

Load Rating Using an In-Service Monitoring System

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SEPTEMBER 2007

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Executive Summary

The goal of this project was to investigate the use of in-situ strain data to directly calculate load ratings for bridges. By monitoring a network of bridges and by collecting new data sets every two years, bridge owners will be better able to track the health of their bridges.

The original load rating for a bridge is determined using a numerical analysis based on as-built properties. Since numerical models often incorporate conservative assumptions regarding bridge behavior, the load ratings that result from the numerical analysis are often more conservative than they would be if they were based on actual bridge response data.

Adding a quantitative aspect to the bridge inspection process would be helpful in more precisely indicating the capacity of a bridge. Measures of actual live load stresses induced by ambient traffic would provide inspectors with quantitative data that would indicate the condition of the bridge and could augment visual evaluations, which are primarily qualitative.

For this project, which was the master's thesis work for a Bridge Center student at the University of Delaware, an In-Service Bridge Monitoring System (ISBMS) was deployed on selected steel bridges in Delaware for two-week periods during regularly scheduled bi-annual inspections. Two advantages of in-service monitoring over a diagnostic load test are that (1) no traffic control is needed to conduct the test, and (2) the data collected provides information about the actual bridge response due to ambient traffic over time.

The six bridges selected were on Interstates 95 and 495, State Routes 7 and 4, Kirkwood Highway, and Newport Gap Pike. These roads were selected because they are major truck traffic routes in the geographical area being studied; the specific bridges were chosen based on a variety of criteria including ease of access.

The ISBMS used in this project was developed at the University of Delaware. The current version consists of a Bridge Diagnostics Inc. (BDI) Strain Transducer and a Snap Shock Plus M4. The BDI gauge is mounted to the bridge using C-clamps, and the Snap Shock Plus collects strain events measured by the BDI.

The student's thesis, which is organized as follows, represents the report for this project:

- **Chapter 1** describes the problem, provides highlights of prior work in the area, and identifies the contributions made by this research.
- **Chapter 2** describes the system used and presents the bridges that were monitored.
- **Chapter 3** presents the data collected for each of the bridge tests and describes the test details.

- **Chapter 4** presents an analysis of the data that was collected and used to develop load ratings for each of the bridges in the study.
- **Chapter 5** discusses details of a novel peak gauge, including the advantages of the gauge and the results of the tests conducted.
- **Chapter 6** discusses future testing and implementation of the described system and makes recommendations for future work.

The following summarizes the conclusions of the project:

- The two-week rating factors derived in this project are generally three to eight times greater than the 50-year rating factors traditionally used. It is believed that the projected two-year rating from this data would lead to an increased rating factor in bridges. Collecting data on a network of bridges every two years would provide not only a better idea of how certain bridges are aging through use but also a comprehensive view of trends in traffic and truck travel in a specific area.
- Using the peak gauge to collect a two-year peak stress would allow the calculation of a two-year rating without projection. One assumption that was made in this project was that the girder being monitored was the site of the highest stress in the bridge. This assumption may need to be corrected for in the future, especially if a two-year rating were to be determined directly from collected data.

The work carried out in this project was the first phase of a project that is being continued by another grad student. Future work will involve incorporating the peak gauges into the ISBMS data collection, enabling prediction of ratings for longer periods of time.

At this time, it is unknown whether the stress data points yielded by the ISBMS represent multiple trucks crossing the bridge at the same time or one heavy truck crossing it alone. To study how the loads that cause the stresses compare to the loads applied in the analysis done by BRASS, Weigh in Motion (WIM) data could be compared to the stresses seen during a certain time period. This would help identify the average weight of truck crossing the bridge and help correlate the weight of truck to the stress in the bridge.

**LOAD RATINGS USING AN IN-SERVICE
BRIDGE MONITORING SYSTEM**

by

Briana O. Brookes

A thesis submitted to the Faculty of the University of Delaware in partial fulfillment of the requirements for the degree of Master of Degree in Major

Spring 2007

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ABSTRACT

This thesis investigates the use of in-situ strain data to directly calculate load ratings for bridges. Six bridges were monitored using an In-Service Bridge Monitoring System (ISBMS) for two week periods. The data collected is presented, along with the resulting load ratings for each bridge. The load ratings calculated using the collected data are compared to ratings computed using the standard practice. In addition to yielding higher load ratings, the ISBMS system has great potential to be used to understand changes in bridge condition over time. By monitoring a network of bridges, and by collecting new data sets every two years, bridge owners will be better able to track the health of their bridges. Also included in this thesis is the testing and evaluation of a peak gauge that can be used for long term strain monitoring of bridges. This system mechanically stores the peak strain experienced by the system over the deployment period. The advantages of the peak gauge are presented, as well as the use of the collected data in calculating load ratings for bridges.

Chapter 1

INTRODUCTION

1.1 Problem Description

To insure the safety of the traveling public, bridges are visually inspected every two years. Data collected during those inspections are used to help track the “health” and capacity of the bridge. If the inspections uncover bridge deteriorated that may affect the way the bridge is able to carry loads, the load rating for the bridge would be reassessed.

The original load rating for a bridge is determined using a numerical analysis of the bridge that is based on the as-built properties of the bridge. Since numerical models of a bridge often incorporate conservative assumptions regarding bridge behavior, the load ratings that result from the numerical analysis are often lower (i.e. more conservative) than what they might have been if actual bridge response data was used to rate the bridge.

Adding a quantitative aspect to the bridge inspection process would be helpful in more precisely indicating the capacity of a bridge. Measures of actual live load stresses induced by ambient traffic would provide inspectors with quantitative data that would indicate the condition of the bridge and could enhance their visual (primarily qualitative) evaluation.

One way to get quantitative data on the condition of a bridge is to conduct a diagnostic load test using a truck of known weight and configuration. The response

of the bridge to the loaded truck, primarily the measured strains, are used to rate the bridge. Diagnostic load tests require a number of sensors be deployed, and also require traffic control during the test. As a result, this type of test is not routinely performed.

Many different strain collection methods and load rating techniques have been developed over the years. The goal of this research was to use quantitative in-service data to aid the inspection and load rating process of bridges. In-service strain data is caused by ambient traffic, and does not require traffic control. Using the data collected, a methodology is developed in which the data is utilized to load rate the bridge. The ability to use in-service strain data with knowing the associated truck weight or truck configuration could potentially change the way bridges are rated and what load limits are set.

Using two weeks of data collected during biannual inspections of bridges, a two-week rating for each bridge in the study is developed. This two-week rating is compared to the 50-year rating which is typically computed for bridges based on design capacities. This research also explores the possibility of collecting an in-service peak stress over a two year period which would enable one to directly determine a two-year load rating. Predicting the next two-year load rating using two weeks of in-service data collected once every two years, combined with a two-year peak value, seems to be a practical and accurate way of rating bridges. Collecting a database of two week snapshots of strain data for selected bridges can also be helpful in monitoring the health of those bridges, as well as providing insight into how traffic patterns and deterioration might be changing for a network of bridges.

1.2 Prior Work

The area of in-service data collection on bridges has been slowly developing over the past few years. New sensor systems have been used to capture many different kinds of events. The development of a system which can be easily deployed and give meaningful data without a great deal of post collection processing is very important. The In-Service Bridge Monitoring System (ISBMS) used in this project was developed during previous research. The first generation system was developed by Eric Holloway and is reported in his 1999 thesis “A Long-term Monitoring System for Highway Bridges” (Holloway, 1999). A second generation ISBMS, which allows data to be downloaded remotely and other data collection options, was developed by Daniel Howell and is reported in his 2003 thesis “Development of a Wireless In-service Bridge Monitoring System” (Howell, 2003). The system developed by Holloway was modified for easier deployment, and this system is the one used in this research.

1.3 Contributions Made By This Research

For this project, the ISBMS developed previously (with minor alterations) was used to collect in-service data from six bridges. The data collected in this study was analyzed and used to determine bridge load rating. In addition to this work, application of a novel peak strain gauge that can be used to collect long term in-service data was explored. The peak gauge mechanically stores the maximum strain experienced by the transducers, and therefore requires no power (except when a reading is taken of the stored value). This gauge only records the maximum strain seen by a member over the time deployed, and can be useful in collecting the long term peaks needed for load rating predictions.

1.4 Organization

During this research, one system of data collection was used to monitor six bridges. In Chapter 2, the system is described, and the bridges that were monitored are presented. In Chapter 3, the data collected for each of the bridge tests is presented, as are the details of the tests. In Chapter 4, the data collected is analyzed and then used to develop load ratings for each of the bridges in the study. In Chapter 5, details of a novel peak gauge are discussed, and the advantages of this peak gauge, as well as testing results, are presented. Finally, a discussion of future testing and implementation of the described systems and other studies that might be investigated in the future are presented in Chapter 6.

Chapter 2

IN-SERVICE BRIDGE MONITORING SYSTEM

2.1 In-Service Program

The goal of this project is to enhance bridge management through the collection and utilization of in-service strain data. This will be accomplished through the use of an In-Service Bridge Monitoring System (ISBMS) that will be deployed on selected Delaware bridges during their regularly scheduled bi-annual inspections. Two advantages of in-service monitoring over a diagnostic load test are (1) no traffic control is needed to conduct the test and (2) the data collected provides information about the actual bridge response due to several days/weeks of ambient traffic. Statistical evaluation of the data will provide valuable input regarding both bridge condition and the nature of local truck traffic. In this step of the project protocols for deploying the system, collecting data, and analyzing the data have been developed and will be explained. The goal is to incorporate the data collection into the current bi-annual inspection process and to utilize the data to enhance management of bridge networks.

2.2 Bridge Selection

In order to select the bridges used for this study a few criteria were selected to narrow the number of bridges down to a set to consider. There are over 1400 bridges in the Delaware Department of Transportation's bridge inventory. Steel bridges were selected for this bridge set because the system being used mounted easily to steel, and the test data obtained of steel bridges tends to be more reliable than that of concrete bridges. For accessibility reasons bridges close to the campus of the University of Delaware were considered; bridges which carried or crossed over railways were excluded from this bridge selection. The bridges chosen for the study were not culverts, so that more National Bridge Inventory (NBI) data was available on these bridges. This part of the study focused on bridges which were due for inspection in the year 2006 which eliminated approximately half of the bridges in Delaware's bridge inventory. Bridges due for inspection in 2007 will be looked at in future testing. Bridges were selected on: Interstate 95, Interstate 495, Kirkwood Highway, Newport Gap Pike, and State Routes 7 and 4. These routes were selected because they were seen as major truck traffic routes in the geographical area being studied. Six bridges were selected for this part of the study and were chosen due to ease of access and location on routes being carried. Being able to access the bridges with ladders and not needing snooper trucks or boats was important so that Delaware Department of Transportation was not needed to retrieve the system after it was deployed.

After site visits the final six bridges were selected which best fit all the criteria. There were two bridges on Interstate 95, one north of, and one south of, the Interstate 495 interchange. One bridge on 495 was selected. One bridge was selected which carries State Routes 7 and 4 together north of the State Route 7 and 1 interchange. One bridge on Kirkwood highway south of Limestone Road was selected.

One bridge on Newport Gap Pike, near the intersection of Kirkwood highway and Newport Gap Pike was also chosen.

2.3 Bridge Details

The details of the six bridges are as follows. A summary of the bridges is presented in **Error! Reference source not found.** A map of the bridge locations can be seen in **Error! Reference source not found.** For all the bridges the superstructure ratings ranged from 5 to 8 and the Average Daily Traffic (ADT) was between 23700 and 38900, with 6 or 15% trucks.

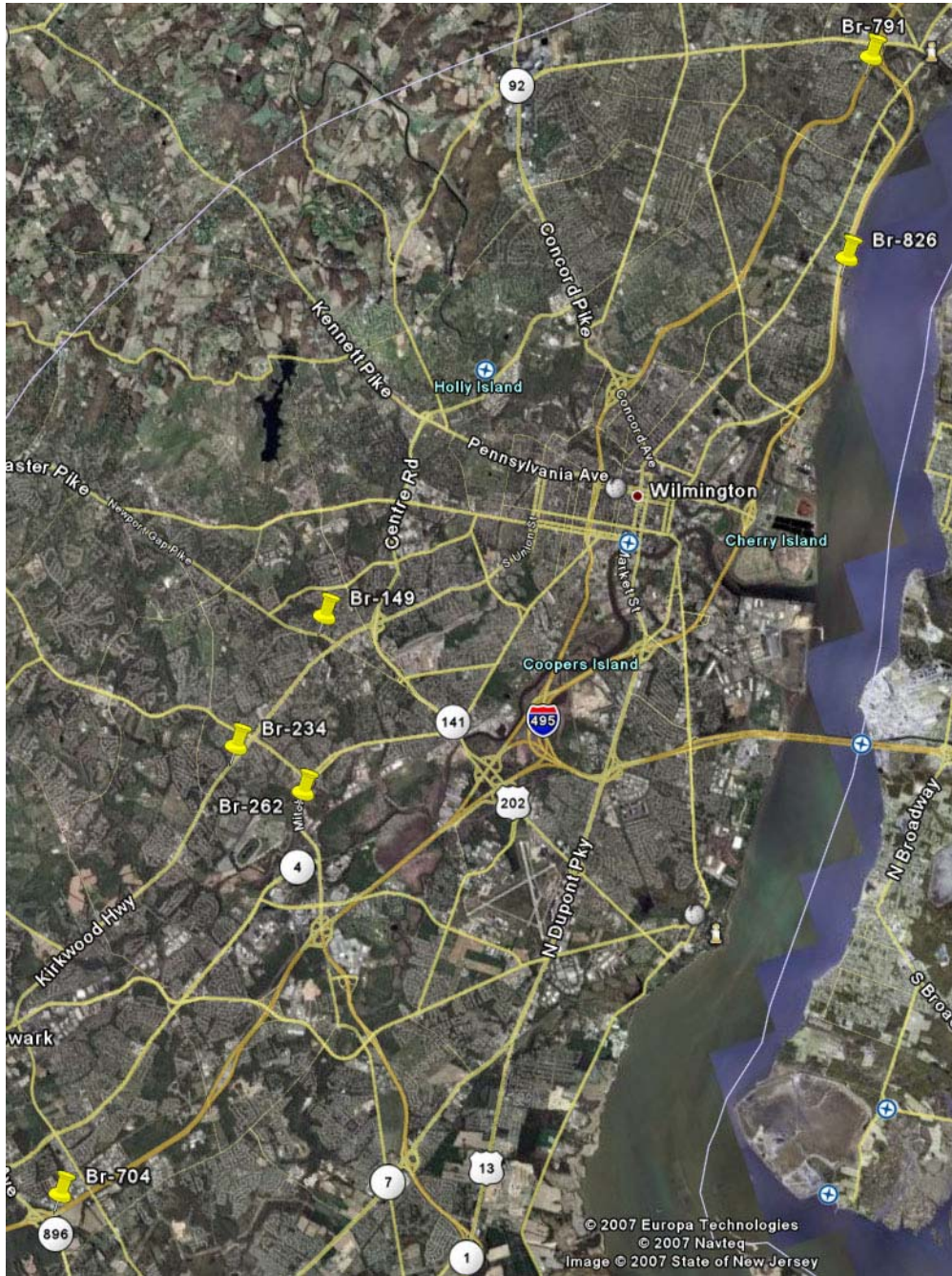


Figure 2.1 Bridge Locations Map

2.3.1 Bridge 1-791 N

The bridge carries Interstate I-95 North into Pennsylvania. The bridge carries three lanes of traffic. There are six rolled steel girders in the cross-section and three continuous spans. The approach span was the one used for this in-service testing. The approach span is 35 feet and the gauge was placed at mid-span on girder number three (see Figure 2.2 and Figure 2.3).

2.3.2 Bridge 1-149

The bridge carries Newport Gap Pike over Red Clay Creek. The Bridge carries 2 lanes of traffic, one in each direction. There is one simple span which is 80 feet long and contains 11 rolled steel girders in the cross-section. The gauge for testing was placed at mid-span on girder number 4 (see Figure 2.4 and Figure 2.5).

2.3.3 Bridge 1-826 N

The bridge carries three lanes of Interstate 495 North over Stoney Creek. There are three continuous spans and 7 rolled steel girders in the cross-section. Girder number 4 on the back span was used for testing. The gauge was mounted at mid span of the girder (see Figure 2.6 and Figure 2.7).

2.3.4 Bridge 1-234

The bridge carries four lanes of Kirkwood Highway, SR 2, over the Mill Creek. The bridge has one simple span of 70 feet and has 13 rolled steel girders in the cross-section. The gauge was placed at mid-span on girder number 3 (see Figure 2.8 and Figure 2.9).

2.3.5 Bridge 1-262 S

The bridge carries two lanes of SR 7 and SR 4 over White Clay Creek. The bridge has 5 continuous spans and 7 steel plate girders in the cross-section. The back span of the bridge was used which spans 90 feet. The gauge was placed at mid-span of girder number 3 (see Figure 2.10 and Figure 2.11).

2.3.6 Bridge 1-704 S

The bridge carries I-95 South to the Delaware/Maryland boarder. The bridge carries 4 lanes of traffic plus the 1B exit ramp lane (see Figure 2.12 and Figure 2.13). The bridge has 3 simple spans. The two approach and back spans are 24'-6" long and the main span is 62'-6". There are 13 rolled steel girders in the cross-section.

Bridge #	Year Built	Facility Carried	Facility Crossed	Load Rating	Superstructure NBI rating	# of Lanes	ADT/ Truck %	# of Girders	Simple/ Continuous	# of Spans	Span used/ length of span	Girder used/ location of gauge on girder
1-791 N	1966	I-95 NB	Darley Rd	0.95	7	2	26955 / 15	6	C	3	1 / 35'-9 1/8"	3 / mid-span
1-149	1989	Newport Gap Pike	Red Clay Creek	1.95	7	2	23750/ 6	11	S	1	1 / 80'-0"	4 NB / mid-span
1-826 NB	1972	I-495	Stoney Creek	1.22	8	3	33632/ 15	7	C	3	3 / 70'-0"	4 / mid-span
1-234	1949	Kirkwood Highway (SR2)	Mill Creek	1.2	5	4	38889/ 6	13	S	1	1 / 62'-4"	3 SB / mid-span
1-262 S	1984	SR 7 / SR 4	White Clay Creek	1.89	7	2	28237/ 5	7	C	5	5 / 90'-0"	3 / mid-span
1-704 Test 1	1962	I-95 SB	Christina Creek	0.96	7	5	38387/ 15	13	S	3	1 / 25'-1 3/4"	7 / mid-span
1-704 Test 2	1962	I-95 SB	Christina Creek	0.96	7	5	38387/ 15	13	S	3	1 / 25'-1 3/4"	5 / mid-span

