Connecticut Permanent Long-Term Bridge Monitoring Network Volume 4: Monitoring of Curved Steel Box-Girder Composite Bridge – I-84 EB Flyover to I-91 NB in Hartford (Bridge #5868)

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Disclaimer

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16. Abstract

This report describes the instrumentation and data acquisition for a continuous curved steel box-girder composite bridge in Connecticut. The computer-based remote monitoring system was installed in 2001, with accelerometers, tilt meters and temperature sensors. The bridge is part of a network of bridges in a long-term research project to evaluate the performance of a variety of bridges in Connecticut. Data has been collected over a multi-year period using normal vehicular traffic. A series of papers has been generated to explore the behavior of this bridge and to provide information to the Department of Transportation. The first study involved the development, implementation and evaluation of the initial data obtained from the monitoring system. This included a study of the large temperature gradients due to both annual climate changes and the position of the sun during the day. The goal was to explain the cause of torsion cracking in the tall slender concrete interior column supports. The second study used data collected over a multi-year period to develop benchmark parameters to use for structural health monitoring. Methods reviewed included natural frequency based methods, the modal assurance criterion, the signature assurance criterion, sensitivity coefficients of natural frequencies, and tilt meter data. The goal was to use ambient field monitoring data to detect changes in the structural integrity of the bridge. In the next study the improvement in bandwidth of the upgraded system is identified. The final study described in this report identifies and quantifies different data qualification measures needed for the structural health monitoring of this bridge.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^{o}F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^{o}C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^{o}C	Celsius	1.8C+32	Fahrenheit	^{o}F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

Table of Contents

LIST OF FIGURES

Monitoring of Curved Steel Box-Girder Composite Bridge – I-84 EB Flyover to I-91 NB in Hartford (Bridge #5868)

INTRODUCTION

Researchers at the University of Connecticut and in the Connecticut Department of Transportation have been using field monitoring to explore the behavior of bridges during the past two and a half decades (Lauzon and DeWolf, 2003). This report is based on the research project that was developed to place long-term monitoring systems on a network of bridges in the state (DeWolf, Lauzon and Culmo, 2002; Olund and DeWolf, 2007; DeWolf, Cardini, Olund and D'Attilio, 2009). The first system was installed in 1999, and since then five other bridges have been added to the network. The bridges have been selected because they are important to the state's highway infrastructure and because they are typical of different bridge types. Each monitoring system has been tailored to the particular bridge, using a variety of sensors, and all data is collected remotely. As with many of our busier highways, it is not possible to close a bridge for monitoring, and thus all systems collect data from normal vehicular traffic. The goal of this research has been to use structural health monitoring to learn about how bridges behave over multi-year periods, to provide information to the Connecticut Department of Transportation on the behavior of the state's bridges, and to develop structural health monitoring techniques that can be used to show if there are major changes in bridges' structural integrity.

The current four-year phase in this long-term project has focused on installation and implementation of monitoring systems on two new bridges, substantial upgrading of the monitoring equipment with the addition of video collection, and development of techniques for long-term structural health monitoring. Specifically for this bridge, during the current project the probabilistic health monitoring approach was further refined (Scianna, et al. 2011) and the monitoring system was replaced, which included removal of the previous data acquisition system and replacement with National Instruments CompactDAQ hardware connected to a Small Form Factor PC. The new data acquisition system allows for enhanced capabilities, including improved sensor resolution, anti-aliasing of accelerometer signals, internet connectivity for viewing and archiving of data, and flexibility for future expansion. This new bridge monitoring system also underwent a full data qualification and error quantification. These efforts are documented within the report.

This report is for a nine span, curved, double steel box-girder bridge with a composite concrete deck. An aerial view of the bridge is shown in Figure 1. The bridge is a flyover off-ramp that carries I-84 eastbound (EB) traffic to I-91 northbound (NB) in Hartford (Bridge #5868). Out of its nine spans, a continuous three-span interior segment was selected for the primary monitoring. The three spans are indicated by the arrows in the figure. The three-span segment is simply supported at both ends.

