1. Report No. FHWA/LA.03/378	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle **Monitoring of the Bonnet Carré Spillway Bridge During Extreme Overload**	5. Report Date **October 2003** 6. Performing Organization Code	
7. Author(s) **Dr. Paul Ziehl and Dr. Anthony Lamana**	8. Performing Organization Report No.	
9. Performing Organization Name and Address **Department of Civil and Environmental Engineering** **Tulane University** **New Orleans, Louisiana 70118**	10. Work Unit No. 11. Contract or Grant No. **State Project Number: 736-99-1146** **LTRC Project Number: 03-6ST**	
12. Sponsoring Agency Name and Address **Louisiana Transportation Research Center** **4101 Gourrier Avenue** **Baton Rouge, LA 70809**	13. Type of Report and Period Covered **Final Report** PERIOD COVERED: NOVEMBER 2002 14. Sponsoring Agency Code	
15. Supplementary Notes		

16. Abstract

This report concerns the monitoring of a portion of the Bonnet Carré Spillway Bridge during an extreme overload. On Tuesday, November 5, 2002, Tulane University was requested to monitor strain at the bottom flange of two adjacent prestressed girders before, during, and after the passing of the overload. On Thursday, November 7, this request was modified to include monitoring of strain in the following regions of interest:

a. Positive moment region on three adjacent girders
b. Negative moment region over the pile cap at the center of the bridge width
c. Positive and negative moment region on one pile cap
d. Shear region on one pile cap
e. Compression region on one square pile

It was originally believed that the overload would cross the bridge at approximately 4:00 a.m. on Saturday morning, November 9. Access in the form of a "Reach-All" vehicle was provided on Friday, November 8, and strain gauges and wires were affixed to appropriate locations of the structure. Due to mechanical difficulties the overload did not pass until approximately 3:00 pm on Sunday, November 10. On Saturday, November 9, access was also provided to the structure, and instrumentation was placed. During the day of Friday, November 8, an additional request was made to monitor the strain on a reinforcing bar in the negative moment region of the concrete deck over a pile cap. Furthermore a request was made to monitor the displacements at midspan of the girders on Saturday, November 9. These strains and displacements were monitored during the passage of the overload. Acoustic emission was monitored on three girders at midspan during the passage of the overload as well. When the overload crossed the bridge on Sunday, November 10, many of the strain gauges were still functional but a few had been lost due to environmental exposure or other factors. Due to the very short notice prior to the monitoring, some compromises were made in the data collection. These included, but were not limited to, the use of a quick curing epoxy for the mounting of the strain gauges, the use of long runs of 18-gauge shielded three-conductor wire from the strain gauges to the data acquisition system, the elimination of weatherproofing in some instances, and the use of portable generators to power the data acquisition system. After the monitoring was performed, efforts were made to quantify and reduce the error that may have arisen from these necessary compromises. These efforts included additional laboratory testing described in this report.

17. Key Words **acoustic emission, strain gauges, instrumentation, data collection, visual inspection, extreme overload.**	18. Distribution Statement **Unrestricted. This document is available through the National Technical Information Service, Springfield, VA 21161.**

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
N/A	N/A	65	

Monitoring of the Bonnet Carré Spillway Bridge During Extreme Overload

by
Paul H. Ziehl, Ph.D., P.E.
Anthony J. Lamanna, Ph.D., P.E.

Dept. of Civil and Environmental Engineering
Tulane University
New Orleans, Louisiana 70118

LTRC Project No. 03-6ST
State Project No. 736-99-1146

conducted for

Louisiana Department of Transportation and Development
Louisiana Transportation Research Center

October 2003

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When the overload crossed the bridge on Sunday, November 10, many of the strain gauges were still functional but a few had been lost due to environmental exposure or other factors. Due to the very short notice prior to the monitoring, some compromises were made in the data collection. These included, but were not limited to, the use of a quick curing epoxy for the mounting of the strain gauges, the use of long runs of 18-gauge shielded three-conductor wire from the strain gauges to the data acquisition system, the elimination of weatherproofing in some instances, and the use of portable generators to power the data acquisition system.

After the monitoring was performed, efforts were made to quantify and reduce the error that may have arisen from these necessary compromises. These efforts included additional laboratory testing described in this report.

ACKNOWLEDGMENTS

We wish to express our sincere appreciation to Paul Fossier of the Louisiana DOTD for meeting us at the site prior to the instrumentation effort and identifying key locations of the structure to be monitored. We also acknowledge the efforts of Shyam Shah for his role in coordinating the instrumentation effort and Dr. Robert Bruce for his coordination efforts throughout the course of this short project. We also thank Walid Alaywan for his efforts and additional photographic information and Harold “Skip” Paul for his input and support.

IMPLEMENTATION STATEMENT

The results of this investigation would be useful for implementation in the event that analysis of the structural system were undertaken. In that event, the live load distribution during the passage of the overload and the degree of fixity in the negative moment region over the bents could be determined and used to optimize future designs or to provide a better understanding of how to rate bridges for extreme overloads. In its current form, the investigation is primarily useful as a description of the instrumentation effort and the results and limitations of that effort. With interpretation from the reader, it may also be used as a qualitative measure of the extent of damage that did or did not occur due to the passage of this particular overload.

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INTRODUCTION

Extreme overloads are a common occurrence in Louisiana due to the prevalence of the petroleum processing industry and the relative ease of water-born deliveries. Some question exists, however, with regard to the damage that may be caused to roads and bridges by these extreme overloads. As a result, concerns exist with regard to basic structural behavior such as live load distribution and restraint offered by the current method of detailing joints between adjacent spans.

To determine the extent of damage that may be caused by such loadings, a portion of the Bonnet Carré Spillway bridge was monitored prior to, during, and after the passing of such an overload. The Bonnet Carré Spillway bridge was subjected to a similar overload approximately two weeks previous to the passage of the overload described herein. The effects on the bridge resulting from overload were not monitored and are unknown. An elevation and plan view of the overload that was not monitored are shown in figure 1. The subject of this report is the passage of a second overload. This second overload was significantly heavier than the first overload, however, in this overload, the load was better distributed. An elevation and plan view of the second overload are shown in figure 2. Photographs of the second overload are shown in figures 3 and 4.

The primary data measured during the passage of the second overload was strain on the surface of various portions of the structure. Secondary data measured were maximum deflections and acoustic emission. Limited visual inspection was also performed prior to the passage of the overload. Due to the short notice given prior to the event and limited access, necessary compromises were made in the data acquisition; these compromises are described in the report. A photograph of the access to the underside of the bridge is shown in figure 5.

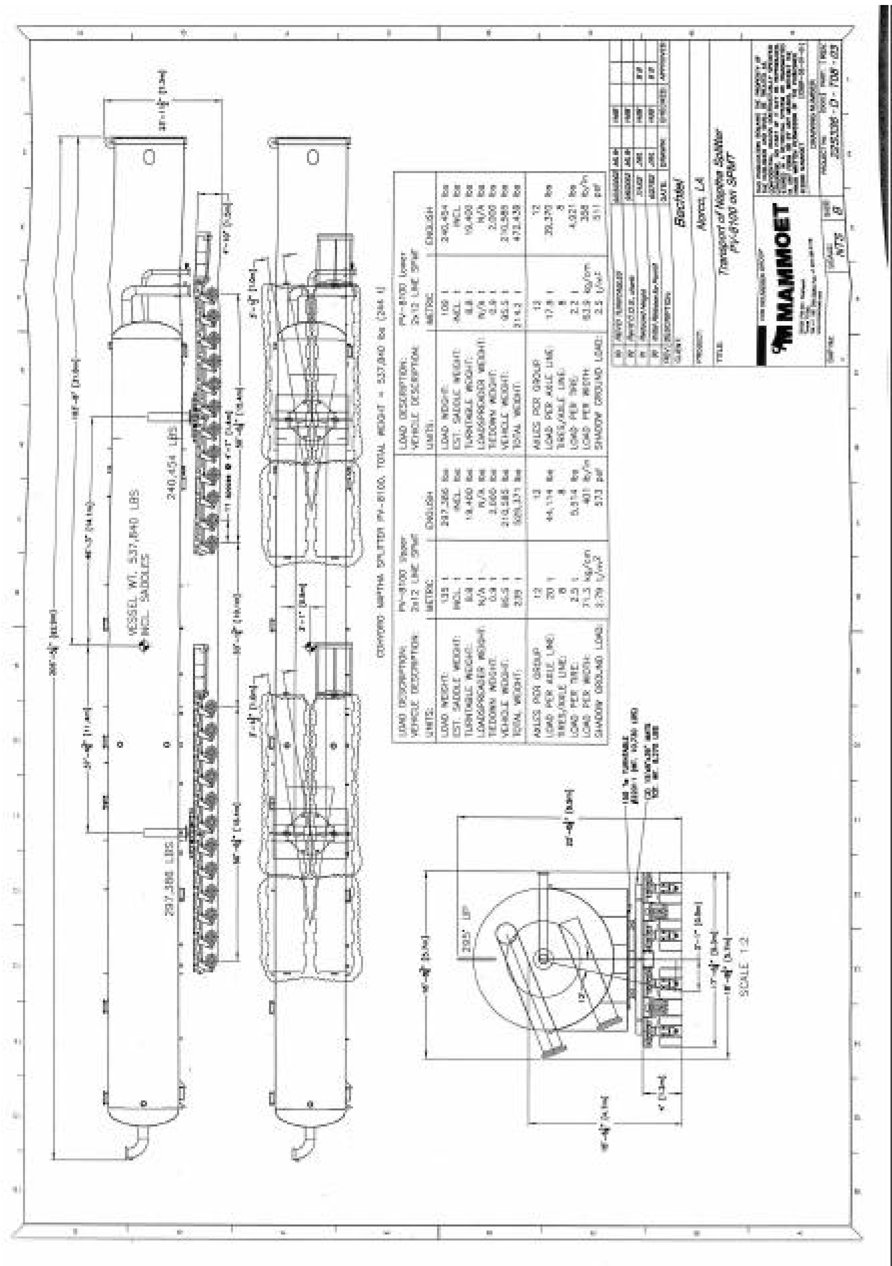


Figure 1
Elevation and plan view of overload no. 1

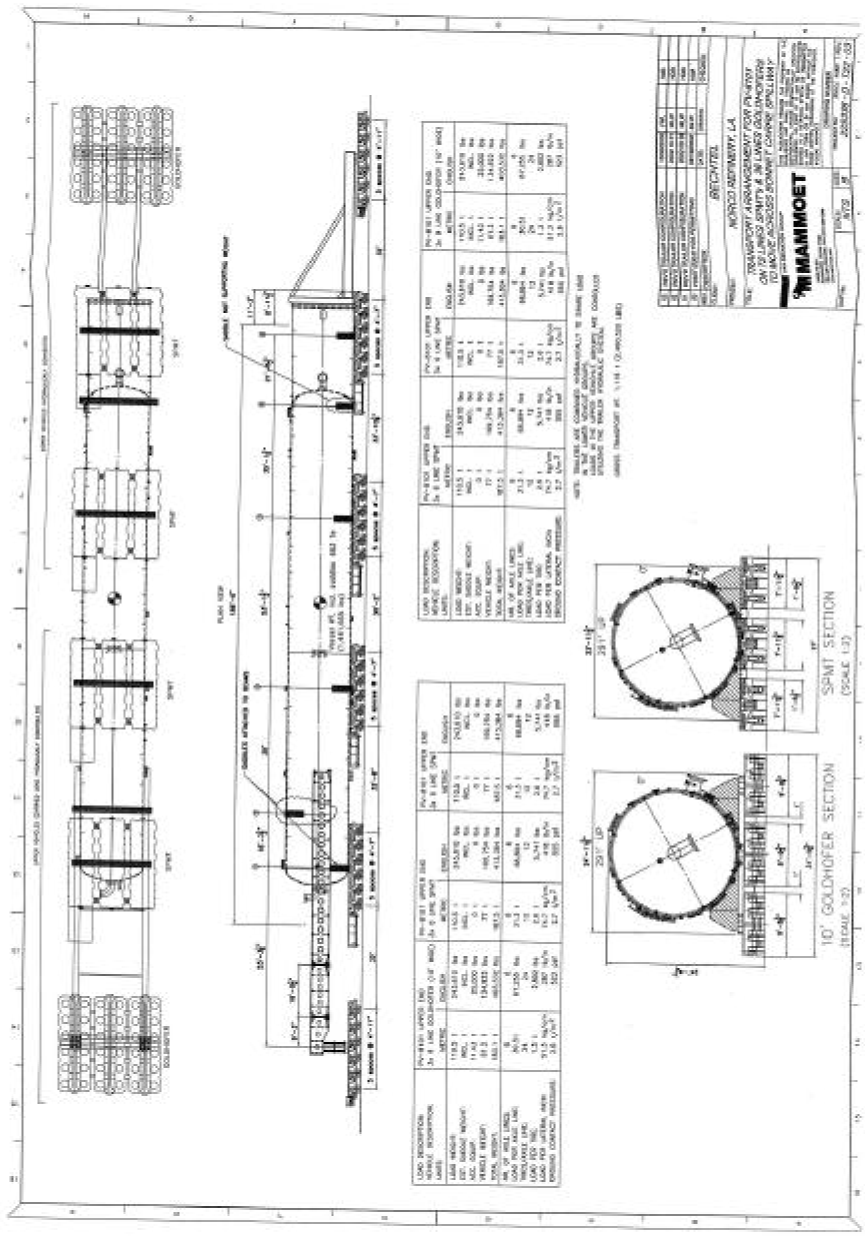


Figure 2
Elevation and plan view of overload no.2

Figure 3
Photograph of overload no.2 , side view

Figure 4
Photograph of overload no. 2, front view

Figure 5
Photograph of access to underside of bridge

OBJECTIVE

The primary objective of this report is to present the results of the field monitoring that was performed and to provide a description of the data acquisition methods. This report also aims to provide a description of the limitations of such methods and recommendations for future research and evaluation in regard to this project and future field monitoring projects in general.