Table 2.1 Bridge Description and Gauge Layout

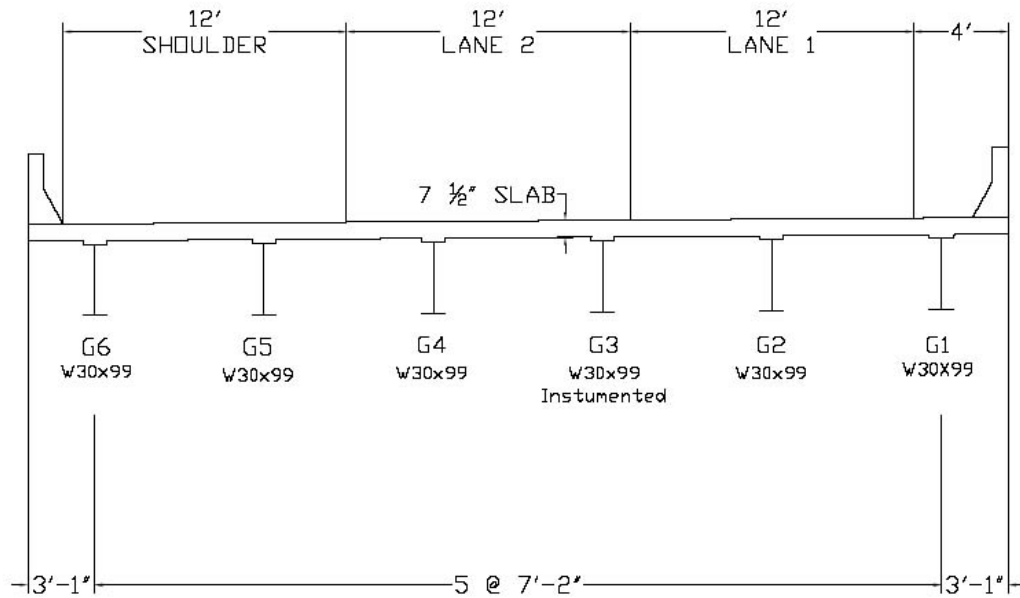


Figure 2.2 Bridge 1-791 Cross-Section of Approach Span

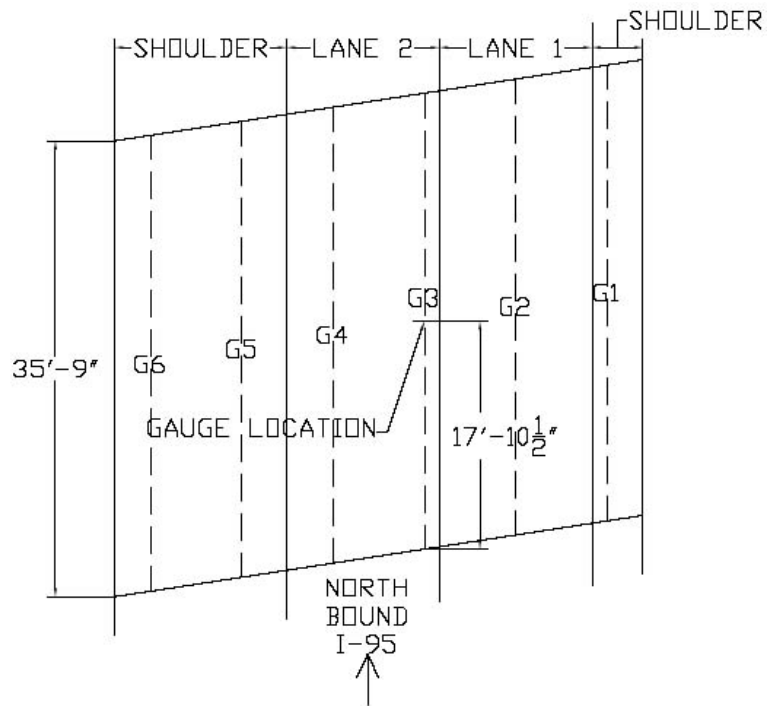


Figure 2.3 Bridge 1-791 Plan View of Approach Span

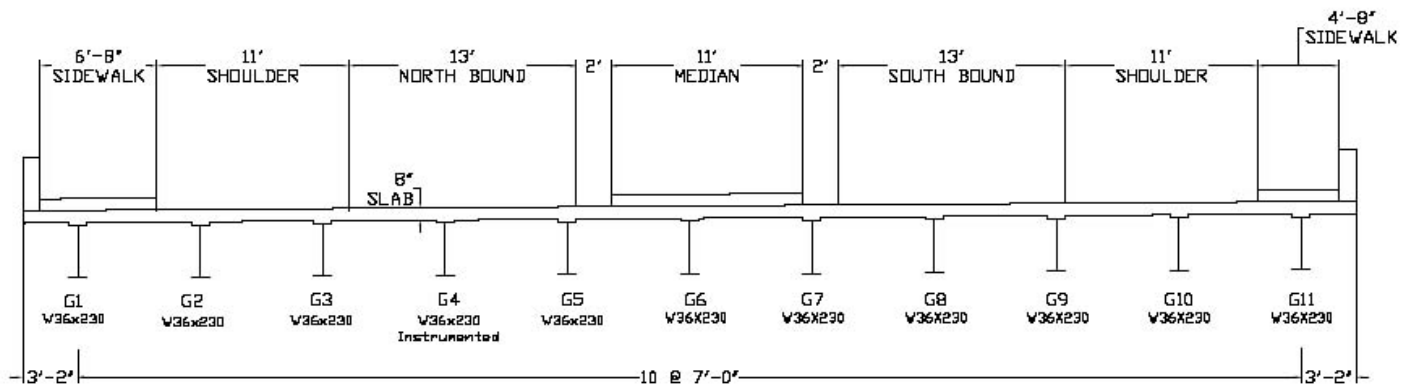


Figure 2.4 Bridge 1-149 Cross-Section

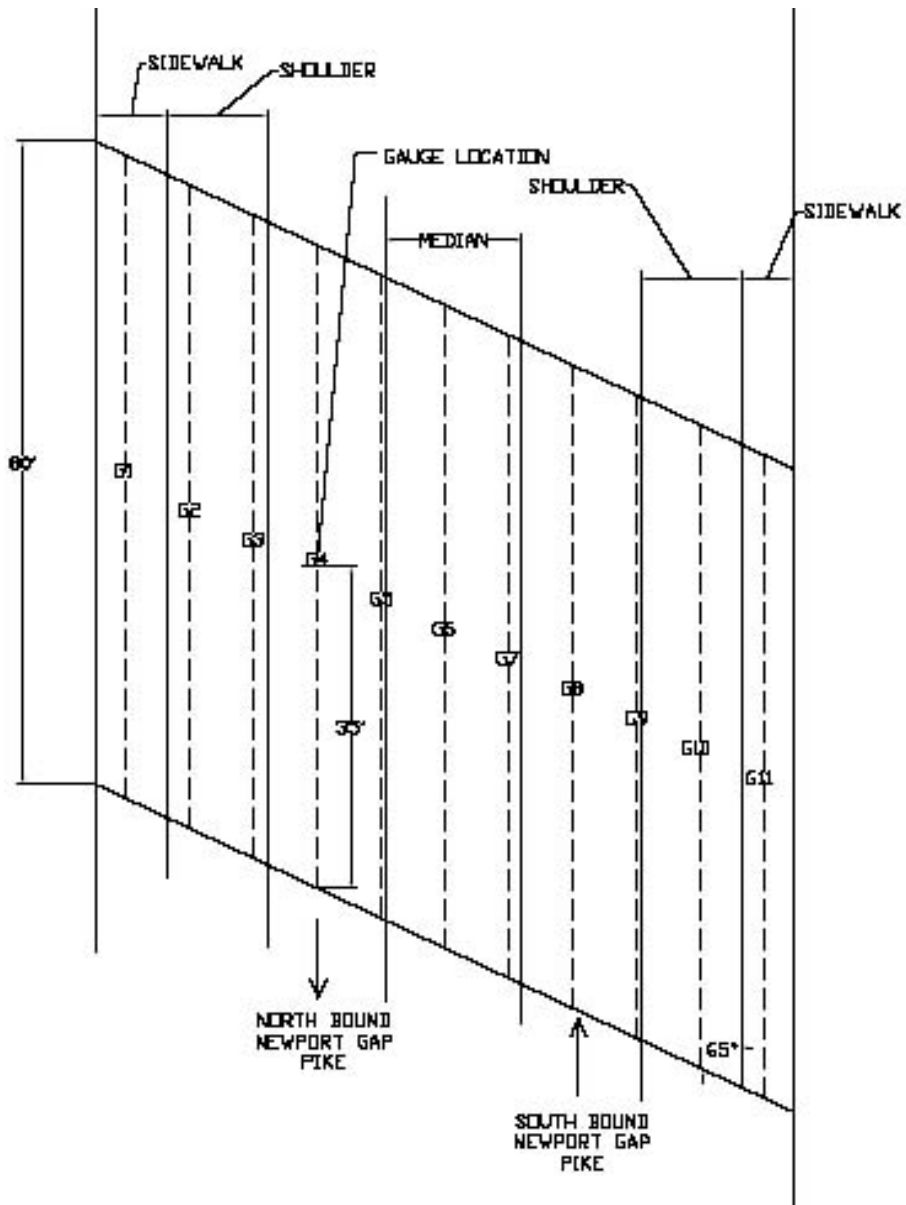


Figure 2.5 Bridge 1-149 Plan View

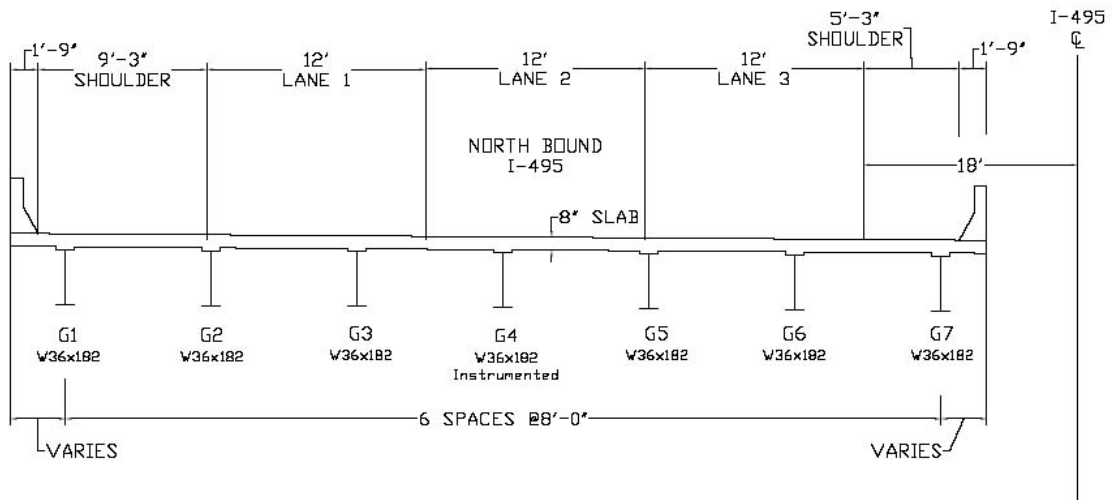


Figure 2.6 Bridge 1-826 Cross-Section at Mid-span of Span #3

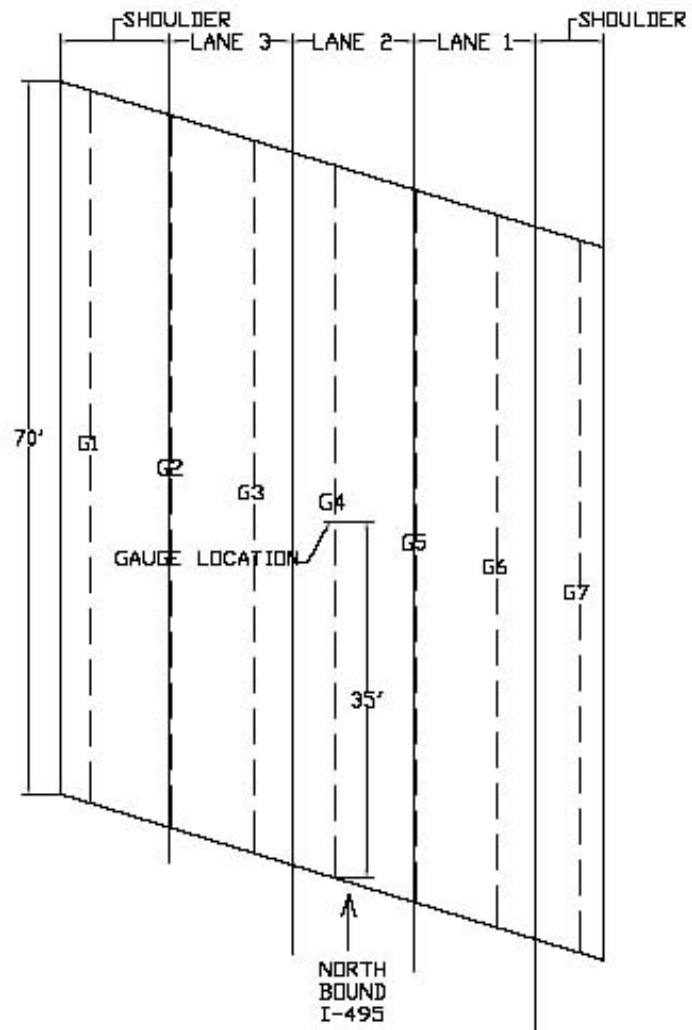


Figure 2.7 Bridge 1-826 Plan View of Span #3

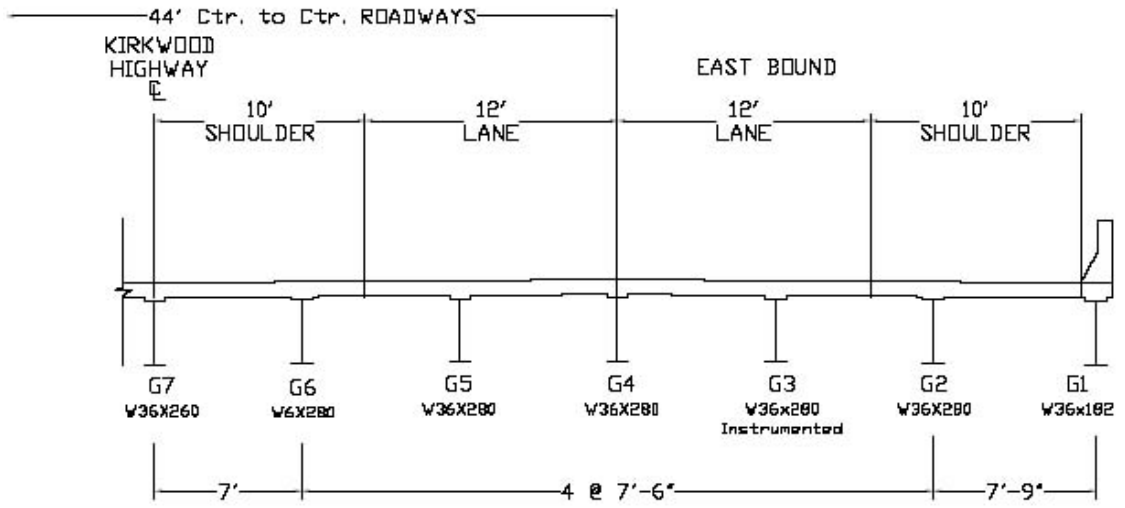


Figure 2.8 Bridge 1-234 Cross-Section

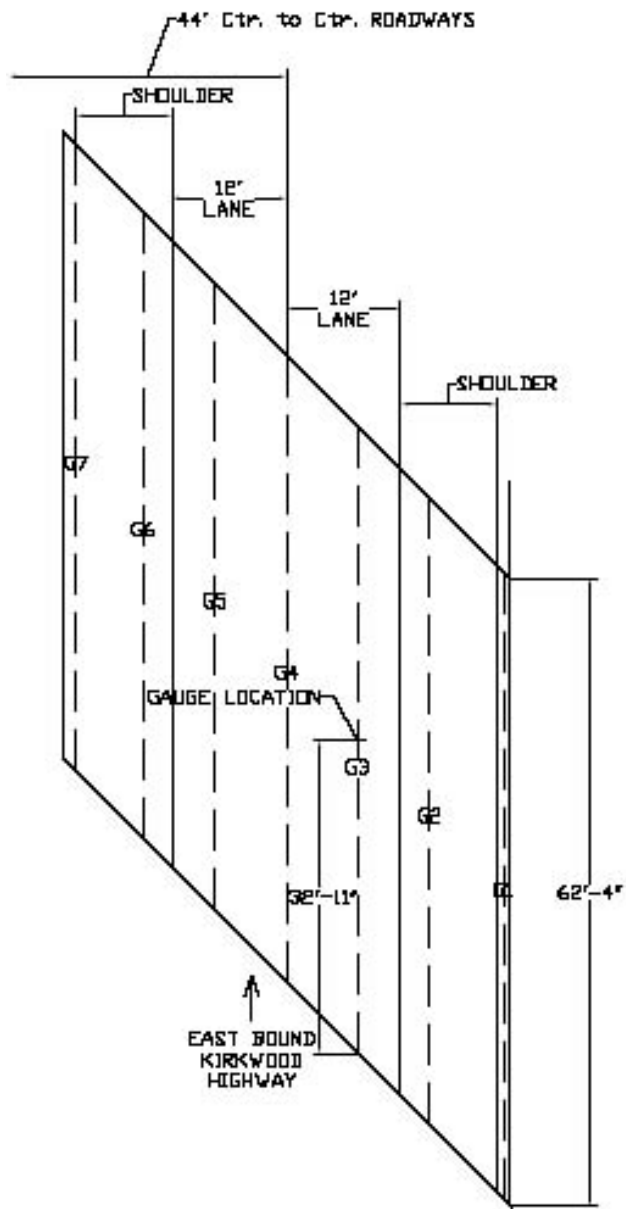


Figure 2.9 Bridge 1-234 Plan View

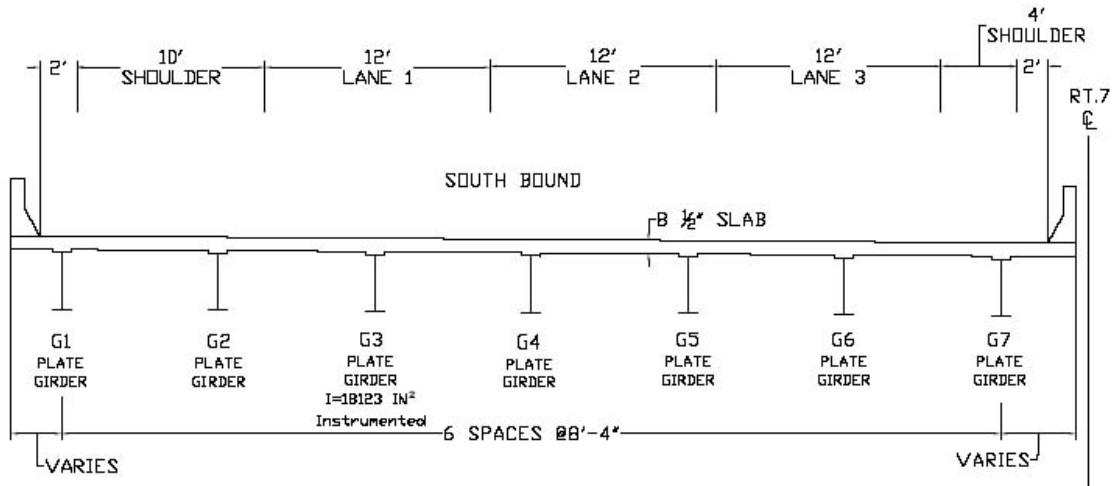


Figure 2.10 Bridge 1-262 Cross-Section at Mid-span of Span #5

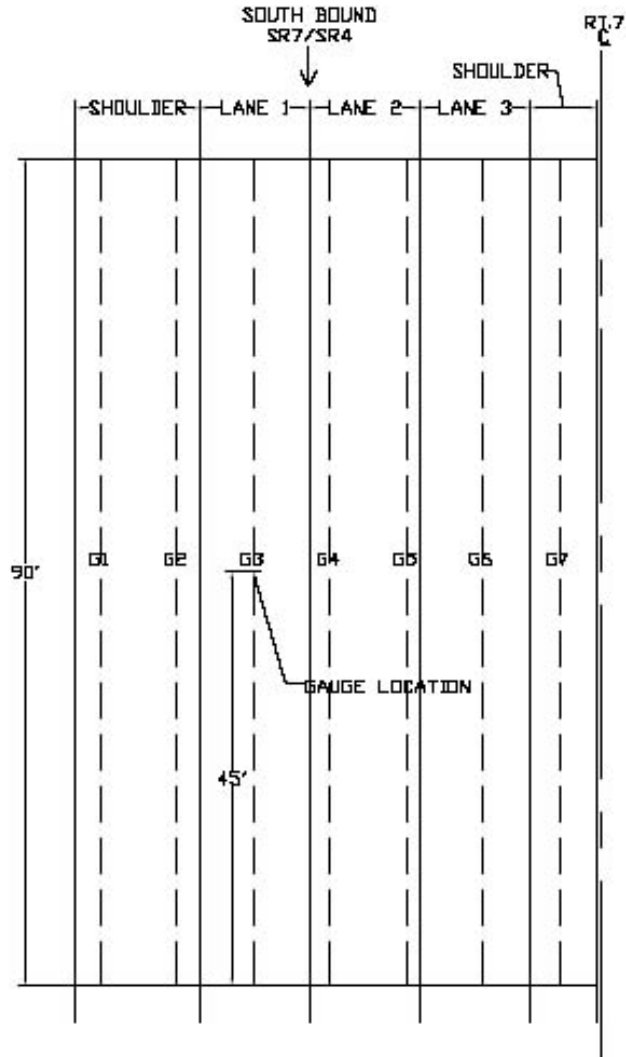


Figure 2.11 Bridge 1-262 Plan View of Span #5

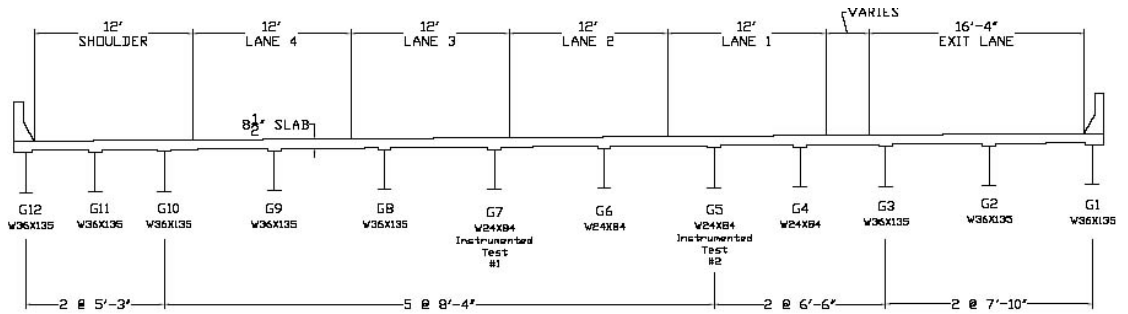


Figure 2.12 Bridge 1-704 Cross-Section of Approach Span

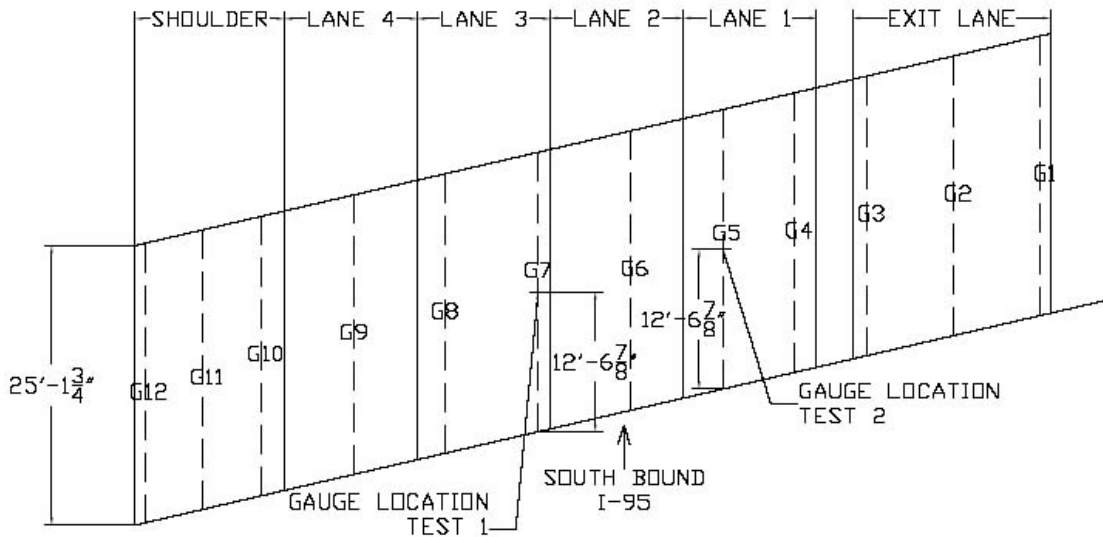


Figure 2.13 Bridge 1-704 Plan View of Approach Span

2.4 ISBMS Description

The ISBMS used in this study was originally presented by Eric Holloway in his masters thesis “A Long-Term Monitoring System For Highway Bridges”. The ISBMS has changed a bit since this initial design. The hardware is the same as described by Holloway but the system has a few new additions that make setup easier. Two changes that were made in the electronics are that the power supply to the strain transducer, which in the original system consisted of six regular D cell batteries, was replaced with a rechargeable 6 volt battery and the voltage regulator was removed. The whole system was repackaged and can be seen in Figure 2.14 and Figure 2.15. The ISBMS operates the same as presented by Holloway except for the shunt calibration and zeroing processes which make installation easier. This section will review set up of the system and will give specifics for installation and guidelines for future deployment.

The two main parts to the ISBMS are a Bridge Diagnostics Incorporated (BDI) Strain Transducer and the Snap Shock Plus M4. The BDI gauge is mounted to the bridge of interest by using C-Clamps and the Snap Shock Plus collects strain events measured by the BDI. Details of setting up the Snap Shock Plus for data collection is in the following section.



Figure 2.14 ISBMS with Gauge



Figure 2.15 ISBMS Open View

2.5 System Set-up

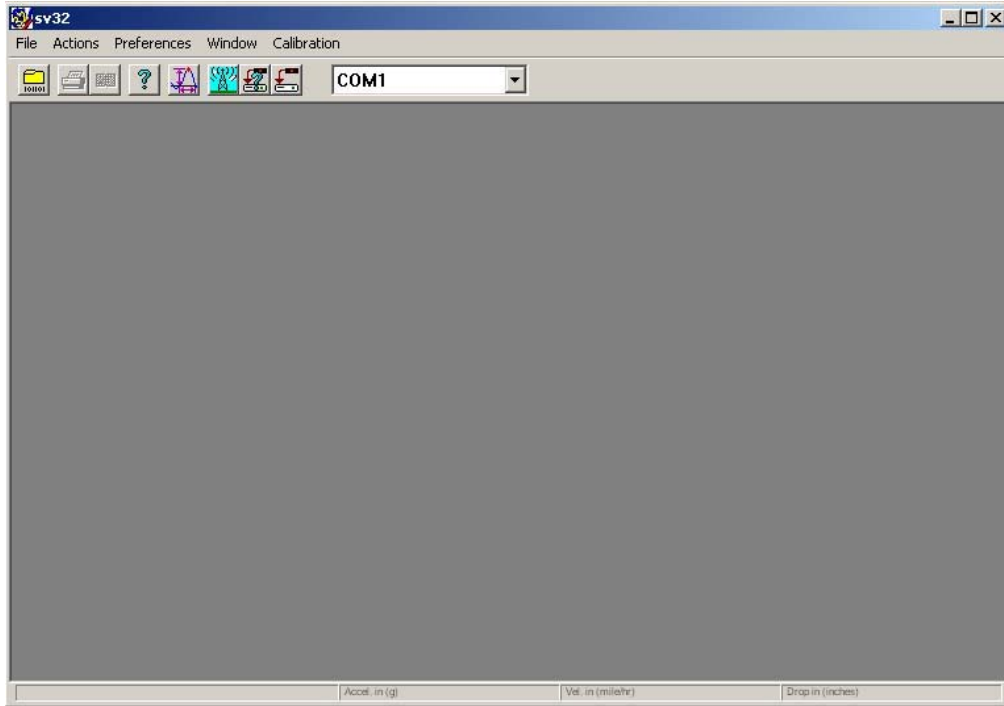
Prior to installation the system is set up with the testing parameters. The ISBMS is set up prior to going to the field with a laptop computer. The program used to set up the ISBMS is SV32 which communicates with and programs the Snap Shock Plus data collector. The following process is used to program the Snap Shock Plus data collector and make the ISBMS ready for installation.

There are a few parameters that need to be decided before the ISBMS is put into the field and even before the ISBMS is programmed. The strain threshold or trigger for the bridge needs to be selected. This strain threshold is the parameter that tells the system the level of strain above which the ISBMS should record. If the threshold is too high, little or no events will be recorded by the ISBMS. If the threshold is too low, there is a possibility that the system memory will fill with data and be unable to collect events for the full length of the collection period. Truck traffic volume and bridge geometry can help in the decision for what the threshold value should be set at. For most of the tests presented in this document the threshold was $50\mu\epsilon$. For some bridges this did not yield sufficient data and the threshold was lowered in order to capture more events. This threshold will be programmed into the ISBMS during the set up programming. The BDI gauge for the system must be chosen prior to installation so the correct shunt calibration can be preformed on the system so the system will be able collect data correctly. This calibration can be done easily and the process will follow. The shunt calibration only needs to be preformed once for a single gauge in order to obtain the calibration constant for the Snap Shock Plus.

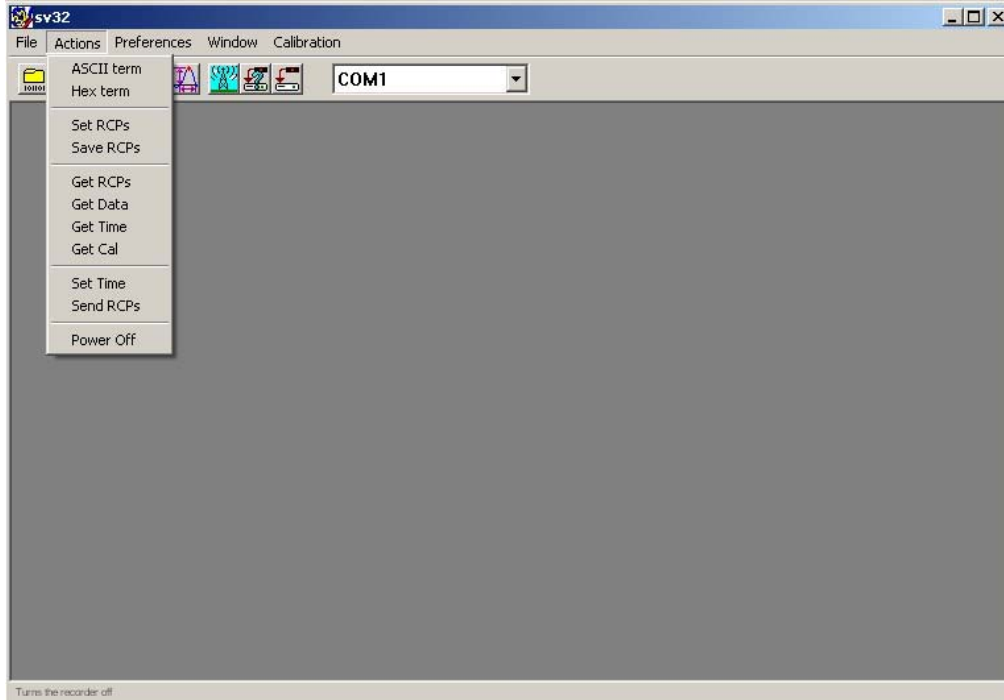
2.6 Snap Shock Plus Programming

First one must connect the system to the computer with the serial cable. Next one must cycle the Snap Shock Plus system to the Communications 1 mode by holding the mode button until a single red light comes on (see Figure 2.15). In this mode the system can communicate with the computer and is able to be programmed. The following steps are taking to program the system,

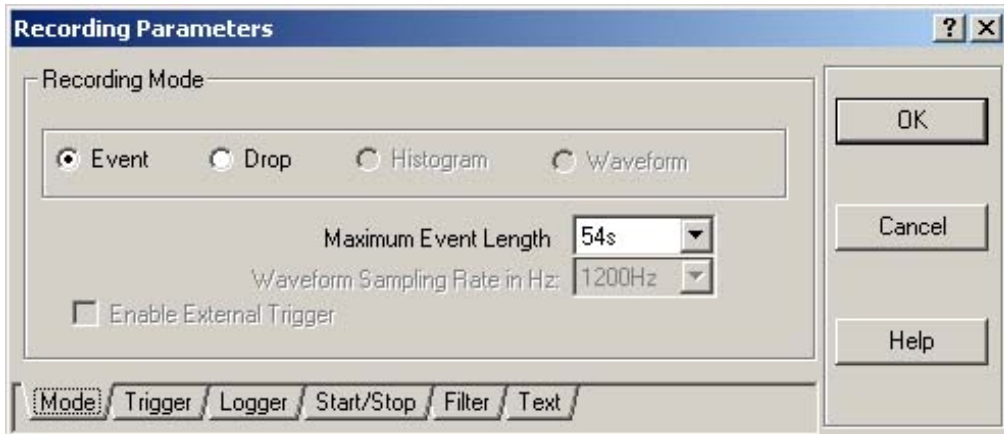
1. Open the sv32 program.



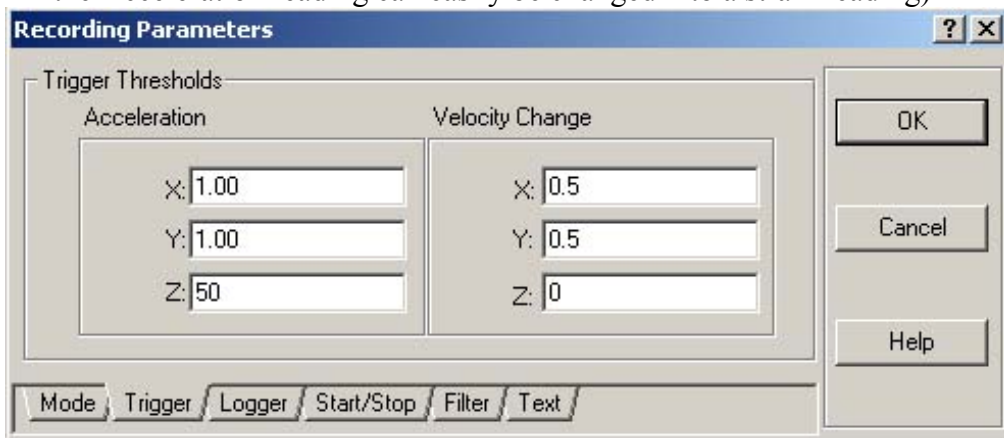
2. Under the Actions menu select Set RCP's



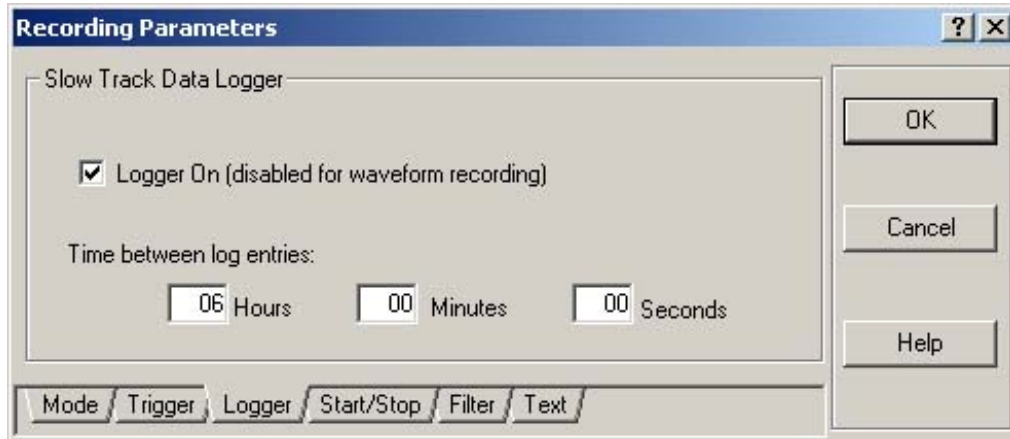
3. Chose Event Mode on the mode tab



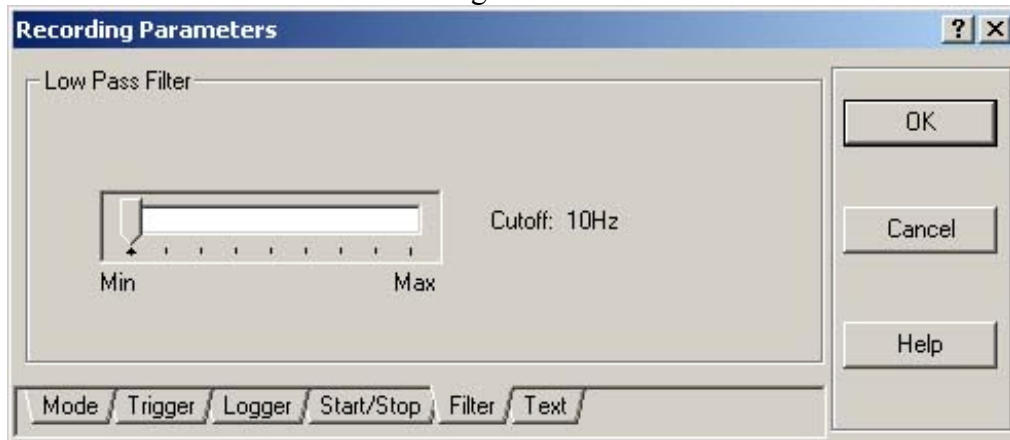
- Click the Trigger Tab and input the strain threshold value in Acceleration Z: value and set the Velocity Change Z: value to zero. (Acceleration is what the Snap Shock Plus is designed to measure but by using the shunt calibration the Acceleration reading can easily be changed into a strain reading)



- Set the temperature logger options on the Logger tab. For the tests performed, the temperature logger was set to take 4 measurements each day (every 6 hours)



6. Set Filter to the lowest setting with the Filter Tab



7. Use the Text tab to label the test data for later use
8. Under Actions Menu select "Set Time" to synchronize the time on the system with real time
9. Under Actions Menu select "Send RCP's"
10. Repeat step number 8

After completing the set-up turn the system off by cycling it to the position where both the red and green lights come on. The system is now ready to be installed in the field.

2.7 Field Installation

Once in the field the BDI gauge is mounted to the beam using C-Clamps at the desired location and then connected to the ISBMS. The system then needs to be zeroed. To zero the system the following steps are used:

1. Use a multimeter to measure the voltage between the S+ and S- terminals (see Figure 2.15)
2. If the voltage is more or less than 0.000, turn the Null Pot until a zero reading is indicated.
3. Once the system is zeroed, lock the Null Pot by moving the lock switch clockwise

Once the system is zeroed it can be turned on. To turn on the system hold the Mode button until the single green light turns on. The green light should stay on for a few seconds and then turn off. This means the system is now ready to collect data. When an event occurs, i.e., a vehicle with sufficient weight to cause a strain in excess of the specified threshold crosses the bridge, the green light will come on and then go off. To ensure the system zeroed correctly and is on, press the mode button again until a light comes on and then release. If the green light comes on and goes out the ISBMS is on and ready to collect data. If the green and red lights both come on the ISBMS did not zero correctly and the zeroing steps should be repeated until the system properly zeros. Sometimes the BDI gauge may need to be remounted and the system re-zeroed. If the BDI is clamped too tightly or on an uneven surface where the BDI is pre-strained, the system may not be able to zero and this would require re-mounting the transducer to allow the system to zero. The BDI should be clamped tight enough to the beam so that it will strain with the beam, but not so tight to introduce pre-strain into the transducer at its resting position. After the BDI gauge is installed and the system is correctly set up the system box can then be mounted to the beam in a place that is out of reach of any persons who might be under the bridge. The system box has magnets on the bottom of the box for easy mounting and can easily be mounted to any steel girder.

2.8 Shunt Calibration for New BDI Gauge

For each BDI transducer, prior to deployment, a calibration needs to be performed to get the calibration constant used in the set up of the system. To perform the shunt calibration the following steps should be used.

1. Connect BDI to ISBMS and lay on flat surface
2. Cycle ISBMS to Communications 1
3. Connect Computer to ISBMS through serial cable
4. Open SV32
5. Under Calibration Menu select Get Z counts
6. Repeat step 5 until average Z count reading is close to 2048.
7. Write down average Z count reading
8. Flip Shunt Cal switch ON
9. Repeat step 5
10. Subtract Z count reading from step 7 from reading in step 9
11. Divide result from step 10 by 125
12. Under Calibration Menu select Set Cal
13. Enter result from step 11 in Accel Calibration Constants Z:
14. Click ok
15. Calibration is now complete for specified BDI gauge.

Example of Calibration Procedure:

$$\text{Z-Count reading} = 1953.54 \qquad \text{Shunt Z-Count reading} = 2433.46$$

$$\begin{aligned} \text{Shunt Z-Count reading} - \text{Z-Count reading} &= \text{counts} \\ 2433.46 - 1953.54 &= 479.92 \text{ counts} \end{aligned}$$

$$\text{Counts}/\mu\epsilon = 479.92/125 = 3.839 \text{ counts}/\mu\epsilon$$

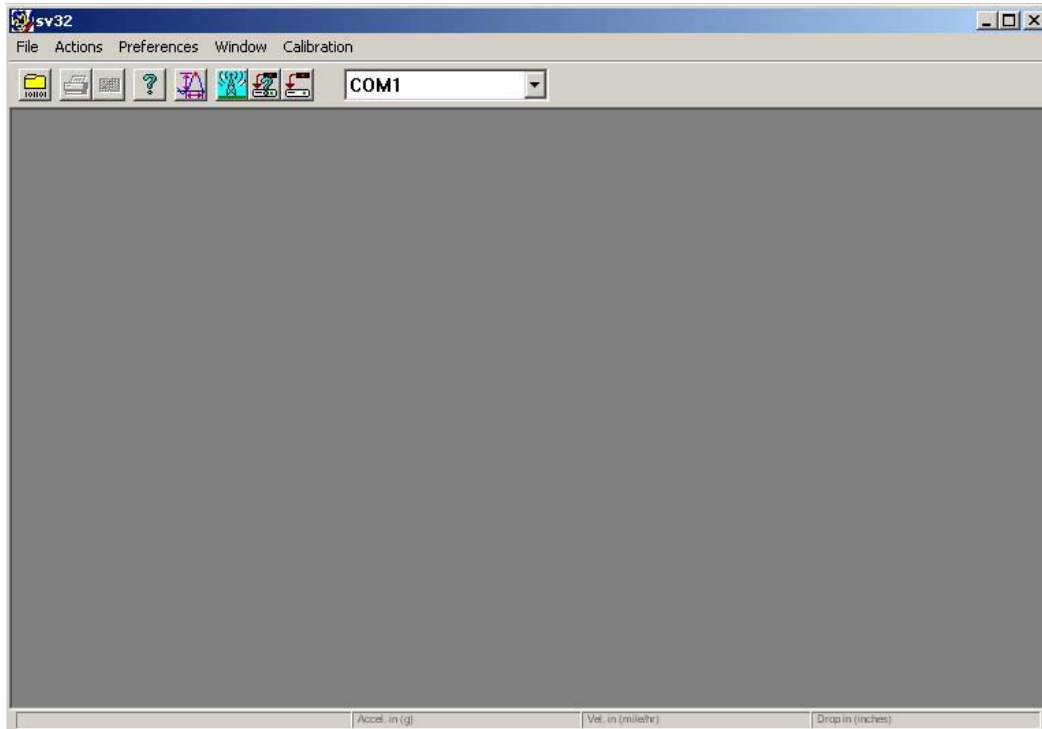
3.839 is the Calibration constant to be used for the gauge in this case.

