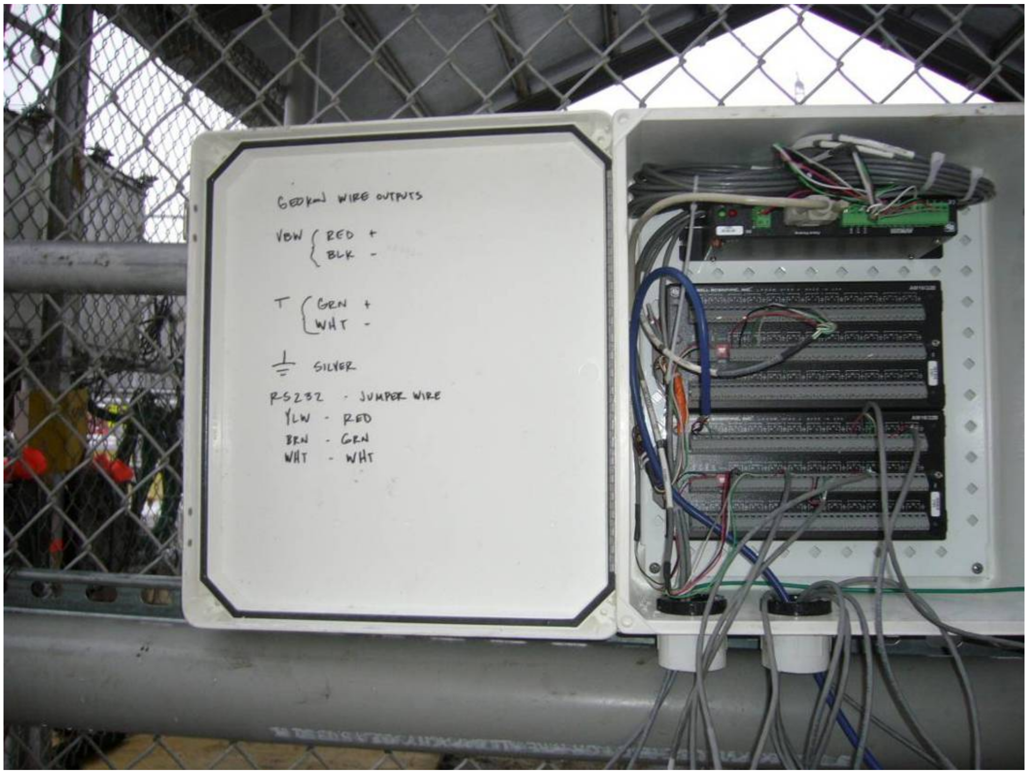

Figure 4-31: Gage Wires Connection to System 2.

Figure 4-32: Construction Load Monitoring Systems. (VW – Blue, RT – Green)

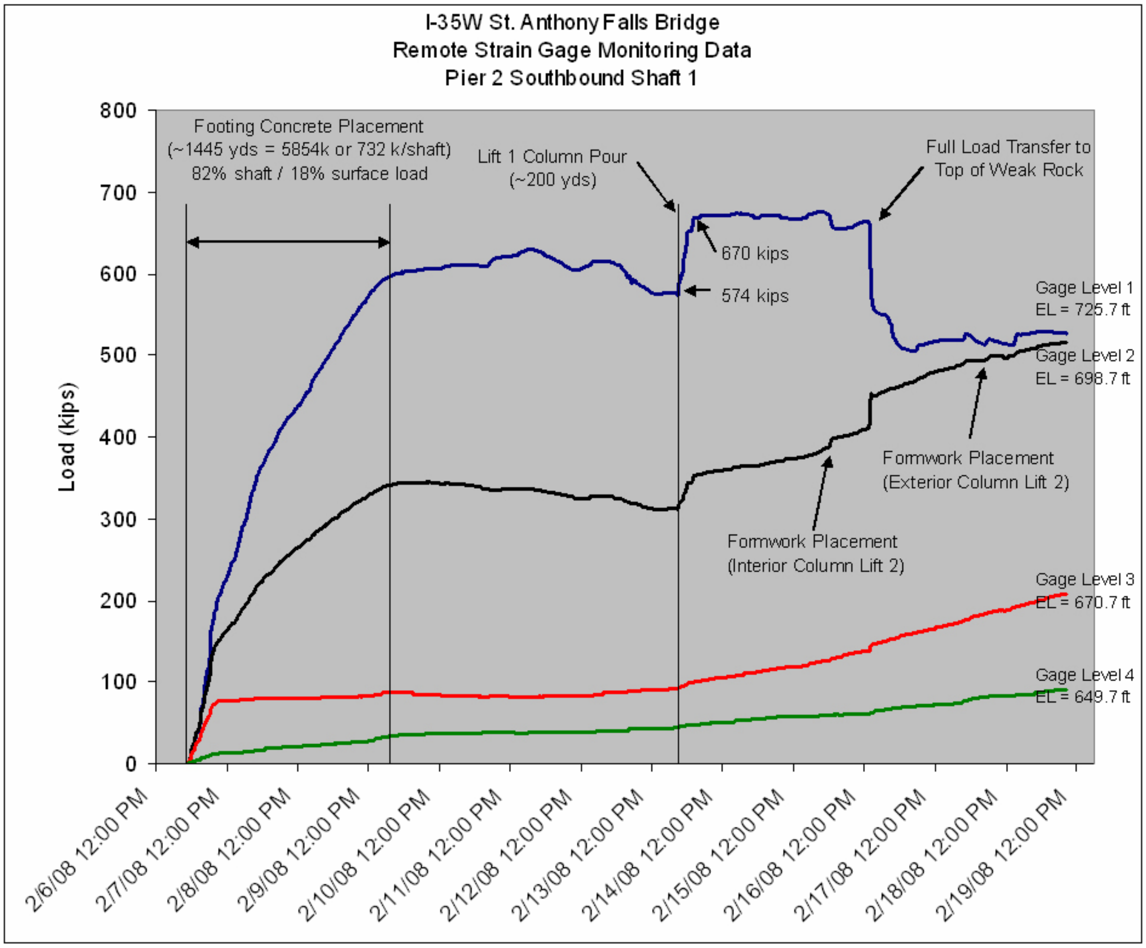

Figure 4-33: Annotated Graph of Shaft Construction Loads and Events.

Figure 4-34: I-35W Bridge Southbound Pier Footing Concrete Placement.

Figure 4-35: I-35W Bridge Lift 1 Column Concrete Placement.

Figure 4-36: I-35W Bridge Interior Column Lift 2 Formwork Placement.

Figure 4-37: I-35W Bridge Exterior Column Lift 2 Formwork Placement.

Figure 4-38: I-35W Bridge South Perspective CC640 Field Camera.