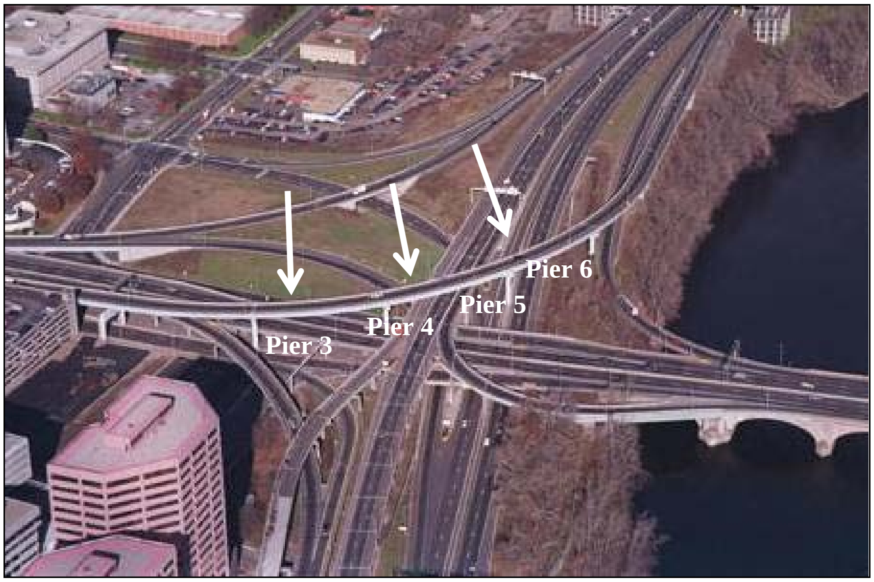


Figure 1. Aerial View of Steel Box-Girder Bridge

The bridge is supported by tall slender circular reinforced concrete columns. The supporting column at the beginning of the segment studied, Pier 3 in Figure 1, is shown in Figure 2.

Figure 2. Column Support at Start of Segment Monitored

The bridge plan and cross section for the monitored segment are shown in Figures 3 and 4, respectively.