2.9 Retrieving Data

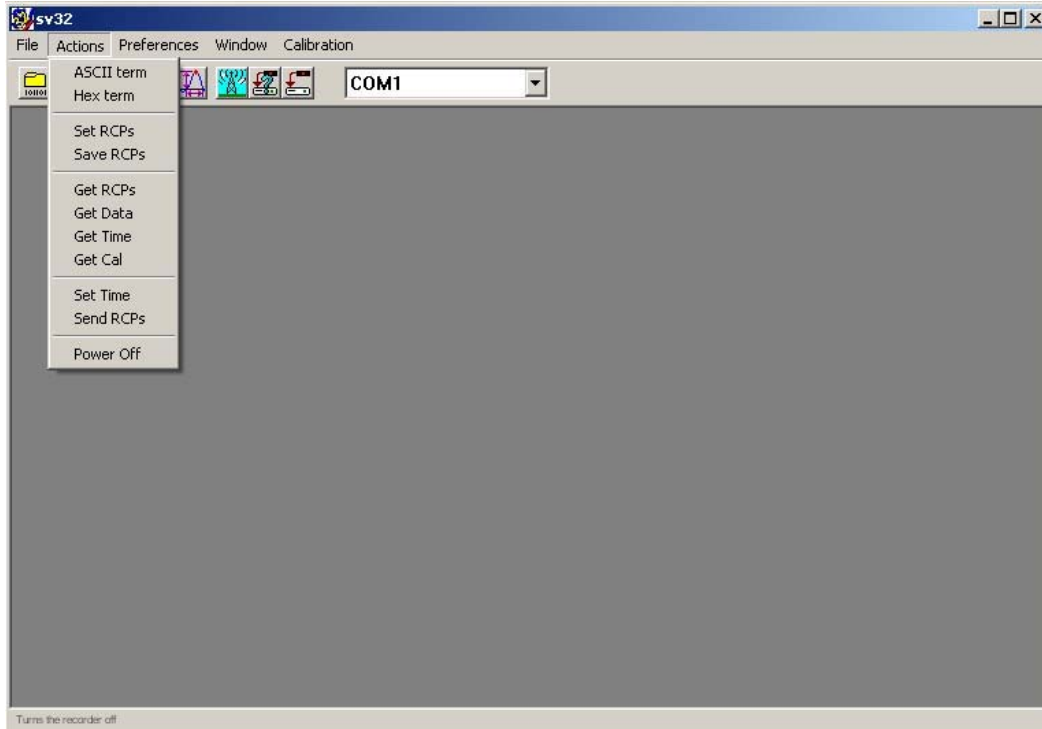
After data is collected the ISBMS can be retrieved simply by pressing the Mode button until both the green and red lights come on. This will turn the ISBMS

system off. At this point the BDI gauge can be unclamped and disconnected from the ISBMS. To download the data, follow these steps:

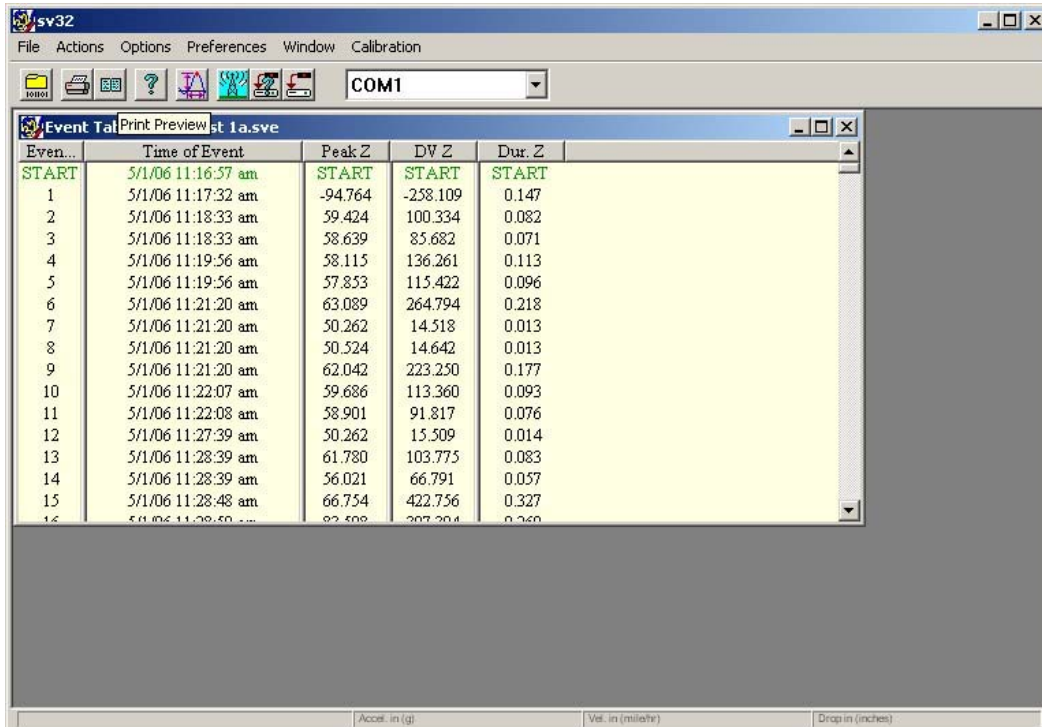
1. Connect the serial cable from the ISBMS to the computer
2. Open SV32



3. Under the Actions Menu select Get Data



4. Save the File to desired location (the data will appear in the frame)



5. The data file can be copied to an Excel file simply by right clicking on the file and selecting copy to clipboard
6. Open Excel and paste the data file directly into spread sheet by right clicking and selecting paste
7. SV32 can present some quick views if Excel analysis is not needed. To produce these views use the Options menu. Select add view to get a time line plot, statistical summary, or even a histogram of events.

For each of the bridges described earlier the ISBMS was installed and approximately two weeks of data was collected. The data collected is presented in Chapter 3 and the data analysis can be seen in Chapter 4.

Chapter 3

IN-SERVICE DATA

3.1 Collected Data

This chapter outlines the data collected from each of the six bridges that were tested. Data was collected on each bridge for approximately two weeks. The location of the gauge and instrumentation details can be seen in Section 2.3. A summary of key data values for all tests done during this study can be seen in Table 3.1.

For each bridge test a time-line of recorded events was created (see Figure 3.1, Figure 3.3, Figure 3.5, Figure 3.7, Figure 3.9, Figure 3.11, Figure 3.13, and Figure 3.14). The time-line plots are plots of the stresses recorded versus the date/time the stress occurred. In the time-line plots the bandedness of the data can be seen. From the bands in the data each day can be seen clearly. The weekends on each time line can also be seen as times where the truck traffic was sparse. To help with identifying the weekend days they have been labeled on each timeline. Histograms of the bending stresses seen in the beams monitored during the tests were also created (see Figure 3.2, Figure 3.4, Figure 3.6, Figure 3.8, Figure 3.10, Figure 3.12, and Figure 3.14). The distribution of the data can nicely be seen in the histogram plots. All of the data seems to have a most of the events near the threshold value then the data tails off quickly as

the stress increases. A summary of the results can be seen in Table 3.1. From Table 3.1 it can be seen that the maximum stresses on the six bridges ranged from 0.98 ksi to 4.71 ksi for the bridges studied in 2006, and the number of events range from 62 to 4204. The average stress for all bridges, except bridge 1-234, is very similar, and ranged from 1.56 to 1.81 ksi.

During testing, normal traffic patterns were altered on two of the bridges due to construction. During testing there was construction that led to the closing of the right lane of Interstate 495, approximately a mile north of bridge 1-826. This could have led to more trucks being in the left lane while crossing the bridge than in a normal traffic situation. Likewise, there was construction on I-95 as bridge 1-704 was tested. During the first test of bridge 1-704 the two right hand lanes were closed prior to the bridge which moved the traffic further to the left than usual. Because of the construction, the gauge was placed on the 7th girder to try to catch as much truck traffic as possible with the shifted traffic pattern. During the second test of bridge 1-704 the two left hand lanes of I-95 were closed prior to the bridge. To capture the most truck traffic the sensor was mounted on girder 5. (See Figure 2.12 and Figure 2.13) Looking at the data and the number of events captured in test 2 on 1-704, it is suspected that trucks may have not stayed in the right hand most lane in the construction. If the truck traffic was more in the second lane they were not over the girder selected as often as free flowing truck traffic.

Bridge 1-234 saw very few events for the time it was monitored. The cause of the few events is thought to be due to the location of the bridge. The bridge is located south of Limestone road (Route 7). From observations of this bridge it can be seen that not many heavy trucks travel past the Limestone road intersection on

Kirkwood highway. This low number of events shows that some trucks do travel the road but not as heavy trucks and not as frequently as some of the other roads that were chosen for testing.

Bridge	Min Stress (ksi)	Max Stress (ksi)	Average Stress (ksi)	Number of Events
1-149	1.45	4.2	1.69	1892
1-234	0.44	0.98	0.51	62
1-791	1.02	3.14	1.29	2889
1-262	1.45	3.47	1.76	754
1-826	1.45	4.71	1.69	4204
1-704 (test 1)	1.45	3.02	1.59	1543
1-704 (test 2)	1.45	2.83	1.56	382

Table 3.1 Summary of Collected Data

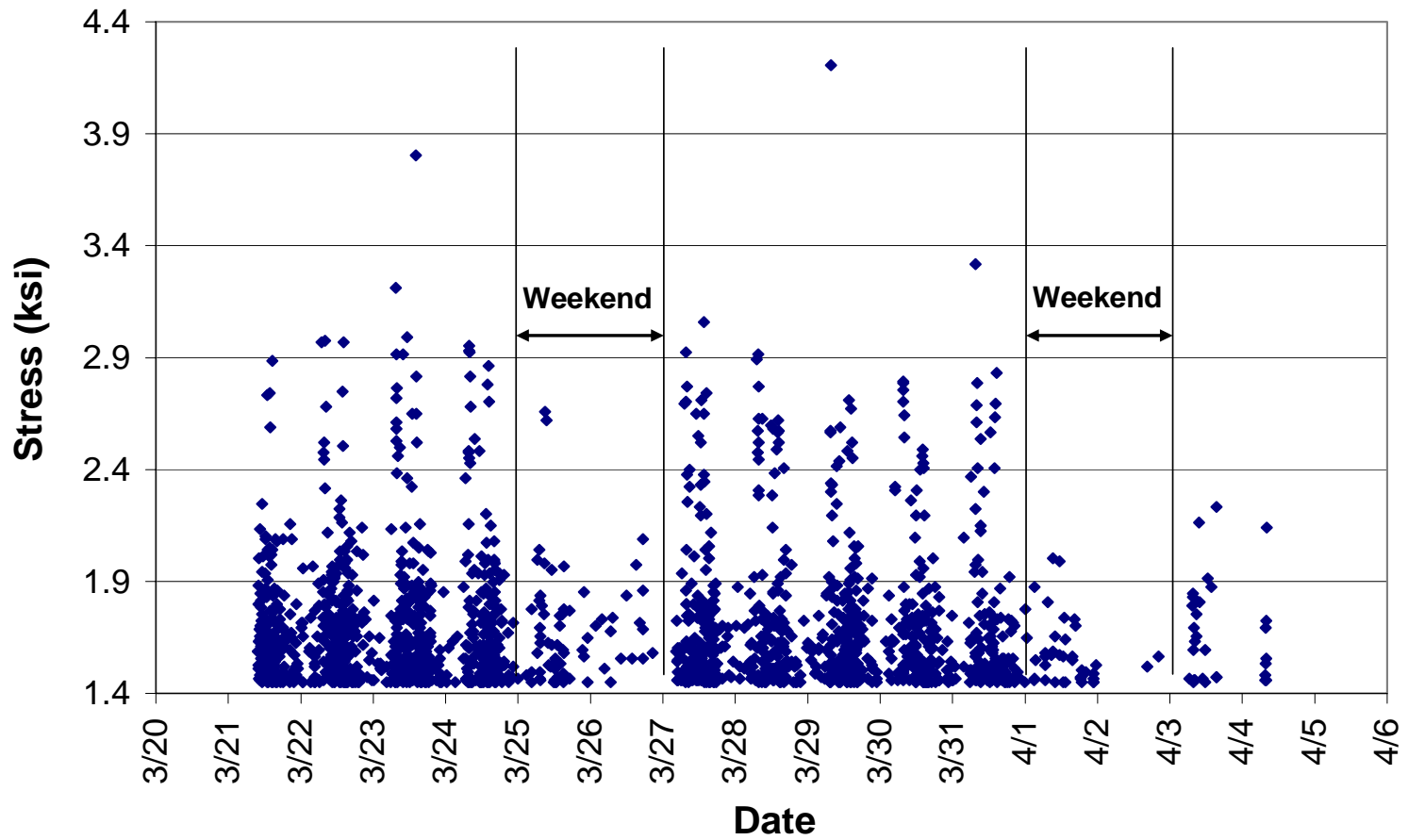


Figure 3.1 Bridge 1-149 Time Line of Recorded Events, 2006

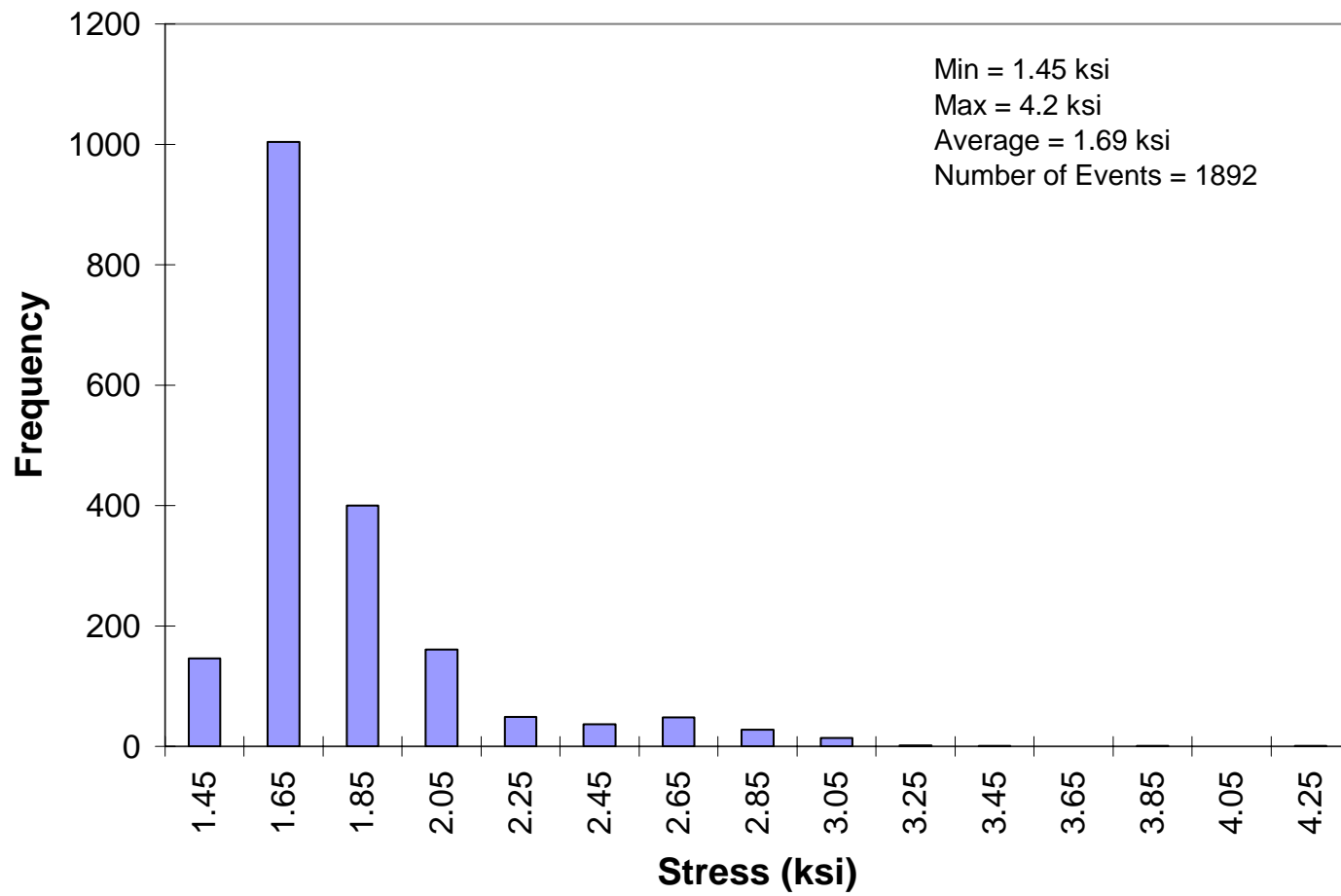


Figure 3.2 Bridge 1-149 Histogram of Interior Girder Bending Stress at Midspan, 2006

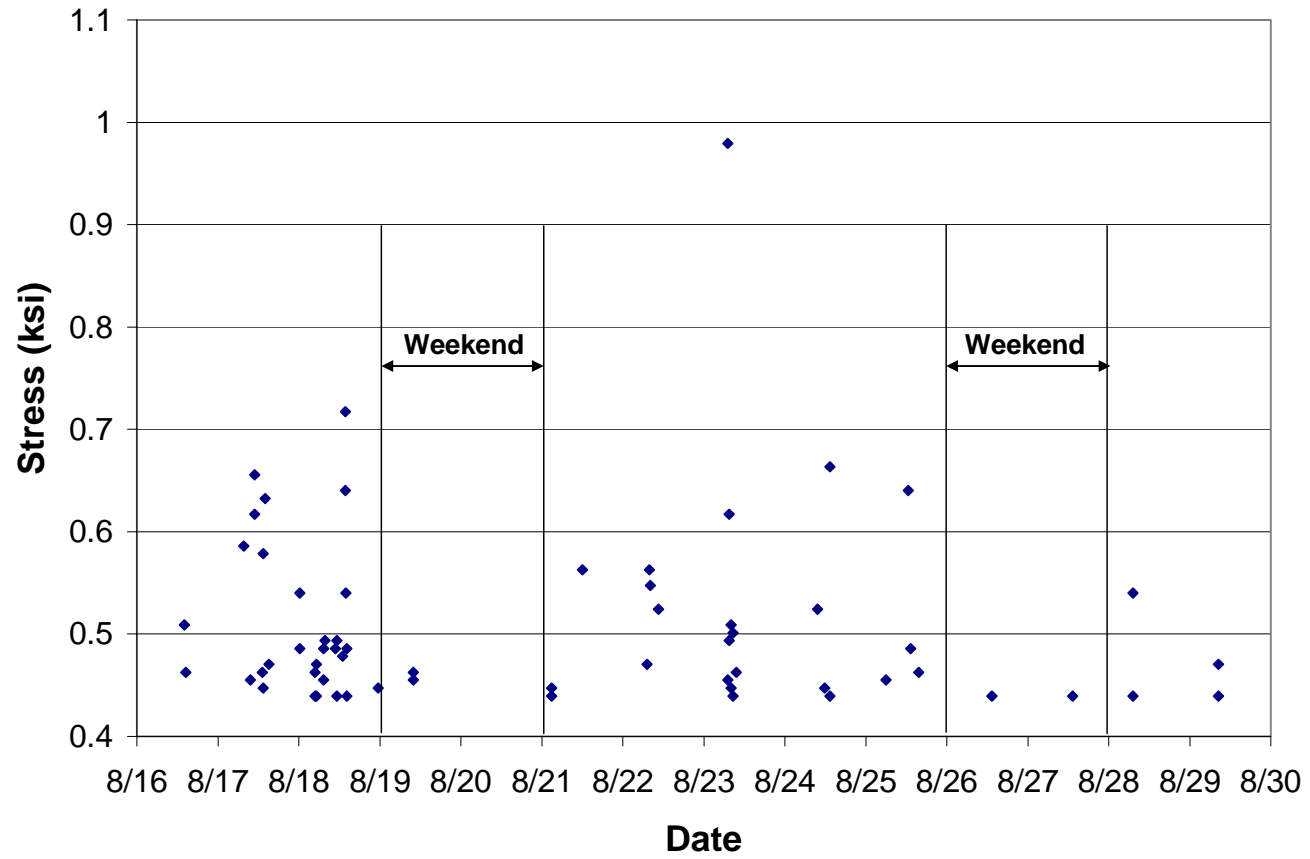


Figure 3.3 Bridge 1-234 Time Line of Recorded Events, 2006

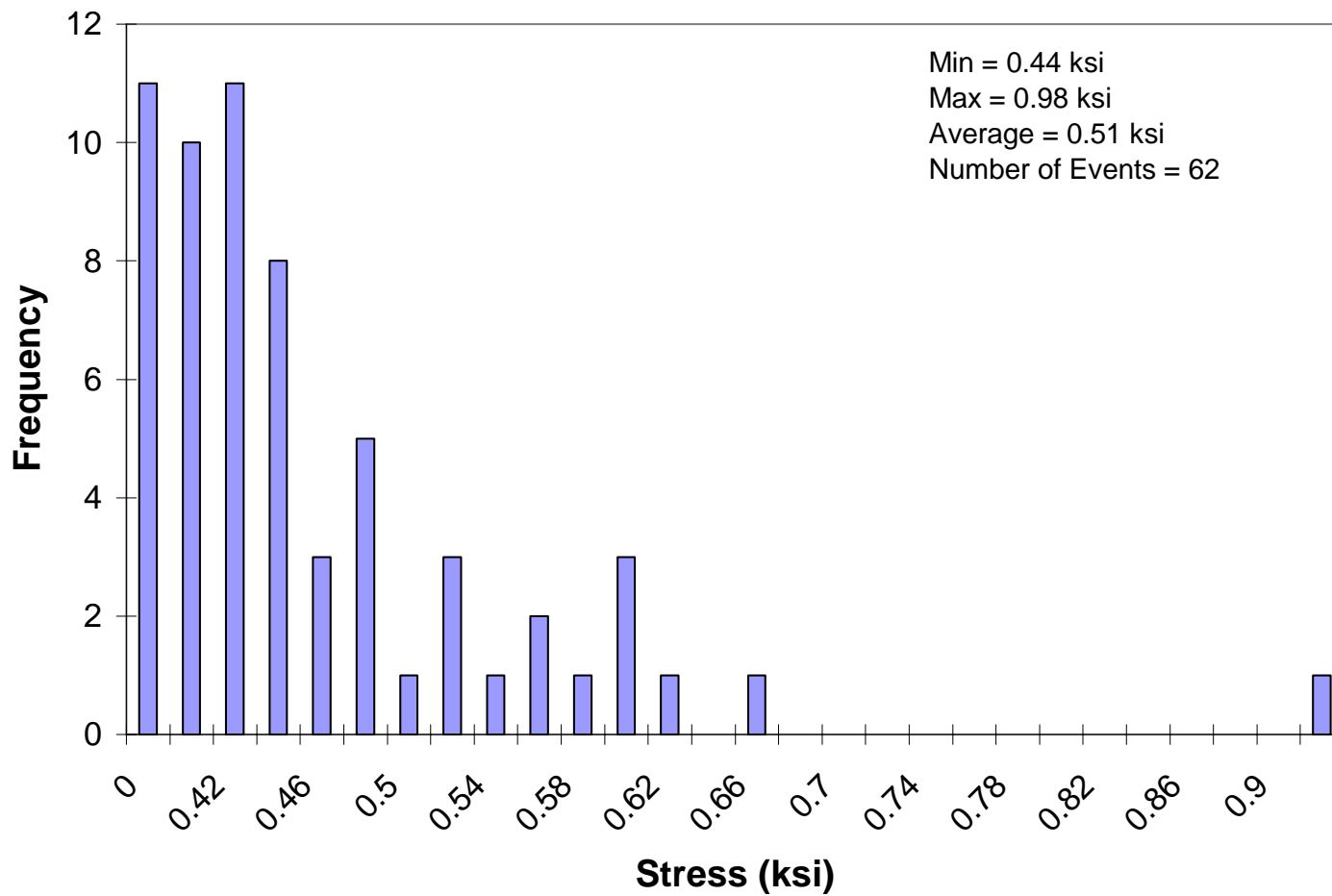


Figure 3.4 Bridge 1-234 Histogram of Interior Girder Bending Stress at Midspan, 2006

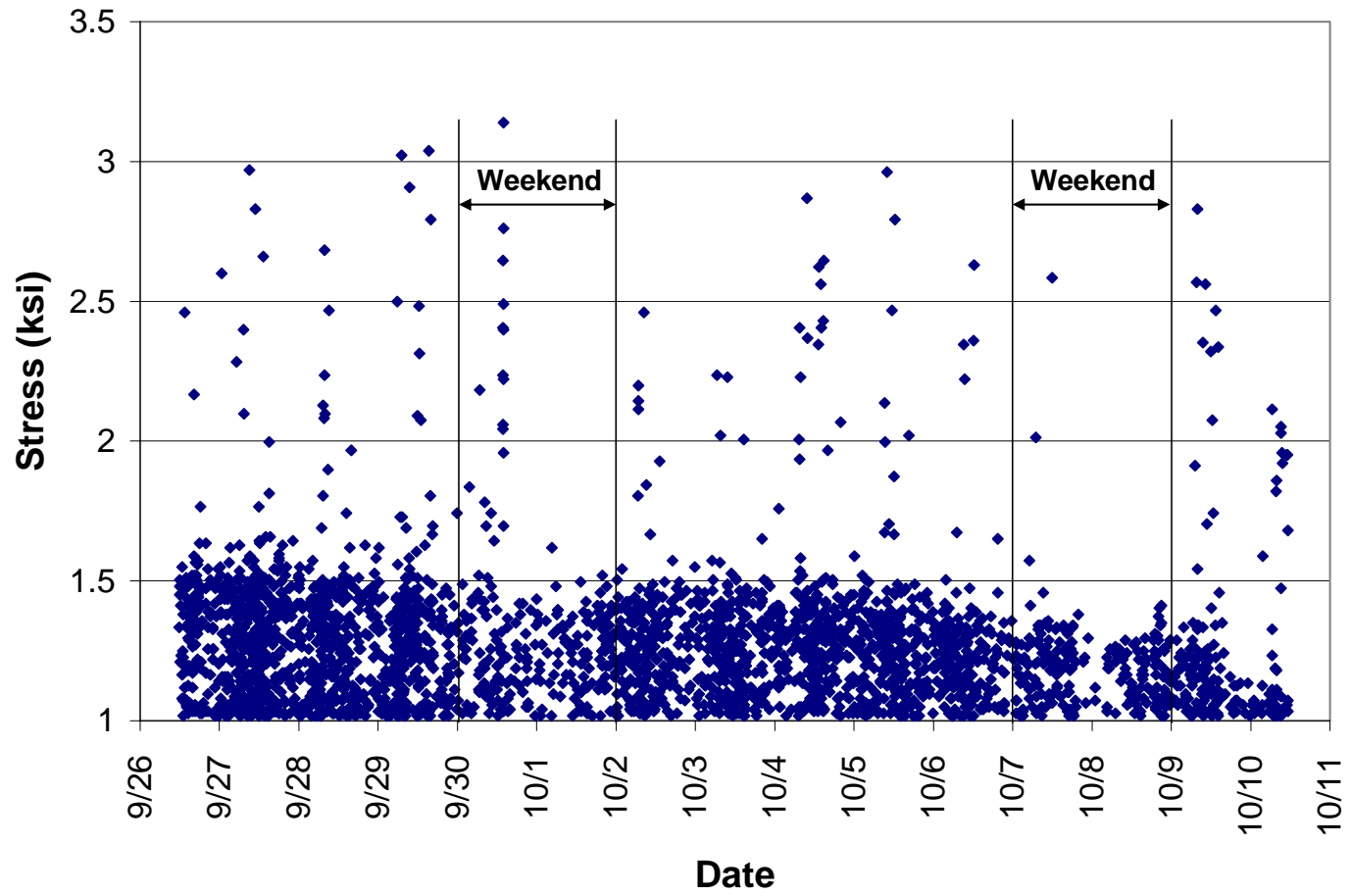


Figure 3.5 Bridge 1-791 Time Line of Recorded Events, 2006

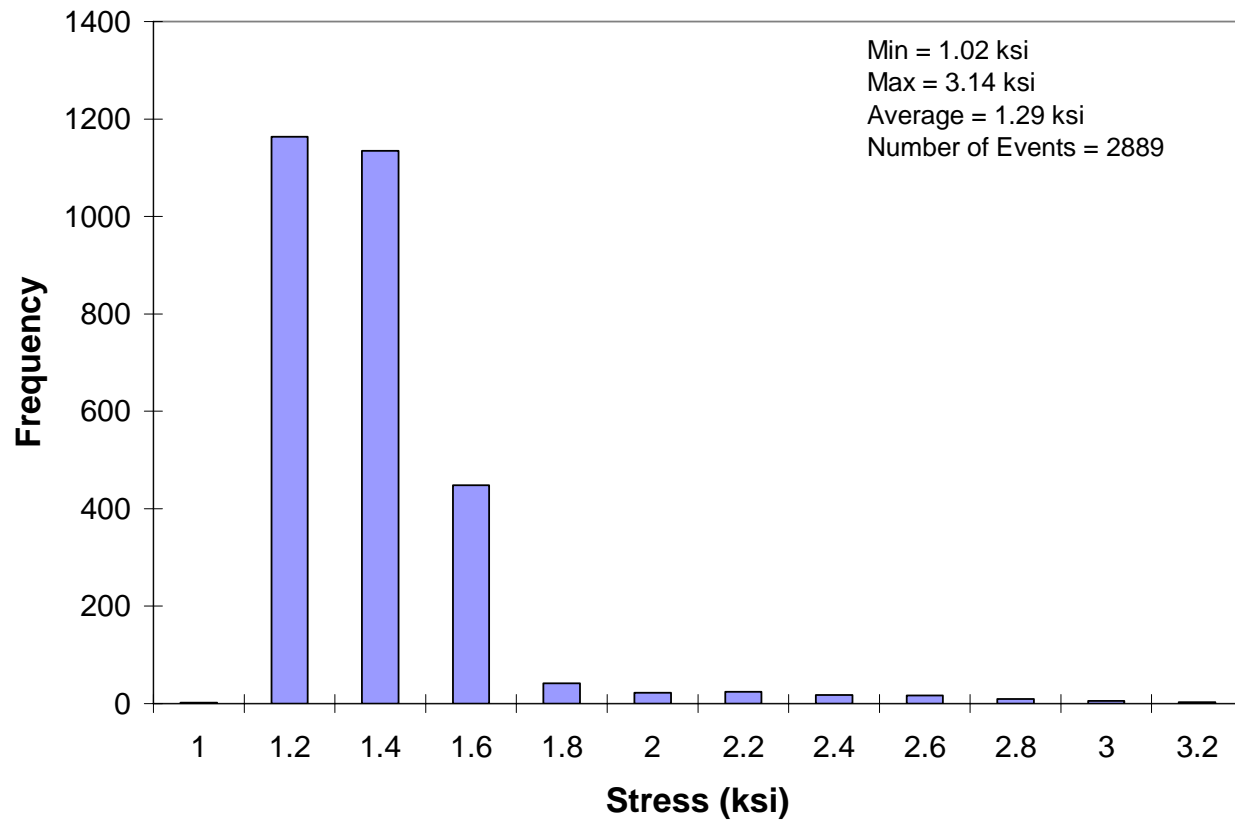


Figure 3.6 Bridge 1-791 Histogram of Interior Girder Bending Stress at Midspan, 2006