SCOPE

The scope of this report is limited to the presentation of the data that was obtained before, during, and after, the extreme overload. Interpretation of the results obtained is beyond the scope of this report, but some general discussion is included.

METHODOLOGY

The monitoring of the Bonnet Carré Spillway Bridge is addressed in four sections. These sections include visual inspection, monitoring of strains, displacements, and acoustic emission.

The visual instrumentation effort was limited to bents 1 and 2 and spans 1 and 2 as shown in figure 6. The locations of strain gauges, deflectometers, and acoustic emission sensors are shown in figures 7 through 11. The strain gauge locations were based primarily on conversations with Paul Fossier and Shyam Shah of the Louisiana DOTD. In some cases, it was not possible to locate strain gauges in the requested locations due to time constraints, inadequate length of wire, and limited access. The general intent of the strain gauge placement was achieved in most cases.

Visual Inspection

A limited visual inspection was performed at the site prior to the passage of the overload. This inspection focused primarily on the possibility of transverse cracking in the reinforced concrete deck due to the negative moment over the bents. This visual inspection was limited to spans 1 through 6 (see figure 6). Limited visual inspection of the underside of the bridge was also conducted during the instrumentation process.

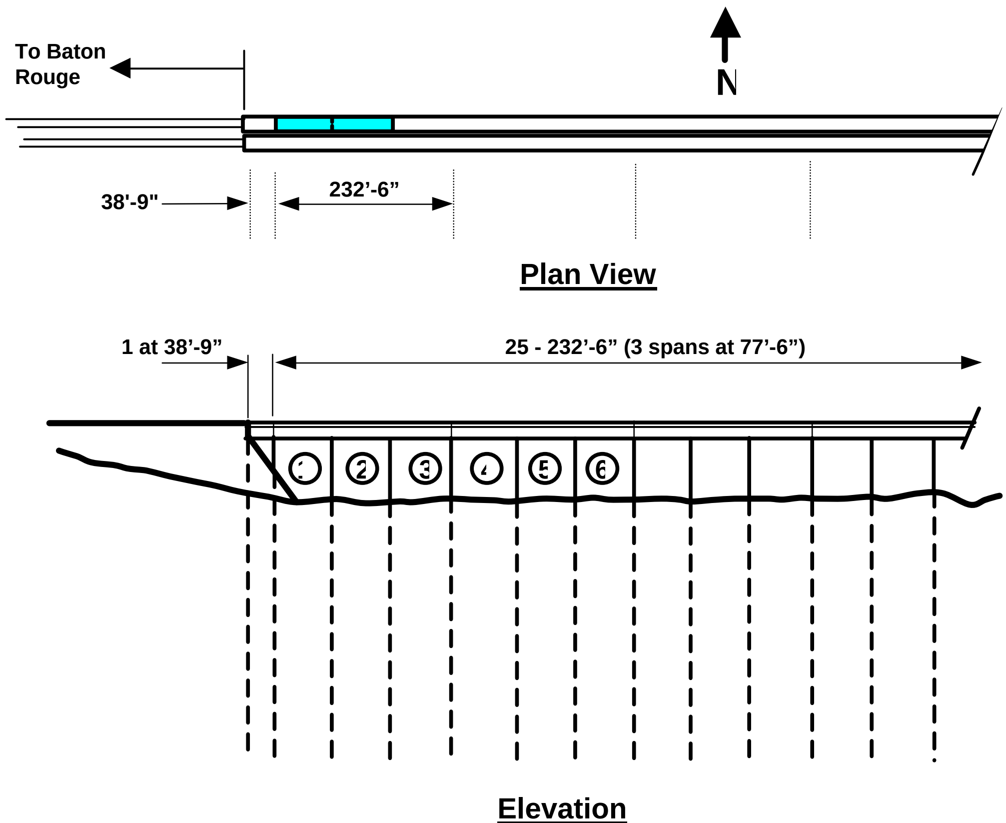


Figure 6
Plan and elevation view of instrumented spans

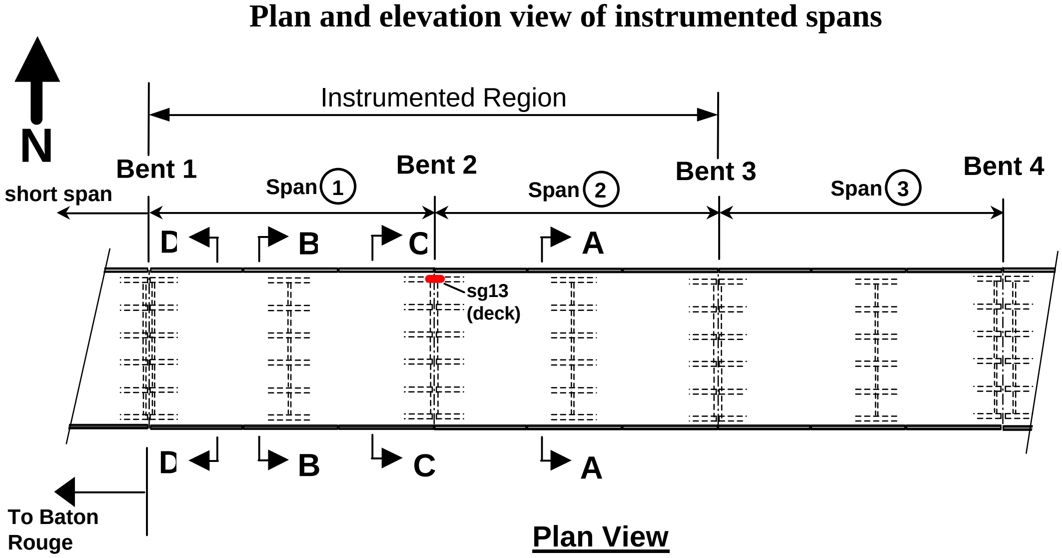
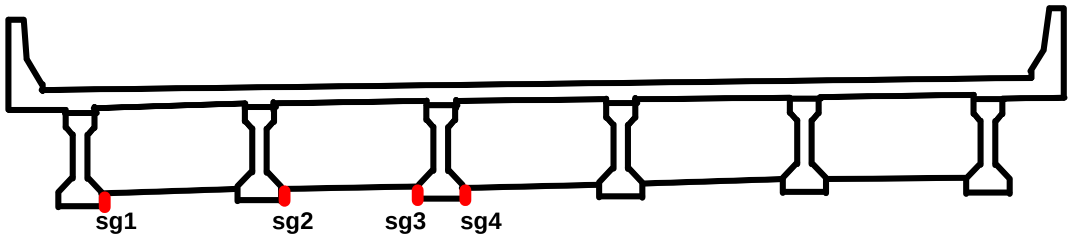
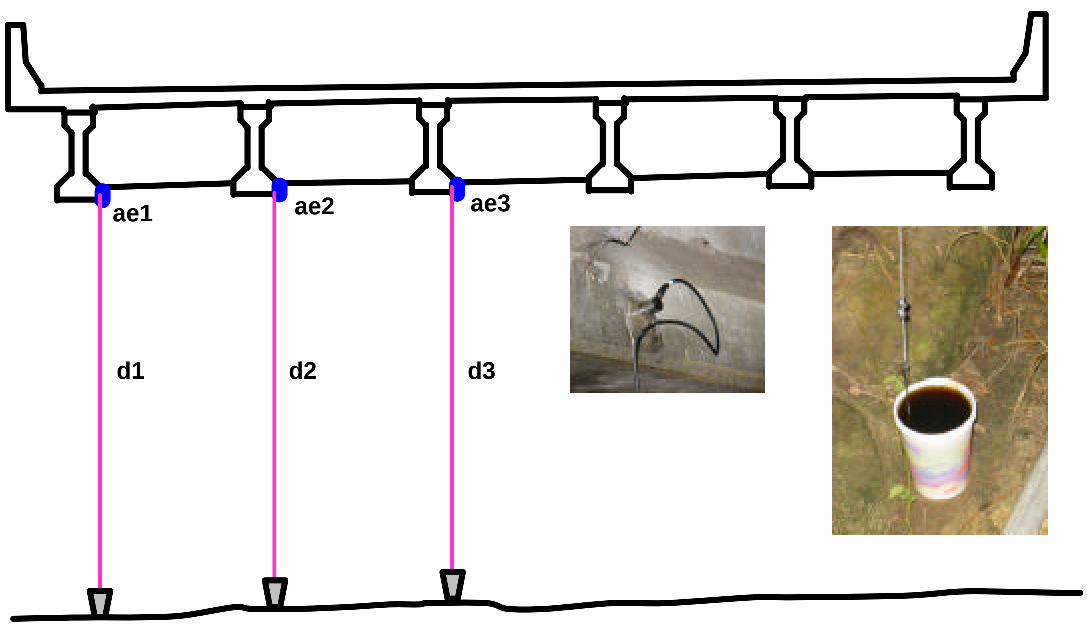


Figure 7
Plan view of instrumented portion of the bridge



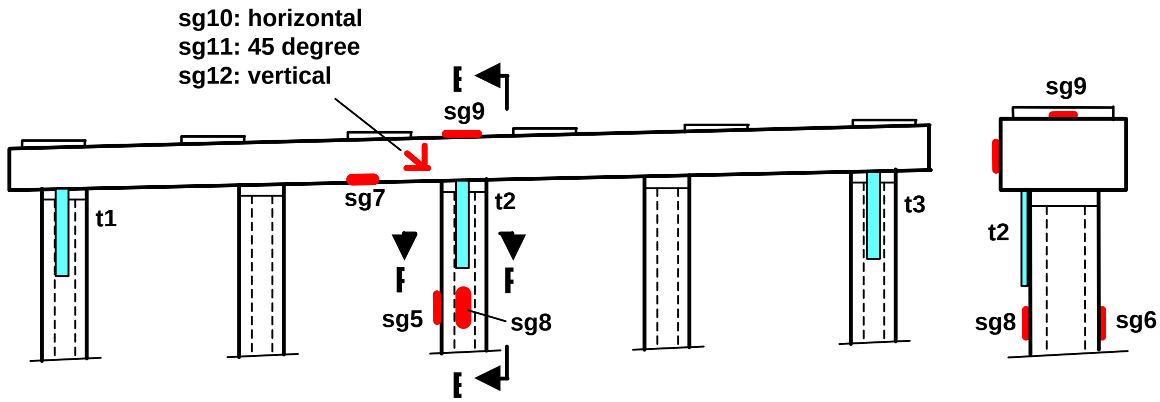
<u>Section A-A (Span 2)</u>
(looking east)

Figure 8
Strain gauge numbering (midspan of girders)

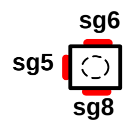


<u>Section B-B (Span 1)</u>
(looking East)

Figure 9
Deflectometer and AE sensor numbering (midspan of girders)

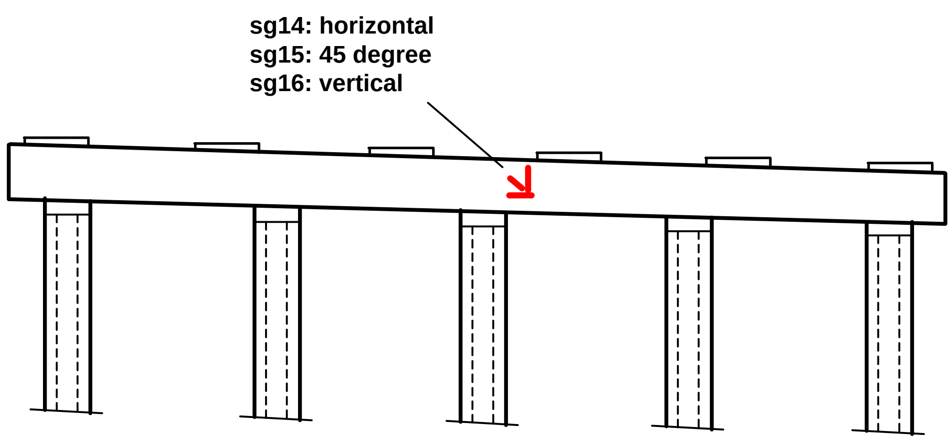


Section C-C (Bent 2)
(looking East)



Section F-F

Figure 10
Strain gauge and target numbering (bent 2)



Section D-D (Bent 1)
(looking west)

Figure 11
Strain gauges numbering (bent 2)

Monitoring of Strains

Description of Data Acquisition System

A 24-channel IOTech conditioned signal system was used for the monitoring of strains. This system is equipped with proprietary commercial software necessary for data reduction and analysis. Moreover, it is suitable for the monitoring of load cells, LVDT's, thermocouples, and other data inputs. In addition to eight channels suited to the monitoring of vibrations, the system has 16 channels of conditioned signal for strain gauge measurements. These channels can be modified to accept either 120 or 350-ohm strain gauge inputs. The system is capable of sampling rates of up to 1 mHz at a 16 bit resolution in addition to simultaneous scanning and holding for all channels. Digital signal processing boards necessary for the analysis of digital waveforms are part of the computer systems in charge of controlling the data acquisition instruments. A photograph of the portable system is shown in figure 12.

Figure 12

Photograph of Portable Data Acquisition System

Application of Strain Gauges in the Field

Strains were monitored with electrical resistance foil strain gauges at a frequency of once each second (1 Hz) in all cases. To obtain results with a high level of confidence, it is imperative that proper installation techniques be used when affixing gauges to the concrete surface. One factor to consider when monitoring strains on concrete structures is the length of the strain gauge to be used. A minimum length of four inches is preferred for concrete due to the heterogeneous nature of the material. The lack of homogeneity results in large, local variations in strain that are averaged to some degree by the use of a long strain gauge. Long strain gauges (type CEA-06-20CBW-120) were used whenever possible. Due to the lack of time for preparation, it was not possible to obtain long gauges in all cases and, therefore, shorter gauges (type CEA-06-500UW-120) were used in some instances. Sorter gauges are commonly used for composite materials, which are also heterogeneous though typically to a lesser degree than concrete. They are also relatively large and, while not ideal for this application, the results obtained from these gauges are considered to be reliable.