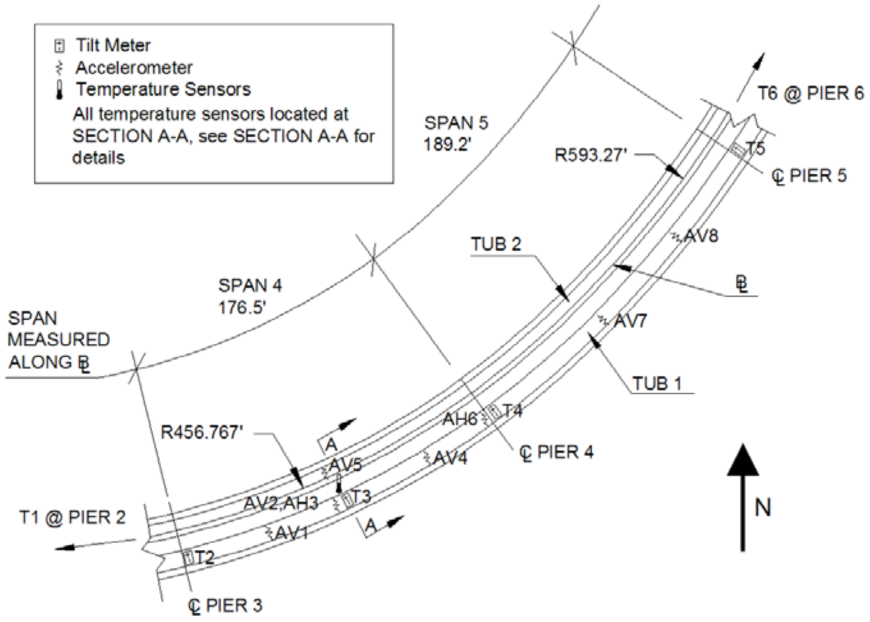


Figure 3. Plan View of Steel Box-Girder Bridge

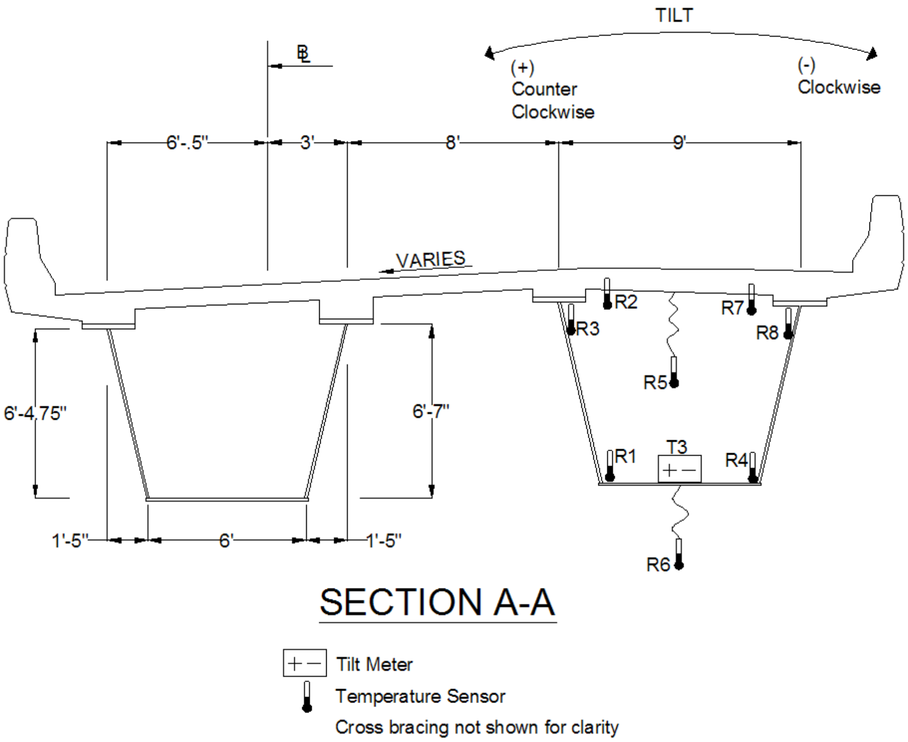


Figure 4. Typical Cross Section of Steel Box Girder Bridge

Prior to installation of the monitoring system, inspection noted that there were substantial cracks in some of the tall interior supporting columns. In addition, monitoring has shown that there were small, permanent changes in tilt in the transverse direction near one of the interior supports. These tilt changes occurred during two winter periods.

OBJECTIVES AND SCOPE OF STUDY

This bridge was selected as part of an overall research project, designed to implement long-term monitoring systems on a network of different bridges in Connecticut, using different bridge types and sensor combinations. This bridge was added to the project because it is representative of curved steel box-girder bridges and because it is on the interstate and subject to significant automobile and truck traffic. Also of interest was the desire to use the data developed from the monitoring system to explain the cracking behavior and to evaluate its long-term influence on the overall behavior of the bridge.

The design of the monitoring system was based on meeting these objectives. Accelerometers were used to study structural health monitoring techniques, temperature sensors were used to study the variation in temperatures that were thought to be the major cause of the column cracking, and tilt meters were used to correlate the column displacements with the overall structure.

INSTRUMENTATION AND DATA ACQUISITION

The monitoring system has eight accelerometers, eight temperature gages, and six tilt meters. The sensors have been distributed in two of the three spans as shown in Figure 3. Installation of sensors, hardware and wiring was completed in the late summer of 2001.

The temperature sensors were installed at the center of the tub on the side that receives solar gain. It was felt that this would offer the most valuable information to determine horizontal and vertical temperature differentials. Of interest was the variation in the behavior due to the changing angle of incidence of the sunlight over the daily cycle. The location of the temperature sensors over the cross section is shown in Figure 4.

Six accelerometers were placed to gather vertical acceleration data and two were oriented horizontally. The two horizontal accelerometers are located in one tub, one at the mid-span and one at the pier. The vertical accelerometers were placed to best match the lowest mode shapes generated from a finite element analysis. They were positioned at the mid-span, quarter-points and three-quarter points in two spans.

Six tilt meters were placed at the piers and at the mid-span of one of the spans. All tilt meters measure the tilt in the direction perpendicular to the bridge centerline. One of the primary interests was to address potential cracking causes, and thus tilt meters were located at the piers to get information on the supporting column behavior.