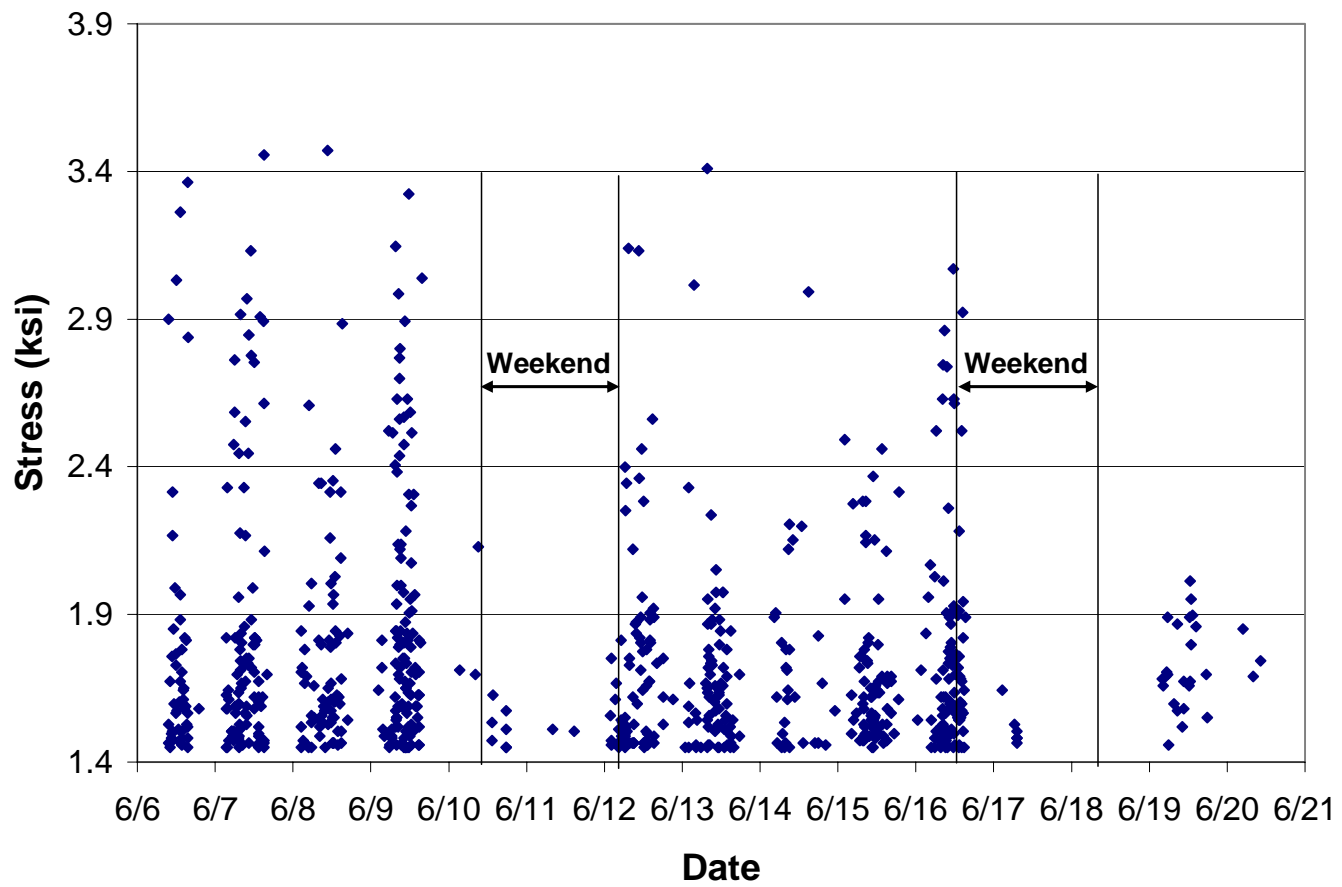


Figure 3.7 Bridge 1-262 Time line of recorded events, 2006

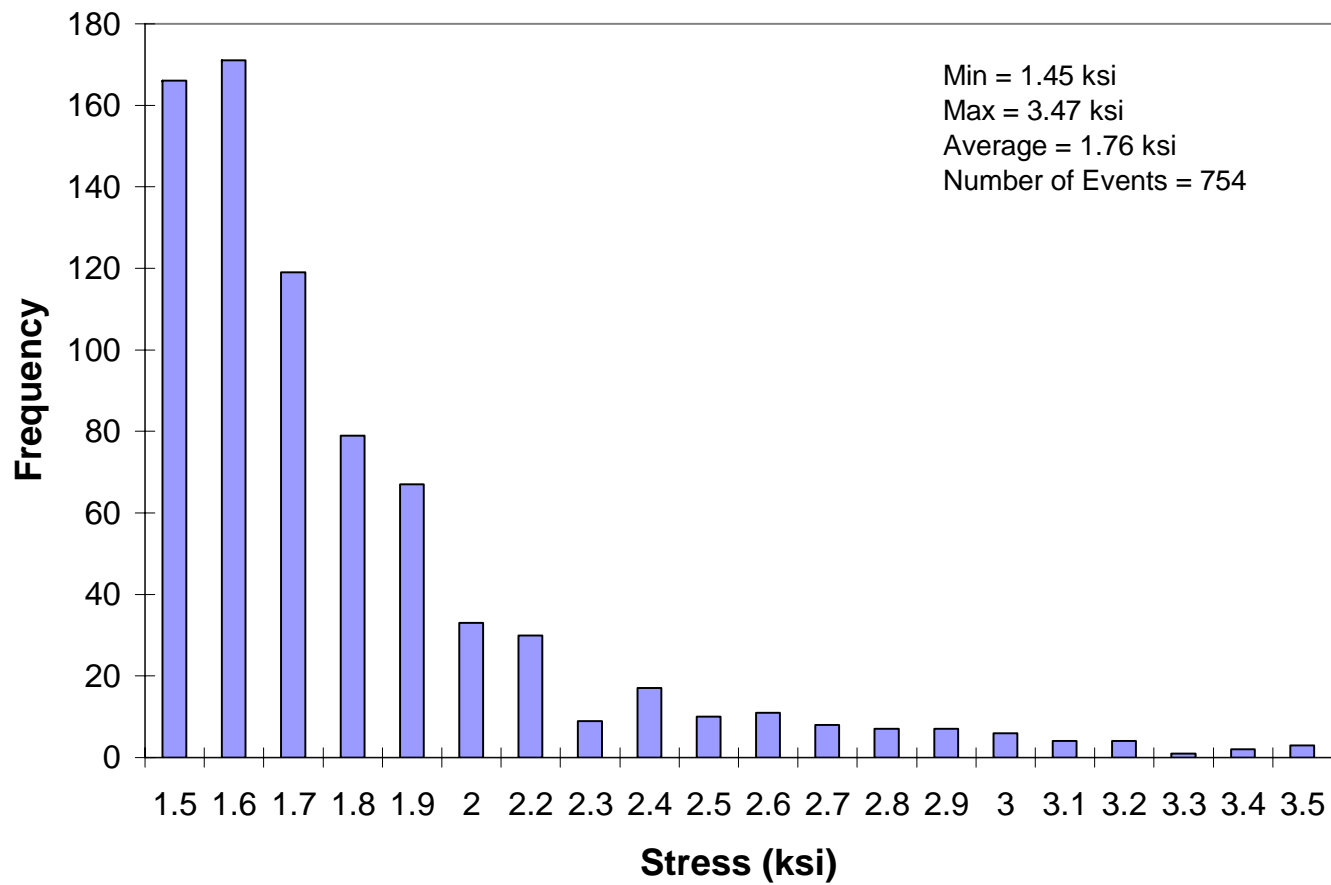


Figure 3.8 Bridge 1-262 Histogram of Interior Girder Bending Stress at Midspan, 2006

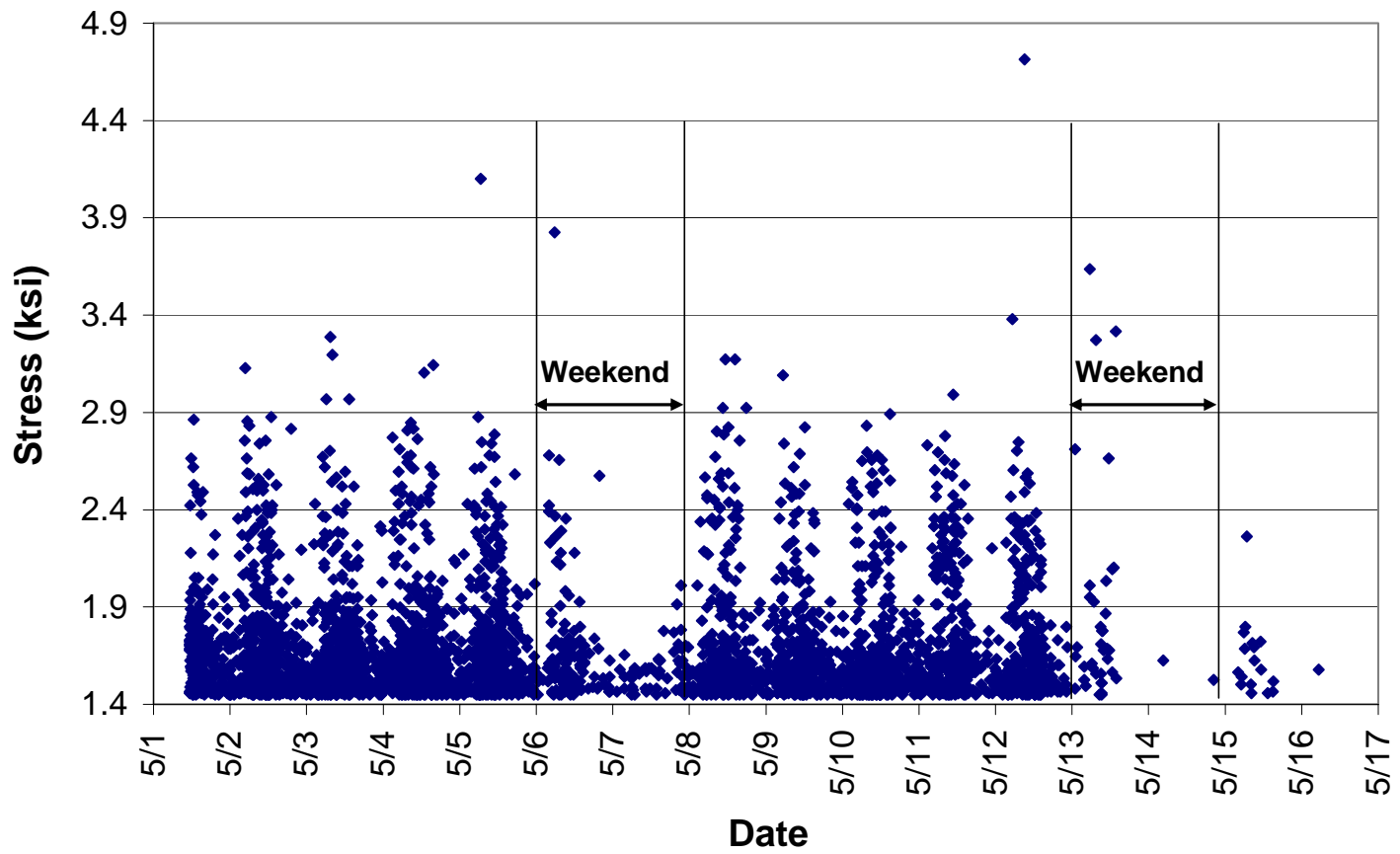


Figure 3.9 Bridge 1-826 Time line of Recorded Events, 2006

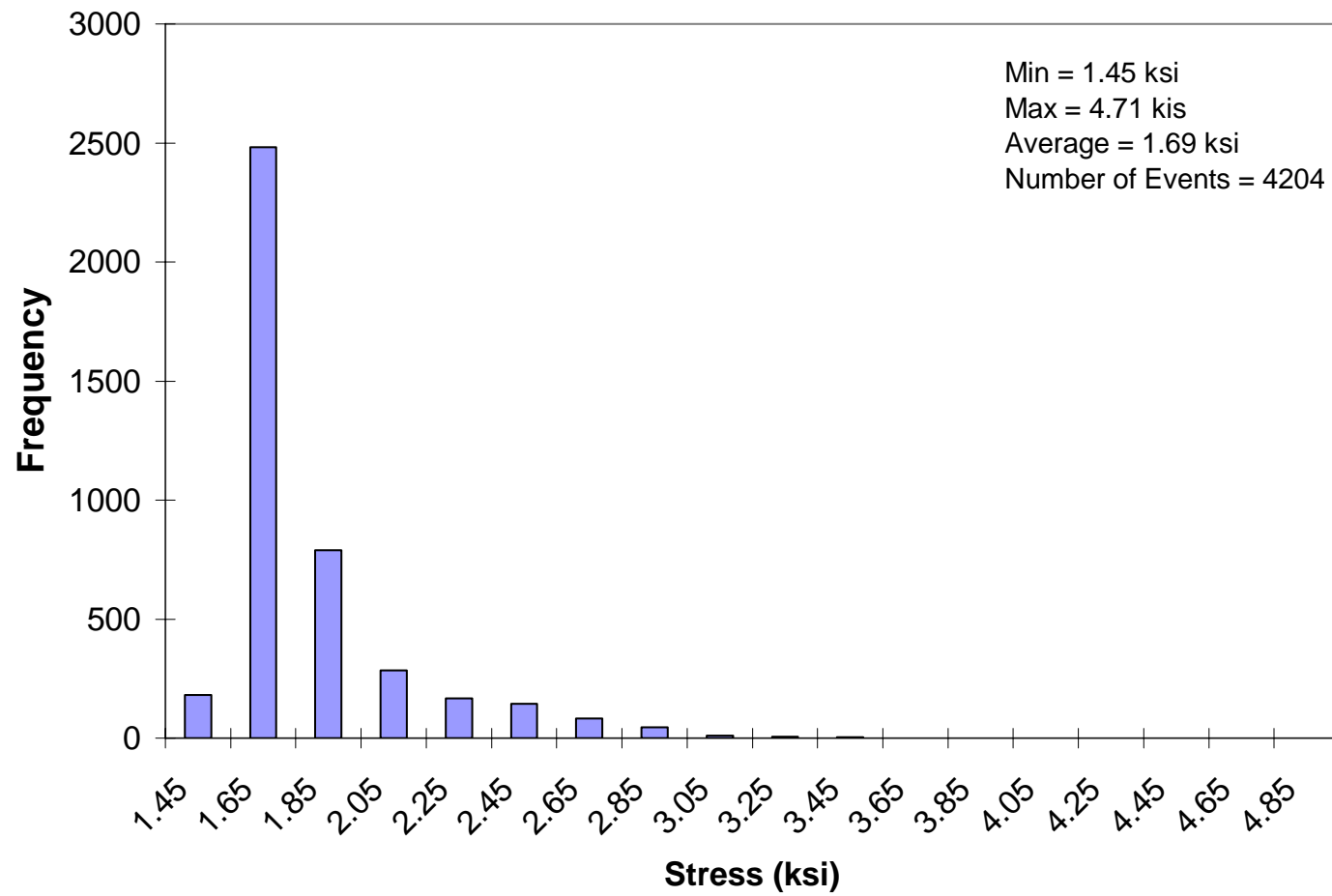


Figure 3.10 Bridge 1-826 Histogram of Interior Girder Bending Stress at Midspan, 2006

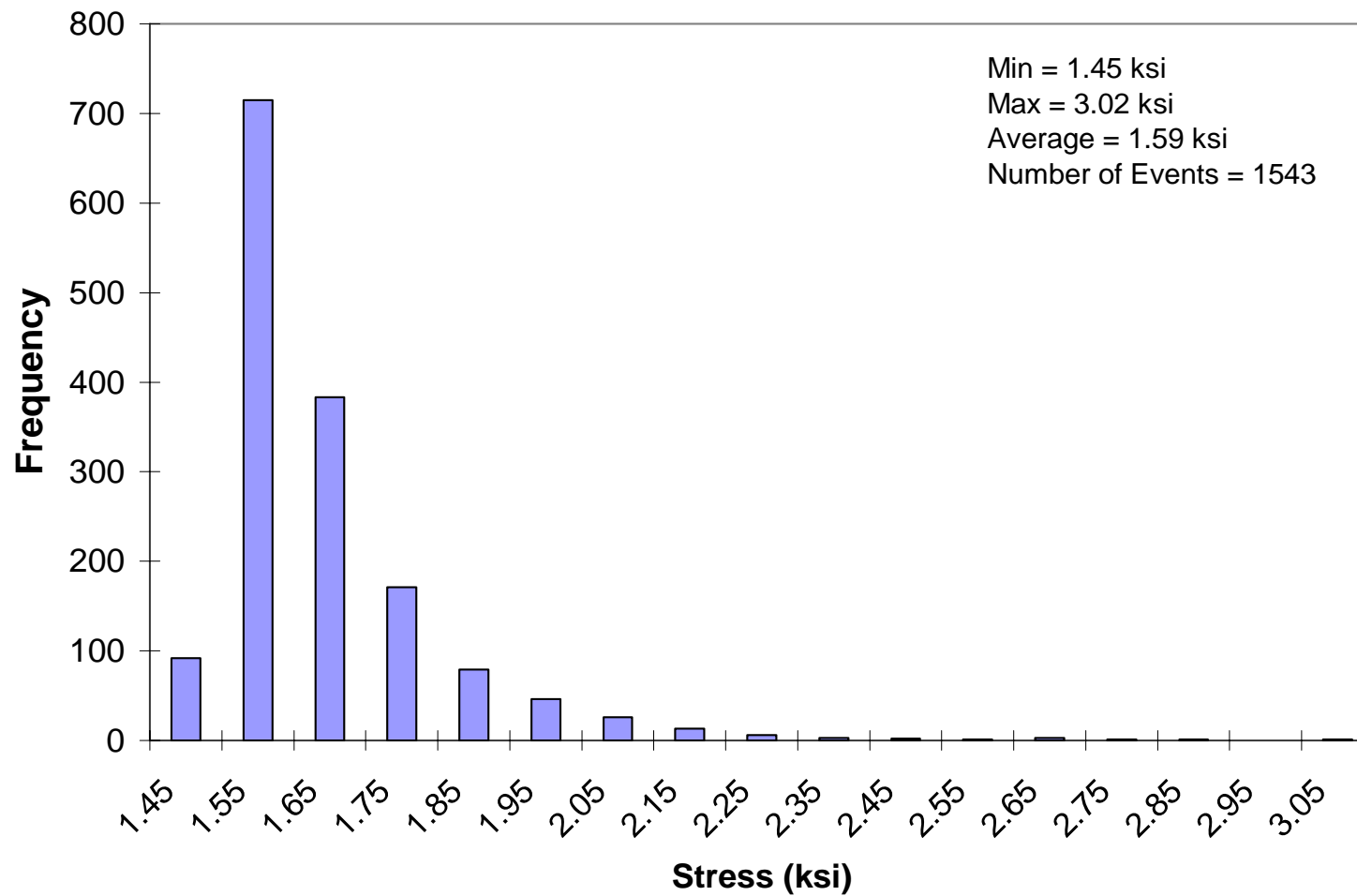


Figure 3.12 Bridge 1-704 Test 1 Histogram of Interior Girder Bending Stress at Midspan, 2006

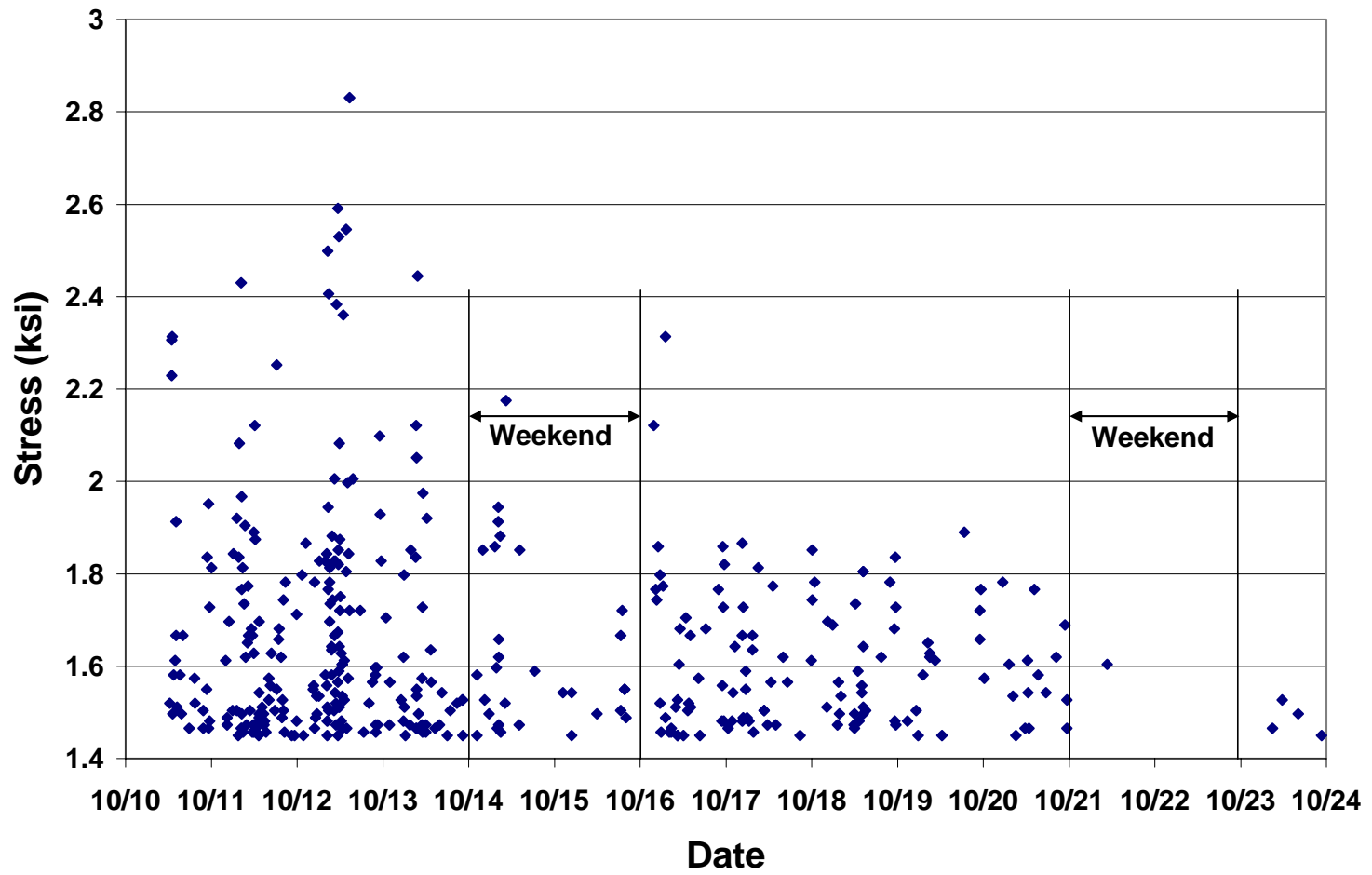


Figure 3.13 Bridge 1-704 Test 2 Time line of Recorded events, 2006

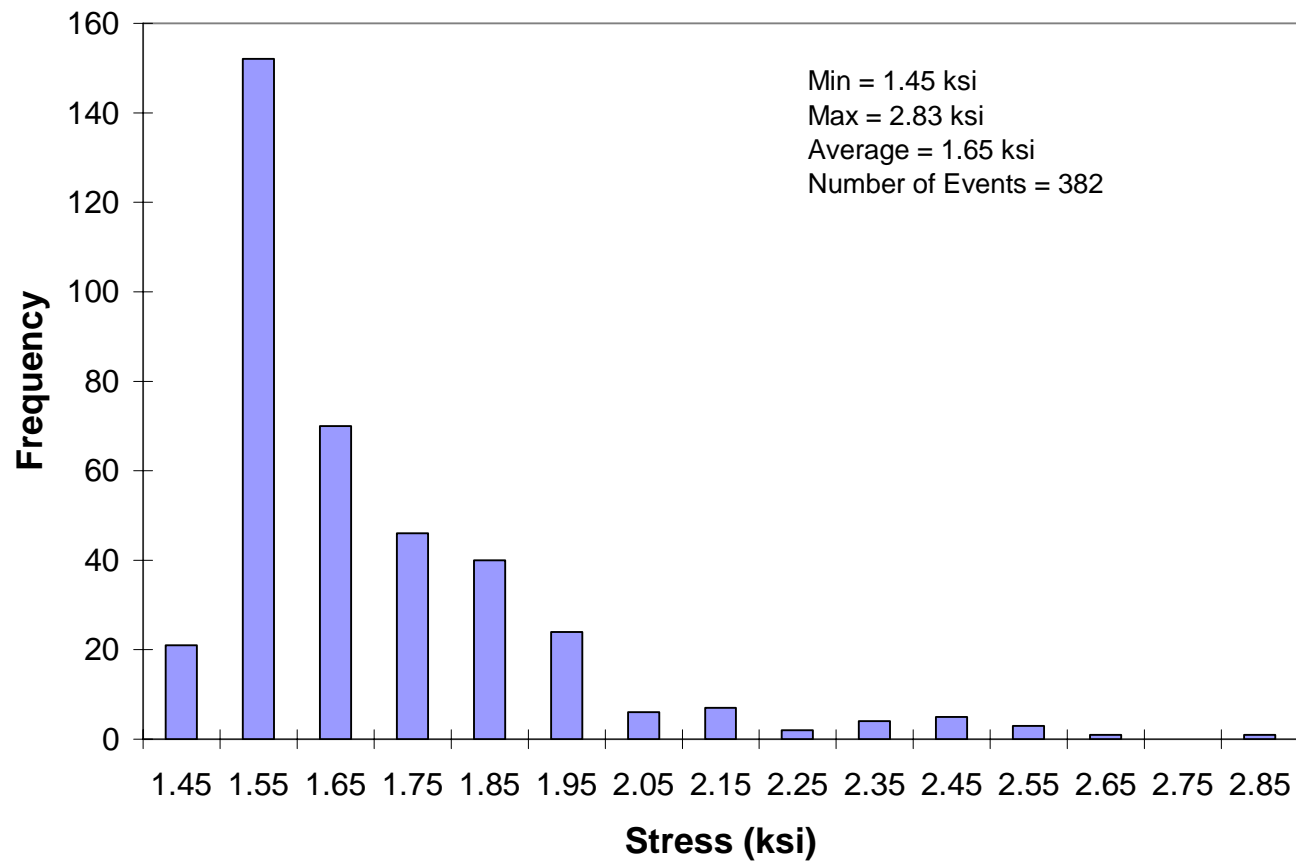


Figure 3.14 Bridge 1-704 Test 2 Histogram of Interior Girder Bending Stress at Midspan, 2006