The preparation of the concrete surface is also critical. Favorable results are generally obtained when a high quality two-part epoxy system such as M-Bond AE-10 epoxy (manufactured by Micro-Measurements, Inc.) is used to prepare the surface. Preparation of the surface functions by grinding the concrete surface to remove loose debris, applying a thin layer of M-Bond AE-10 epoxy, allowing it to cure, and then sanding the thin layer of adhesive with a fine-grit (200 or finer) sandpaper. The AE-10 epoxy requires a 24- hour cure at room temperature. This curing time can be accelerated by the use of a hand held heating source such as a halogen lamp or heat gun. Due to short notice and lack of access to the structure, a 24-hour curing period was not possible. Acceleration of the cure was attempted with a heat gun, but this approach failed due to the low temperature of the concrete surfaces on the underside of the bridge. Therefore, a quick-drying two-part epoxy (Devcon two-part epoxy) was used to prepare the concrete surfaces. When used in combination with the heat gun, this approach allowed for application of the gauges within 2 hours after the epoxy was applied. The gauges were affixed to the prepared surface with M-Bond 100 adhesive (also by Micro-Measurements, Inc.). This adhesive is less desirable than M-Bond AE-10 because of the lack of durability over time. However, it has the significant advantage of obtaining a full cure within 10 minutes after application.

For exterior applications, it is desirable to weatherproof the gauges in all instances. The weatherproofing process was somewhat time consuming. The first gauges that were installed (gauges 1, 2, 3 and 4 at midspan of span 2) were weatherproofed. The next set of gauges installed (gauges 5, 6, 8 and 9 on the central pile and in the negative moment region of bent 2) was also weatherproofed. Time did not allow for weatherproofing of the remaining gauges.

Factors Affecting Apparent Strain Results

When monitoring strain gauges in the field, consideration should be given to additional resistance imposed by the use of long runs of wire from the strain gauge to the data acquisition system. Very long runs in conjunction with a small-diameter wire will significantly alter the apparent resistance of the strain gauge. This results in inaccurate strain readings. To minimize this problem, a heavy gauge (16-gauge or heavier), high quality three-conductor shielded wire is preferable for monitoring of strains in the field. Eighteen-gauge three-conductor shielded wire was the largest available at the time of the project, and this was used in all cases for the runs from the gauge to the data acquisition system. The shielding of this wire was not of the highest quality, which may have contributed to the problems with electrical noise discussed later in this report. The leads from the gauge itself to the 18-gauge wire were made from 26-gauge wire. These leads were kept less than six inches in all cases to minimize similar problems with increased resistance.

Due to the short notice given for this project, gas powered generators were used to supply electrical power to the data acquisition system and the auxiliary systems such as floodlights, heat guns, etc. Two problems can arise when portable generators are used in conjunction with electrical resistance strain gauges. First, the strain gauge data acquisition is a sensitive instrument and relies on a relatively "clean" power supply to provide consistent results.

Second, the electrical field from the portable generator itself can interfere with the strain readings. This is particularly true if the wire from the strain gauge to the data acquisition is insufficiently shielded. A better option for power supply to the data acquisition is battery power. Battery power adapters are available for most portable data acquisition systems, including the one used for this project.

Laboratory Testing Performed to Address Necessary Compromises in Field Instrumentation

The necessary compromises listed above were cause for concern regarding the validity of the data acquired in the field. Therefore, additional laboratory testing was performed to address three concerns: 1) the effect of long runs of 18-gauge wire, 2) the effect of Devcon epoxy in place of the preferred AE-10 epoxy, and 3) the effect of the portable generators on the strain data.

The effect of the long runs of wire and the Devcon epoxy were investigated with a single compression test of a 6-inch by 12-inch concrete cylinder. Four strain gauges were affixed to the cylinder as shown in figure 13. The effects of the Devcon versus AE-10 epoxy were analyzed on one side of the cylinder and the effects of short versus long runs of 18-gauge wire were explored on the opposite side of the cylinder. The results of this investigation are shown in figure 14. As can be seen in the figure, neither the epoxy type nor the length of the wire run significantly affected the results. When viewing the results, it should be remembered that concrete is heterogeneous, strain gauges are never perfectly aligned, and some eccentricity in loading is to be expected. Therefore, it is normal to have in the data. The effect of the noise in the data due to the long run (80-feet) of wire from gauge 4 is apparent in figure 14. Power was provided to the data acquisition from an AC wall outlet for this test.

The effect of providing power to the data acquisition system with a portable generator was investigated in a separate test. For this investigation, 80-foot long wire runs of 18-gauge wire were connected to two strain gauges. One gauge was mounted on a small (1 in by 7 in by 3/8 in fiber-reinforced polymer specimen with Devcon epoxy while the other gauge was mounted to the same specimen with AE-10 adhesive. Strains were then monitored under a condition of no load for a period of two to three minutes. The results of this investigation are shown in figure 15. Some background noise was noticed in the 70-foot run of wire when power was provided from the wall outlet. This noise typically stayed in the range of ! 30 micro-strain. A significant amount of background noise occurred when the power was provided by the portable generator. This noise lied, for the most part, in the range of ! 100 micro-strain. The noise experienced by the portable generator shows greater variability when compared to the noise data gathered during the wall outlet investigation.

For both of these laboratory tests, an attempt was made to match the field conditions and treatment of the field data. Due to an oversight, the data acquisition system in the field was improperly grounded. This may have also contributed to the background noise in the data. The laboratory tests were run without grounding the data acquisition system so that meaningful comparisons could be made between the laboratory and field data.

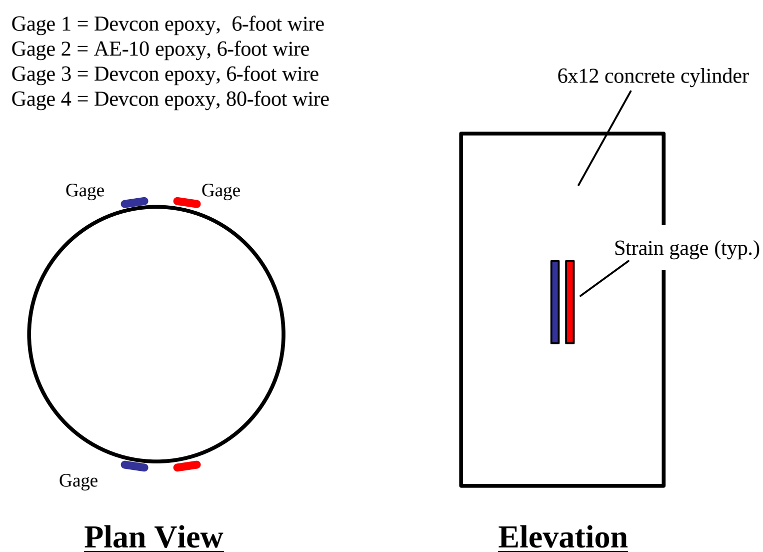


Figure 13
Concrete cylinder test for validation of strain measurements

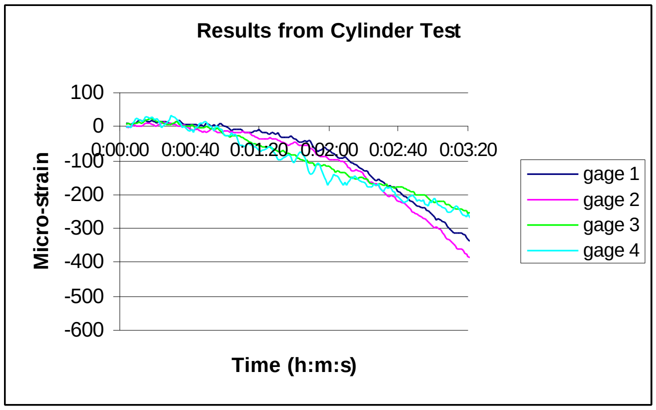


Figure 14
Strain results from concrete cylinder test

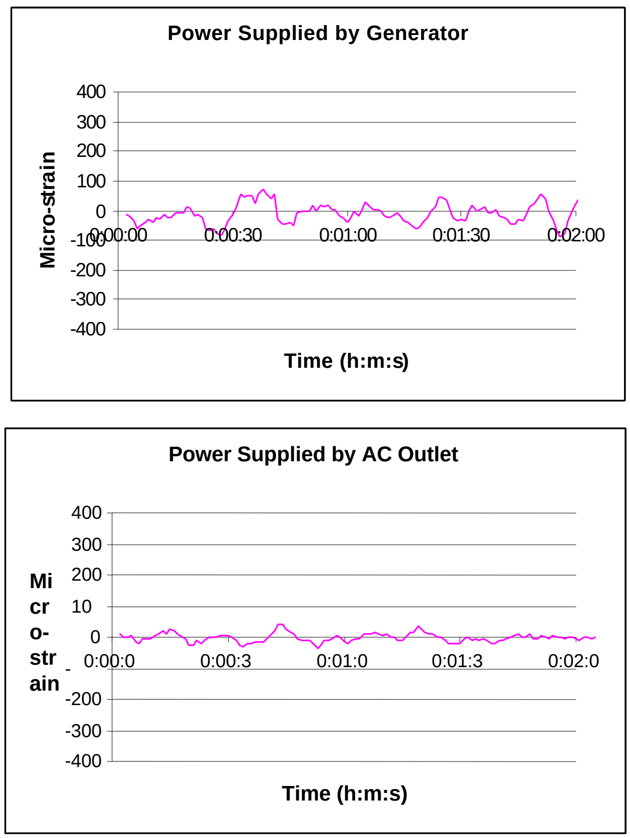


Figure 15
Interference due to portable generator

Monitoring of Deflections

Description of Data Acquisition

One concern expressed on Friday, November 8, was that a global plunging failure of the piles may take place with the overload. To investigate this possibility, fiberglass targets were affixed to three of the piles on bent 2 with "liquid nails" as shown previously in figure 10. The fiberglass targets were shot with a surveying level on the night of Saturday, November 9, and the elevations relative to a benchmark were recorded. The level was protected from movement until the overload crossed on Sunday, November 10. The targets were then shot again shot and the relative elevations were recorded and compared to those recorded prior to the passage of the overload.

A request for deflection information at the midspan of the beams was made on Saturday, November 9. The instrumentation requests to this point had been limited to strain; therefore, it was not possible to set up LVDTs, string potentiometers, or other electronic data acquisition within the time allotted. An improvised approach was used that involved a Dixie cup filled with motor oil, a kite string, and fishing weights. The kite string was affixed to the side of three adjacent girders at the midspan of span 1. A 6-inch long 26-gauge wire with a black vinyl jacket was hot-glued to the bottom portion of the kite string. The string with 26-gauge wire attached was weighted down with fishing weights (often referred to as BB's or split-shot) and was then placed in the Dixie cup filled with motor oil. The original location of the oil on the 26-gauge wire was marked by indenting the wire with a metal tool. The maximum displacement of the beams at midspan could then be determined by observing the excursion of the oil line on the 26-gauge wire. The point of this excursion was marked by indenting the wire with a metal tool immediately following the passage of the overload. A photograph of the device and a schematic diagram are shown in figure 9. While this method was undeniably crude, it did serve the purpose of recording the maximum deflection due to the passage of the overload.

Monitoring of Acoustic Emission

Description of Data Acquisition System

Acoustic emission can be described as transient elastic waves produced in a medium by a sudden release of energy, such as that produced by crack growth. Common acoustic emission terms can be found in [*1*] and [*2*]. Acoustic emission data was collected with three R6I sensors (resonant at approximately 60 kHz). These sensors were affixed near the bottom of the girders with hot-melt glue (figure 9). The acoustic emission data was recorded with a portable DISP-Main-24 acoustic emission system. This system has two PCI/DSP-4P modules with a waveform option and source location capability. All AE equipment was manufactured by Physical Acoustics Corporation (PAC). In addition to acoustic emission, the AE system can monitor up to 8 different parametric inputs such as strain, load, temperature, etc. The system is designed in a modular fashion (either 4 or 8 channels per module). Modules can be added to attain a total of 48 channels. A photograph of the acoustic emission monitoring system is shown in figure 16.

Figure 16
Photograph of Portable acoustic Emission Monitoring System

Frequency of Acoustic Emission Data Collection

Acoustic emission was gathered during routine traffic loading prior to the passing of the overload to determine a baseline of acoustic emission activity from the girders. This was done on at least two occasions. They were then gathered prior to, during, and after the passage of the overload. Emissions were gathered continuously in all cases, as opposed to sampling at a particular time interval as, occurred with the strain gauge data.

DISCUSSION OF RESULTS

As was the case for the Methodology, the results of the Bonnet Carré Spillway Bridge are presented in the following four sections. These are visual inspection, recorded strains, displacements, and acoustic emission.

Visual Inspection

Visual inspection of the concrete deck was performed prior to the passage of the overload. It revealed cracks in the concrete deck in the negative moment region over the bents (figures 17 and 18). The cracks generally spanned the entire width of the bridge. Approximately five negative moment regions were inspected, and the cracks were observed in the deck at each region. The cracks were estimated to be approximately 1/32 to 1/16-inch wide. A crack width-measuring device was not available for a more accurate determination of crack width. The overload passed on the afternoon of Sunday, November 10. Time did not allow for another visual inspection of the cracks after the passage of the overload. Therefore, it is not known if the crack widths were increased by the overload.