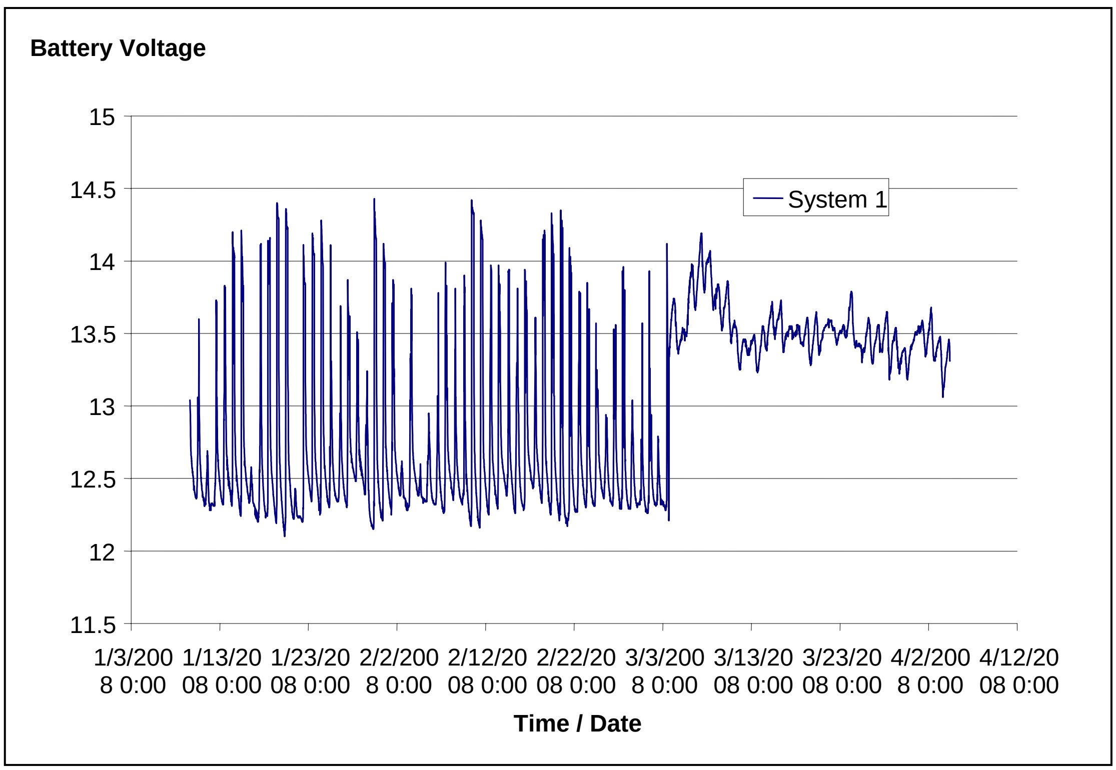

Figure 4-39: I-35W System 1 Battery Voltage over Time.

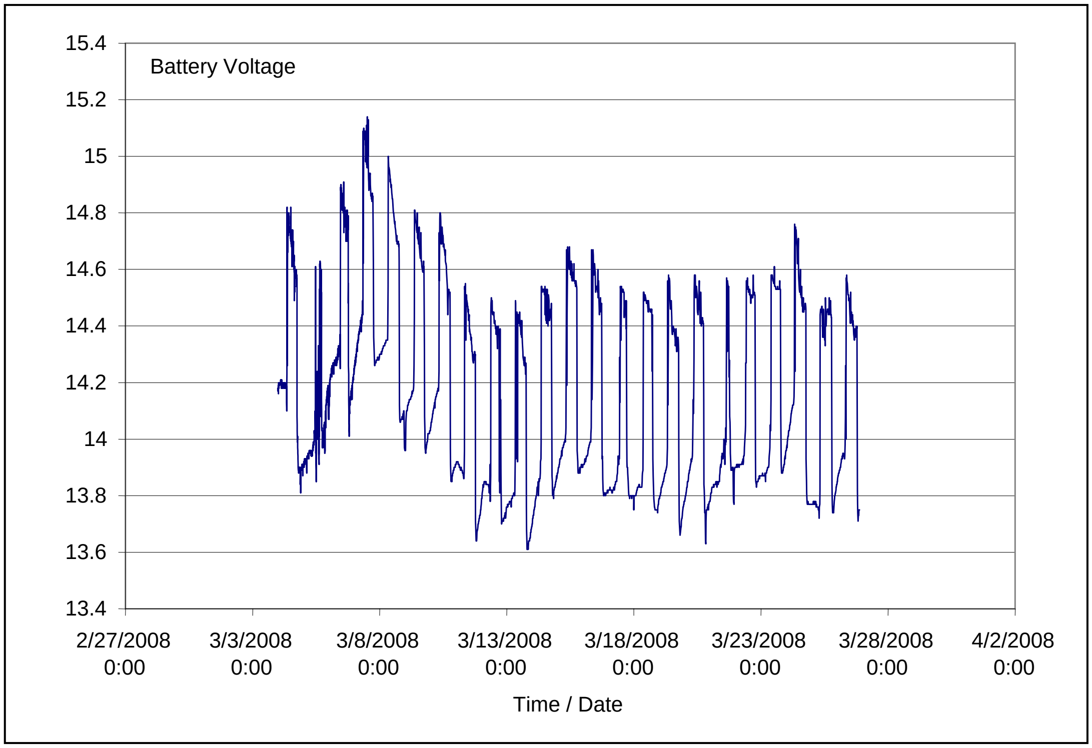

Figure 4-40: I-35W System 2 Battery Voltage over Time.

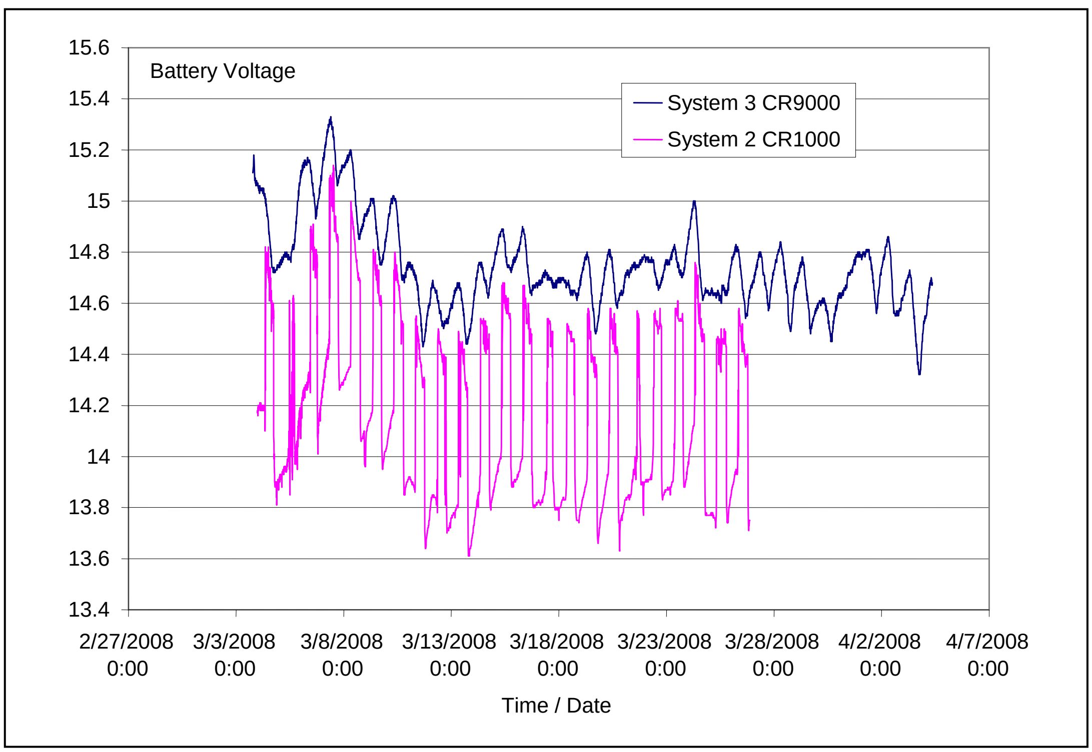

Figure 4-41: I-35W System 2 Battery Voltage Compared to System 3.