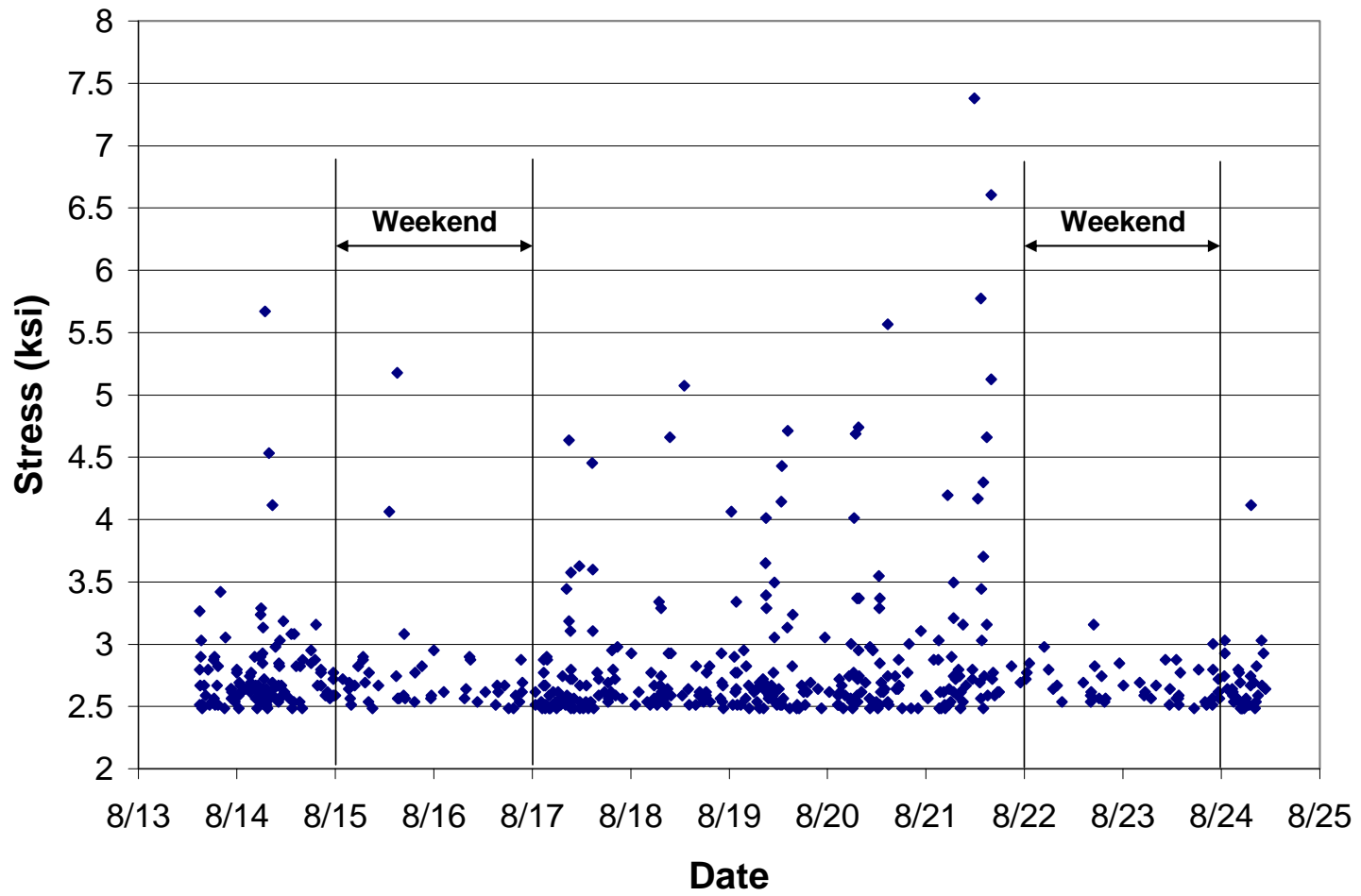


Figure 3.15 Bridge 1-791 Time line of Recorded Events, 1998

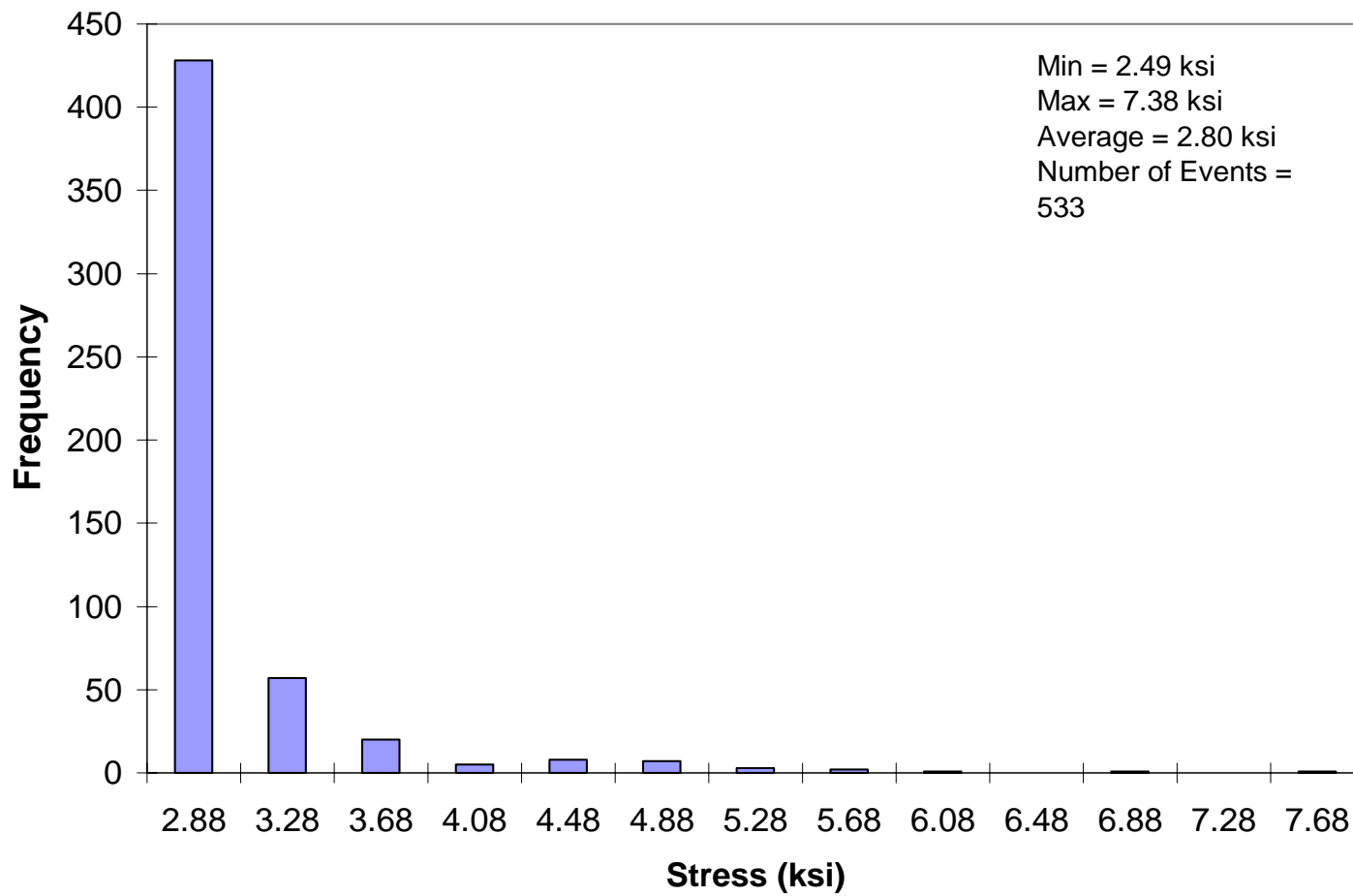


Figure 3.16 Bridge 1-791 Histogram of Interior Girder Bending Moment at Midspan, 1998

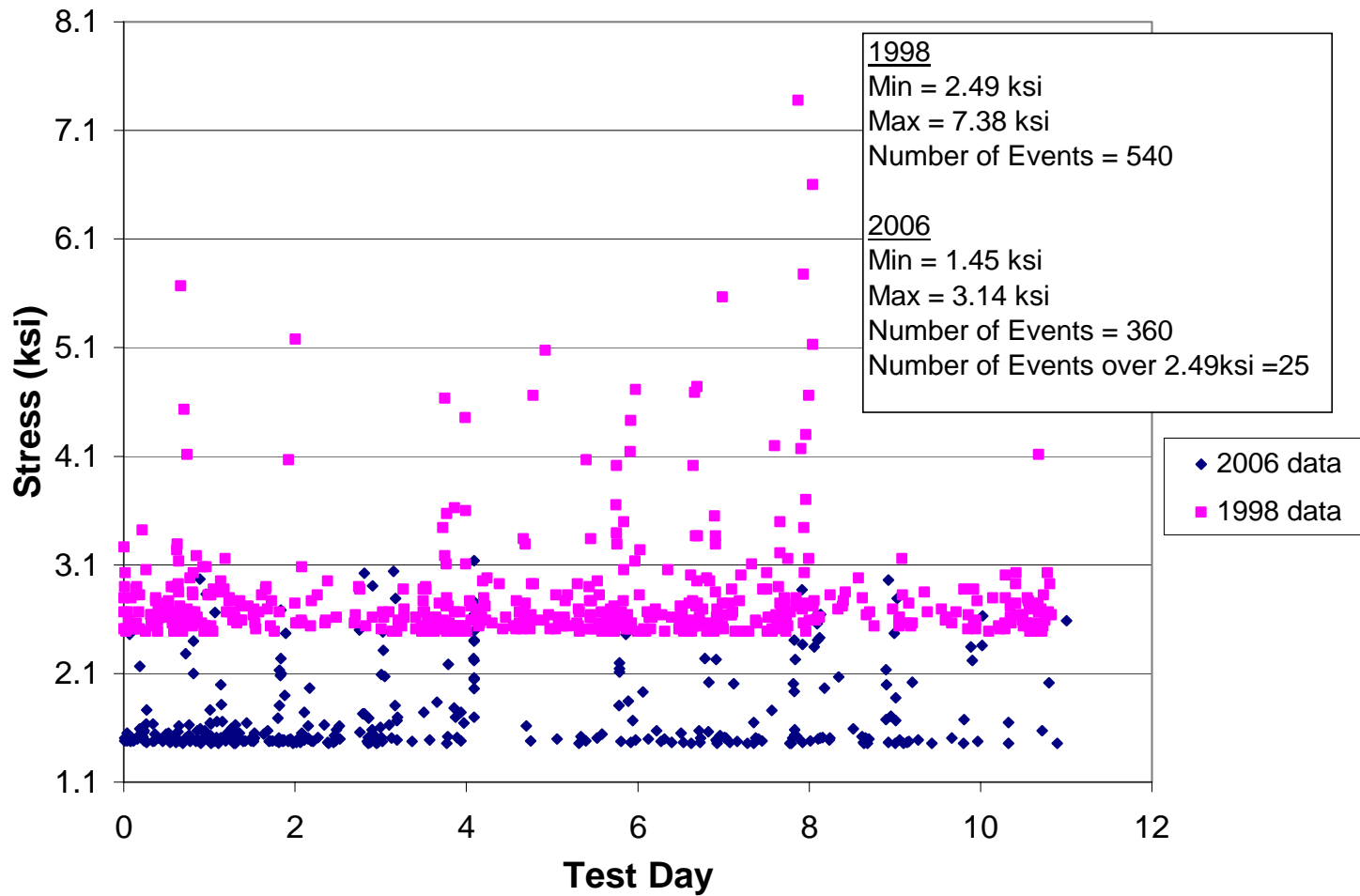


Figure 3.17 1998 vs. 2006 Time line of Events for Bridge 1-791

3.2 Comparing Collected Data Sets

In a previous study Holloway collected data on Bridge 1-791 (on I-95) which was one of the bridges used in this study. The Holloway study was done in 1998 so if the two data sets are compared they can show some different trends and give more insight to what the maximum stress in the bridge might be. The comparisons of the two data sets can be seen in Figure 3.17. The data from each collection period was zeroed and plotted versus the number of days the data was collected. The summarizing data for both the 1998 and 2006 tests on bridge 1-791 can be seen in Table 3.2.

Year	Min Stress (ksi)	Max Stress (ksi)	Average Stress (ksi)	# of Event s	Threshold
1998	2.49	7.38	2.8	533	2.49 ksi
2006	1.45	2.58	1.81	179	1.45 ksi

Table 3.2 Bridge 1-791 1998 vs 2006 data comparison

In Figure 3.17 the trend seen is that the stress values seen by the bridge in the eleven day period in 1998 are higher than in the eleven day period in 2006. The events in the 1998 data are more numerous than in the 2006 data. The 2006 data saw fewer than 3 events per day, using the same trigger threshold set in 1998. To compare data sets with equal numbers of events the 2006 test would have had to have been run for over 230 days. In a test run for 230 days it is more likely than in an 11 day test, that an event as large as the largest in 1998 would be collected.

One possible explanation for the difference in the data is that the frequency of large trucks on northbound I-95 north of Wilmington is much lower now, than it was in 1998. The difference could be due to more trucks using I-495, instead of I-95, to travel into Pennsylvania now, than were in 1998. Unless a heavy truck had reason to go through Wilmington or the truck driver wanted to travel I-95 instead of I-495, trucks would choose I-495 for travel thus reducing the number of events seen by the system on bridge 1-791. This theory is also supported by the fact that bridge 1-826 experienced many more events than bridge 1-791.

Chapter 4

ISBMS DATA ANALYSIS

4.1 Load Ratings Factors for Bridges

In this chapter, the in-service data collected for each bridge is used to calculate a load rating factor for the bridge. This is, in effect, a two-week rating factor, which will then be compared to the rating factor obtained using the AASHTO procedure, which is a 50 year rating.

The Delaware Department of Transportation rates their bridges using a computer program called Bridge Rating and Analysis of Structural Systems (BRASS). This program models a bridge using AASHTO load rating specifications to evaluate the load rating for a bridge. BRASS analyzes a bridge by modeling it as a beam and moves a load across the beam and calculates the moments, shears, and deflections. A BRASS analysis is done based on as-built or design properties. For this analysis the BRASS output files from each of the bridges tested were used to get rating information.

The rating factor equation used in BRASS is:

$$RF = \frac{C - DL}{LL(1 + I)} \quad (1)$$

C = Capacity DL = Dead Load LL = Live Load I = Impact

The variables needed to calculate the rating factor using the in-service data are all obtained from the BRASS output files. The design load truck used for comparison was the HS20 (Truck 1 in BRASS output files). The BRASS input files used to produce the output files can be seen in Appendix 1. For each bridge the rating limit state was found and then the rating factor calculations were carried out to get the same ratings as those given by BRASS.

In order to use the in-service data to compute a load rating, the following procedure was followed. First, the bending moment corresponding to the maximum stress seen by the bridge in our two week test was calculated and was used in place of the LL(1+I) term in the rating equation. The calculation can be seen in the following sections for each bridge and a summary of all results is presented in Table 4.1.

The stress collected from the two week tests is the true two week maximum load seen in the beam that is monitored. In the rating factor equation the live load values have distribution factors applied which incorporate multiple presence factors and number of lanes for the bridge. The governing limit state factors applied to both live and dead loads are based on using the design criteria to give a rating which is safe for 50 years after the bridge is built. The impact factor is also applied to live load values in the rating factor equation. Since the measured values are not based on static analysis and are based on actual events, no distribution factor or impact factors are applied to the live load values in the rating equation. The limit state factors are still applied to the dead loads because design values are being used. The moment used in the two-week rating factors was calculated by the following equation:

$$M = \frac{\sigma I}{y} \quad M = LL(1+I) \quad (2)$$

σ = measured maximum stress y = distance to neutral axis I = moment of inertia

To obtain the two-week rating factor (RF*) the following equation is used:

$$RF^* = \frac{C - DL}{M} \quad (3)$$

4.1.1 Bridge 1-149

From the Brass output the following information was extracted:

Governing Limit State for Inventory rating is Flexure, therefore the load levels are:

$$1.3 (1.0 DL + 1.67 LL) \quad (4)$$

Controlling point= mid-span of span 1 (same as data collecting point)

C = 4402.8 k-ft DL = 1136 k-ft LL = 689.5 k-ft (including Impact)

y = 31.22 in I = 36931 in⁴ σ = 4.205 ksi (from Table 3.1)

From Equation 1:

$$RF = \frac{4402.8 - (1.3 \times 1136)}{1.3(1.67 \times 689.6)} = \frac{4402.8 - (1.3 \times 1136)}{1497} = 1.95$$

From Equation 2:

$$M = \frac{4.205 \times 36931}{31.22} \div 12 = 414.52 \text{ k-ft}$$

From Equation 3:

$$RF^* = \frac{4402.8 - (1.3 \times 1136)}{414.52} = 7.06$$

4.1.2 Bridge 1-234

From the Brass output the following information was extracted:

Governing Limit State for Inventory rating is Serviceability, therefore the load levels are:

$$1.0 (1.0 \text{ DL} + 1.67 \text{ LL}) \quad (5)$$

Controlling point= mid-span of the span (same as data collecting point)

$$C = 2099.3 \text{ k-ft} \quad \text{DL} = 627.0 \text{ k-ft} \quad \text{LL} = 735.5 \text{ k-ft (including Impact)}$$

$$y = 18.13 \text{ in} \quad I = 17300 \text{ in}^4 \quad \sigma = 0.9795 \text{ ksi (from Table 3.1)}$$

From Equation 1:

$$RF = \frac{2099.3 - (1.0 \times 627.0)}{1.0(1.67 \times 735.5)} = \frac{2099.3 - (1.0 \times 627.0)}{1497} = 1.2$$

From Equation 2:

$$M = \frac{0.9795 \times 17300}{18.13} \div 12 = 77.89 \text{ k-ft}$$

From Equation 3:

$$RF^* = \frac{2099.3 - (1.0 \times 627.0)}{77.89} = 18.9$$

4.1.3 Bridge 1-791

From the Brass output the following information was extracted:

Governing Limit State for Inventory rating is Flexure, therefore the load levels are:

$$1.3 (1.0 \text{ DL} + 1.67 \text{ LL})$$

Controlling point= mid-span of span 2 (not the same as collected data)

$$C = 1163.7 \text{ k-ft} \quad \text{DL} = 198.1 \text{ k-ft} \quad \text{LL} = 438.8 \text{ k-ft (including Impact)}$$

$$y = 28.32 \text{ in} \quad I = 11974 \text{ in}^4$$

From Equation 1:

$$RF = \frac{1163.7 - (1.3 \times 198.1)}{1.3(1.67 \times 438.8)} = \frac{1163.7 - (1.3 \times 198.1)}{953} = 0.95$$

For bridge 1-791 span 1 was built non-composite and span 2 was built composite. The location governing rating for the bridge is at mid-span of span 2, not mid-span of span 1 where the data was collected. To transfer the load rating from the mid-span of span 1 to the mid-span of span 2, the rating factor for span 1 was calculated using composite section properties. This composite rating factor was then used to develop a ratio of load rating of span 1 to span 2. The ratio calculated will allow the two-week rating factor for span 2 to be calculated using data collected on span 1. So the following information was used for the composite section rating factor for span 1:

$$C = 1163.7 \text{ k-ft} \quad \text{DL} = 54.2 \text{ k-ft} \quad \text{LL} = 241.1 \text{ k-ft (including Impact)}$$

$$y = 28.32 \text{ in} \quad I = 11974 \text{ in}^4 \quad \sigma = 3.14 \text{ ksi (from Table 3.1)}$$

From Equation 1:

$$RF = \frac{1163.7 - (1.3 \times 54.2)}{1.3(1.67 \times 241.1)} = \frac{1163.7 - (1.3 \times 54.2)}{523} = 2.09$$

The ratio of the rating factors of span 2 to span 1 is:

$$\frac{0.95}{2.09} = .45$$

This ratio will help convert the two-week rating factor from span 1 to span 2. For span 1

using Equation 2:

$$M = \frac{3.14 \times 11974}{28.32} \div 12 = 110.64 \text{ k-ft}$$

From Equation 3:

$$RF^* = \frac{1163.7 - (1.3 \times 54.2)}{110.64} = 9.88$$

For span 2 using the ratio of rating factors from span 1 to span 2 the two-week rating factor is:

$$RF^* = 9.88 \times .45 = 4.5$$

4.1.4 Bridge 1-262

From the BRASS output the following information was extracted:

Governing Limit State for Inventory rating is Flexure, therefore the load levels are:

$$1.3 (1.0 \text{ DL} + 1.67 \text{ LL})$$

Controlling point= mid-span of span 1 (not the same as collected data)

$$C = 4192.4 \text{ k-ft} \quad \text{DL} = 656.6.1 \text{ k-ft} \quad \text{LL} = 839.9 \text{ k-ft (including Impact)}$$

$$y = 45.25 \text{ in} \quad I = 49319 \text{ in}^4$$

From Equation 1:

$$RF = \frac{4192.4 - (1.3 \times 656.6)}{1.3(1.67 \times 839.9)} = \frac{4192.4 - (1.3 \times 656.6)}{1823} = 1.83$$

Since the data was collected at mid-span of span 5 the rating factor for that span needs to be calculated. The following information was used for the rating factor for span 5:

$$C = 4192.4 \text{ k-ft} \quad DL = 656.7 \text{ k-ft} \quad LL = 814.3 \text{ k-ft (including Impact)}$$

$$y = 45.25 \text{ in} \quad I = 49319 \text{ in}^4 \quad \sigma = 3.47 \text{ ksi (from Table 3.1)}$$

From Equation 1:

$$RF = \frac{4192.4 - (1.3 \times 656.7)}{1.3(1.67 \times 814.3)} = \frac{4192.4 - (1.3 \times 656.7)}{1768} = 1.89$$

The ratio of the rating factors of span 1 to span 5 is:

$$\frac{1.83}{1.89} = .968$$

This ratio will help convert the two-week rating factor from span 5 to span 1. For span 5

using Equation 2 :

$$M = \frac{3.47 \times 49319}{45.25} \div 12 = 315.29 \text{ k-ft}$$

From Equation 3:

$$RF^* = \frac{4192.4 - (1.3 \times 656.7)}{315.29} = 10.6$$

For span 1 using the ratio of rating factors from span 1 to span 5 the two-week rating factor is:

$$RF^* = 10.6 \times 0.968 = 10.26$$

4.1.5 Bridge 1-826

From the Brass output the following information was extracted:

Governing Limit State for Inventory rating is Flexure, therefore the load levels are:

$$1.3 (1.0 \text{ DL} + 1.67 \text{ LL})$$

Controlling point= mid-span of span 1 (span 1 is geometrically the same as span 3 where data was collected so the information for span 1 can be used for span 3)

$$C = 2365.2 \text{ k-ft} \quad \text{DL} = 484.2 \text{ k-ft} \quad \text{LL} = 658.0 \text{ k-ft (including Impact)}$$

$$y = 31.28 \text{ in} \quad I = 27135 \text{ in}^4 \quad \sigma = 4.714 \text{ ksi (from Figure Table 3.1)}$$

From Equation 1:

$$RF = \frac{2365.2 - (1.3 \times 484.2)}{1.3(1.67 \times 658.0)} = \frac{2365.2 - (1.3 \times 484.2)}{1429} = 1.22$$

From Equation 2:

$$M = \frac{4.714 \times 27135}{31.28} \div 12 = 340.8 \text{ k-ft}$$

From Equation 3:

$$RF^* = \frac{2365.2 - (1.3 \times 484.2)}{340.8} = 5.09$$

4.1.6 Bridge 1-704

From the Brass output the following information was extracted:

Governing Limit State for Inventory rating is Flexure, therefore the load levels are:

$$1.3 (1.0 \text{ DL} + 1.67 \text{ LL})$$

Controlling point= mid-span of span 1 (same as data collecting point)

$$C = 524.5 \text{ k-ft} \quad \text{DL} = 86.2 \text{ k-ft} \quad \text{LL} = 198.5 \text{ k-ft (including Impact)}$$

$$y = 12.05 \text{ in} \quad I = 2370 \text{ in}^4 \quad \sigma = 3.016 \text{ ksi (from Table 3.1)}$$

From Equation 1:

$$RF = \frac{524.5 - (1.3 \times 86.2)}{1.3(1.67 \times 198.5)} = \frac{524.5 - (1.3 \times 86.2)}{431} = 0.96$$

From Equation 2:

$$M = \frac{3.016 \times 2370}{12.05} \div 12 = 49.43 \text{ k-ft}$$

From Equation 3:

$$RF^* = \frac{524.5 - (1.3 \times 86.2)}{49.43} = 8.34$$

4.2 Summary of Ratings

The ratings for the bridges all increased when the collected data was used to obtain the two-week rating factor. The theoretical ratings range from 0.96 to 1.95, the two-week ratings based on the in-service data range from 4.5 to 18.9. The increase in ratings is over 4 times. The comparisons of these ratings can be seen in Table 4.1. It is important to remember when comparing the ratings that the In-Service Rating is a two-week rating and the BRASS rating is a 50-year rating.

Bridge	Measured Max Stress (ksi)	BRASS Rating Factor (RF)	In-Service Rating Factor (*RF)	Ratio (In-Service vs BRASS)
1-149	4.205	1.95	7.06	3.62
1-234	0.9795	1.2	18.9	15.75
1-791	3.14	0.95	4.5	4.74
1-262	3.47	1.89	10.26	5.43
1-826	4.714	1.22	5.09	4.17
1-704	3.016	0.96	8.34	8.69

Table 4.1 Rating Factors

4.3 Trigger level and Number of Events Collected

Before deployment of the ISBMS a trigger level has to be chosen for each bridge. To do this can be a guess of what the trigger level should be to capture enough events to get meaningful results. To try to find better way of choosing a trigger level the stress caused by an HS20 truck on each of the bridges was calculated and compared to the threshold used during the test.

The HS20 stress level can be calculated for each case using an altered form of equation 2:

$$\sigma = \frac{My}{I} \quad M = \text{Unfactored Live load moment due to HS20 truck (from BRASS outputs)}$$

I = moment of inertia (from BRASS outputs)

y = distance to neutral axis (from BRASS outputs)

The threshold stresses are seen in Chapter 3. The comparison of the different bridges can be seen in Table 4.2.

Bridge	Threshold Stress	HS20 Stress	Ratio (threshold/HS20 stress)	Number of Events
1-149	1.45 ksi	6.99 ksi	0.207	1892
1-234	0.44 ksi	9.25 ksi	0.047	62
1-791	1.02 ksi	6.84 ksi	0.149	2889
1-262	1.45 ksi	8.96 ksi	0.162	754
1-826	1.45 ksi	9.10 ksi	0.159	4204
1-704 test 1	1.45 ksi	12.11 ksi	0.120	1543

Table 4.2 Threshold Stresses, HS20 Stresses and Number of Events

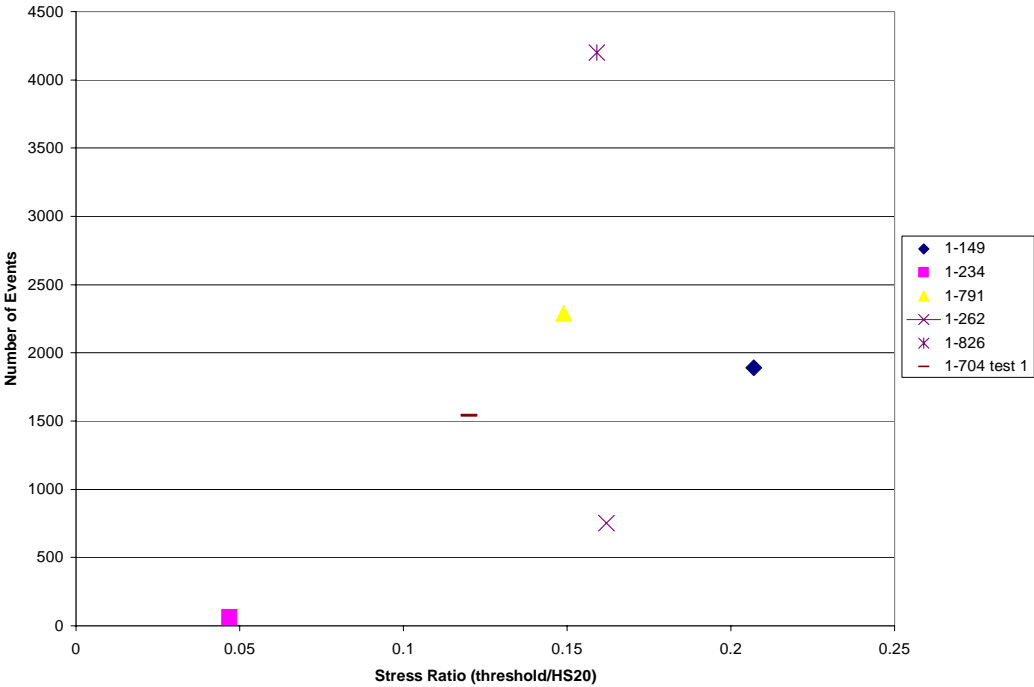


Figure 4.1 Stress Ratios vs Number of Events

Figure 4.1 Stress Ratios vs Number of Events shows the relationship between the ratio of threshold values vs HS20 values and the number of events collected. The ideal set up is really bridge 1-826 and a ratio that is between 0.15 and 0.16 seems to be a good stress ratio. 1-826 also seems to have collected the number of data points that would have been expected. The ADT for bridge 1-826 is 33,632 with 15% trucks. That means that 5045 trucks per day pass over the bridge if there are 3 lanes on the bridge and they are dispersed evenly and only half of the trucks are loaded and only half of those are above the selected trigger the system should see 420 trucks each day which means just under 5900 trucks in 14 days. Given that 4 of those days were weekends and less trucks travel over the roads on weekends the 4204 collected events for this project seems reasonable. Since the other bridges have similar ADT and %trucks some explanation might be needed for why there weren't as many events recorded for those bridges. Part of the reason could be due to the ratio of threshold to HS20 as seen in Figure 4.1 other reasons were that some of the bridges were effected greatly by construction or change in traffic patterns from when ADT numbers were last collected. On bridges off of main highways the %truck numbers do not always mean HS20 trucks pass over the bridge and maybe the %truck represents mainly local delivery trucks which do not weigh as much as and HS20.

4.3.1 Rating Summary

All of the in-service ratings are greater than one which is good to see. If the ratings were less than one it would mean that the bridge was seeing stresses greater than the design capacity of the bridge which would not be encouraging. If bridge 1-234 is taken out of the set of bridges the ratios of the in-service rating versus the BRASS rating are between 3.62 and 8.69. This is a significant increase in rating.

Knowing that the BRASS rating is a 50 year rating and the in-service rating is a two week rating there is a sense that if the two week rating were projected to a 50 year rating the ratio between the two ratings would be smaller. For inspection purposes a two year rating is sufficient and a 50 year rating is not necessary. The two week rating may be closer to a two year rating than the 50 year rating. Using instrumentation to measure the peak stress seen on a bridge in two years to calculate a two year rating and comparing that to a projected two year rating using two weeks of data would help verify if a two week snap shot of data is enough to correctly predict a two year rating. In Chapter 5 a gauge that has the potential to serve as instrumentation to collect a two year stress is discussed.

Chapter 5

PEAK GAUGE

5.1 Introduction

MicroStrain Inc. of Williston, Vermont, has developed a peak strain detect transducer. The gauge mechanically records the maximum strain seen by the gauge over a period of time. The gauge mechanically changes when it experiences strain and will hold that position until a larger strain is seen by the gauge. Under normal operation the peak detect gauge does not require power to record the strain; only when the reading taken is power required. This is convenient for long term peak detection in the field. Further discussion on this process will be seen later. Initial testing of the peak strain detect gauge was done by Ryan Giles in the summer of 2002 (Giles, 2002). Repeat of these initial tests and more extensive testing was done for this project.

5.2 Description and Operation

The peak sensor is able to use a mechanical system to record the peak strain because of the way the Differential Variable Reluctance Transducer works. The DVRT converts linear displacements into linear variable electrical output. The displacement is detected by the movement of the core within the coil. The DVRT has two coils and the core which gives the signal. The basic electrical set up can be seen in Figure 5.1 . The coil is energized using an AC excitation through the center tap.

When the core is at the central location (null position) signals V_a and V_b are equal.
When the core moves V_a and V_b vary proportionally.

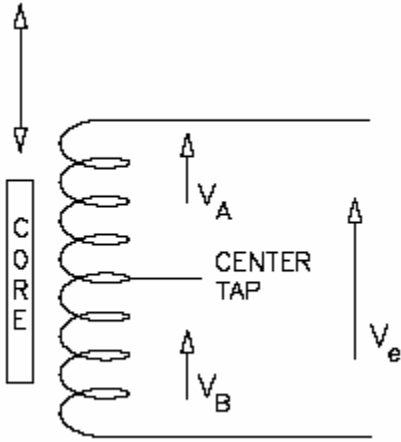


Figure 5.1 DVRT Schematic

The peak gauge set up can be seen in Figure 5.2. There are two mounting blocks, one which holds the DVRT and one which creates a clamp for the core rod. The core for the DVRT is mounted to a rod which is run through one of the mounting blocks. The core is positioned in the DVRT so that the output signal is in the null position for the sensor and then the collar is tightened against the mounting block to hold the core in place. The other mounting block with the DVRT also has a tension spring which allows the core to slide in the DVRT but creates enough friction so that the core only moves when strained and not when vibrated or tipped.