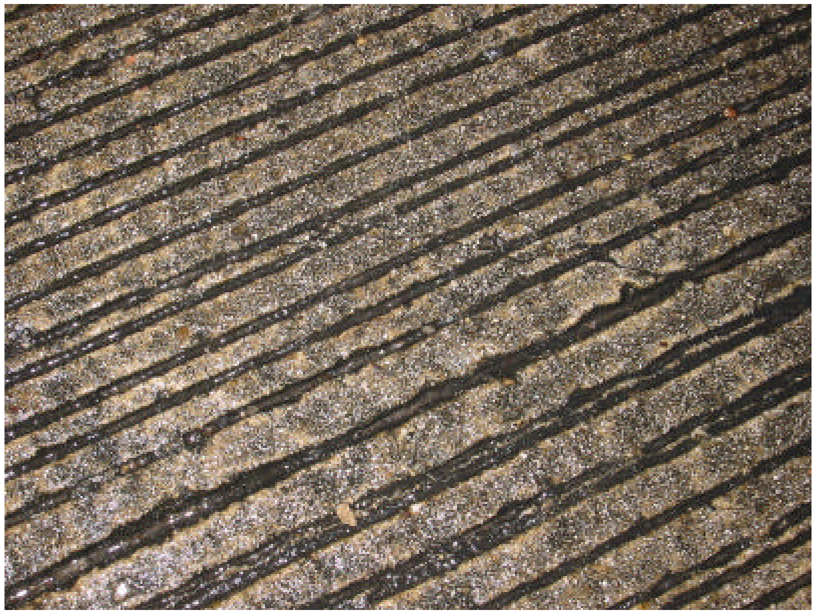


Figure 17
Photograph of cracks in concrete slabs

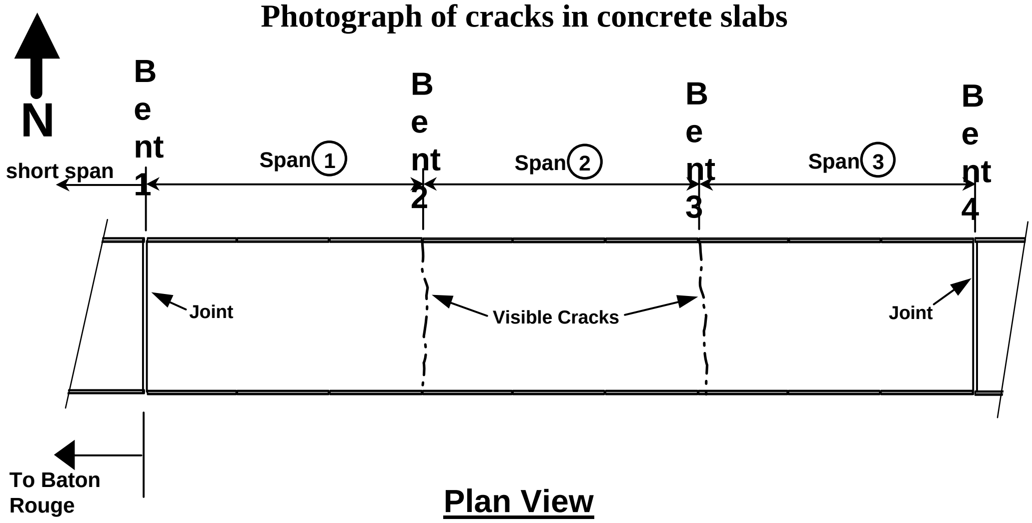


Figure 18
Plan view including typical crack location

Recorded Strains

As aforementioned, the recorded strains were adversely affected due to the use of a portable generator as a power source as well as several other factors. To minimize the effects of the noise that was caused in the strain data each reading was averaged over a five second time interval. This method resulted in smoothed strain readings. The smoothed readings are presented in addition to the actual readings in all cases.

When considering the strains recorded, it should be remembered that the strain gauge is affixed to the surface of the concrete member and is then electronically "zeroed" to indicate a value of zero strain. Any values recorded after the gauge is zeroed are in relation to the zero value. In other words, strain gauges give relative as opposed to absolute strain readings. Since the initial value of strain (or stress) in the concrete member at the location of the gauge is unknown, it is not possible to obtain absolute values of strain (or stress). Rather, changes in strain are recorded. Absolute values of strain could be obtained with strain gauges only if the gauge were zeroed under a state of no load when the member in question was fabricated. However, the presence of tensile cracks in the concrete would still complicate the interpretation of strain results.

If the initial state of strain prior to application of the gauges were known, calculation of the stress in the concrete would still be complicated by a number of factors including creep, shrinkage, and differential settlement. In the case of the girders, bent caps, and piles, the level of initial strain in the member could be estimated by conducting a structural analysis that considered the time, temperature and loading history of the elements in question. If the material properties of the concrete and steel were also known, including the creep and shrinkage behavior, then an estimate of absolute stress in the concrete member could be made. This type of analysis is beyond the scope of this report and therefore was not conducted.

Gauges at Midspan of Girders - Span 2

The recorded results for the strains during passage of the overload at gauge locations 1, 2, 3 and 4 (midspan of the girders) are shown in figure 19. The smoothed results for the same gauge locations are shown in figure 20. Figure 20 gives a clearer picture of the strain gauge data. These results are plotted for the time span of 35 to 45 minutes after the start of data acquisition. The strains are positive as would be expected due to tension in the bottom portion of the girder during the passage of the overload. The strains increase from the outermost girder to the innermost as would be expected from the nature of the overload. The strains on the innermost girder (gauges 3 and 4) are of similar magnitude, however gauge 3 recorded slightly higher strains than gauge 4. This is to be expected even though these gauges are placed on opposite sides of the same girder. Reasons for disagreement between the two gauges include misalignment of gauge 4, heterogeneity of the concrete, and the possible presence of micro-cracks crossing gauge 3 but not gauge 4. The maximum recorded strain due to the overload in these locations occurred in gauge 3 and was approximately 300 micro-strain (micro-strain is defined as engineering strain x 10^{-6}). This compares to the value of approximately 100 micro-strain recorded in the outer girder by gauge 1. With the exception of gauge 3, all of the gauges began in the zero position prior to the load and returned to the zero position after the load passed. Gauge 3, however, began slightly below zero and returned to a reading of slightly above zero after the load passed. This may indicate the malfunction of this gauge or the development of a crack during the passage of the load. The results from this set of gauges (1, 2, 3 and 4) are considered to be the most reliable due to the fact that these were weatherproofed, and, thus, there was no adverse effect of moisture due to weeping as was the case for most of the other gauges.

Gauges on Pile and Pile Cap - Bent 2

The recorded results for the strains during the passage of the overload at gauge locations 5, 6, 8 (central pile of bent 2), and 9 (negative moment region of the pile cap of bent 2) are shown in figure 21. The smoothed results for the same gauge locations are shown in figure 22. Figure 22 gives a clearer picture of the strain gauge data. The locations of these gauges are shown in Figure 10. These results are plotted for the time span of 33 to 43 minutes after the start of data acquisition. Regarding the recorded strains on the central pile, the strains are negative as expected due to compression in the pile. If the strains from gauge 6 and 8 are compared at similar times, some evidence of bending about the axis of the pile cap can be inferred. The maximum recorded strain on the central pile occurred in gauge 9 and was approximately 100 micro-strain (negative). Similar maximum values were obtained from gauges 6 and 8. All three gauges began in the zero position and returned to the zero position once the load had passed. The results from this set of gauges are believed to be reliable because those gauges were weatherproofed and the data were generally inspected. The readings from gauge 9 (negative moment region on pile cap of bent 2) alternate between positive and negative. The readings contradict the expected results of positive or zero micro-strain in all cases. Gauge 9 begins at approximately 50 micro-strain (negative) and does not return to this value or to zero after the load has passed. Based on this behavior the results from this gauge are not considered.

Gauge in Positive Moment Region of Pile Cap - Bent 2

Gauge 7 was placed in the positive moment region of the pile cap of bent 2. The location of this gauge is shown in figure 10. This gauge malfunctioned prior to the passage of the overload. Time did not allow for weatherproofing of this gauge.

Gauge on Longitudinal Reinforcing Bar in Deck - Bent 2

The recorded results for the strains during passage of the overload at gauge location 13 (gauge placed on top longitudinal reinforcing bar in the deck above bent 2) are shown in figure 23. These results are plotted for the time frame of 33 minutes to 43 minutes after the start of data acquisition. The smoothed results are shown in the same figure. In this case, the smoothed results give only slightly better insight into the behavior of this gauge during the passage of the overload. The noise in the data from this gauge location is of a high magnitude when compared to the noise from other locations. This high volume of noise may be due to the following factors. First, the wire for this gauge was located on top of the bridge as opposed to beneath the bridge. This may have exposed the wire to electrical fields from the generators or the overload itself. Next, the gauge itself was not weatherproofed and water did come into contact with the gauge and the leads. This may have caused the unusual behavior of this gauge. When viewing the data, it appears that the maximum strain due to the overload in this gauge was approximately 200 micro-strain (positive). The strain measured as expected and the gauge began approximately at zero and returned to zero after the passage of the overload.

Gauges on Pile Cap (Shear Rosette) - Bent 2

Gauges 10 and 12 of the shear rosette on the pile cap of bent 2 (gauges 10, 11 and 12) malfunctioned (locations shown in figure 10). Therefore, the results from this rosette could not be used. Time did not allow for weatherproofing of these gauges, which may have been the source of the problem. A similar rosette was placed in a similar location on bent 1 prior to the passage of the overload. The results from this rosette are described below.

Gauges on Pile Cap (Shear Rosette) - Bent 1

The recorded results for the strains during passage of the overload at gauge locations 14, 15 and 16 (shear rosette on pile cap of bent 1) are shown in figure 24 as well. The smoothed results for these gauges are shown in the same Figure. These results are plotted for the time period of 15 minutes to 35 minutes after the start of data acquisition. No clear pattern or maximum can be seen in the results for this set of gauges. The actual strains are expected to be small and may be obscured by the noise in the data. Each of the gauges in this set also experienced significant drift (did not return to zero). Due to time constraints these gauges were not weatherproofed and may have malfunctioned. It is recommended that the results from this set of gauges be disregarded.

General Discussion of Recorded Strains

As described above, in most cases the strain readings returned or nearly returned to their initial "zero" value after the passage of the overload. For the gauges that returned to zero, this is an indication that permanent damage did not occur to the monitored portions of the structure. For those cases in which the gauges did not return to zero after the passage of the overload, this is not necessarily an indication to the contrary. A number of factors can affect the apparent strain results, including the development of minor cracks that may have developed due to the passage of the overload but would not adversely affect the performance of the structure. Other possibilities include creep of the epoxy bonding agent or electrical drift of the gauges in question.

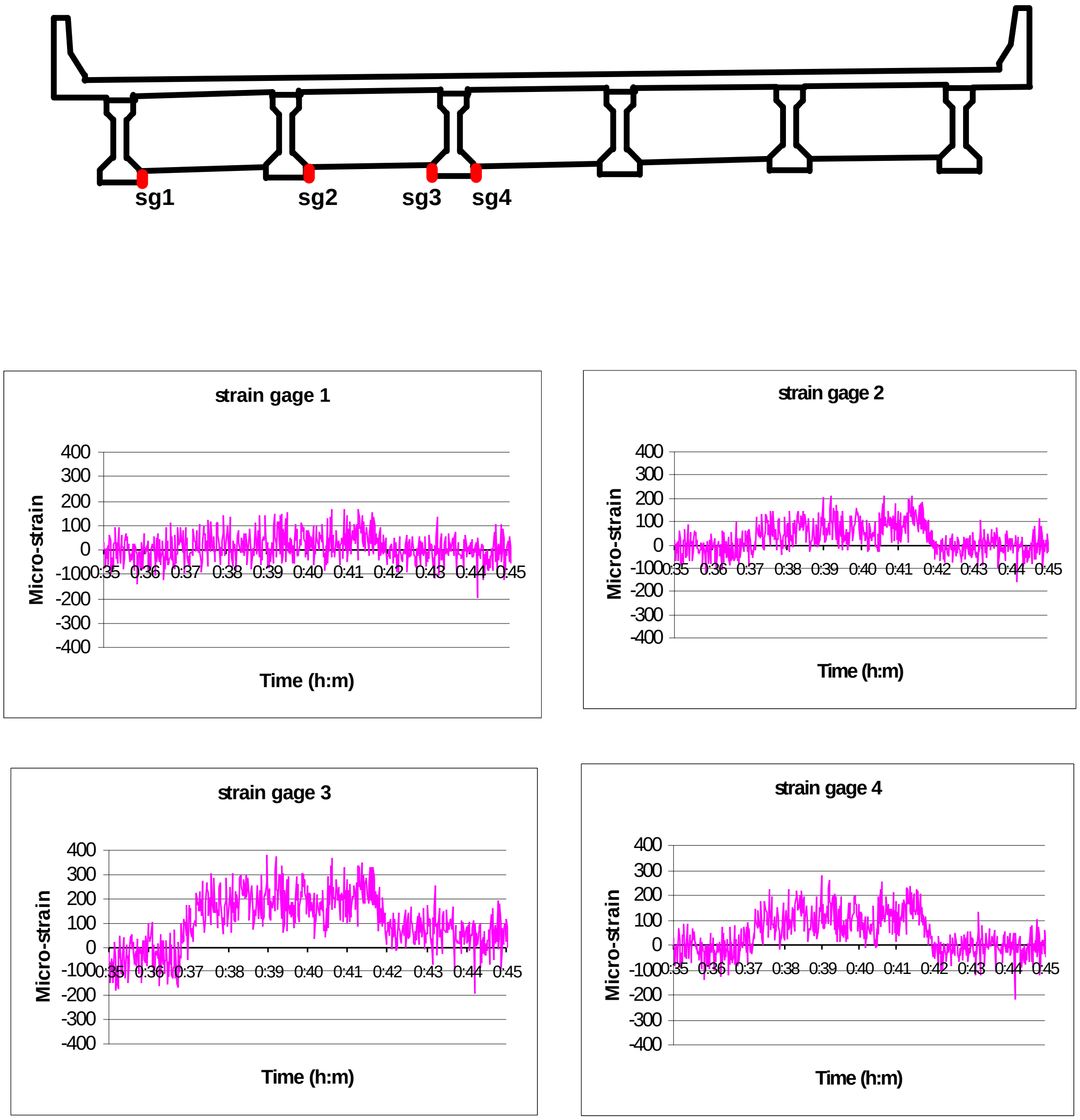


Figure 19
Actual results for strain gauges 1-4 (midspan of girders)