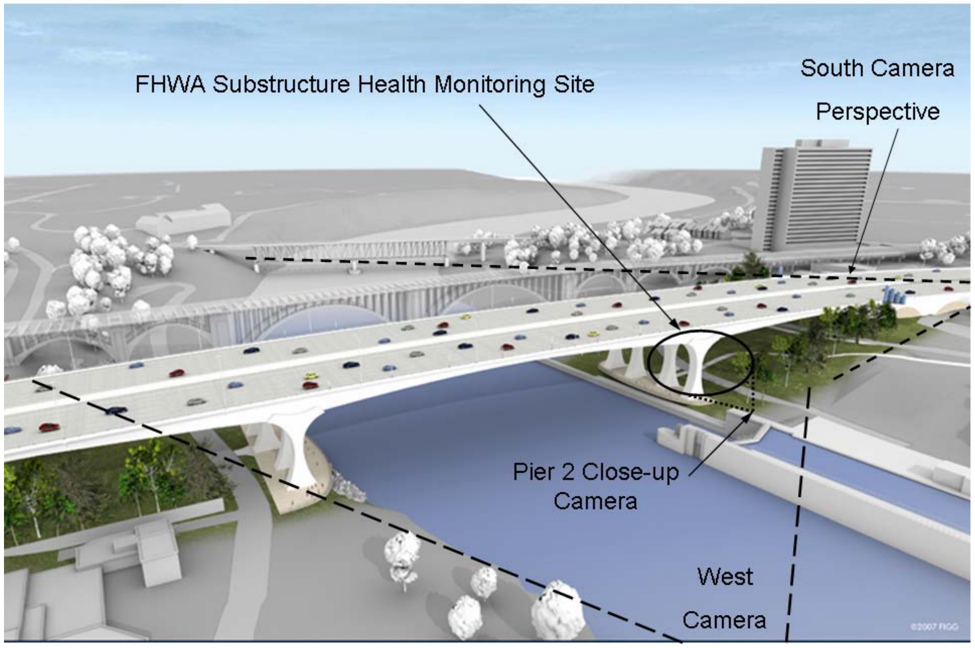

Figure 4-42: Main Page of St. Anthony Falls Bridge Study Website.

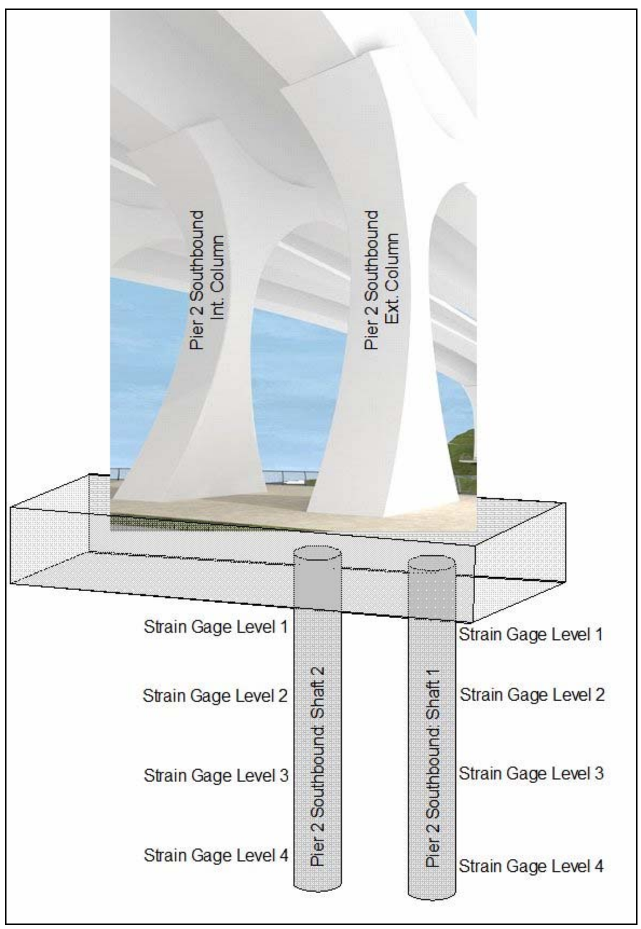

Figure 4-43: Secondary Page of St. Anthony Falls Bridge Study Website.

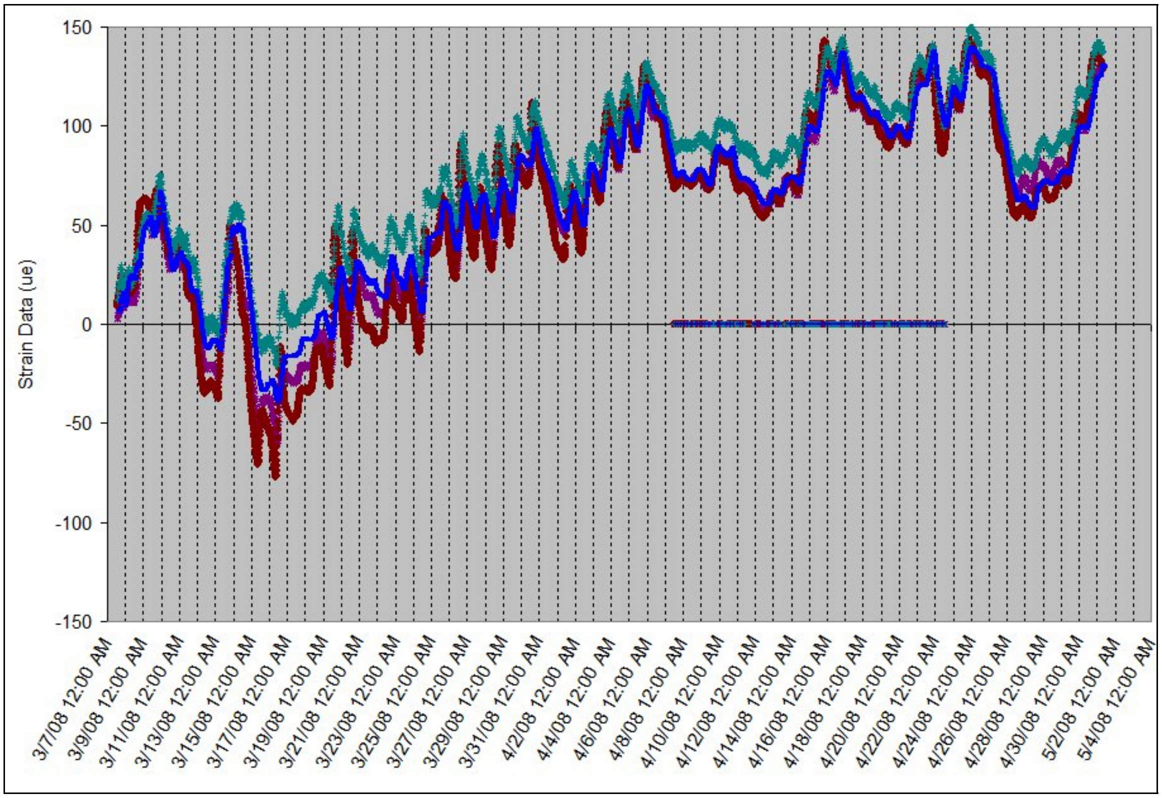

Figure 4-44: I-35W Bridge Pier 2 Interior Column Strain Data with Links.