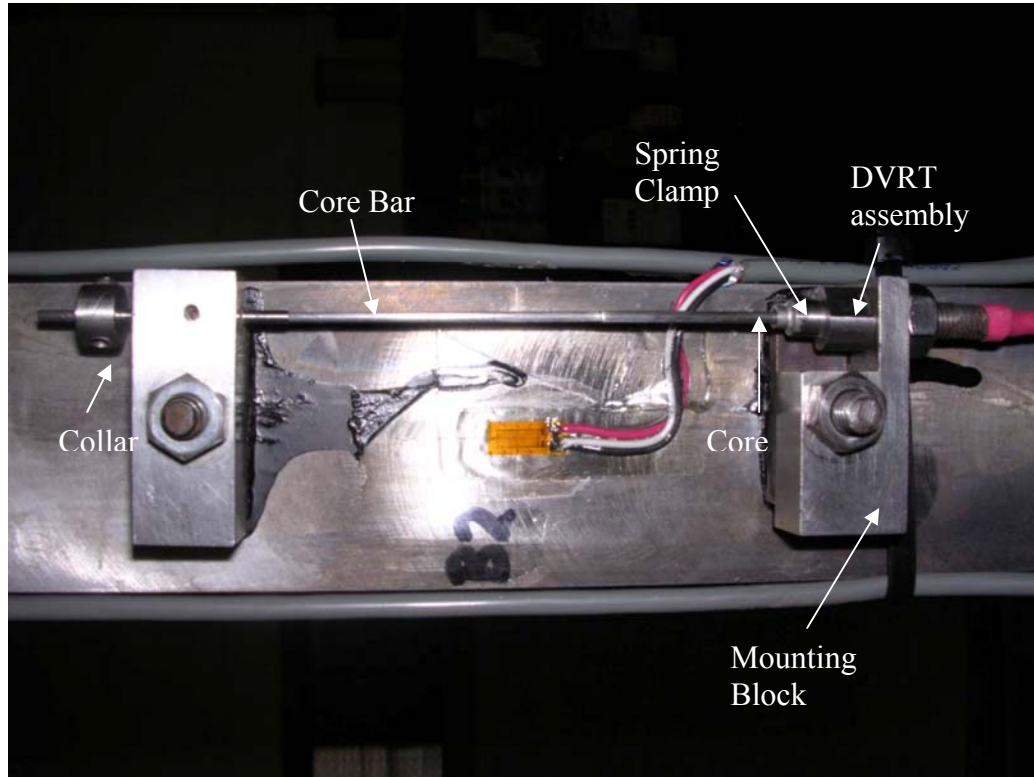


Figure 5.2 Peak Gauge Mounted

When installed the core on the peak gauge is set as close to the null position as possible and the voltage reading is taken for the gauge. The voltage reading is then taken after the desired period of time and the peak strain from that time period can be determined.

MicroStrain has given the University of Delaware two peak gauges to use for testing. The two gauges have been labeled Red and Yellow.

5.3 Peak Gauge Calibration

Each peak gauge has a unique calibration constant that depends on the instrumentation used to read the gauge. To obtain the calibration constant, which allows the conversion of voltage to displacement and in-turn strain/stress, a calibration

of the gauge must be preformed. MicroStrain calibrates each gauge but calibration can also be done in the lab, to check if the calibration constant has changed for any reason or if any parameters change.

To calibrate the gauges the core and sensor are mounted in a transducer calibration fixture, which consists of a fixture for mounting the gauge and a micrometer, as seen

in

Figure 5.3 and Figure 5.4. The gauges are set up so that a decrease in voltage indicates a tensile strain on the gauge. The displacement and voltage readings are taken and the linear range is used as the sensitive region and used for testing. The slope of the linear range is used as the calibration constant. The calibration curves for the gauges being read by the Smart Motherboard reader box can be seen in Figure 5.5, Figure 5.6, and Figure 5.7.

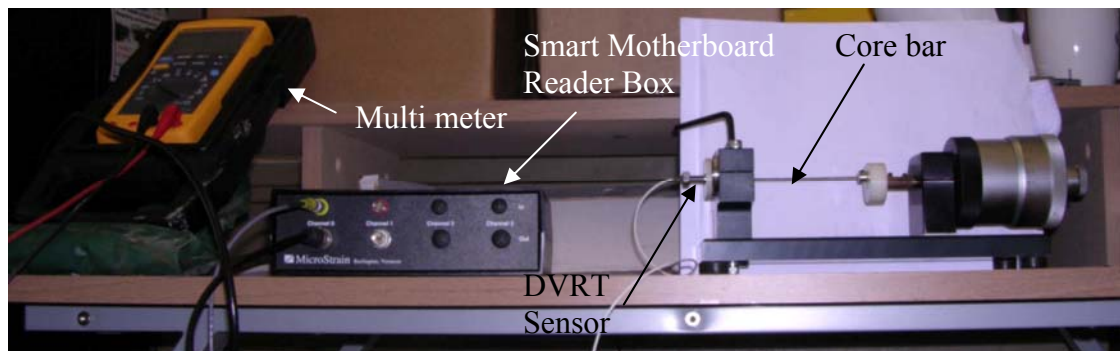


Figure 5.3 Calibration Set-up with Smart Motherboard Reader Box

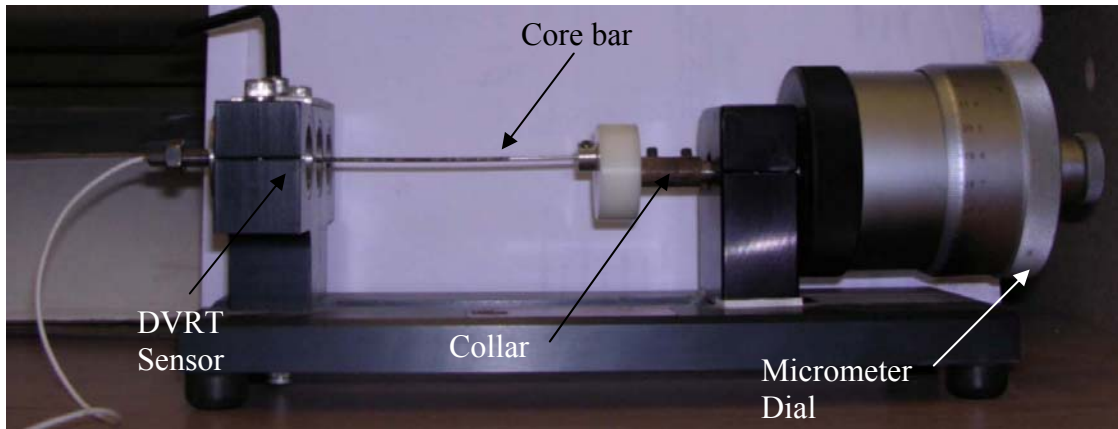


Figure 5.4 Calibration Set-up Micrometer Close up View

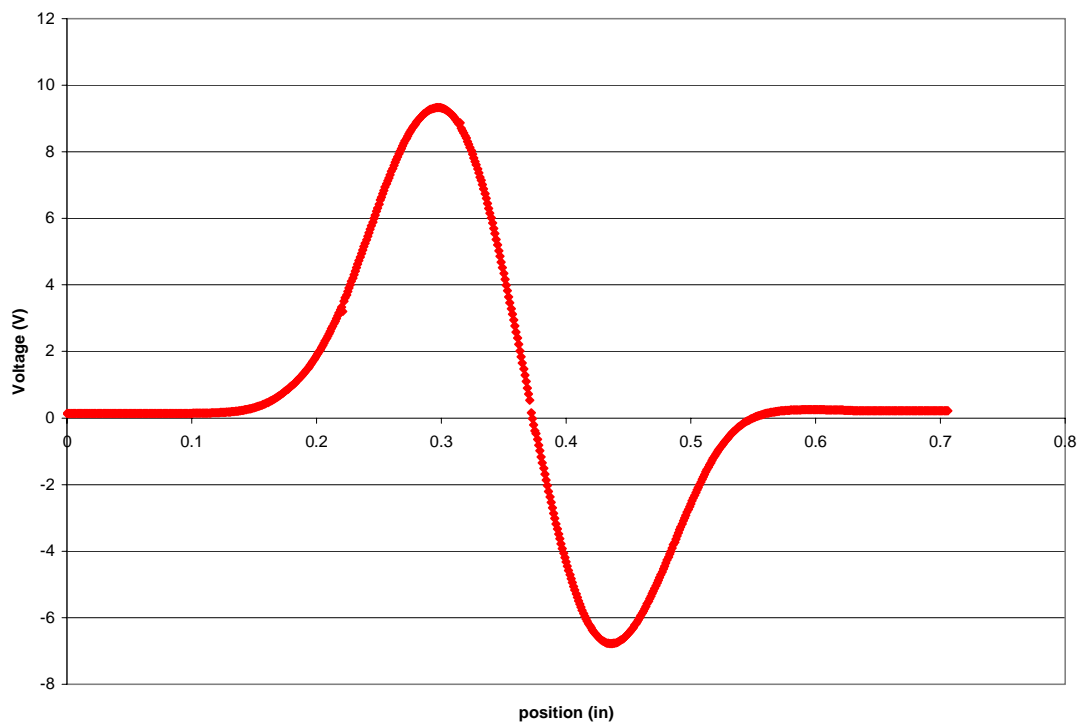


Figure 5.5 Red Gauge Calibration Curve Collected Using Smart Motherboard Reader Box at University of Delaware

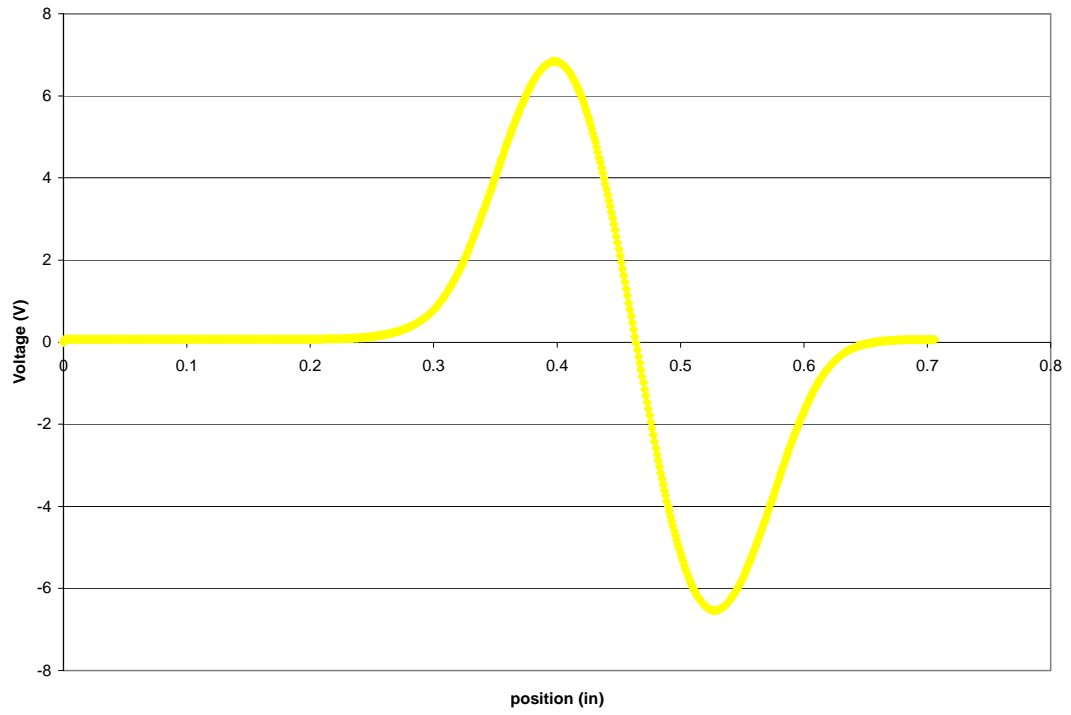


Figure 5.6 Yellow Gauge Calibration Curve Collected Using Smart Motherboard Reader Box at the University of Delaware

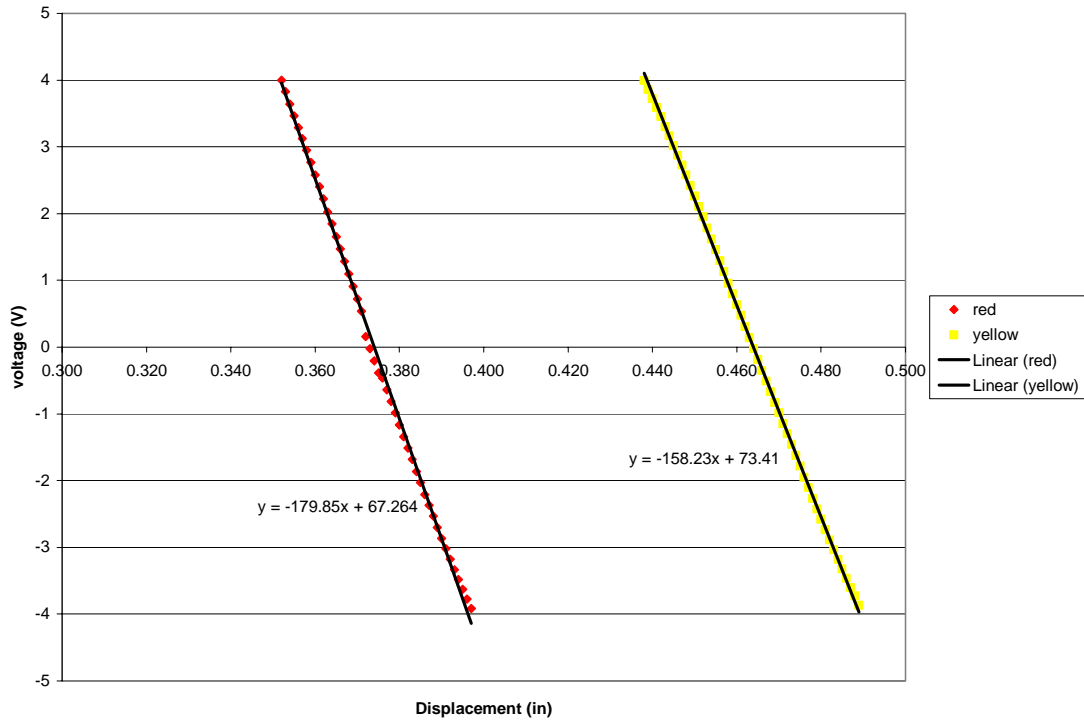


Figure 5.7 Linear Range of Calibration for Red and Yellow Gauge Collected Using Smart Motherboard Reader Box at the University of Delaware

The calibration constants are slightly different for each type of instrumentation used to read the gauge. The plots shown in Figure 5.5, Figure 5.6, and Figure 5.7 are typical results for calibration of the peak gauge. After these plots were obtained the peak gauges were adjusted by MicroStrain to increase sensitivity and new calibration constants were obtained. For the testing done in the following sections the calibration constants given by MicroStrain, $1.1311 \mu\epsilon/mV$ for the Red Gauge and $0.6067 \mu\epsilon/mV$ for the Yellow Gauge, were used.

5.4 Temperature Testing

One problem that arises with any mechanical or electrical system is temperature sensitivity. The peak gauge was made so that temperature variations would not play a huge role in the variation of the readings. There are different pieces that could affect the readings when temperature is concerned. When a temperature change takes place the specimen the gauge is mounted on can expand or contract. If the core bar is not made of the same material as the specimen, the core may move in the DVRT causing a change in reading and an apparent strain to be seen. This was compensated for by making the rod out of a material which has a similar thermal expansion coefficient as the steel that most bridges are made out of. To check the temperature effects of the gauges temperature testing was preformed as described in the following section.

A few different sets of temperature tests were preformed on the two peak gauges to see the full effect of temperature on the sensors. The first set of temperature tests were conducted at the University of Delaware in a temperature chamber in the Center for Composite Materials laboratory. The chamber was set up to run a temperature cycle from room temperature down to as close to 0° F as possible and then up to as close to 120° F as possible. To optimize the extreme temperature values the chamber was set to hold the peak temperatures for 10min to allow the chamber to reach as close to those peak values as possible. The increase in temperature from 0 to 120 was set to take 60 minutes. The chamber was then set to cool down to room temperature over 20 minutes. This temperature cycle was then repeated as necessary for each test. The peak gauges were mounted to a steel plate using steel tabs welded to the plate. These steel plates were hung on a bent metal bar and placed in the chamber. The gauge set ups can be seen in Figure 5.8.

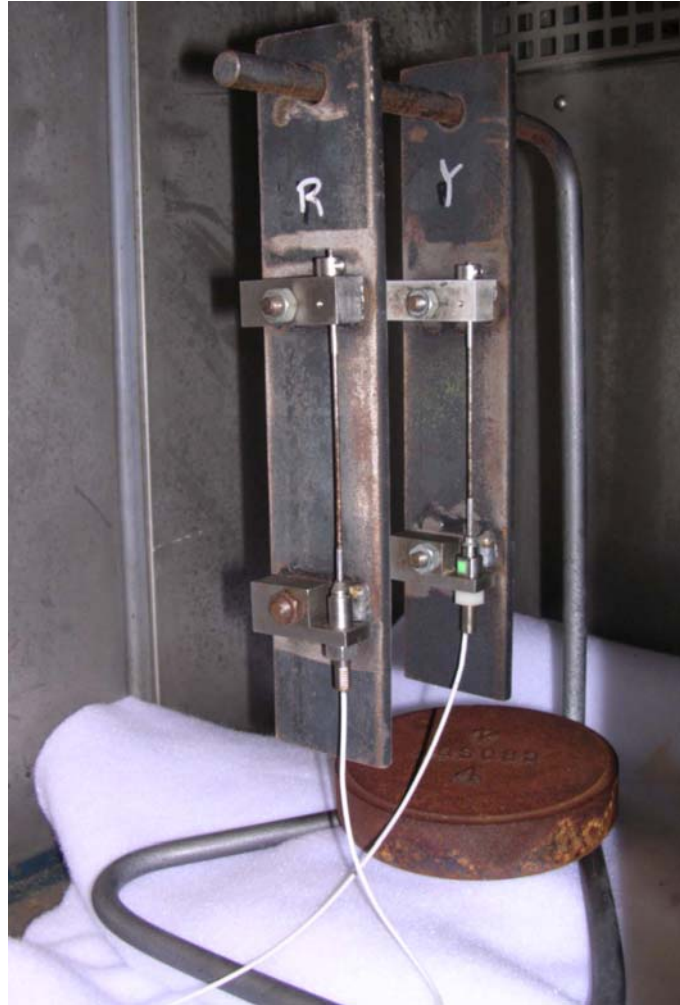


Figure 5.8 Peak Gauge Temperature Testing Set-up

To read the peak gauge voltage the gauges were connected to MicroStrain's Smart Motherboard system, which is in the reader box seen in Figure 5.9 and Figure 5.10. The Smart Motherboard system is a signal conditioning system made by MicroStrain that allows the reading from the peak gauges to be read by a multimeter or other voltage reading devices. The Smart Motherboard is supplied with voltage from either two 9 volt batteries or a 12 volt DC power supply. This Smart Motherboard system can also be hooked up to MicroStrains Smart Motherboard

Software by way of a serial cable located on the back of the reader box. The software can be downloaded from MicroStrain's website

<http://www.microstrain.com/updates/software.htm>.

For the first set of tests each gauge was hooked up to a channel on the reader box and then the channel output was connected to the Micro Measurements System 5000 data acquisition system. The System 5000 system allows different voltage outputs to be recorded in one data file, all on the same time scale. Using the System 5000 system allowed us to also collect temperature data along with voltage readings in the same data file for easy comparison after the tests.



Figure 5.9 Peak Gauge Reader Box Front View



Figure 5.10 Peak Gauge Reader Box Back View

5.4.1 Temperature Test 1

For the first temperature test the peak gauges were placed in the chamber but the cores were removed from the DVRTs so that the temperature effects of just the instrumentation would be collected and not the effect of the mechanical system being subjected to temperature changes. This test took the DVRTs through one temperature cycle from a minimum temperature of approximately 9°F to a maximum of approximately 120°F. These results can be seen in Figure 5.11. From these results it can be seen that there is some hysteresis or drain in the data occurring. We used 9 volt batteries in the power supply for the reader box for this test and investigating the cause of the unpredictability in the data we found the batteries in the reader box were very low and read around only 7volts each. The batteries were replaced and the test was redone as shown in 5.4.2.

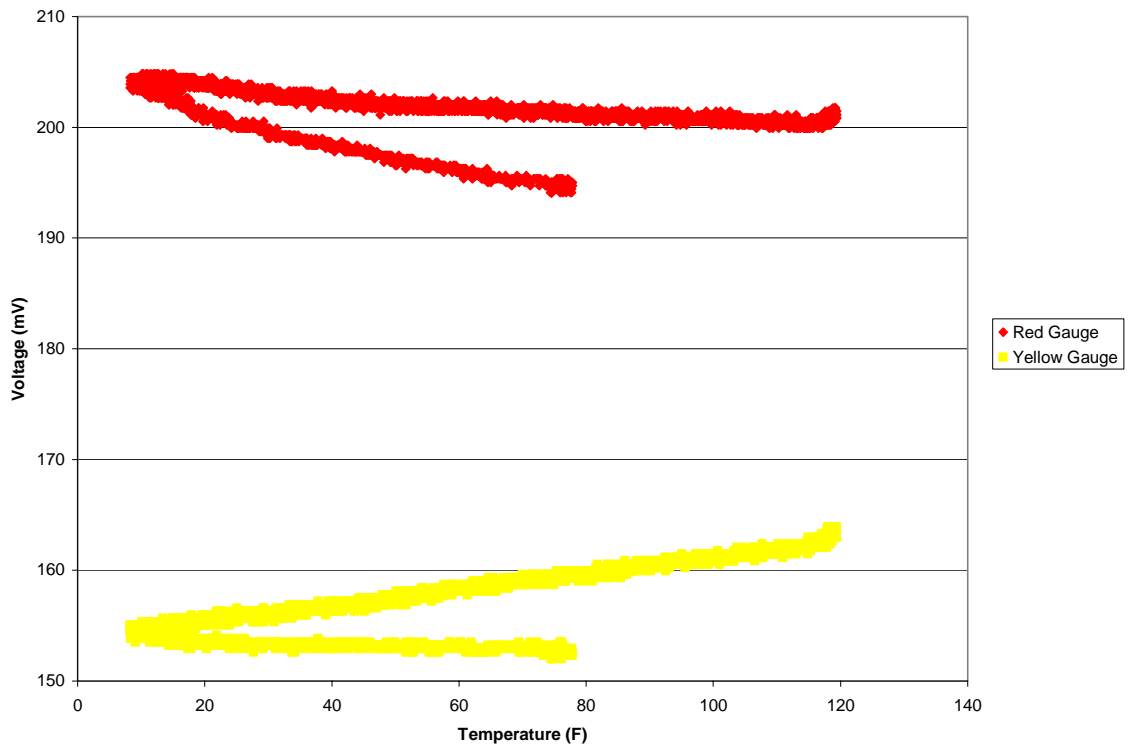


Figure 5.11 DVRT Temperature Test

5.4.2 Temperature Test 2

For this test we repeated temperature test 1 but we also recorded the voltage readings of the two batteries which were the power supplies for the reader box. This was to see how the voltage dropped through the test and how this may affect the results given by the peak gauges. The results for Test 2 were similar to Test 1 as seen in Figure 5.12. There was suspicion that the unpredicted data was due to the voltage of the batteries. To find out if the voltage problem is the cause of the unpredictable data a voltage supply was obtained from MicroStrain to repeat tests.

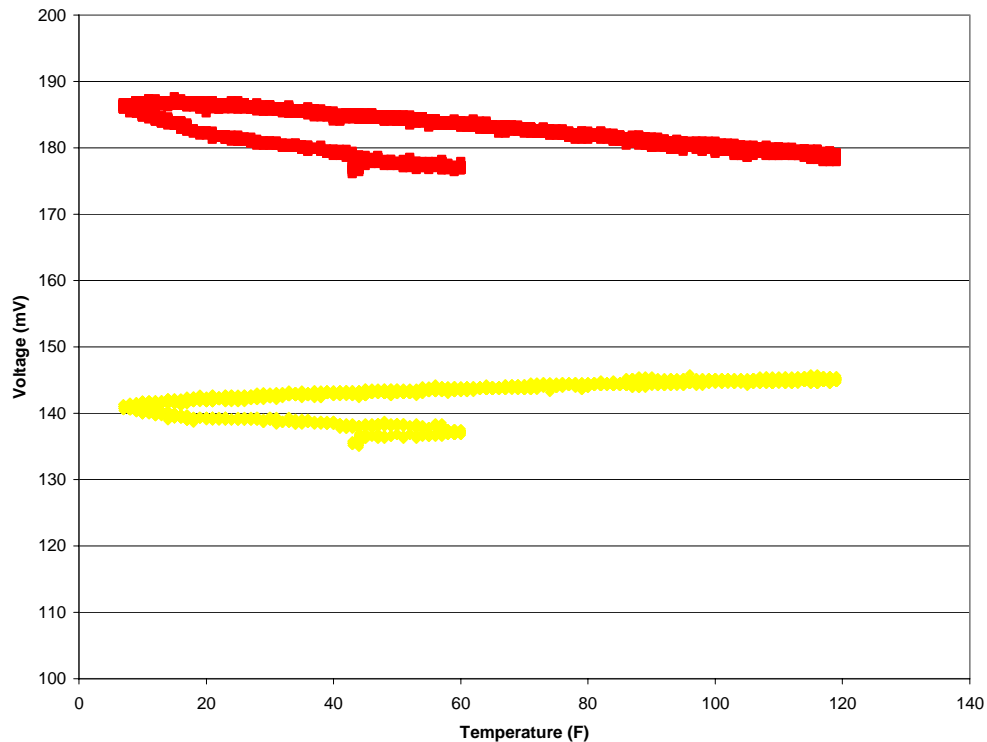


Figure 5.12 DVRT Temperature Test 2

5.4.3 Temperature Test 3

A DC power supply was connected to the reader box for this test and the test run for Tests 1 and 2 was repeated. The temperature cycle for this test was repeated for 4 cycles. The results of this test can be seen in Figure 5.13. The results are more linear for this test than the previous. There is still some hysteresis in the data, but much less than the previous tests. The data can be fit with a linear fit to determine the temperature calibration for the DVRTs.

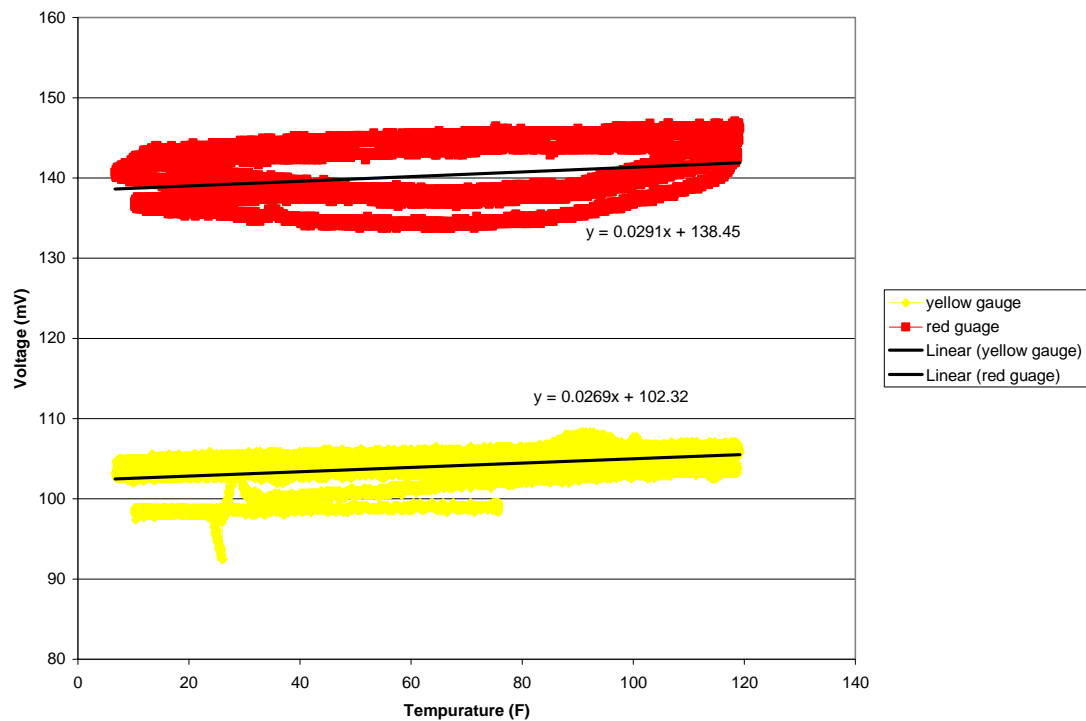


Figure 5.13 DVRT Temperature Test with Power Supply

5.4.4 Temperature Test 4

The next test that was performed was setting the peak gauges up in the chamber with the cores inserted but the collars not secured on them so that there is no strain on the core bar but the effect of the core material on the voltage of the gauges during the temperature sweep can be seen. The test was run through almost 3

temperature cycles before stopping. The results from this test can be seen in Figure 5.14. These results have again a linear trend that can be fit and a temperature calibration constant can be determined. This calibration constant is different from that of the DVRT alone but is still linear.