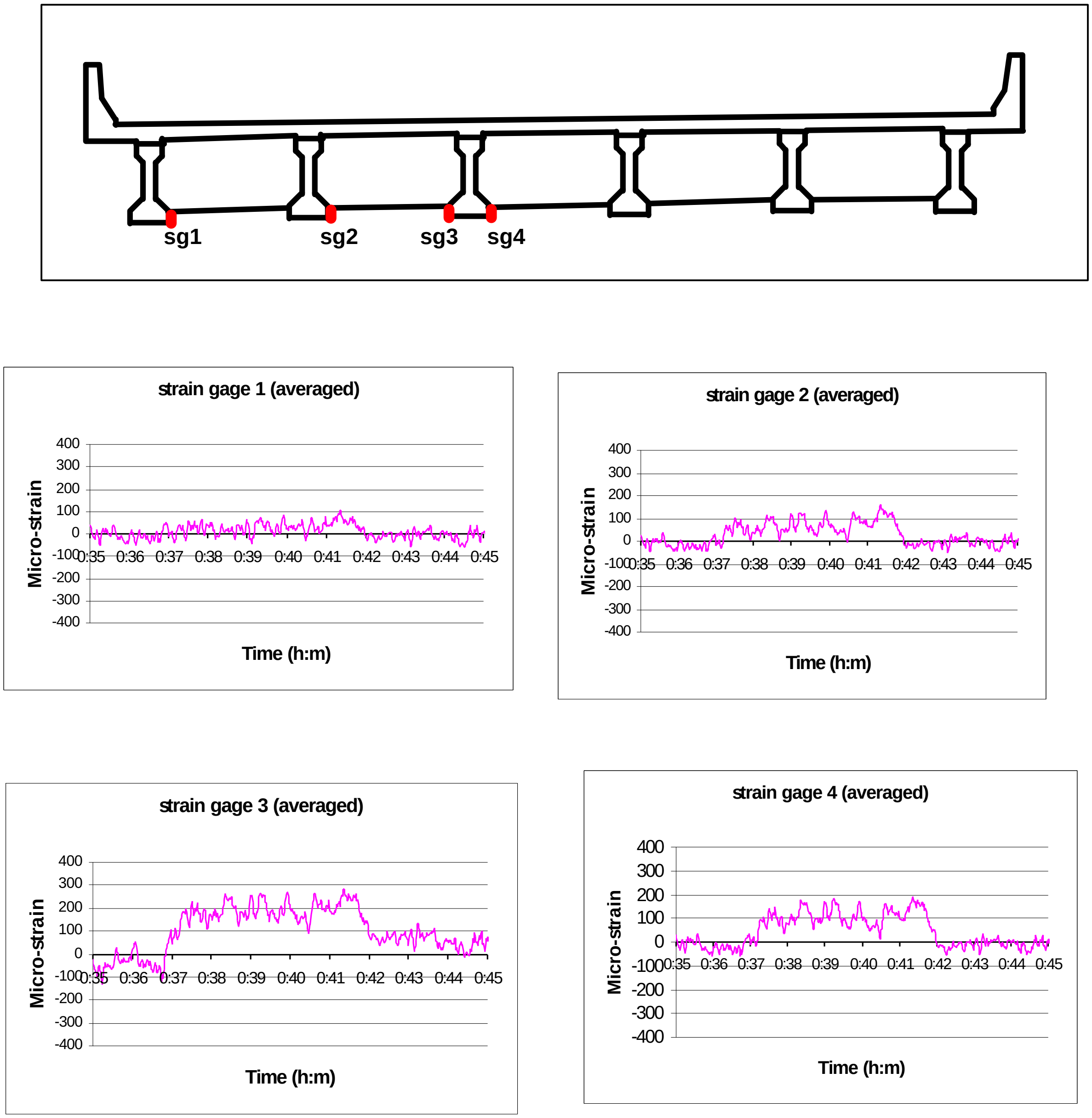
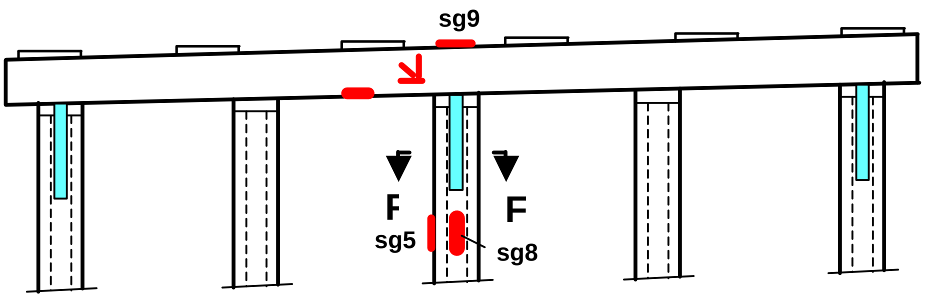
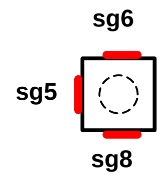


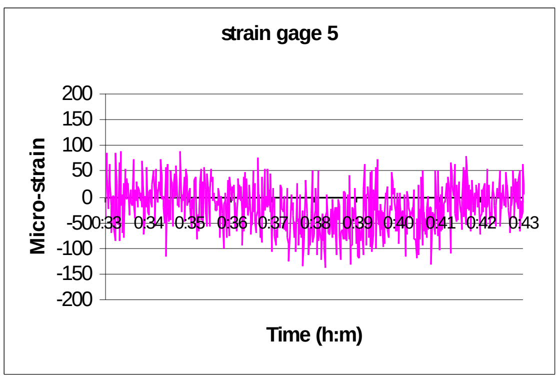
Figure 20
Smoothed results for strain gauges 1-4 (midspan of girders)

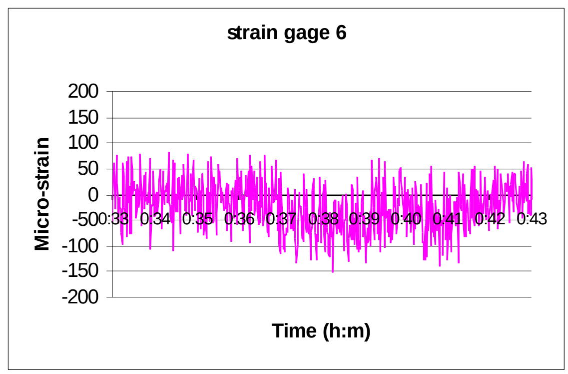


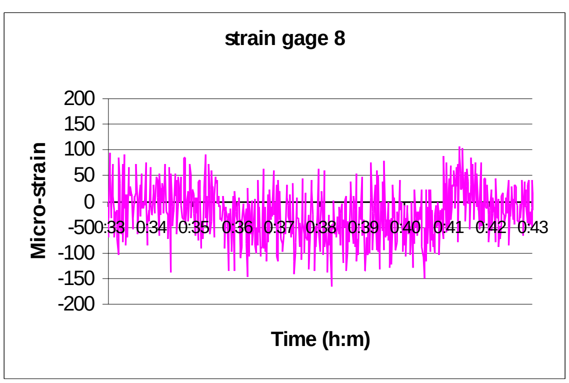
Section C-C (Bent 2)
(looking East)



Section F-F







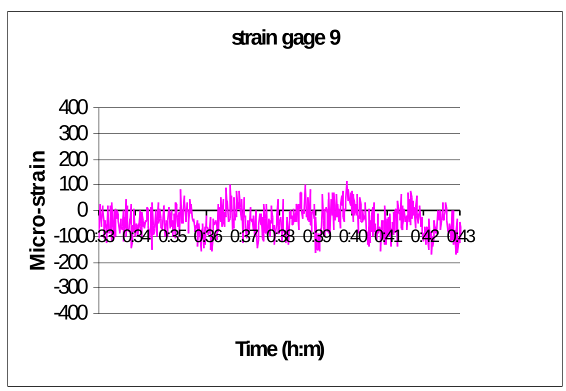
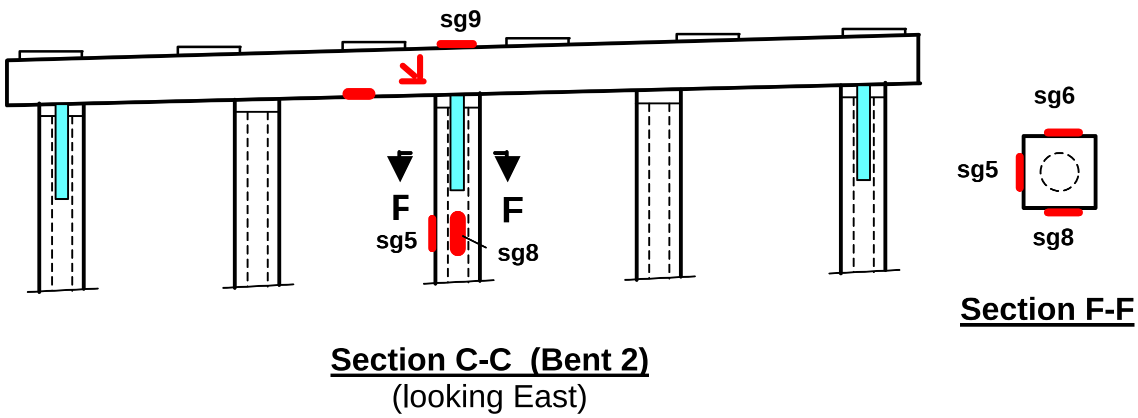


Figure 21
Actual results of strain gauges 5,6,8, and 9 (bent 2)



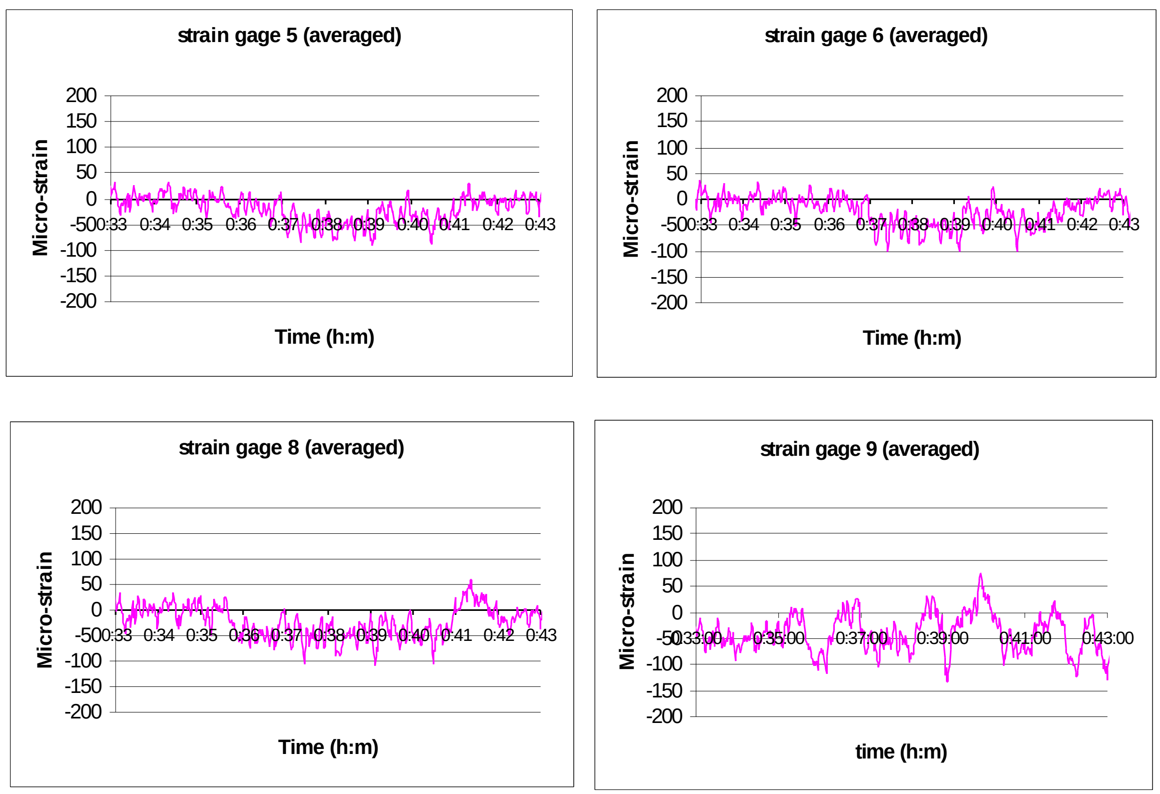
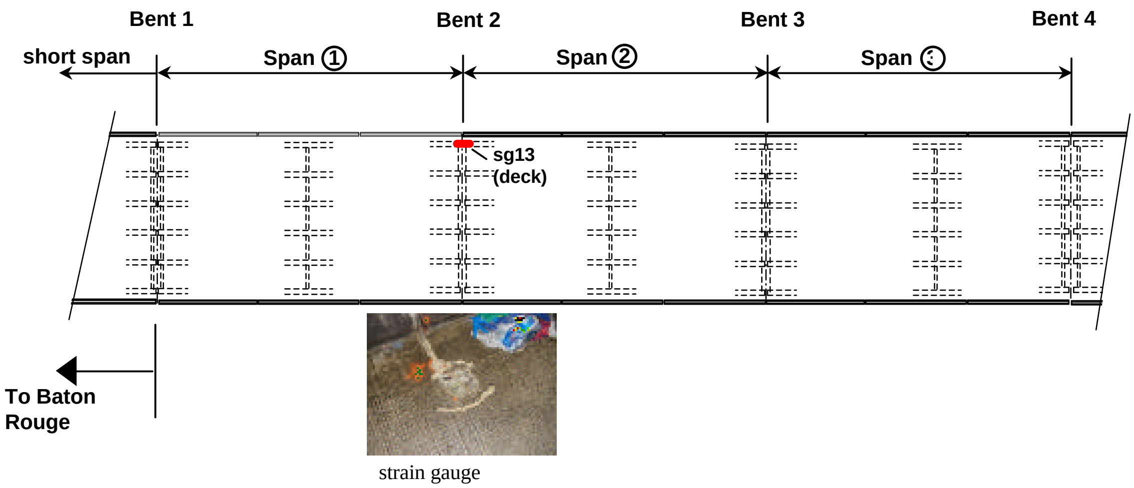
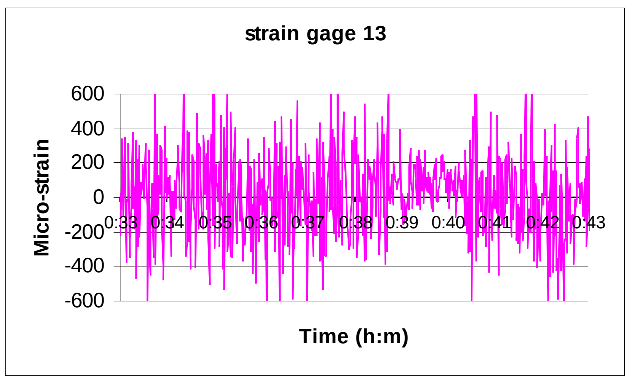