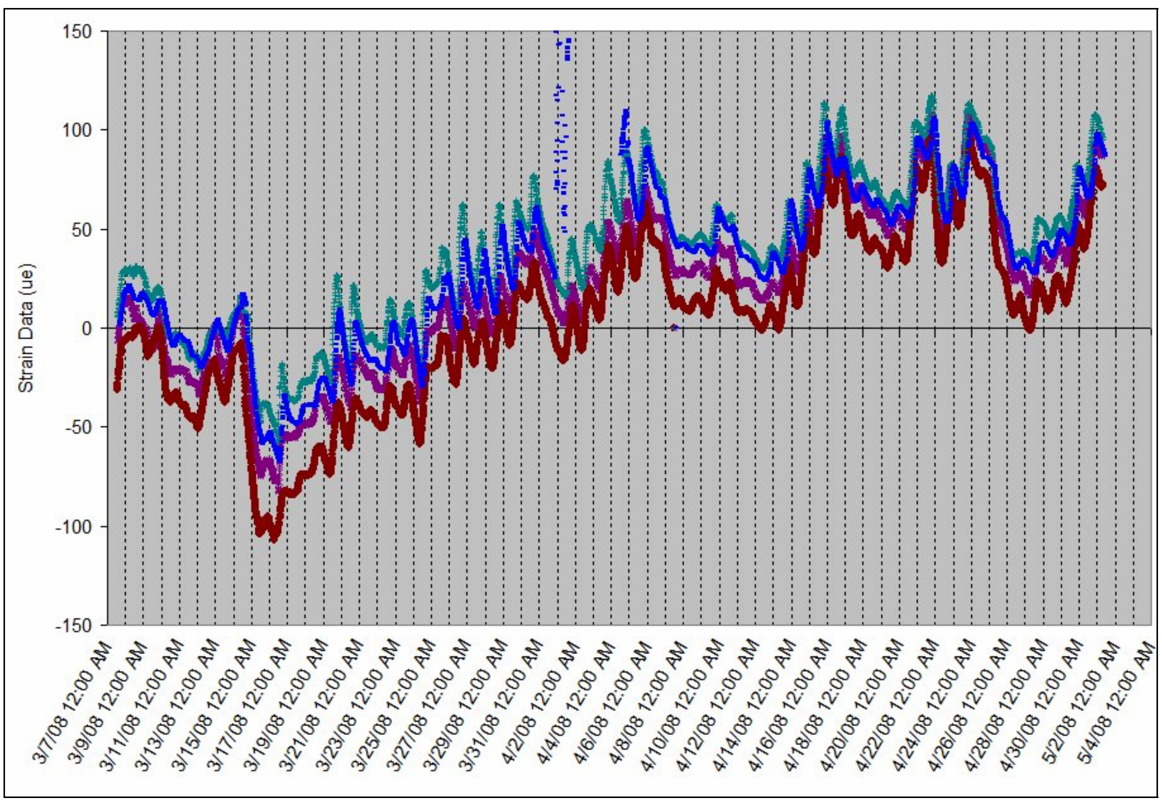

Figure 4-45: I-35W Bridge Pier 2 Exterior Column Strain Data with Links.

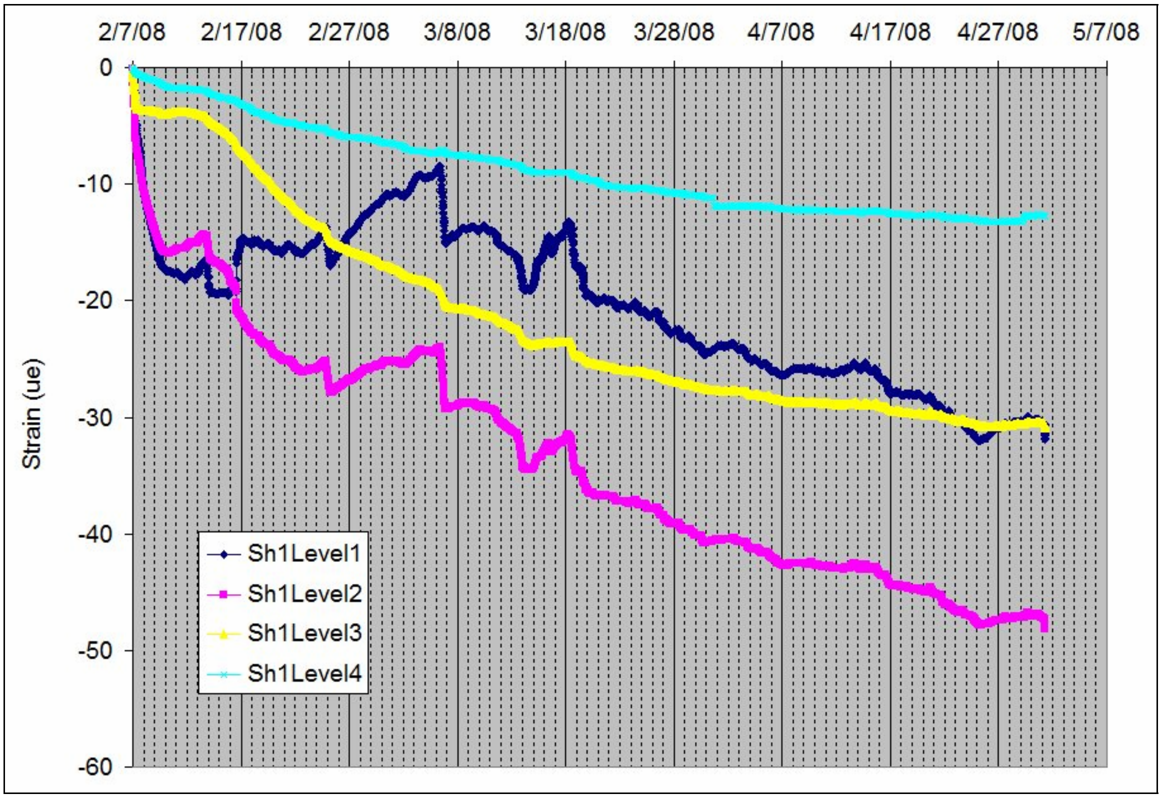

Figure 4-46: I-35W Bridge Pier 2 Shaft 1 All Levels Strain Data with Links.

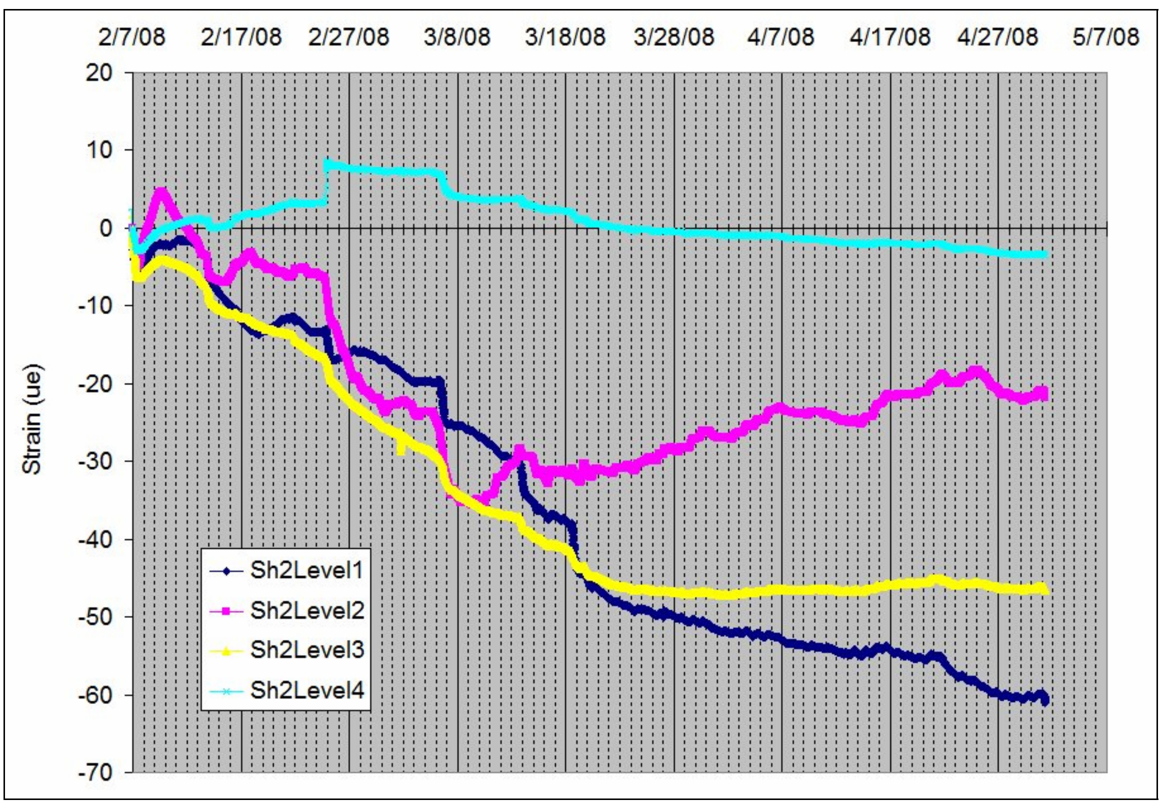

Figure 4-47: I-35W Bridge Pier 2 Shaft 2 All Levels Strain Data with Links.