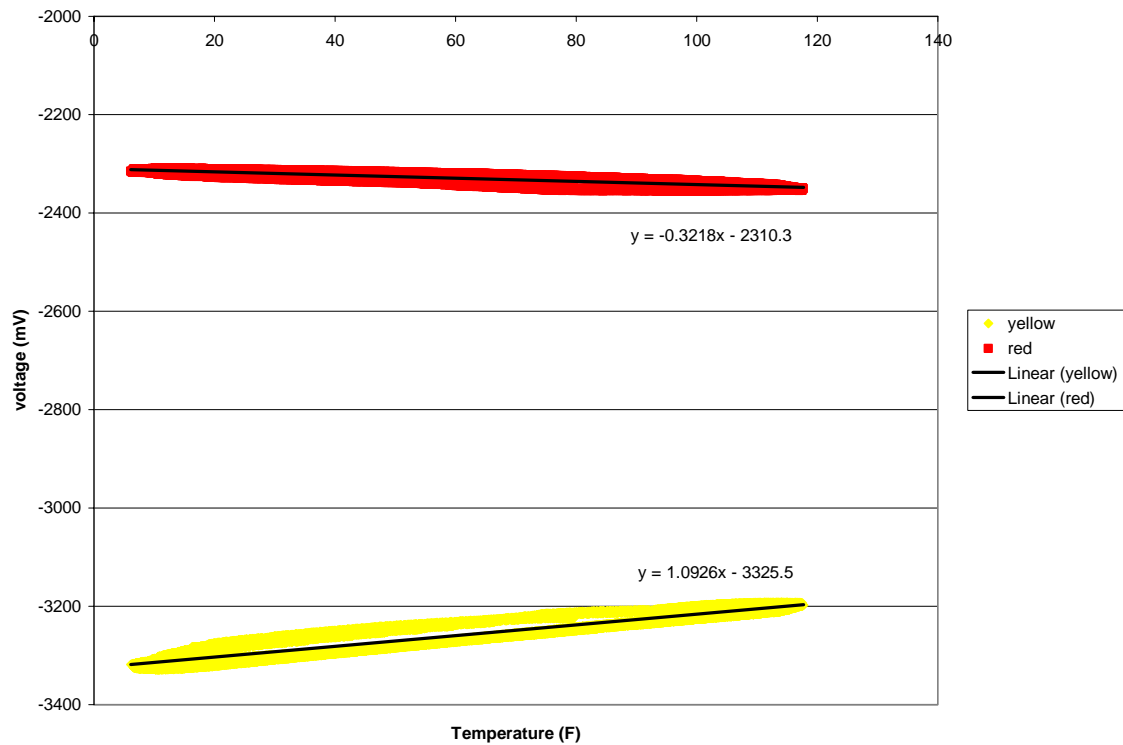


Figure 5.14 Peak Gauge Temperature Test without Collar Fastened

5.4.5 Temperature Test 5

The final test in this set of temperature tests was to set up the peak gauges as they would be in the field. The cores were inserted and the gauges were set to their null positions at room temperature with the collar securely fastened to hold the null position. Setting the gauges to their null positions at the lowest temperature was difficult since the chamber had to be opened in order to accomplish this so for this round of testing the gauges were zeroed at room temperature and then run through the

temperature cycles. This test ran through almost 3 temperature cycles and the results are seen in Figure 5.15. The results again show a linear trend in the data and the nearly same calibration constant as in Test 4.

The temperature sensitivity results for the three tests are presented in Table 5.1. From seeing that the constants from Test 4 and 5 match it can be seen that the instrumentation itself is affected by temperature but can be corrected for if the temperatures when the gauge is installed and when the reading is taken are known. If the gauge was straining due to temperature we should have seen the readings reach a peak value and then stay constant for the remainder of Test 5, or an offset would be apparent in Test 5's results. This means that as long as the temperature is known at the time of deployment of the gauges and the temperature at the time of reading of the gauges the correction in voltage readings can be made and an accurate reading can be made in the field. An example temperature correction and stress calculation can be seen in Section 5.4.11.

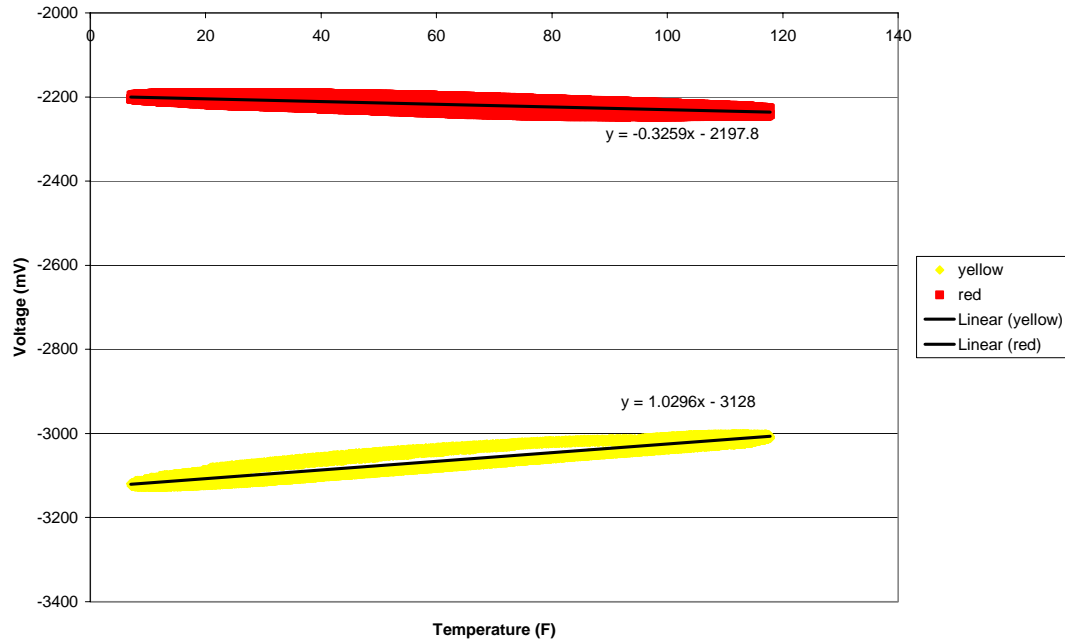


Figure 5.15 Peak Gauge Temperature Test as Installed in Field

Test Number	Gauge	Temperature Calibration	Test Description
3	Red	0.0291 mV/°F	DVRT alone
	Yellow	0.0269 mV/°F	
4	Red	-0.322 mV/°F	Gauge with Collar not attached
	Yellow	1.093 mV/°F	
5	Red	-0.326 mV/°F	Gauge In-Field Installed
	Yellow	1.030 mV/°F	

Table 5.1 CCM Peak Gauge Temperature Testing Results

After completing this first set of temperature tests MicroStrain suggested switching to DEMOD-DC signal conditioners (Figure 5.16) for readings instead of the

Smart Motherboard reader box. Using the DEMOD-DC's helps with collecting data in the field by eliminating a power supply. This DEMOD is wired to supply the necessary voltage for the DVRT and then returns the output voltage in a DC form to be read by a multimeter. This DEMOD for the peak detect gauge was wired and set up in a box like seen in Figure 5.17 and Figure 5.18. This box provides an easy way to take readings in the field. The BNC connection creates a quick connect for the multimeter to the DEMOD and then the DEMOD connection connects either directly into the DVRT or an extension cable that can be connected to the DVRT and run to a convenient location for readings to be taken. The DEMOD uses one 9 volt battery to supply power to the gauges, but because of the circuitry does not have the same power drain problems as seen with the reader box, and can also be used for long-term data collection like the temperature tests.



Figure 5.16 DEMOD-DC



Figure 5.17 DEMOD Reader Box for Red Peak Gauge (Closed View)

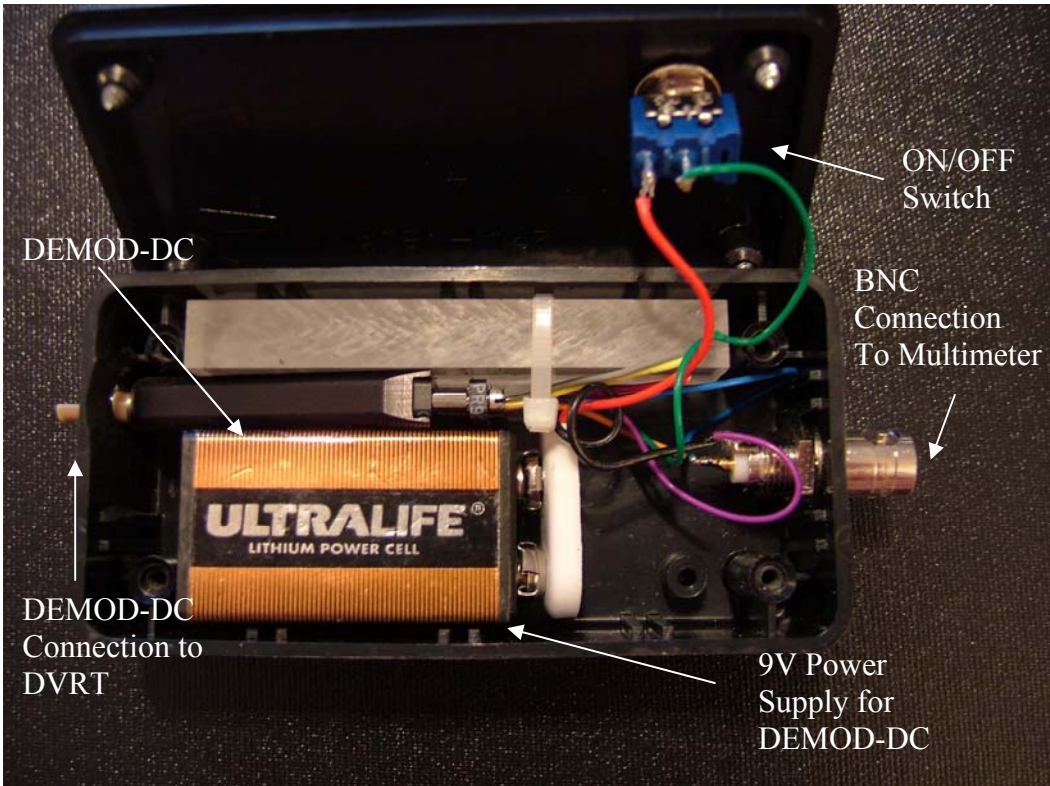


Figure 5.18 DEMOD Reader Box (open view)

After switching to the DEMOD-DC's the temperature testing was redone to get the new temperature calibration for each gauge with the new instrumentation. These tests were done at MicroStrain in their temperature chamber.

5.4.6 MicroStrain Temperature Test 1

This test was done like Temperature Test 1 performed in the CCM chamber. This test had only the DVRT's in the chamber. The test ran through one temperature sweep where the chamber was set to step through a temperature cycle at 10°C steps and hold each step for 10min. The cycle ran from room temperature or the chamber temperature down to around -15°C then up to 54°C and back to room temperature. The results of this test can be seen in Figure 5.19.

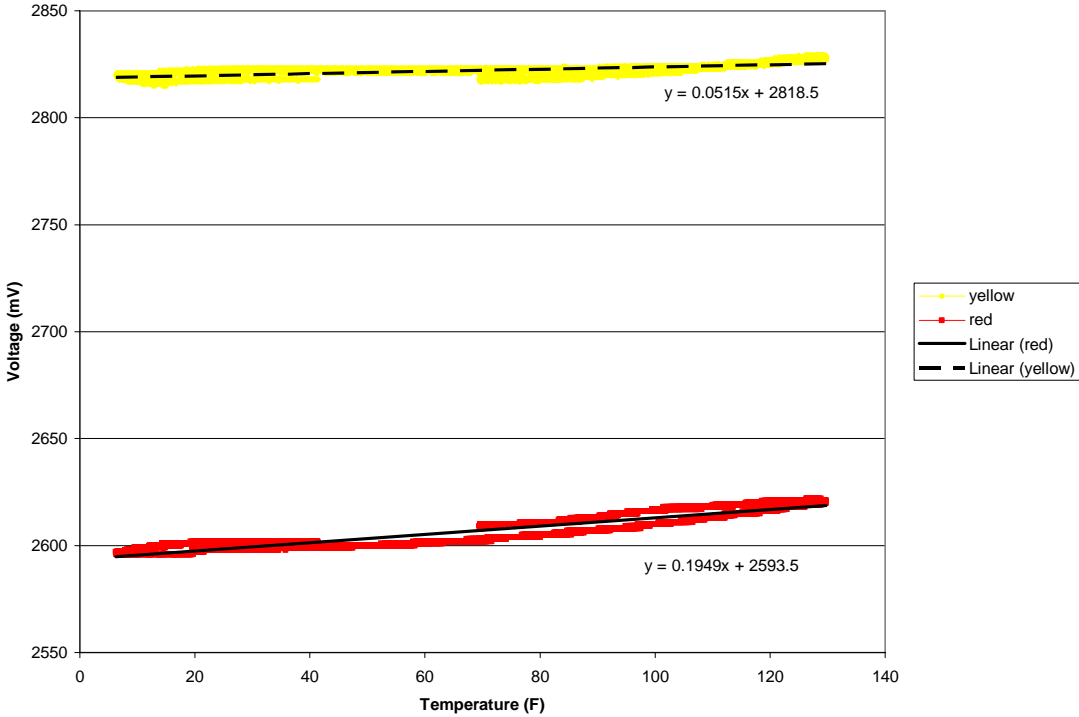


Figure 5.19 DVRT Temperature Test with DEMOD-DC Reader Box

5.4.7 MicroStrain Temperature Test 2

This test was similar to that of Temperature Test 4 performed in the CCM chamber. The peak gauges were installed as in the field except for the collar being left off. The test ran through one temperature sweep like in the MicroStrain Temperature Test 1. The results can be seen in Figure 5.20.

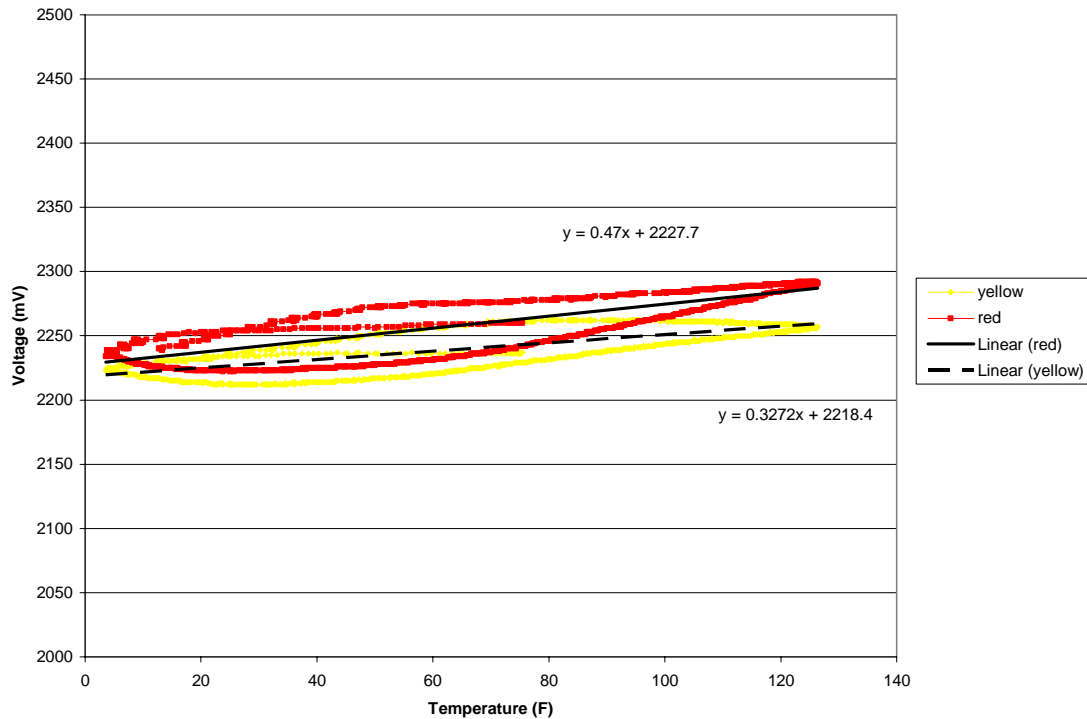


Figure 5.20 Peak Gauge Temperature Test without Collar Fastened with DEMOD DC Reader Box

5.4.8 MicroStrain Temperature Test 3

This test was run like that of the test performed at the CCM Temperature Test 3. This test had the peak gauge installed as in the field. The test ran through 2.5 temperature cycles like that done in the previous MicroStrain temperature tests. The results can be seen in Figure 5.21.

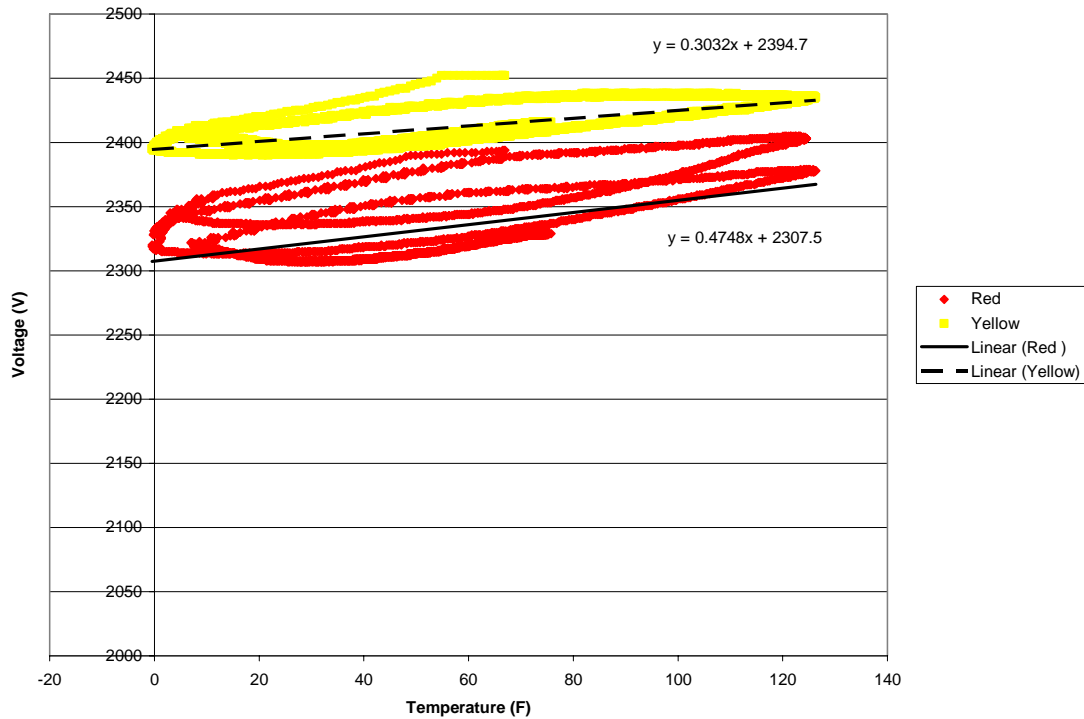


Figure 5.21 Peak Gauge Temperature Test as Installed in Field with DEMOD-DC Reader Box

5.4.9 MicroStrain Temperature Test 4

The data seen in the MicroStrain Temperature Test 3 caused us to investigate the Red Peak Gauge more closely. This gauge had a drift or hysteresis in the data collected. The cause of this drift was assumed to be mechanical. The one cause we could think of was that the core in the Red Peak Gauge was too big and friction is causing the drift in the data. To test this theory the cores were swapped in the two gauges. Since the core in the red gauge was larger in size than the yellow gauge the test should look the same as the previous test except the yellow should drift and the red should not drift if the problem was with the core size. This test was run the same as MicroStrain Temperature Test 3 but only went through 1.5 temperature sweeps. The results can be seen in Figure 5.22

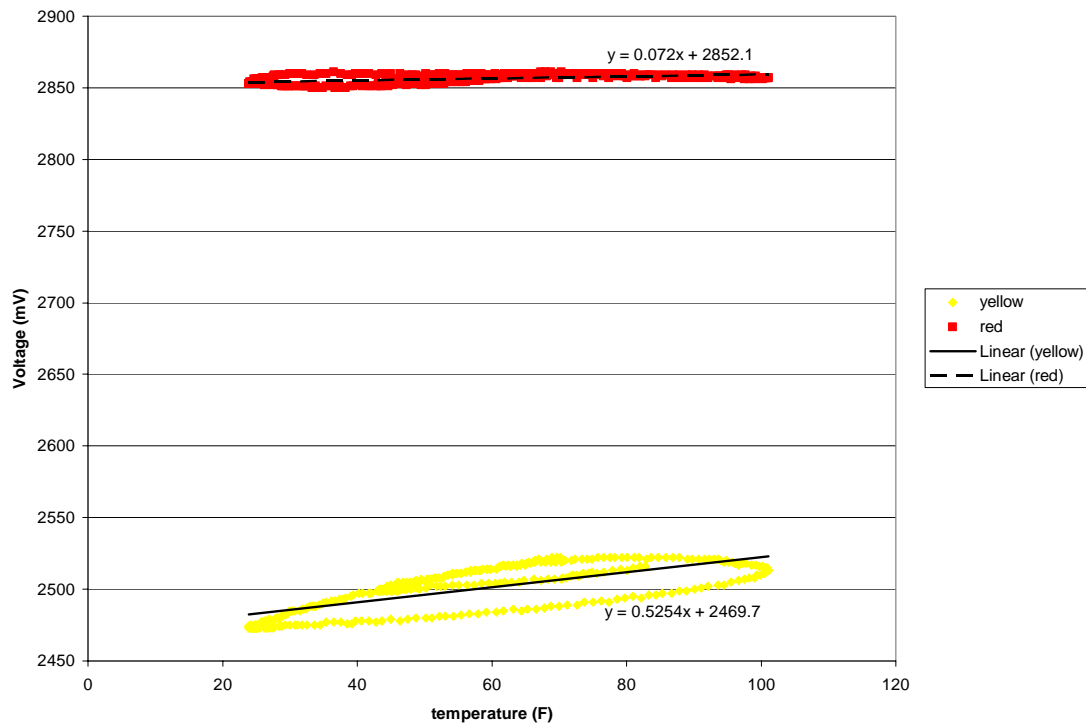


Figure 5.22 Peak Gauge Temperature Test as Installed in Field with Cores Swapped

The results of Test 4 look the same as Test 3 except swapped therefore the size of the core in the red gauge was thought to be the cause of the drift in the results.

5.4.10 MicroStrain Temperature Test 5

Because of the results of the previous test the core on the red gauge was rebuilt. A final temperature test was done to get the temperature calibration constant for the red gauge with the new core and to make sure the drift problem was fixed. The test was done as in the past three tests with the gauges installed as in the field and run through 2.5 temperature cycles. The results can be seen in Figure 5.23.

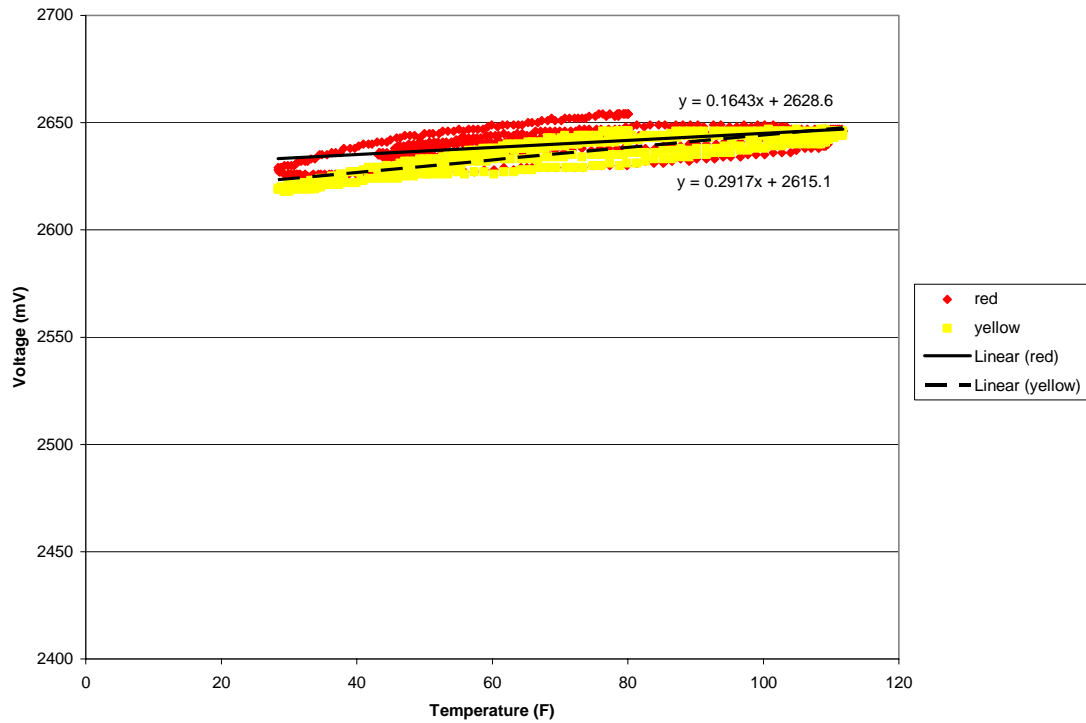


Figure 5.23 Peak Gauge Temperature Test Installed as in Field, Red Gauge with New Core

With the DVRT set up the temperature calibrations are 0.164 mV/°F for the Red Gauge and 0.292 mV/°F for the Yellow Gauge. The calibration constants given by MicroStrain are 1.1311 $\mu\epsilon$ /mV for the Red Gauge and 0.6067 $\mu\epsilon$ /mV for the Yellow Gauge. These calibration factors should be used for all future testing. See Section 5.4.11 for example of strain calculations.

5.4.11 Strain and Stress Calculation Example for Peak Gauge

If the following field data was collected where the Red gauge was placed on one bridge and the yellow gauge on another bridge the following processes would apply:

Gauge	Initial		Final	
	Temperature (°F)	Voltage (V)	Temperature (°F)	Voltage (V)
RED	67	2.327	75	2.151
YELLOW	78	2.660	64	2.342

Red Gauge:

$$\Delta T = 8 \quad \Delta V = 0.176 \text{ V} = 176 \text{ mV}$$

$$\text{Voltage due to temperature swing} = 0.164 \text{ mV/}^\circ\text{F} * 8^\circ\text{F} = 1.312 \text{ mV}$$

ΔV^* = voltage change due to strain alone

$$\Delta V^* = \Delta V - \text{voltage due to temperature} = 176 - 1.312 = 174.688 \text{ mV}$$

$$\epsilon = 174.688 \text{ mV} * 1.1311 \mu\epsilon/\text{mV} = 197.6 \mu\epsilon$$

$$\sigma = 197.6 \mu\epsilon * 29000 \text{ ksi} = 5.7 \text{ ksi}$$

Yellow Gauge:

$$\Delta T = -14 \quad \Delta V = 0.318 \text{ V} = 318 \text{ mV}$$

$$\text{Voltage due to temperature swing} = 0.292 \text{ mV/}^\circ\text{F} * -14^\circ\text{F} = -4.088 \text{ mV}$$

ΔV^* = voltage change due to strain alone

$$\Delta V^* = \Delta V - \text{voltage due to temperature} = 318 - (-4.088) = 322.088 \text{ mV}$$

$$\epsilon = 322.088 \text{ mV} * 0.6067 \mu\epsilon/\text{mV} = 195.41 \mu\epsilon$$

$$\sigma = 195.41 \mu\epsilon * 29000 \text{ ksi} = 5.67 \text{ ksi}$$

If the temperature swing is negative the voltage due to temperature swing would be added to the voltage change seen instead of subtracted.

5.5 Tension Testing

After temperature testing the next lab test done was a tensile test. A flat steel bar was chosen for the test. The specimen was gauged with one foil gauge on each side at the middle of the bar, and the peak gauges were mounted centered over the foil gauges on each side of the bar. The peak gauges were installed on the bar with the center of the transducer centered over the foil gauges as seen in Figure 5.2. Six tests were run where the load on the bar was increased to approximately 3500 lbs. For test 1, 2 and 5 the Red gauge was mounted on Side B of the bar and the Yellow gauge was mounted to Side A of the bar. For tests 3 and 4 the Red gauge was mounted to Side A and the Yellow gauge was mounted to Side B. The results were plotted for each test which can be seen in Figure 5.24, Figure 5.25, Figure 5.26, Figure 5.27, Figure 5.28, and Figure 5.29. The results are summarized in Table 5.2.