The monitoring system is remotely accessed from both the University of Connecticut and the Connecticut Department of Transportation. Tilt and temperature readings are made at specific time intervals, which can be changed as desired. Accelerations are measured according to a set trigger level. When one of the accelerometers measures a preset acceleration, data is saved for all accelerometers from a time just prior to this time until a short time after this time. In this way acceleration data is saved for each truck passage over the three span segments.

As explained subsequently, the system was upgraded in 2010.

DATA ANALYSIS FOR STUDIES PRIOR TO UPGRADING OF MONITORING SYSTEM

There has been a series of studies using the extensive data collected over multi-year periods on this bridge. The initial task was to set up the data collection following what was done with other bridges in the project. The data was used with the field data and an extensive finite element model to evaluate the global deformations. Of particular interest was the determination of the cause of cracking in the columns, and this involved a study of the longitudinal deformations and those due to differential temperatures through the cross section. The second study used the extensive data to create benchmark parameters for use in structural health monitoring. The goal was to determine how the data could be used to determine if there are major changes to the structural integrity that could be cause for alarm.

The following presents summaries and examples taken from research conducted by graduate students who have been assigned to work on this bridge. The references with each of the studies have the complete information.

Development of Data Collection Approach with Information on Deformations and Cause of Cracking

The initial monitoring and development of the data collection approach is described by Virkler (2004) and Virkler and DeWolf (2005). The field data were used to define the overall bridge behavior. Key interests were to investigate the reliability of the data over an extended time period, define a base set of data that could be used for long-term monitoring, and determine the cause of cracking in the tall interior support columns.

Temperature data was used to determine thermal gradients across the bridge and through the depth. A data collection rate of 15-minute intervals was used and proved to be more than sufficient to determine the temperature variations. When the sun is lower in the sky, it strikes a larger portion of the steel box-girder web. In the summer, the concrete deck overhang effectively shades the steel, reducing the temperature differentials. The transverse, or horizontal, temperature differentials were considerably smaller. The greatest temperature differentials were observed in winter months.

Figure 5 shows a typical example of the temperature gradient through the cross section. The figure is based on the month of February, using sensor number 4, located at the bottom of the tub and temperature sensor number 7, located in the concrete deck on the side exposed to the sun. Both are located at the mid-span. The maximum temperature difference between these two sensors during the initial multi-year monitoring period was approximately 30ºF.