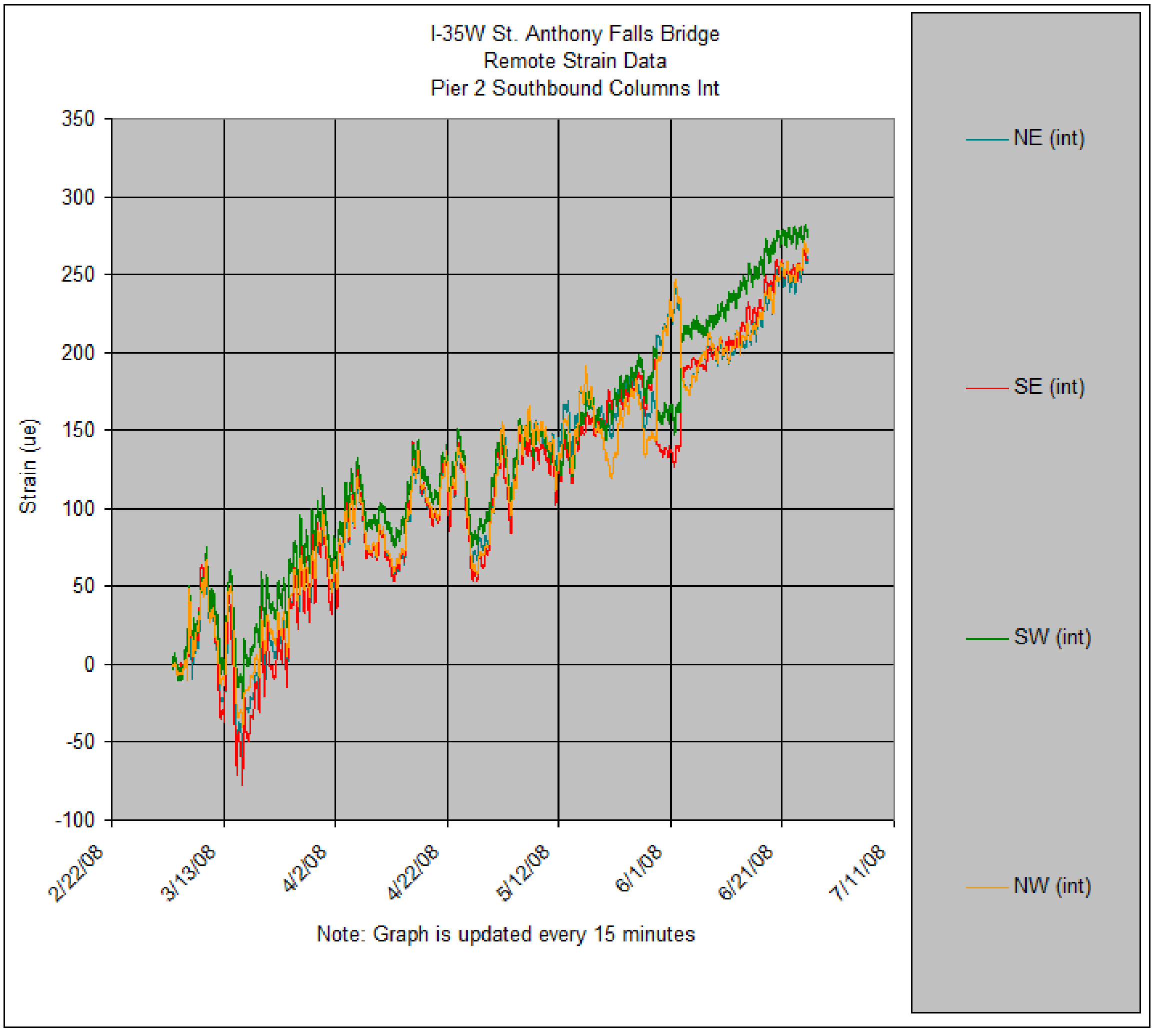

Figure 4-48: I-35W Bridge Pier 2 Interior Column Strain Data.

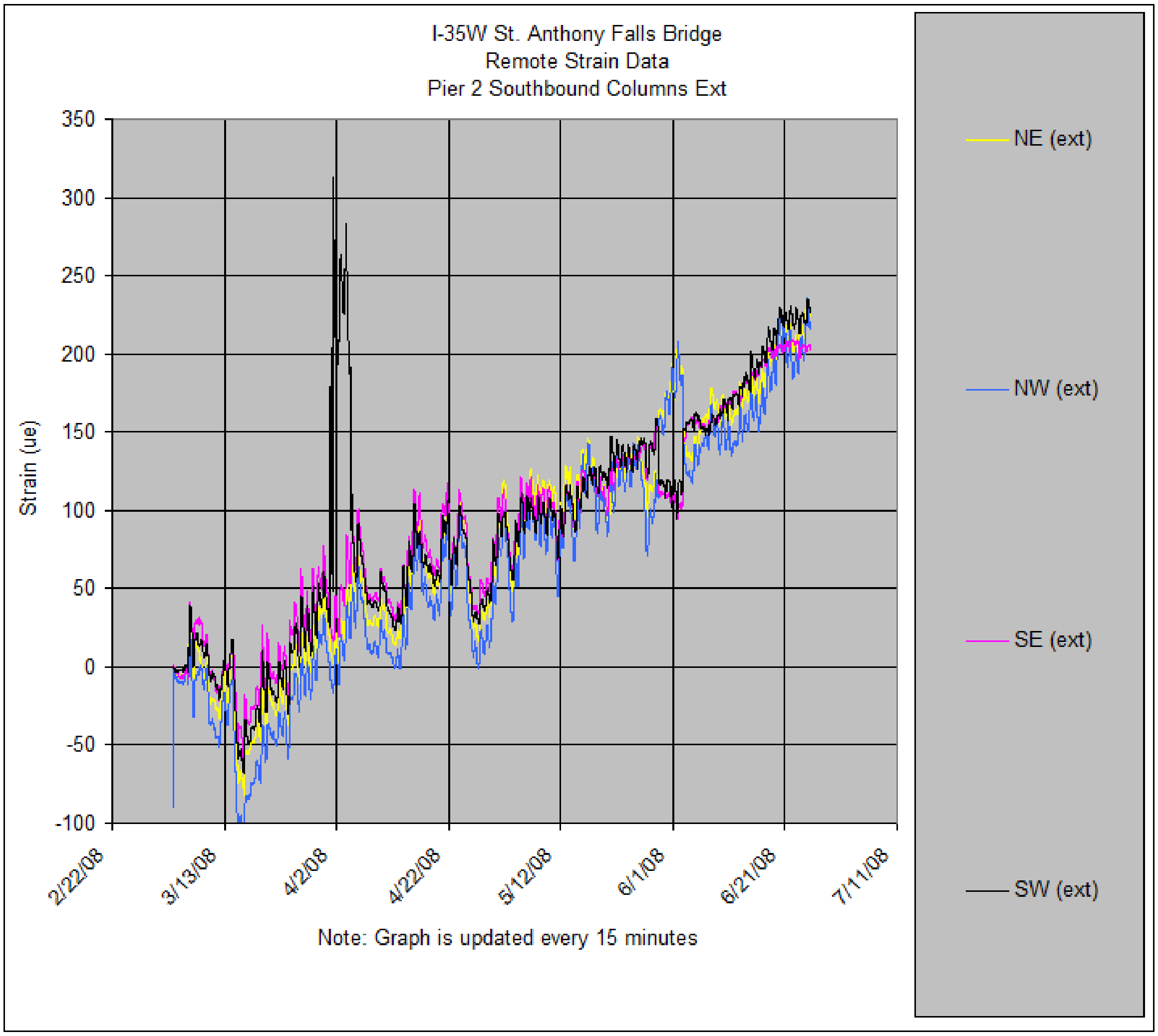

Figure 4-49: I-35W Bridge Pier 2 Exterior Column Strain Data.