Figure 22
Smoothed results for strain gauges 5,6,8, and 9 (bent 2)





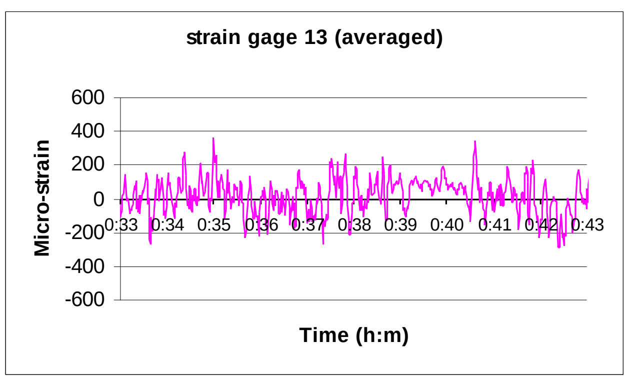
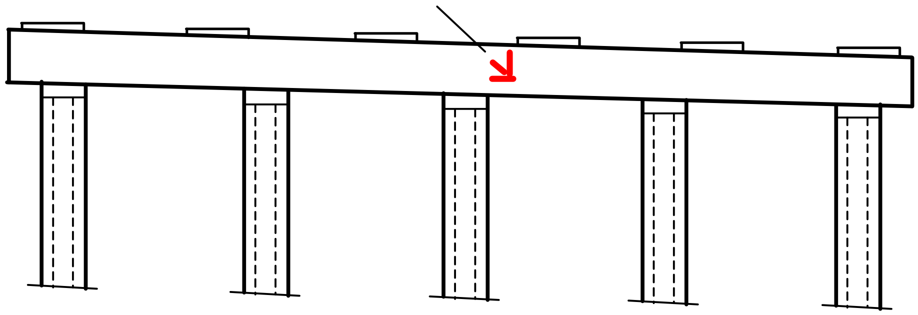


Figure 23
Actual and smoothed results for strain gauge 13 (in deck over bent 2)



Section D-D (Bent 1)
(looking West)

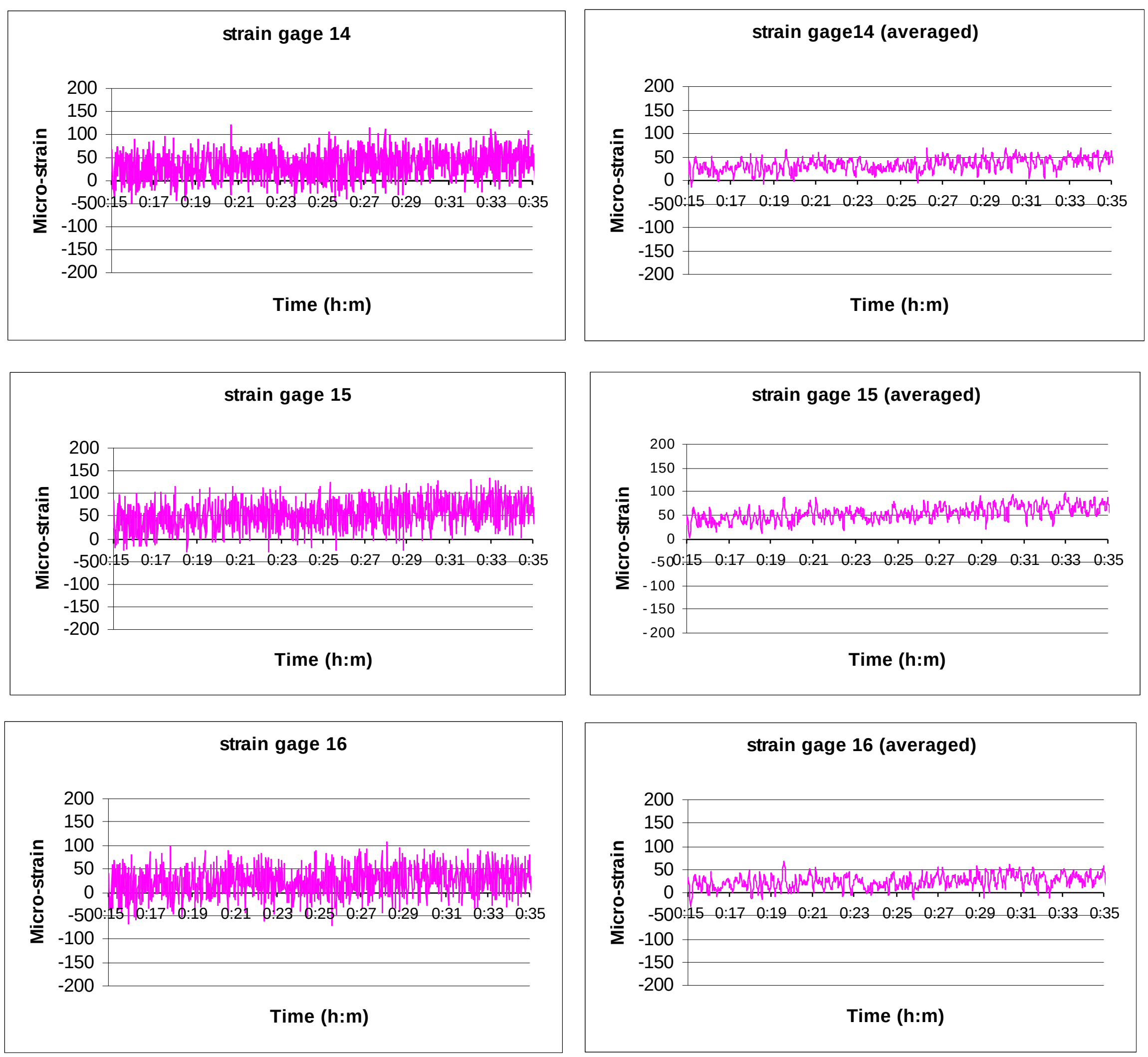


Figure 24
Actual and smoothed results for strain gauges 14, 15, and 16

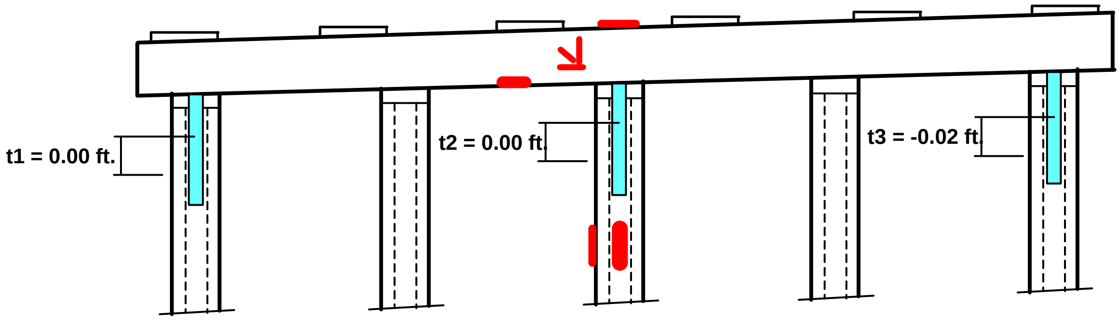
Recorded Displacements

Targets Placed on Piles - Bent 2

Recorded deflection of the piles of bent 2 after the passage of the overload is shown in figure 25. These results indicate that two of the piles experienced no measurable deflection. The recorded deflection of the third pile was 0.02 feet downward. It is possible that this apparent deflection was due to improper reading or recording of the elevation prior to the passage of the overload. The readings taken prior to and after the passage of the overload were taken by different individuals.

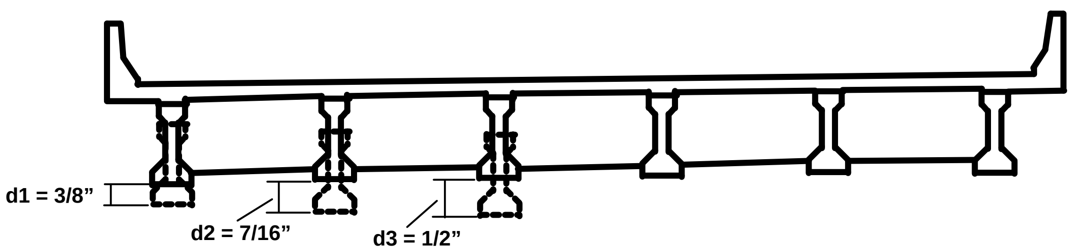
Deflection Measured with Deflectometers - Midspan of Girders (Span 1)

Deflection as recorded by the deflectometers after passage of the overload is shown in figure 26. This figure indicates that the deflection of the interior girders was greater than the exterior girders as expected for a load pattern of this type. The maximum recorded deflection was 1/2-inch.



<u>Section C-C (Bent 2)</u>
(looking toward East)

Figure 25
Deflection results for pier (bent 2)



<u>Section B-B (Span 1)</u>
(looking East)

Figure 26
Deflection results for girders at midspan (span 1)

Recorded Acoustic Emission

Acoustic emission data can provide valuable insight into the behavior and damage mechanisms and the behavior of reinforced concrete structures [*3*, *4*]. As mentioned earlier, acoustic emission can be described as transient elastic waves produced in a medium by a sudden release of energy, such as that produced by crack growth. It is a global, passive method of nondestructive evaluation. In other words, entire structures or portions of structures can be monitored, and merely loading the structure can generate the necessary data. Due to increases in the number of aging structures and damage due to earthquakes, acoustic emission is being proposed as a standard for the in-situ evaluation of reinforced concrete structures [5]. A schematic diagram of a typical acoustic emission "hit" is shown in figure 27.

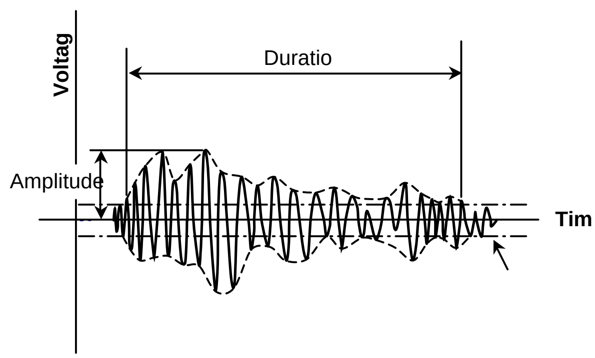


Figure 27
Schematic of Acoustic emission hit recorded by resonant sensor

Obstacles to widespread implementation of acoustic emission monitoring as a means of evaluation for civil engineering structures include the cost of the instrumentation and the fact that the interpretation of results is dependent on operator experience and loading protocol. Because acoustic emission is caused by damage growth, it is heavily dependent on load history [*6*]. Interpretation of acoustic emission results frequently focuses on the patterns of emission that are recorded during reloading of a given structure. In fact, many codes do not require the monitoring of structures during the initial loading and, instead, focus on the evaluation entirely on the data gathered during reloading [*2*]. Therefore, to obtain meaningful results, controlled loading and reloading is preferred. Reloading was not feasible in the case of the passage of the overload. However, loading and reloading with trucks of a known load is feasible and would provide more meaningful insight into the behavior of the structure should this approach be implemented in the future. Even though reloading was not conducted in this case, some general discussion of the acoustic emission monitoring approach and the data gathered is given below.

To establish a baseline of acoustic emission activity, emissions were monitored continuously for a period of approximately 30 minutes while traffic crossed the bridge. Plots of amplitude of each recorded acoustic emission hit versus time for all three sensors and for two different periods of time are shown in figures 28 and 29. These acoustic emission scans took place during the night of Saturday, November 9, and the early morning of Sunday, November 10. Therefore, the acoustic emission recorded is not representative of normal traffic loading. For traffic scan 1, high amplitude hits of up to 80 dB were recorded by sensor ae3. This sensor was affixed to the innermost of the three monitored girders. The outer girders showed significantly less activity. For traffic scan 2, high amplitude hits of approximately 90 dB were recorded by sensor ae3. In this traffic scan, the outer girders again showed significantly less activity than the innermost girder. This is to be expected and is indicative of larger portions of the traffic load being carried by the innermost girders.