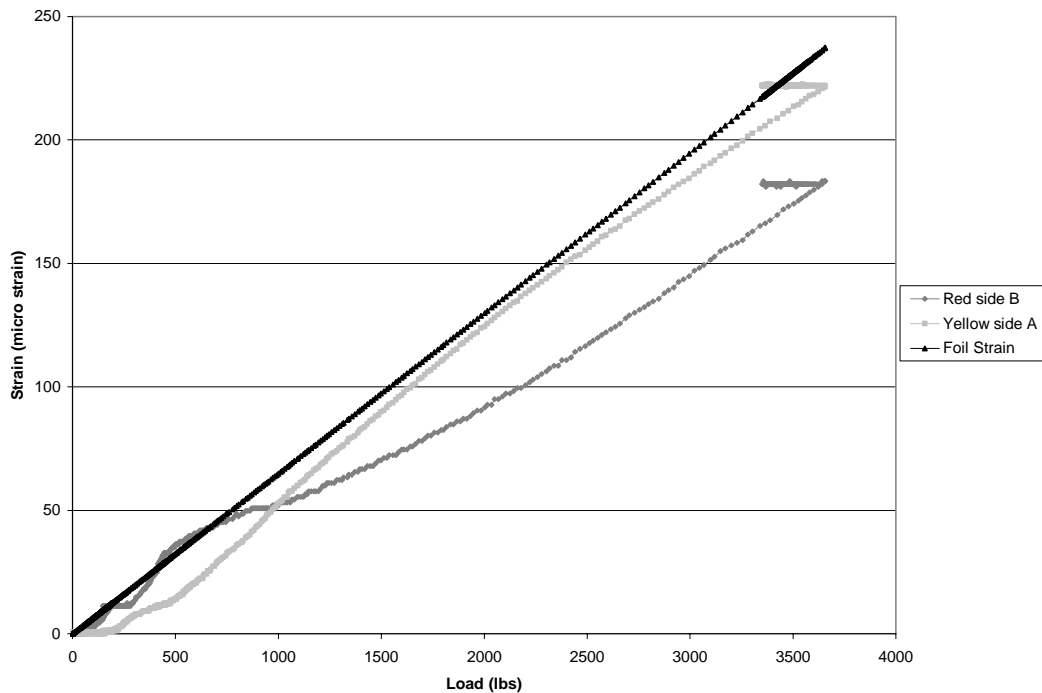


Figure 5.24 Strain Data for Test 1 of Tensile Specimen

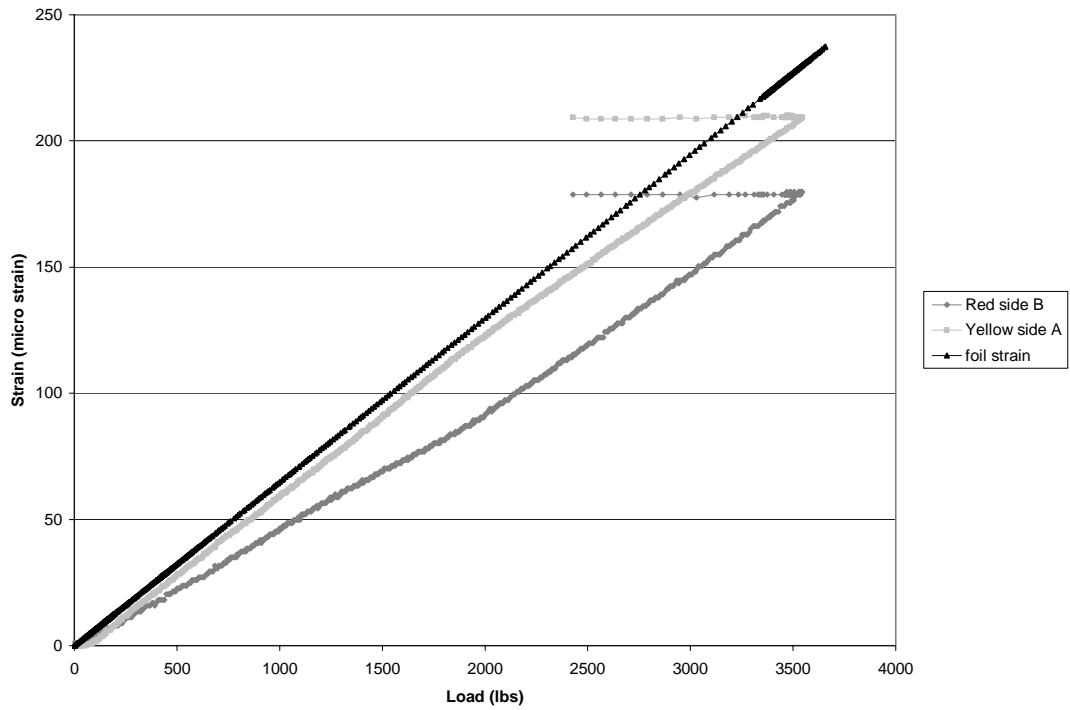


Figure 5.25 Strain Data for Test 2 of Tensile Specimen

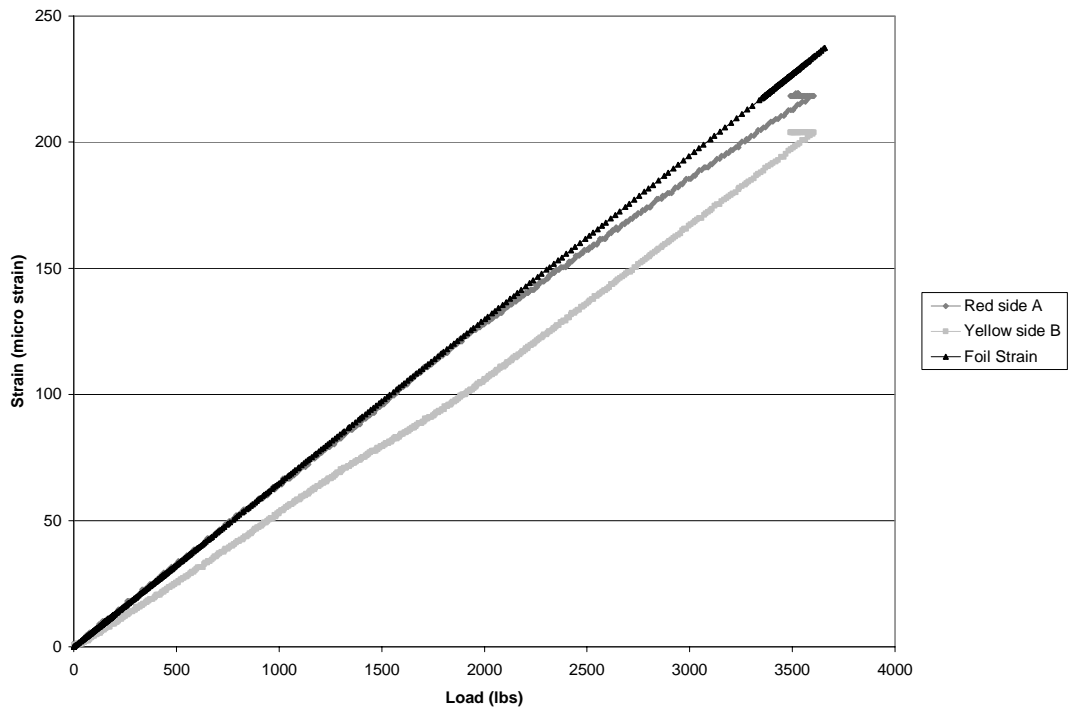


Figure 5.26 Strain Data for Test 3 of Tensile Specimen

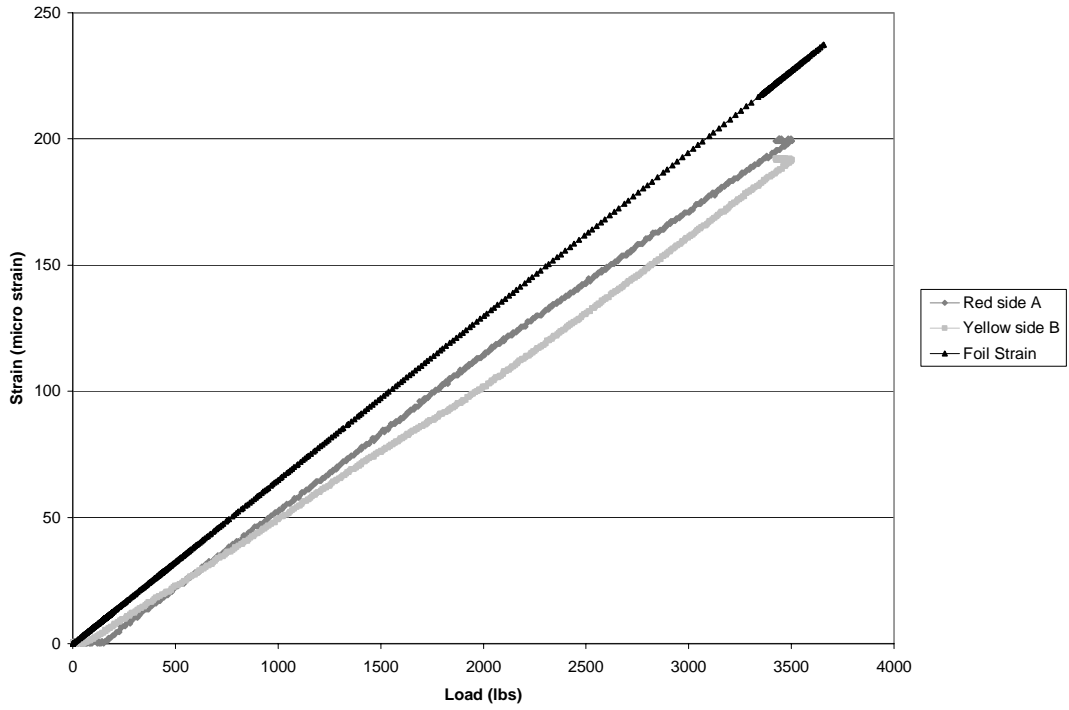


Figure 5.27 Strain Data for Test 4 of Tensile Specimen

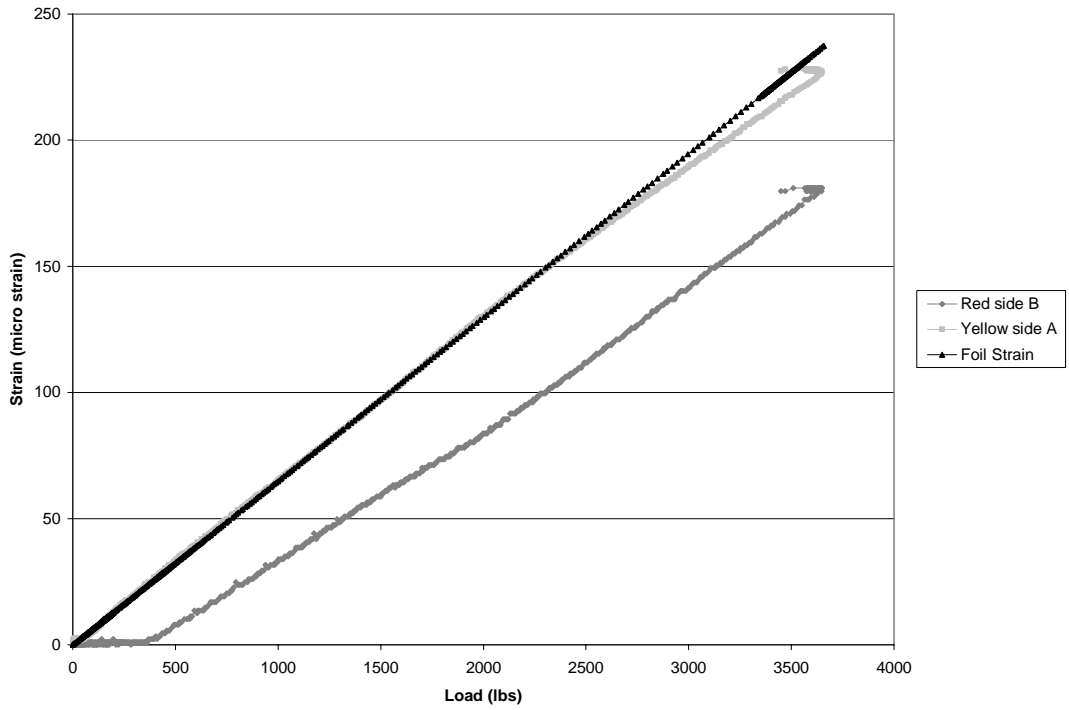


Figure 5.28 Strain Data for Test 5 of Tensile Specimen

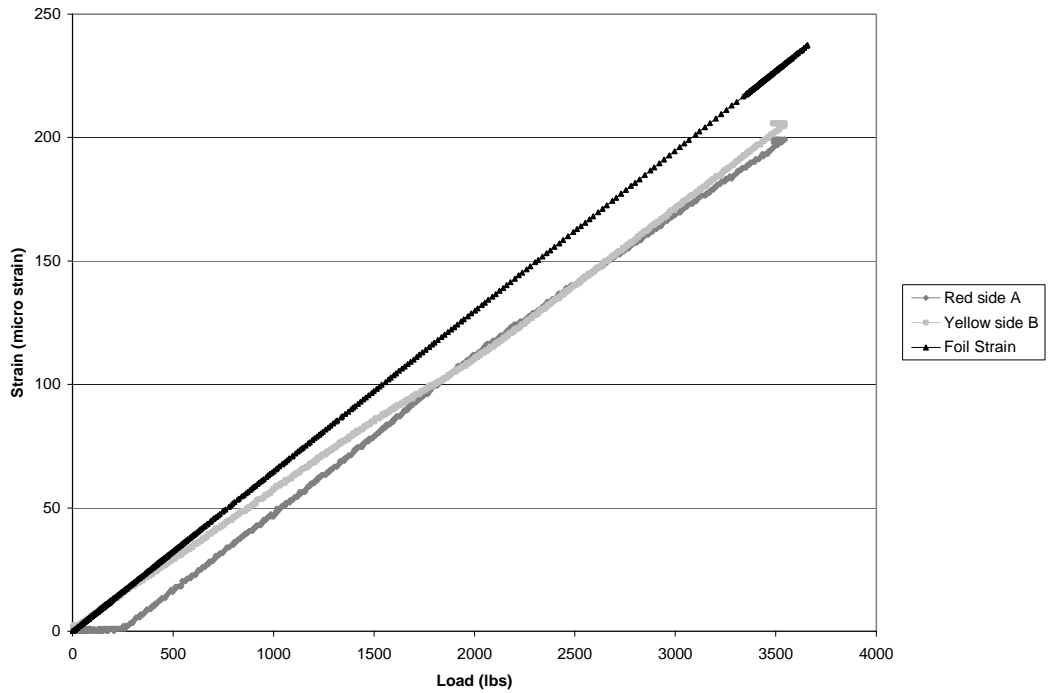


Figure 5.29 Strain Data for Test 6 of Tensile Specimen

Test	Peak Gauge/Side	Peak Strain ($\mu\epsilon$)	Foil Gauge Reading ($\mu\epsilon$)/ Side	% difference
1	Red / B	183	243/B	24.60
	Yellow / A	223	239/A	6.84
2	Red / B	180	227/B	20.77
	Yellow / A	210	228/A	7.93
3	Red / A	219	236/A	7.07
	Yellow / B	204	236/B	13.25
4	Red / A	200	225/A	11.00
	Yellow / B	192	225/B	14.50
5	Red / B	181	234/B	22.70
	Yellow / A	228	237/A	3.75
6	Red / A	199	228/A	12.17
	Yellow / B	206	229/B	10.20

Table 5.2 Tension Test Data

From the results in Table 5.2 the average percent difference for each side of the bar was calculated. The average percent difference from the peak gauges to the foil strain gauges for side A was 8.21%. The average percent difference from the peak gauges to the foil strain gauges for side B was 17.67%. These results tend to imply that there was something with the specimen which caused the peak gauges to not read correctly on side B. If the specimen was bent or some induced torsion or bending was in the bar since the peak gauge has a much larger gauge length than the foil gauge the peak gauge would not read the same as the foil gauge. The readings from side A are all within 10% of the strain gauge readings, which are good results. In each of the test there seems to be a bit of an offset for at least one of the gauges and this could be attributed to the mounting method and if the collar was not correctly seated against the mounting blocks would not start to read strain until a enough movement occurred to cause the collar to come in contact with the mounting blocks correctly.

5.6 Future Testing

The next step in testing is to take the peak gauges out to the field and put the gauges on a bridge with another data collecting system and see how the peak readings compare. One condition that may be seen in the field but not in the lab is vibration. Some testing should be done on vibration to see if it will affect the readings from the peak gauges when installed on a live structure.

Some preliminary testing was done on vibrations. The spring clamp is a simple assembly which has metal springs which hold the core so that it does not slide out with gravity or vibration. The peak gauge assembly was mounted on a plate and then placed on a concrete cylinder vibrator and the voltage reading for the gauges

were monitored. It was seen that with larger/looser spring clamps the vibration caused the voltage readings to change. This in the field would correspond to a strain that did not actually occur but the gauge would record. This vibration would cause readings to be larger than should have been recorded because the core would vibrate out of the sensor. With the vibrations seen on most of the test bridges the vibration issue may need to be investigated more prior to field testing in order to fix the vibration or quantify the amount of vibration required to cause erroneous data.

Chapter 6

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

6.1 Rating Factors

Rating factors calculated with traditional rating equations represent a projected 50 year rating. This 50 year rating is calculated using a design load which represents the heaviest load that the bridge is expected to see in 50 years. The rating of the bridge is very dependent upon how the bridge is modeled, and the numerical model is based upon original as-built properties combined with inspection findings. As a result of this process, ratings found can be quite conservative. On the other hand, rating calculated using in-service data and the proposed method eliminates the uncertainty of the numerical model. If two weeks of data is collected, a definitive two-week rating can be computed. By using extreme value theory, the method can be extended to compute a two-year rating. While projected ratings such as two-year ratings were not computed here, the method can be extended to accomplish this in the future. This two year rating could meet federal standards and give the bridge a realistic rating for the time between biannual inspections. The new rating method might allow larger loads to pass over a bridge and will produce a more quantitative way of measuring decreased load carrying capacity.

From this study it can be seen that the two week rating factors are, for the most part, between three and eight times greater than the 50 year rating factor. It is

believed that the projected two year rating from this data would also lead to an increased rating factor. Collecting data on a network of bridges every two years not only would give a better idea of how certain bridges are aging, but also would give a comprehensive view of trends in truck volume and weights in a specific area.

Another alternative to projecting a two week data set to yield a two year rating is to collect a two year peak stress. Using the peak gauge, a two year peak stress can be measured, and a two year rating can be computed without projection. One assumption that was made throughout this research was that the girder being monitored (and the location on that girder) was the location of the highest stress. This assumption may not be correct and may need to be corrected for in the future, especially if a two year rating was to be determined directly from collected data.

Rating Factors for bridges in Delaware are currently being computed using the Load Factor method. In the near future, Delaware will be switching to the Load and Resistance Factor method, and the calculations would have to be recomputed for the new rating method.

6.2 Future Studies and Recommendations

Use of the ISBMS allows histograms of stressed to be determined, and patterns in truck traffic to be observed for a two week period. If the peak gauge data is added to the data collected by the ISBMS, a better prediction of peak stress over longer periods of time can be determined. The load rating computed using the peak data would allow a two year load rating to be directly computed instead of the two week load rating which can be found from the ISBM system. This longer window will yield a very accurate estimate of what the bridge might see in the next two year period. A two year load rating would help DelDOT achieve a reliable load rating

between biennial inspections. In future studies, the peak gauge data will be incorporated with the ISBMS data, and load ratings for longer periods of time will be calculated.

Another area to study is how actual truck weight corresponds to the measured strains. At the present, the truck weights that cause the stresses collected by the ISBMS are unknown. All that is collected is a stress that is caused by some load crossing the bridge. This load may be caused by multiple trucks crossing the bridge at the same time or one heavy truck, and the weight of the trucks is completely unknown. To study how the loads that cause the stresses compare to the loads applied in the BRASS analysis, it would be interesting to compare Weigh in Motion (WIM) Data to the stresses measured during the monitoring. This would help identify the average weight of trucks crossing the bridge and help correlate the weight of the trucks to the measured stresses in the bridge. These measured stresses can then be compared to what stresses would be predicted by the same loads if the BRASS model is used. The comparison can enable the BRASS model to be calibrated and used to yield more accurate ratings.

Appendix I

BRIDGE 1-149 BRASS IN-PUT FILE

TLE BRIDGE NO. 1-149, S.R. 41 OVER RED CLAY CREEK, 76-10-007
TLE SIMPLE SPAN, STEEL-CONCRETE COMPOSITE, INTERIOR, BUILT 1989
COM *NOTE*: This File was modified on 08/08/95. Refer to Further Comments
ANL 1,0,4
XST 1,WN36X230
XSA 1,50
XSC 84.0,8.0,2.0,0,0,0
SPA 1,80.0,5
SPC 1,80.0,1
FIX 0,1,0,1,1,0
PS1 , ,4.5
PS2 8, ,60
COM DEAD LOADS STAGE 1: SLAB = 788 #', HAUNCH = 34 #/FT, SIP = 28
#/FT
COM DEAD LOADS STAGE 1: DIAPHRAGMS AND UTILITY HANGERS = 60
#/FT
COM DEAD LOADS STAGE 2: MEDIAN, SIDEWALK, PARAPET = 280 #/FT
DLD 1,0.910,0.280
COM *NOTE*: Live Load Dist. Factor Changed to 1.171 from 1.273
COM as per 1994 AASHTO Specs.
LLD 3, 1.171, 0.0, 50.0
TR1 HS20T,S220,S335,S437,T330,T435
TR2 T540
DES 3, 1
COM WS Parameters were Replaced with LF on 08/08/95
INV 1.3, 1.0, 1.67, 1.0, 1.0, 1.0
OPG 1.0, 1.0, 1.67, 1.0, 1.0, 1.0
COM POSTING LEVEL 3
PST 1.0, 1.0, 1.00, 1.0, 1.0, 1.0
SLD 1.0, 1.0, 1.0, 1.0, 1.0, 1.0
SL1 100,4
SL4 ,,1,0.5,8
SL1 104,4
SL1 105,4

BRIDGE 1-234 BRASS IN-PUT FILE

BRIDGE 1-234 BRASS IN-PUT FILE

TLE BRIDGE # 1-234, RTE 2 OVER MILL CREEK
TLE 1 SPAN NON-COMP., CONTRACT # 1000
COM *NOTE*: This File was modified on 08/08/95. Refer to Further Comments
ANL 1,0,3
XST 1,WN36X260
XSA 1,33
SPA 1,65.83,5
SPC 1,65.83,1
FIX 0,1,0,1,1,0
PS1 , ,2.5
PS2 9, ,40
COM DEAD LOADS: SLAB = 750 #/', WEARING SURFACE = 141 #/'
DLD 1,0.891
LDE 1, , ,DIAPHRAGM DEAD LOAD
PTD ,0.247,1,5.0
PTD ,0.247,1,23.0
COM *NOTE*: Live Load Dist. Factor Changed to 1.272 from 1.364
COM as per 1994 AASHTO Specs.
LLD 3, 1.272, ,50.0
TR1 HS20T,S220,S335,S437,T330,T435
TR2 T540
DES 3, 1
COM WS Parameters were Replaced with LF on 08/08/95
INV 1.3, 1.0, 1.67, 1.0, 1.0, 1.0
OPG 1.0, 1.0, 1.67, 1.0, 1.0, 1.0
COM POSTING LEVEL 3
PST 1.0, 1.0, 1.00, 1.0, 1.0, 1.0
SLD 1.0, 1.0, 1.0, 1.0, 1.0, 1.0
SL1 100,2
SL4 ,,1
SL1 104,2
SL4 ,,1
SL1 105,2
SL4 ,,1

BRIDGE 1-262N BRASS IN-PUT FILE

TLE BRIDGE # 1-262, SR 7 NB, CONTRACT # 79-101-01
TLE 5 SPAN CONTINUOUS CSC BRIDGE, DETAILS GIRDER 9
COM *NOTE*: This File was modified on 08/08/95. Refer to Further Comments
ANL 1,0,4
XSA 1,50
XSB 0.500,0.500,14.00,14.00,0.75,0.875
XSC 100.0,8.5,0.75,0,0,0
XSA 2,50
XSB 0.5625,0.5625,16.00,16.00,1.500,1.500
XSC 100.0,8.5,0,0,0,0
COM XSG 6,8,5,3.06
COM XSG 7,18.5,5,5.8
XSA 3,50
XSB 0.4375,0.4375,14.00,12.00,0.625,1.00
XSC 100.0,8.5,0.875,0,0,0
SPA 1,89.8,1,48,48
SPC 1,64.86,1,2
SPD 89.8,2
FIX 0,1,0,0,1,0
SPA 2,114.75,1,48,48
SPC 2,24.9,2,3
SPD 89.8,3,2,114.75,2
FIX 0,1,0,0,1,0
SPA 3,114.75,1,48,48
SPC 2,24.9,2,3
SPD 89.8,3,2,114.75,2
FIX 0,1,0,1,1,0
SPA 4,114.75,1,48,48
SPC 2,24.9,2,3
SPD 89.8,3,2,114.75,2
FIX 1,1,0,0,1,0
SPA 5,89.8,1,48,48
SPC 2,24.9,2,1
SPD 89.8,1
PS1 , ,4
PS2 8, ,60
COM DEAD LOADS STAGE 1: SLAB = 990 #/', SIP = 70 #/', HAUNCH = 20 #/'
COM DEAD LOADS STAGE 2: PARAPET = 116 #/'
DLD 1,1.08,0.116
LDE 1, , ,CROSS FRAME DEAD LOAD
PTD ,0.304,1,21.4

PTD ,0.304,1,44.1
PTD ,0.304,1,68.0
PTD ,0.304,2,22.67
PTD ,0.304,2,45.34
PTD ,0.304,2,68.01
PTD ,0.304,2,90.70
PTD ,0.304,3,22.67
PTD ,0.304,3,45.33
PTD ,0.304,3,68.01
PTD ,0.304,3,90.70
PTD ,0.304,4,22.67
PTD ,0.304,4,45.33
PTD ,0.304,4,68.01
PTD ,0.304,4,90.70
PTD ,0.304,5,22.67
PTD ,0.304,5,45.33
PTD ,0.304,5,68.00
COM *NOTE*: Live Load Dist. Factor Changed to 1.297 from 1.515
COM as per 1994 AASHTO Specs.
LLD 2, 1.297, ,50.0
TR1 HS20T,S220,S335,S437,T330,T435
TR2 T540
DES 3, 1
COM WS Parameters were Replaced with LF on 08/08/95
INV 1.3, 1.0, 1.67, 1.0, 1.0, 1.0
OPG 1.0, 1.0, 1.67, 1.0, 1.0, 1.0
COM POSTING LEVEL 3
PST 1.0, 1.0, 1.00, 1.0, 1.0, 1.0
SLD 1.0, 1.0, 1.0, 1.0, 1.0, 1.0
SL1 100,4
SL4 50,2.5,2,0.75,10.0
SL1 104,4
SL1 105,4
SL1 200,5
SL4 50,2.5,2,1.0,10.0
SL1 204,4
SL1 205,4
SL1 206,4
SL1 300,5
SL4 50,2.5,2,1.0,10.0
SL1 304,4
SL1 305,4
SL1 306,4

BRIDGE 1-704 SPAN 1 BRASS IN-PUT

TLE BRIDGE NO. 1-704, TURNPIKE BRIDGE OVER CHRISTINA
(WESTBOUND)

TLE SPAN # 1 OF 3 SPAN NON-COMP.BRIDGE, CONTRACT # 7002 (S2)

COM *NOTE*: This File was modified on 08/08/95. Refer to Further Comments

ANL 1,0,3

XST 1, WN24X84

XSA 1,32

SPA 1,24.6,5

SPC 1,24.6,1

FIX 1,1,0,0,1,0

COM WIDENING CONTRACT 7058 DOESN'T CONTROL.

COM DEAD LOADS STAGE 1: SLAB = 782 LB/FT, HAUNCH = 32 LB/FT

COM DEAD LOADS STAGE 1: P.PET= 143 #/, 2 " H.MIX = 125 LB/FT

DLD 1,1.082

LDE 1, , ,DIAPHRAGM DEAD LOAD

PTD ,0.283,1,10.4

COM *NOTE*: Live Load Dist. Factor WAS NOT Changed to 1.637 from 1.515

COM As per 1994 AASHTO Specs. It would have been 1.637

LLD 2, 1.515, ,50.0

TR1 HS20T,S220,S335,S437,T330,T435

TR2 T540

DES 3, 1

COM WS Parameters were Replaced with LF on 08/08/95

INV 1.3, 1.0, 1.67, 1.0, 1.0, 1.0

OPG 1.0, 1.0, 1.67, 1.0, 1.0, 1.0

COM POSTING LEVEL 3

PST 1.0, 1.0, 1.00, 1.0, 1.0, 1.0

SLD 1.0, 1.0, 1.0, 1.0, 1.0, 1.0

SL1 100,3

SL1 104,3

SL1 105,3

BRIDGE 1-791 BRASS IN-PUT

TLE BRIDGE # 1-791, I-95 NB OVER DARLEY ROAD

TLE 3 SPAN CONTINUOUS STEEL BEAM BRIDGE, FATIGUE SENSITIVE

COM *NOTE*: This File was modified on 08/08/95. Refer to Further Comments

ANL 1,0,4

XST 1,WN30X99

XSA 1,36

XST 2,WN30X99

XSA 2,36,,1

XSC 8.0,0.5,0.0,8.0,0.5,0.0

XST 3,WN30X99

XSA 3,36

XSC 86.0,7.50,1.0,0,0,0

SPA 1,35.0,5

SPC 1,25.5,1,2

SPD 35.0,2,2

FIX 0,1,0,1,1,0

SPA 2,58,5

SPC 2,5.5,2,1

SPD 11.75,1,3,46.25,3,1

SPD 52.5,1,2,58.0,2

FIX 1,1,0,0,1,0

SPA 3,35.0,5

SPC 2,9.5,2,1

SPD 35.0,1,1

FIX 0,1,0,0,1,0

PS1 , ,3

PS2 10, ,40

COM DEAD LOADS STAGE 1: SLAB = 672 LB/FT, HAUNCH = 18 LB/FT

COM DEAD LOADS STAGE 2: PARAPET = 185 LB/FT, WEARING SURFACE =
173 LB/FT

DLD 1,0.690,0.358

LDE 1, , ,DIAPHRAGM DEAD LOAD

PTD ,0.240,1,17.5

PTD ,0.240,2,14.5

PTD ,0.240,2,29.0

PTD ,0.240,2,43.5

PTD ,0.240,3,17.5

COM *NOTE*: Live Load Dist. Factor WAS NOT Changed to 1.378 from 1.303

COM As per 1994 AASHTO Specs. It would have been 1.378

LLD 2, 1.303, ,50.0

TR1 HS20T,S220,S335,S437,T330,T435

TR2 T540
DES 3, 1
COM WS Parameters were Replaced with LF on 08/08/95
INV 1.3, 1.0, 1.67, 1.0, 1.0, 1.0
OPG 1.0, 1.0, 1.67, 1.0, 1.0, 1.0
COM POSTING LEVEL 3
PST 1.0, 1.0, 1.00, 1.0, 1.0, 1.0
SLD 1.0, 1.0, 1.0, 1.0, 1.0, 1.0
SL1 100,2
SL4 ,,1
SL1 104,2
SL4 ,,1
SL1 105,2
SL4 ,,1
SL1 200,2
SL4 ,,1
SL1 205,4
SL4 ,,1

BRIDGE 1-826 BRASS IN-PUT FILE

TLE BRIDGE # 1-826, I-495 NB OVER STONEY CREEK,
TLE 3 SPAN CONTINUOUS COMPOSITE STEEL & CONCRETE BRIDGE
COM *NOTE*: This File was modified on 08/08/95. Refer to Further Comments
ANL 1,0,4
XSA 1,36
XSB 0.725,0.725,12.075,12.075,1.18,1.18
XSC 90.0,7.5,0.5,0,0,0
XSA 2,36
XSB 0.6,0.6,11.95,11.95,0.79,0.79
XSC 90.0,7.5,0.5,0,0,0
XSA 3,36
XSB 0.600,0.600,11.950,11.950,1.415,0.79
XSC 90.0,7.5,0.5,10.0,0.625,0
COM XSG 6,9,5,2.44
COM XSG 7,12,5,5.56
SPA 1,70.0,1,33.96,33.96
SPC 1,55.83,1,2
SPD 63.2,2,3,70.0,3
FIX 1,1,0,1,1,0
SPA 2,70.0,1,33.96,33.96
SPC 3,6.17,3,2
SPD 63.83,2,3,70.0,3
FIX 1,1,0,1,1,0
SPA 3,70.0,1,33.96,33.96
SPC 3,6.8,3,2
SPD 14.17,2,1,70.0,1
FIX 1,1,0,0,1,0
PS1 , ,3
PS2 9, ,40
COM DEAD LOADS STAGE 1: SLAB = 850 #/FT, SIP = 40 #/FT.HAUNCH = 25
#/FT
COM DEAD LOADS STAGE 2: PARAPET = 138 #/FT
DLD 1,0.890,0.138
LDE 1, , ,DIAPHRAGM DEAD LOAD
PTD ,0.342,1,22.08
PTD ,0.342,1,45.42
PTD ,0.342,2,22.08
PTD ,0.342,2,45.42
PTD ,0.342,3,22.08
PTD ,0.342,3,45.22
COM *NOTE*: Live Load Dist. Factor Changed to 1.317 from 1.454

COM as per 1994 AASHTO Specs.
LLD 2, 1.317, ,50.0
TR1 HS20T,S220,S335,S437,T330,T435
DES 3, 1
COM WS Parameters were Replaced with LF on 08/08/95
INV 1.3, 1.0, 1.67, 1.0, 1.0, 1.0
OPG 1.0, 1.0, 1.67, 1.0, 1.0, 1.0
COM POSTING LEVEL 3
PST 1.0, 1.0, 1.00, 1.0, 1.0, 1.0
SLD 1.0, 1.0, 1.0, 1.0, 1.0, 1.0
SL1 100,4
SL1 104,4
SL1 105,4
SL1 200,5
SL1 204,4
SL1 205,4

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