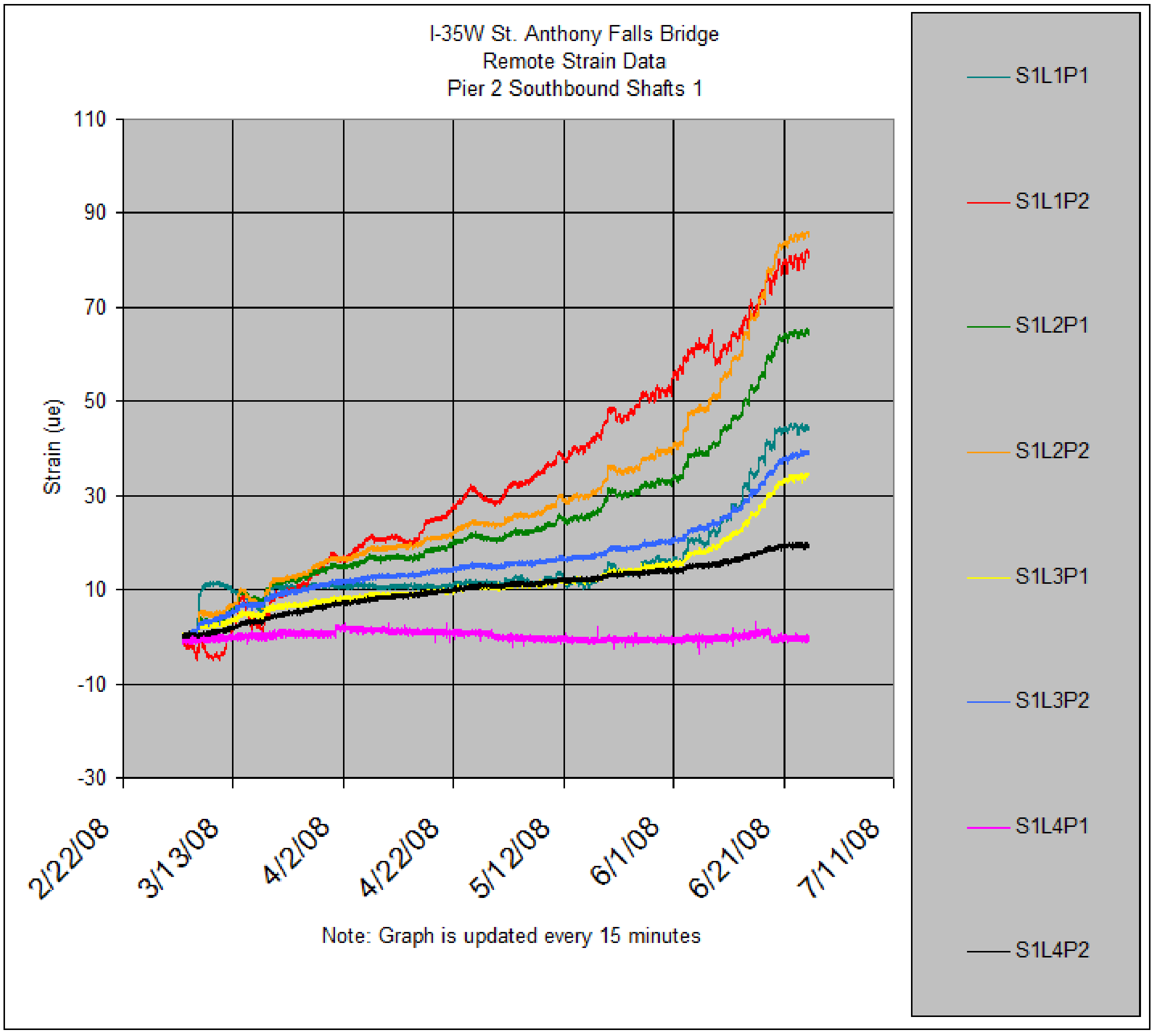

Figure 4-50: I-35W Bridge Pier 2 Shaft 1 All Levels Strain Data.

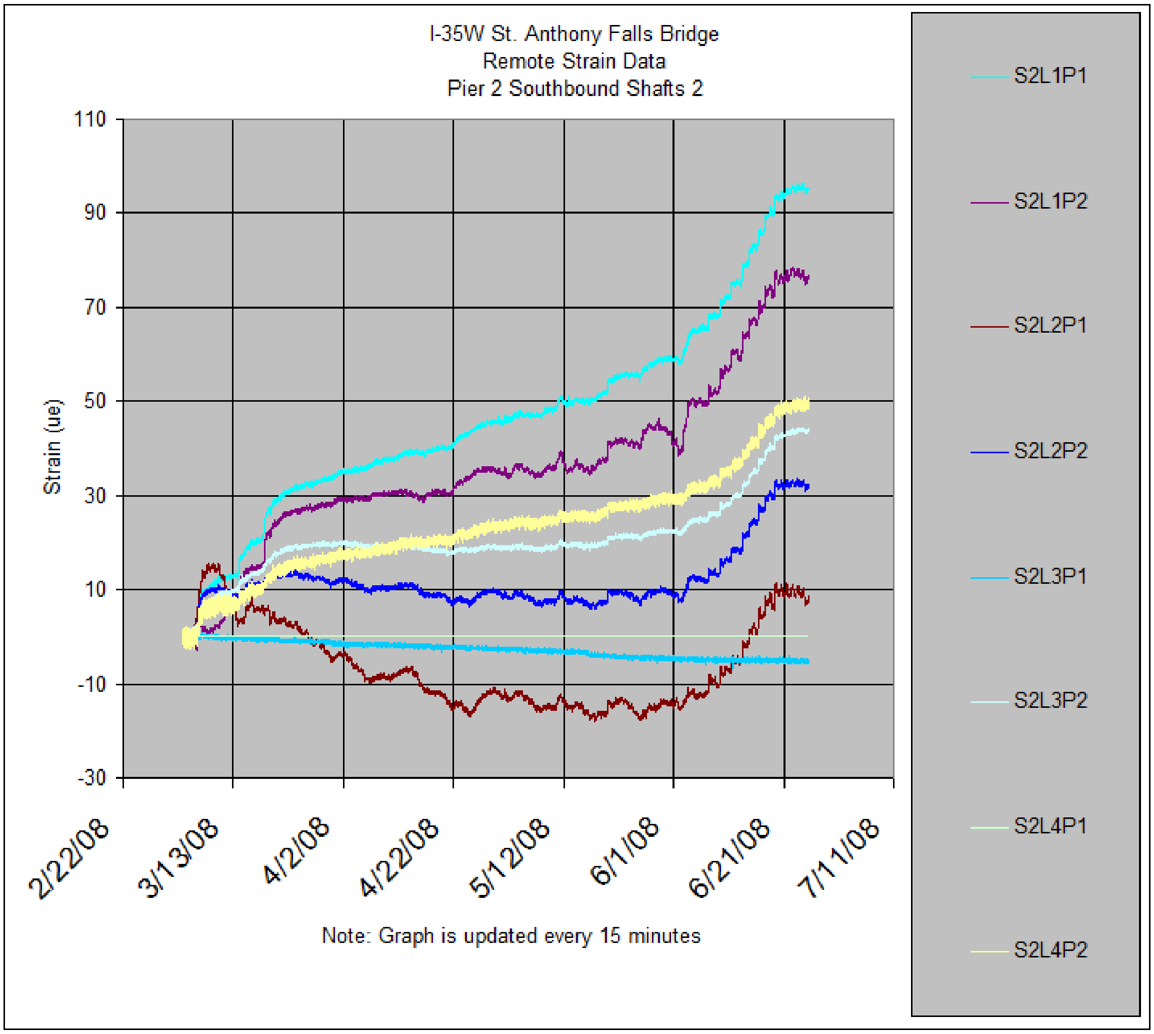

Figure 4-51: I-35W Bridge Pier 2 Shaft 2 All Levels Strain Data.