Another convenient way to view acoustic emission data is to plot cumulative signal strength versus time. Signal strength is essentially the area under the positive and negative envelope of each acoustic emission hit. Signal strength includes the amplitude of the acoustic emission hits while also considering the duration of the hits. Plots of signal strength give an indication of the energy released during crack growth. A plot of cumulative signal strength versus time for traffic load number 2 is shown in figure 30. Similar to the plots for amplitude versus time, this plot also indicates that the majority of the acoustic emission energy is coming from the girder near the center of the bridge under this traffic load.

Plots of amplitude versus time for the time period before, during, and after the passage of overload 2 are shown in figure 31. The effect of this overload on the acoustic emission activity is clear in this figure. From this figure, it appears that the outermost girder is more acoustically active due to the passage of the overload than the interior girders. This is consistent with the formation of new cracks in the outermost girders due to the passage of the overload. Increased acoustic emission activity in the outer girders would be expected since these girders had, most likely, not been as heavily loaded as the innermost girders prior to the passage of the overload. Hits of approximately 80 dB were recorded in the outermost girder, while hits of approximately 90 dB were recorded in the two inner girders. This is consistent with a larger percentage of the overload being carried on the inner girders. The lack of recorded hits above 90 dB indicates that major events such as breakage of prestressing strands did not occur in the monitored portions of the structure.

A plot of cumulative signal strength versus time for the passage of the overload is shown in figure 32. This figure indicates that the acoustic emission energy generated was the largest in the central girder, which is again consistent with a larger percentage of the overload being carried by this girder. Interestingly, the second largest cumulative signal strength value was generated in the outermost girder. This is consistent with a large number of new cracks being formed in this outer girder, even though these cracks were of lesser energy content than those activated in the innermost girder.

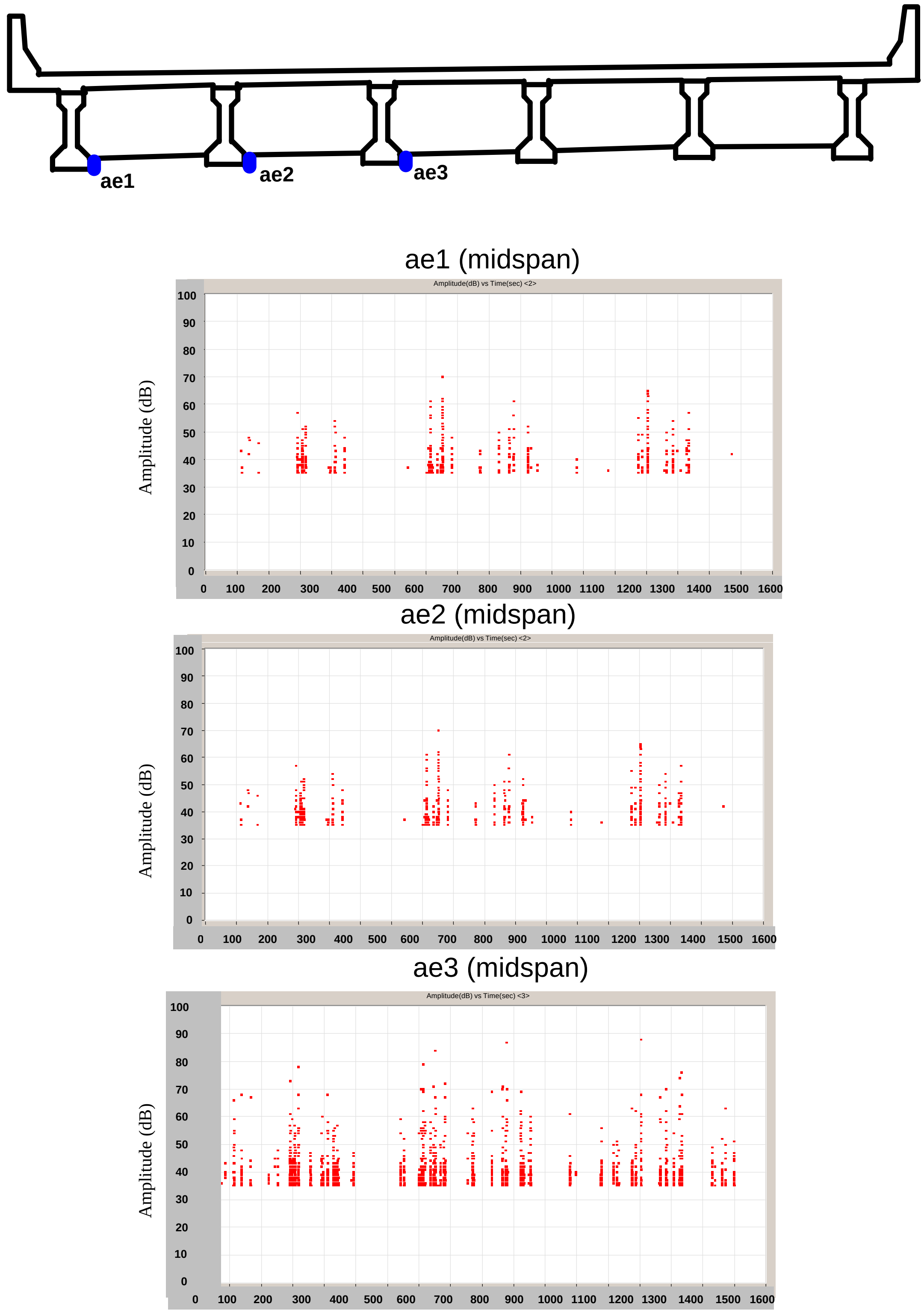


Figure 28
Amplitude vs. time - Traffic Scan 1

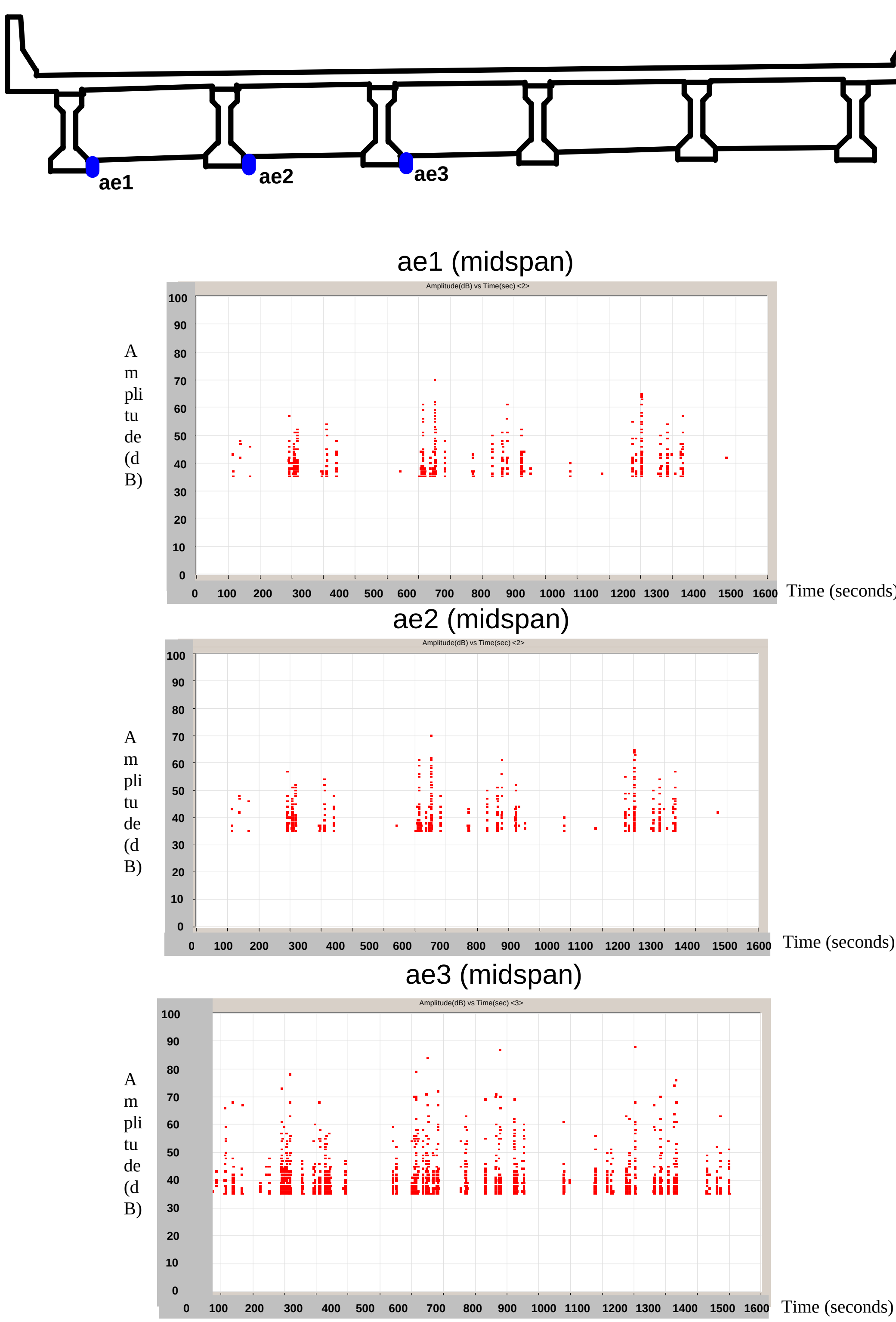


Figure 29
Amplitude vs. time – Traffic Scan 2

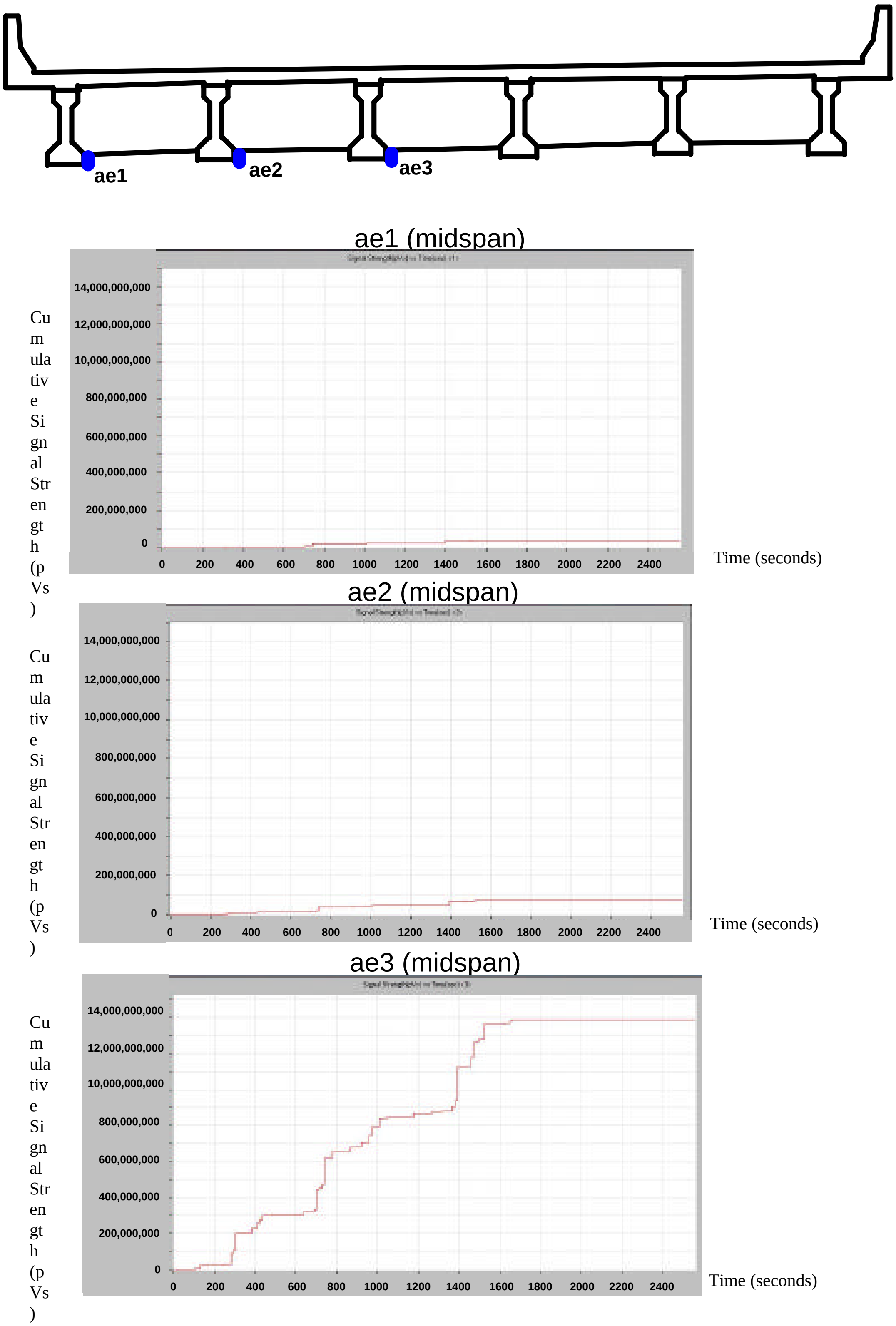


Figure 30
Cumulative Strength Signal - Traffic Scan 2

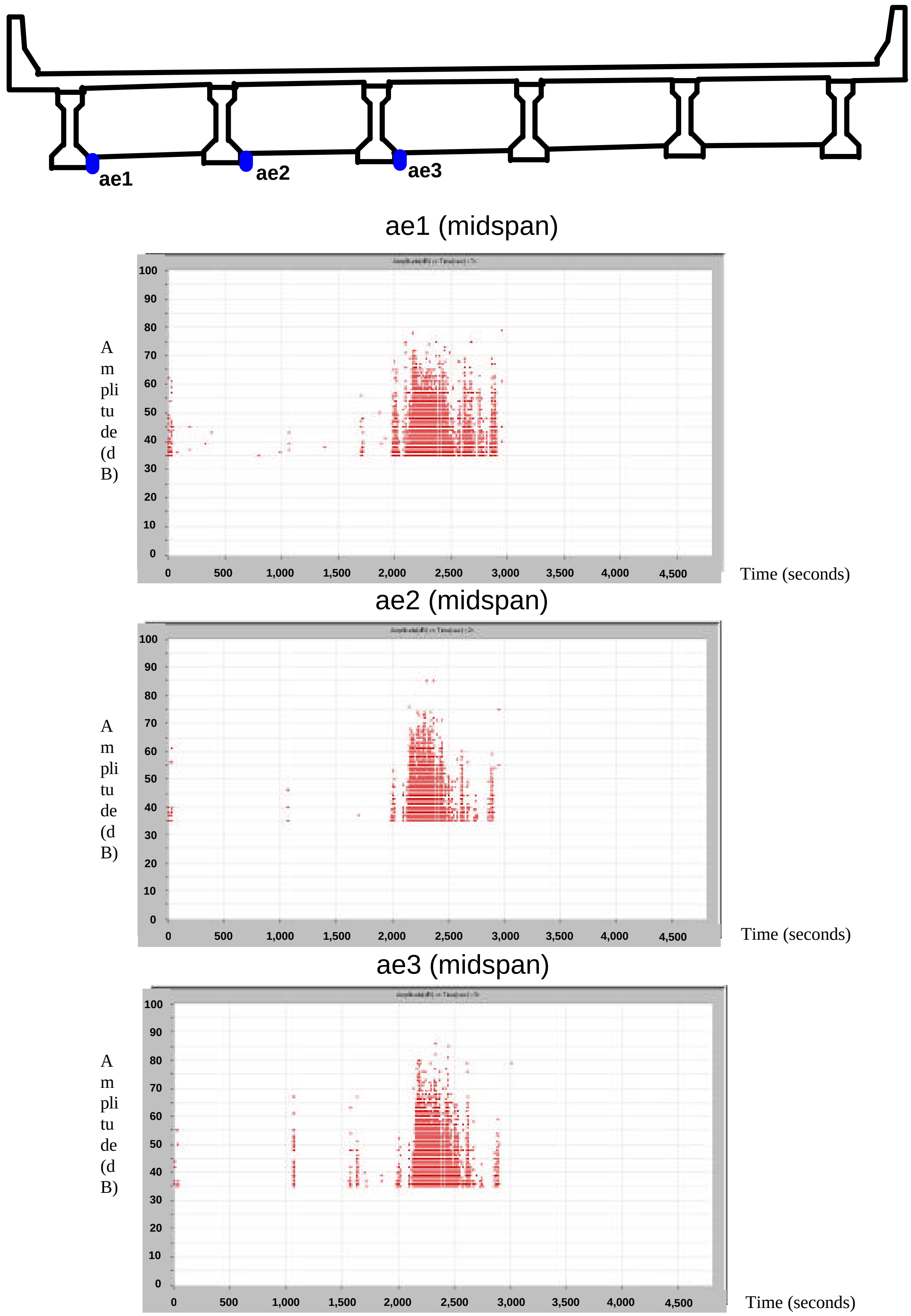


Figure 31
Amplitude vs. Time - Overload 2

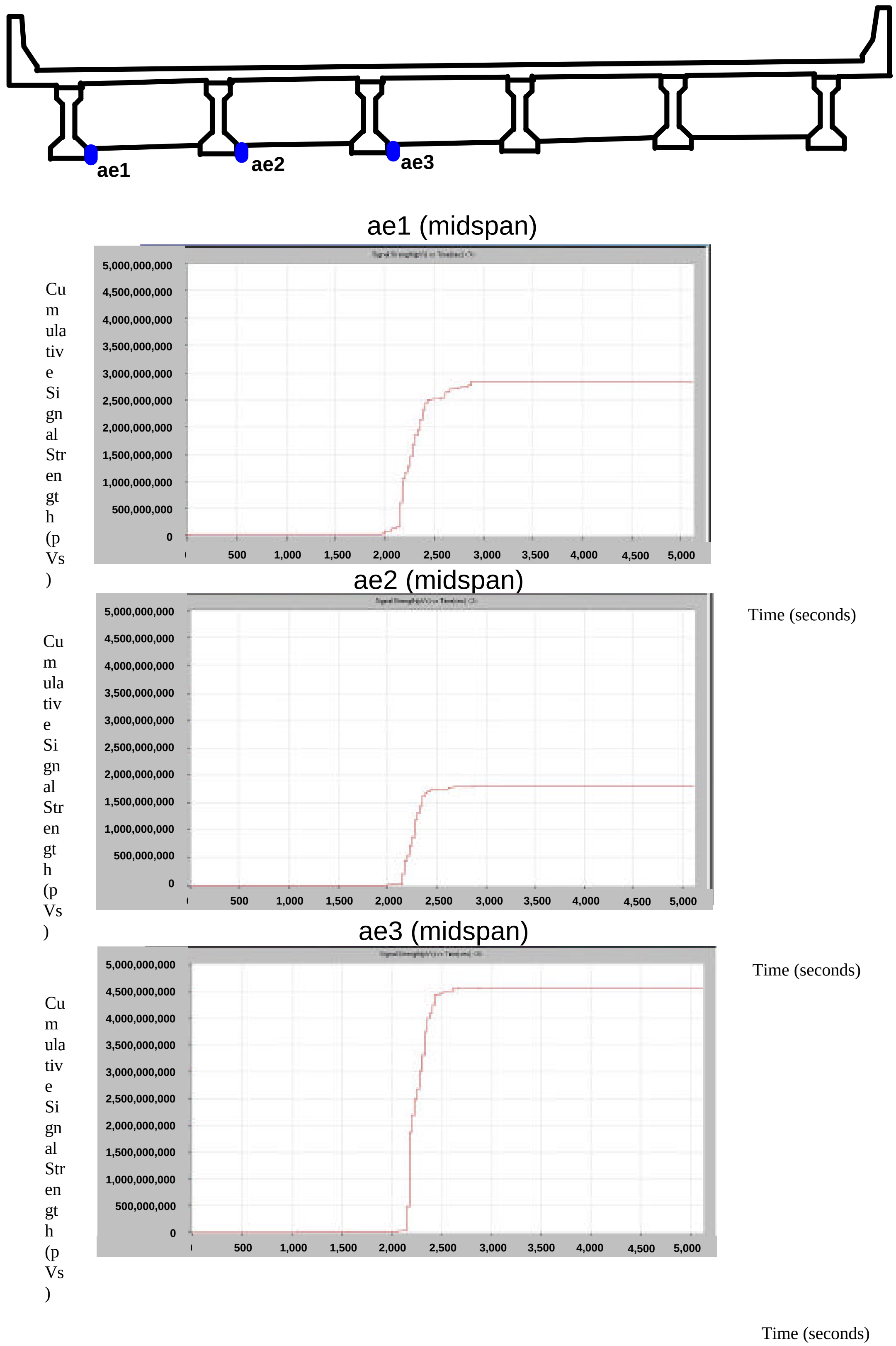


Figure 32
Cumulative Signal vs. Time - Overload 2

CONCLUSIONS

Conclusions based on the limited visual inspection and monitoring of strains, deflections, and acoustic emission performed on Sunday, November 10, during the passage of the overload are beyond the scope of this report. However, some limited analysis is given.

The recorded strain, deflection, and acoustic emission information generally points to the conclusion that major damage did not occur to the monitored portions of the structure due to the passage of overload 2. This conclusion is supported by the following observations:

1. Plunging failure of the piles did not occur as evidenced by the surveying information,
2. The majority of the strain gauges affixed to the bottom of the prestressed girders returned to the zero position after the passage of the overload, and
3. Acoustic emission events greater than 90 dB were not recorded during the passage of the overload. The amplitude of the recorded hits during the passage of the overload was similar to that recorded during light traffic loading.

While live load distribution is beyond the scope of this report, it is generally noted that the inner girders supported a larger percentage of the overload than the outer girders. This conclusion is supported by observations of the recorded strains and displacements during the passage of the overload. The ratio of recorded strain from the innermost to outermost girder was approximately 2 to 1. The ratio of recorded maximum deflection from the innermost to outermost girder was approximately 2 to 1.5.

The acoustic emission data generally indicated that the outer girders were more heavily loaded during the passage of this overload than they had previously been in their history. The data tended to indicate that a considerable amount of new crack growth took place in the outer girder due to the passage of the overload. The acoustic emission data also indicated that a larger percentage of the overload was carried by the inner girders.

RECOMMENDATIONS

Field monitoring of bridges under known loads can provide valuable insight into the behavior of the structure. This insight can, in turn, be used to better evaluate the effects of both normal traffic and overloads.

The monitoring of this particular overload was done with very little advance notice. However, much of the data that was gathered is thought to be reliable. This is particularly true for the deflection, strain, and acoustic emission data gathered at midspan of the prestressed girders. Although the information gathered seems to be reliable, it is recommended that the reliability of this data be further verified. Verification could be achieved with the use of surface-mounted full-bridge strain gauges such as those manufactured by Bridge Diagnostics, Inc. Such gauges are intended for use on concrete structures; extensions to the gauges can be added to provide sufficient gauge length to obtain more reliable readings. The use of the full-bridge configuration is helpful in minimizing background noise. It is recommended that these or similar gauges be placed adjacent to the existing electrical resistance surface-mounted foil strain gauges and that the structure be loaded with a truck (or trucks) of known weight. The results gathered with the two different gauges can then be compared, and the results obtained by the electrical resistance foil strain gauges can be either verified or discounted. The electrical resistance foil gauges were weatherproofed and should be viable for approximately two years from the time of installation.

Should the results obtained by the electrical resistance foil gauges used for this project prove to be reliable, it is recommended that the general assumptions (degree of fixity at the joints, live load distribution, etc.) made for the structural response due to the passage of the overload be examined to determine their validity. This could provide valuable information in regard to the structural capacity of bridges subjected to future overloads.

It is also recommended that the monitored portion of the bridge be loaded with trucks of known weight and that the strain and displacement response be monitored and recorded. A two-dimensional grillage or three-dimensional finite element model of the monitored portion of the structure can then be developed and the model can be correlated to the results obtained. A calibrated model of this kind would be useful in verifying design assumptions made for conventional loading (HS-20, etc.) or future overloads on this or similar structures.

For future load testing on the monitored portion of this structure, it is recommended that acoustic emission monitoring be used to assess the level of damage that takes place in the structure due to normal truck loading. This information should then be compared to the level of damage caused by the passage of the overload. Reloading with the load trucks should be employed to aid in the analysis of the acoustic emission data.

In all cases of field monitoring prior to extreme overloads, it is recommended that a thorough visual inspection of the structure be performed and the results of the visual inspection be documented in photographic and written form both before and after passage of the overload. To allow for adequate time to set up instrumentation properly, access to the structure should be provided a minimum of five days prior to the passage of the load to be monitored. Since notice is often very short prior to this type of project, it is recommended that a retainer contract for this type of monitoring be considered.

Strain is a sensitive and readily measurable quantity for the assessment of structural behavior. When monitoring strain in reinforced concrete structures, the possibility of existing cracks should be considered. In the case of prestressed concrete structures, cracks may not be present in the tension region. In this case, conventional electrical resistance strain gauges can provide satisfactory results. When electrical resistance strain gauges are used, the surface preparation is critical. The type of wire used is also important, and high quality three-conductor 16-gauge (or better) shielded wire should be used for runs of over 50 feet in length. If the strains to be measured are small, power should be provided to the data acquisition with a battery, the system should be well grounded, low-pass noise rejection filters should be used, and averaging of the data over time should be considered. Vibrating wire strain gauges are more stable over time than electrical resistance gauges and are therefore preferable for long term monitoring (though at higher cost). Fiber optic strain gauges offer the additional advantage of stability and no electrical interference. Units for the interpretation of fiber optic sensors are costly at this time. Other options that have recently been developed or are in development include distributed sensor arrays that can be remotely accessed.

Deflection information may be more useful for certain structures, such as reinforced concrete structures with a high degree of cracking. Where the underside of the structure is accessible from land, string potentiometers or LVDTs can be used to accurately measure deflection. Where this access is not available, conventional surveying techniques can be used to provide limited information under static loads. Laser systems are available that can provide better information at increased cost. Digital photography can also be used and the images can then be "pixelated" to determine deflection.

The options mentioned above concern the overall mechanical behavior of the structure. This information can be extended to portions of the structure that are not monitored through computer modeling, such as finite element analysis.

Acoustic emission is a very sensitive measure of damage in a structure. In comparison with other nondestructive evaluation methods it has the advantage of providing global information. Another advantage is the fact that it is a passive method, and therefore, only loading of the structure is required for the generation of data. As aforementioned, acoustic emission depends on load history, and this should be considered in the analysis of the data.

Structural health monitoring is an area of active research and the previously explained points discuss common and useful possibilities. Recommendations for particular monitoring projects will depend on many factors including the time period to be monitored, the investment to be made in the monitoring system, and the type of structure.

ACRONYMS, ABBREVIATIONS, & SYMBOLS

ae = acoustic emission

d = deflectometer

dB = decibel

Hz = hertz

LVDT = linear voltage displacement transducer

micro-strain = engineering strain x 10^{-6}

pico = 1 x 10^{-12}

pVs = pico-volt*second

sg = strain gauge

t = target

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