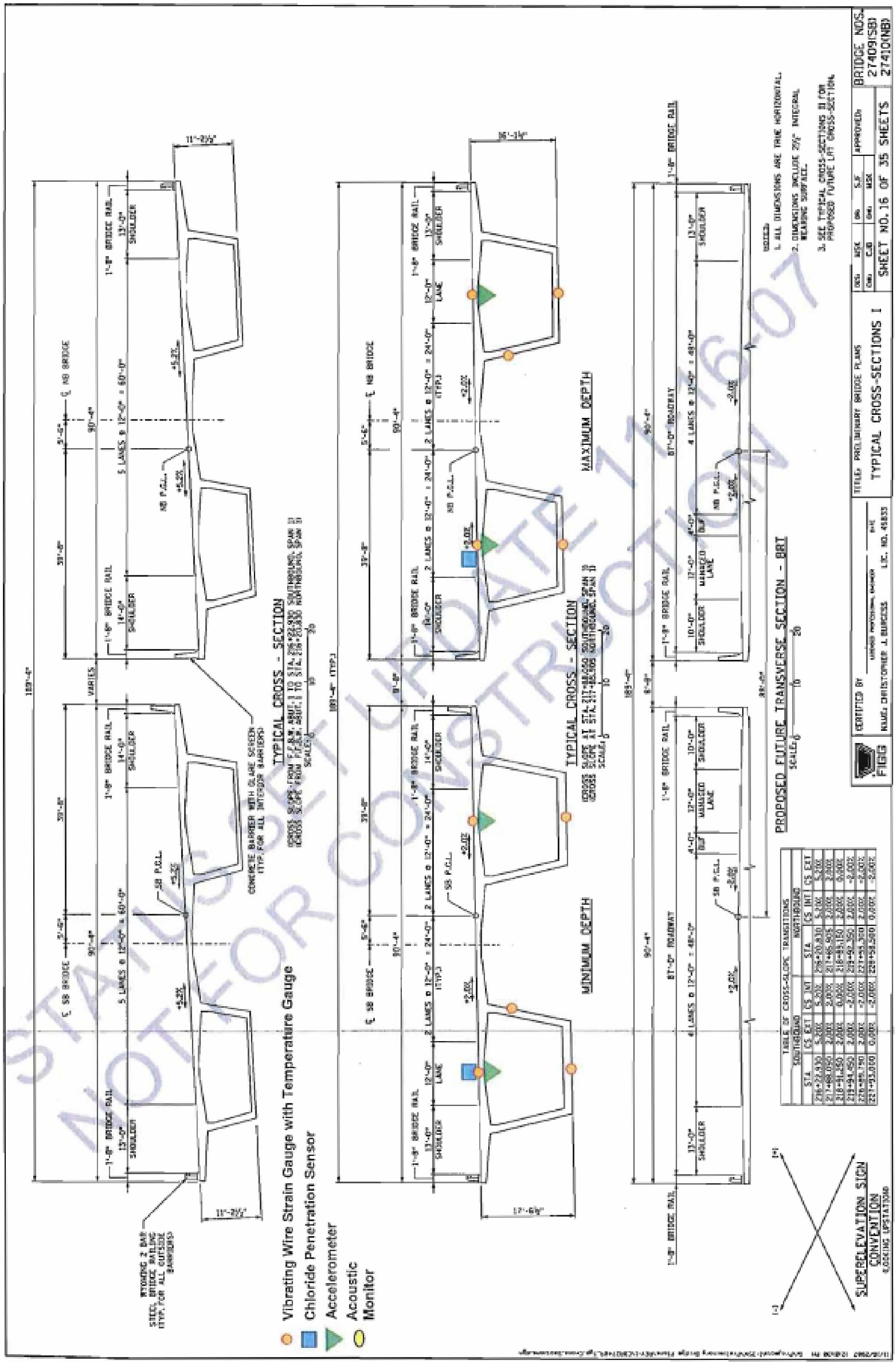

Figure 4-52: Superstructure Gage Locations I. (Courtesy of “Smart-Bridge” Concepts)

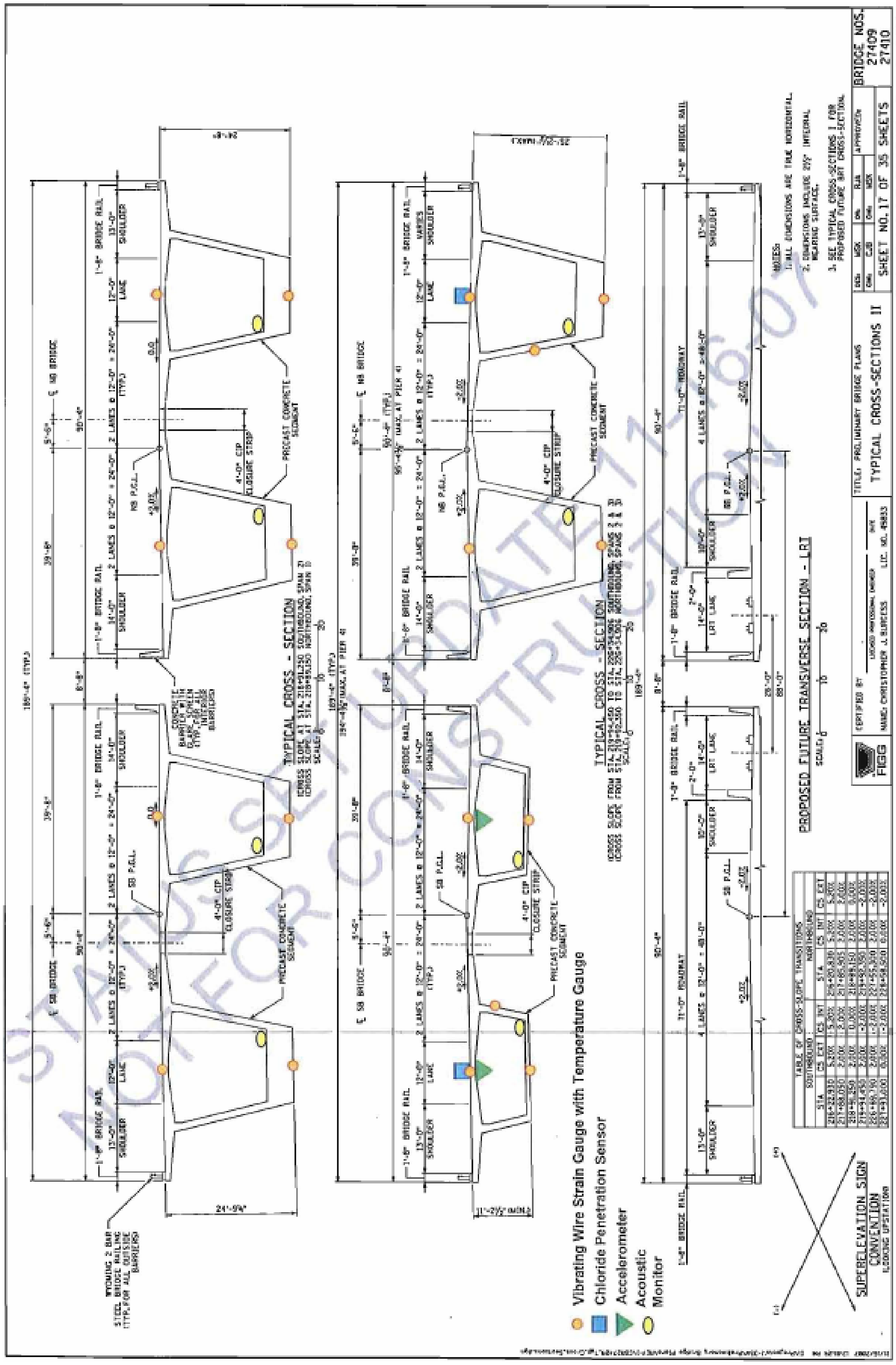

Figure 4-53: Superstructure Gage Locations II. (Courtesy of "Smart-Bridge" Concepts)

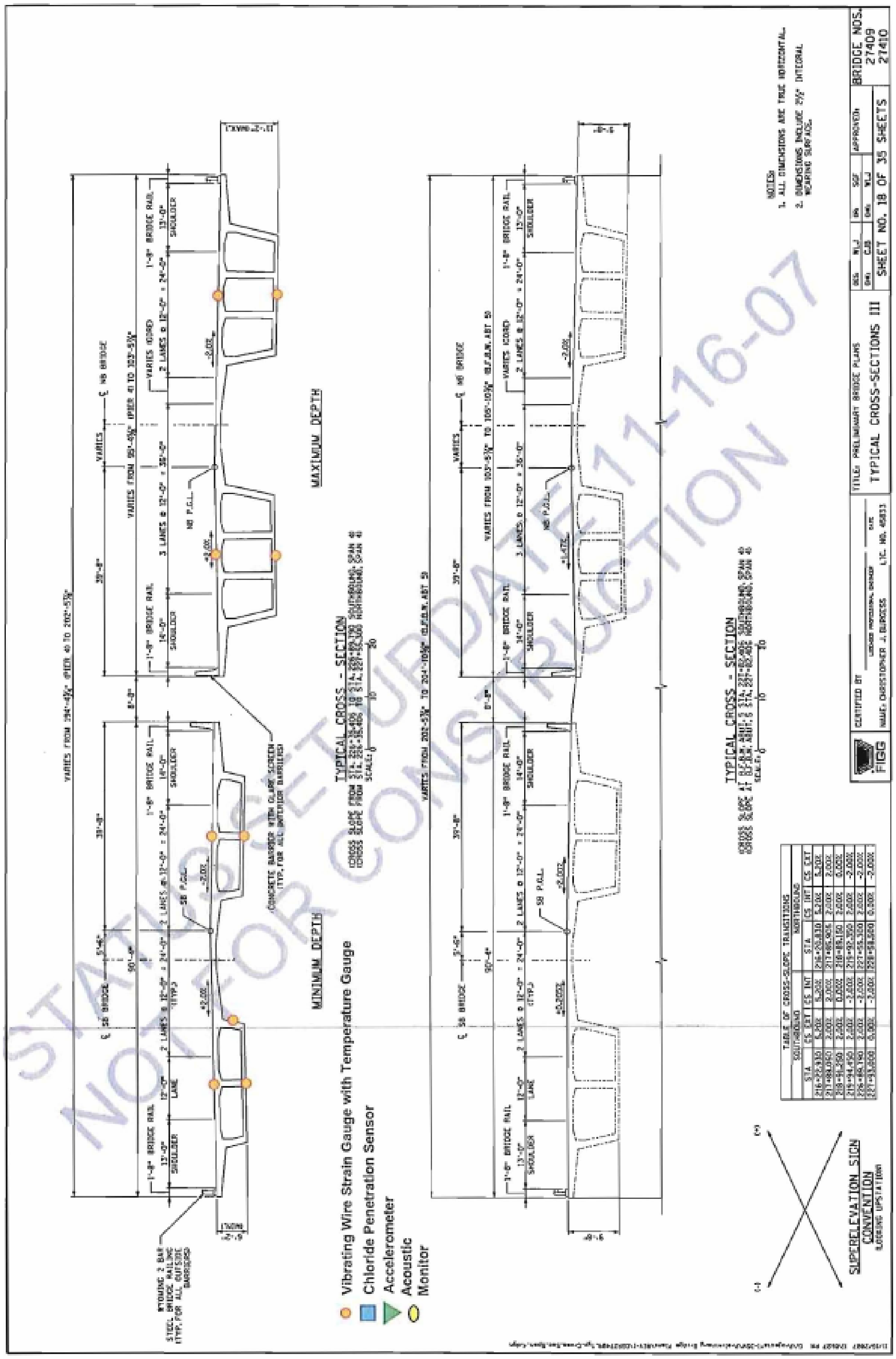

Figure 4-54: Superstructure Gage Locations III. (Courtesy of “Smart-Bridge” Concepts)

Chapter 5

Conclusions

The research in this thesis has shown that a reliable, remote Substructure Health Monitoring (SSHM) system can be implemented on any number of structures. The available options for data logging and strain gage installation, as well as the advancement of wireless technologies for cellular uplink and data transmission, have made remote monitoring systems a viable alternative to the typical monitoring system that is generally used on most structures.

Generally speaking, there were no great problems encountered throughout both the Voided Shaft study and the I-35W Bridge study. There is one important aspect in the implementation of a remote monitoring system for any project: Power. How to get power, how to maintain power, and how to verify that if power is lost, back-up power will be accessible and is ready to take over for the system. Without this verification, the monitoring system becomes fragile and cannot be relied upon.

5.1 Conclusions from Tested Systems

From the Voided Shaft study, it was learned that it is necessary to determine, prior to system installation and start-up, how much power all of the instruments in the monitoring system will require. It was learned early on that the Raven 100 CDMA

Airlink Modem required an extremely large amount of power. However if it is allowed to fall asleep and only awake when transmitting data, that power consumption is greatly reduced. Second, the back-up power source in the form of a solar cell panel, while capable of providing some back-up power, was not strong enough to combat the power consumption of the modem.

From the I-35W Bridge study, it was learned that the power source, even if provided on site, may not be sufficient to run the system. It was learned that the PS100 Power Supply could only receive power from either an A/C power source or a solar cell panel, but not both. This means that this limitation had to be circumvented with extra equipment (and therefore extra money) in order to provide for a fully remote system.

5.2 Future Work for I-35W Bridge Study

At the time of this report, the University of South Florida Geotechnical Research Department has been granted a 2 year extension on its involvement in the I-35W project. In the short term, this will include the installation and wiring of the gages that will be installed in the bridge superstructure as well as the full wiring of the installed substructure gages to the permanent data acquisition system (DAS). This system is planned to be located at a monitoring site that is located on the north bank of the Mississippi river that is approximately1500 feet away from the temporary DAS at the base of the southbound pier as described in this report. This will be the instrumentation of Phase III of the study (Figure 4-2).

In the long term, all of the substructure and superstructure gages will be monitored for the proposed extension of 2 years. The monitoring of these gages will include the general data collection and data analysis that has been described in this report. In addition, it will involve the process of making the analyzed data available to select parties by means of the USF Geotechnical Research website (http://geotech.eng.usf.edu). This will include the strain and load data from the instrumented shafts, columns, and superstructure as well as the other data collected from the superstructure gages. It will also include up-to-date images from the bridge site as well as a reference that will relate strain and load events to real-time traffic and bridge loading events.

5.3 Possibilities for Remote Substructural Health Monitoring Systems

Obviously, while the study of remote data collection and analysis is a complete study in itself, there are no limits to where this type of project can lead. The data provided by this instrumentation can be used by governmental institutions or by engineering societies for furthering the profession.

For agencies such as state departments of transportation (DOTs) and the Federal Highway Administration (FHWA) the data will most likely be used as an early warning or protection system. Threshold limits can be placed on the data collection or monitoring system and can be used to alert the user when certain limits are met, such as a percentage of capacity or an extreme event. For engineering societies, this information could be used for increased information for future analysis. This can include a back-check of future bridge designs to determine the design assumptions of the engineers of record. It could also be used as a verification of the statistical loads that are used in bridge design, such as the HL93 Truck. This is a specified loading that is denoted by the American Association of State Highway and Transportation Officials (AASHTO) and is not actually a truck that exists. From 2 years worth of data collected from a bridge, this loading can be verified to be a worst case scenario that is experienced by an actual structure. Finally, the analyzed data could be used to propose a possible increase in the specified resistance factors that are used in bridge design. The more that is learned about what forces an actual bridge experiences, the more streamlined the design approach can become. This can result in a decrease in the amount of materials used and therefore a more cost effective and efficient design.

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