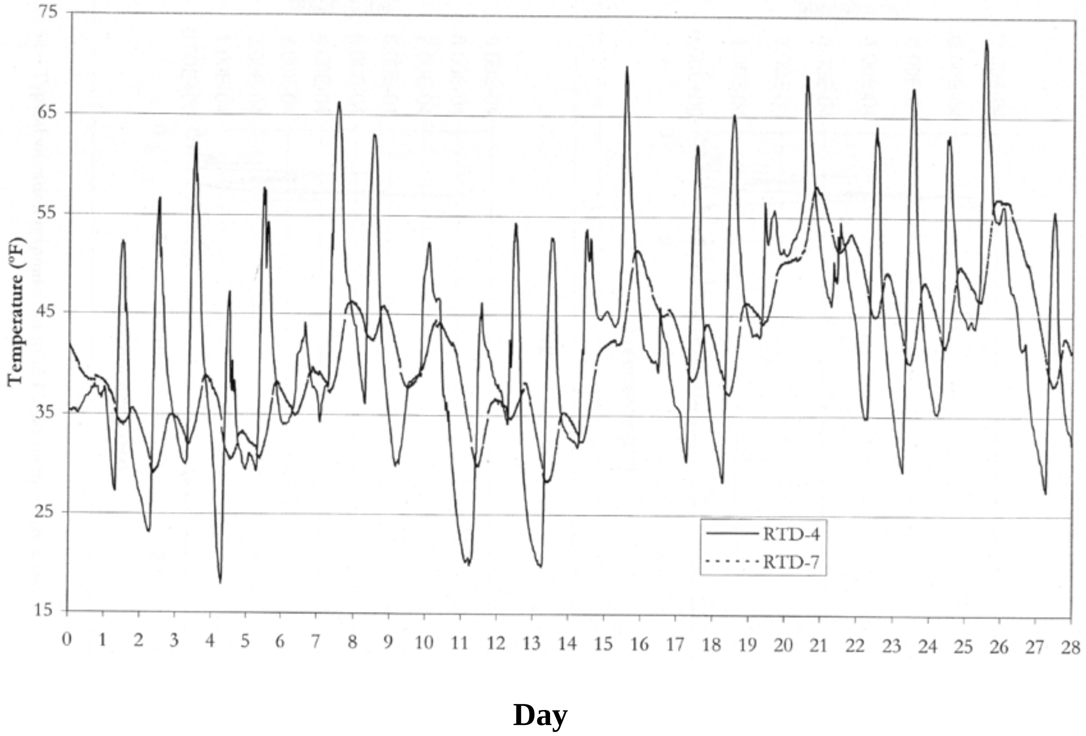


Figure 5. Temperature Changes for Month of February

In addition, the change in longitudinal temperatures can have a major effect on the structure. The bridge is curved such that one side of one tub receives solar gain for much of the day, while the other side is never exposed to sunlight. This creates a tendency for the horizontal radius of the curve for the bridge to increase and decrease over time, i.e. the bridge tends to straighten out as the radius increases.

The relation between the temperature differences and tilt data were used to explore the cause of cracking. A linear regression analysis was performed on this data collected from different sets of gage. Figure 6 shows an example for these comparisons. It is based on the temperature differential between sensors number 4 and number 7, and the tilt data for tilt meter number 3, located in the same cross section as the temperature gages. As shown, there is a small increase in the mean tilt with temperature. This is typical of the results obtained from other sensors sets, as well as for other time periods during the year.

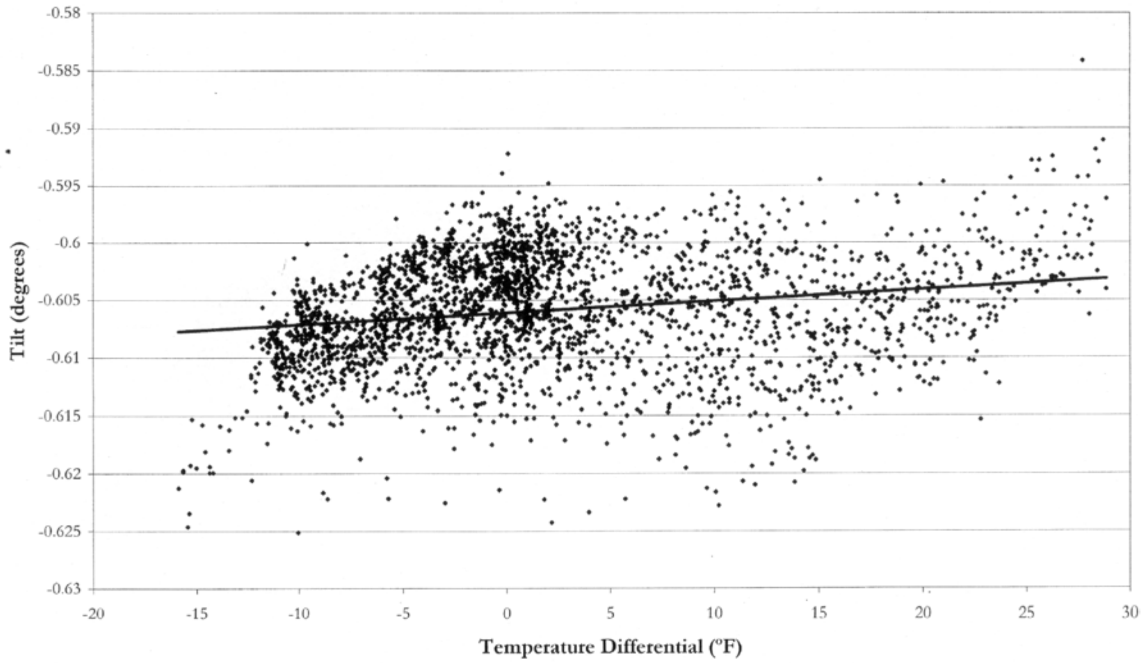


Figure 6. Comparison of Vertical Temperature Differential with Tilt

Further comparisons between the temperature and tilt were carried out. Figure 7 shows a comparison between the temperature for sensor number 6 and the tilt at this point. This figure shows that the temperature and tilt are out of phase.

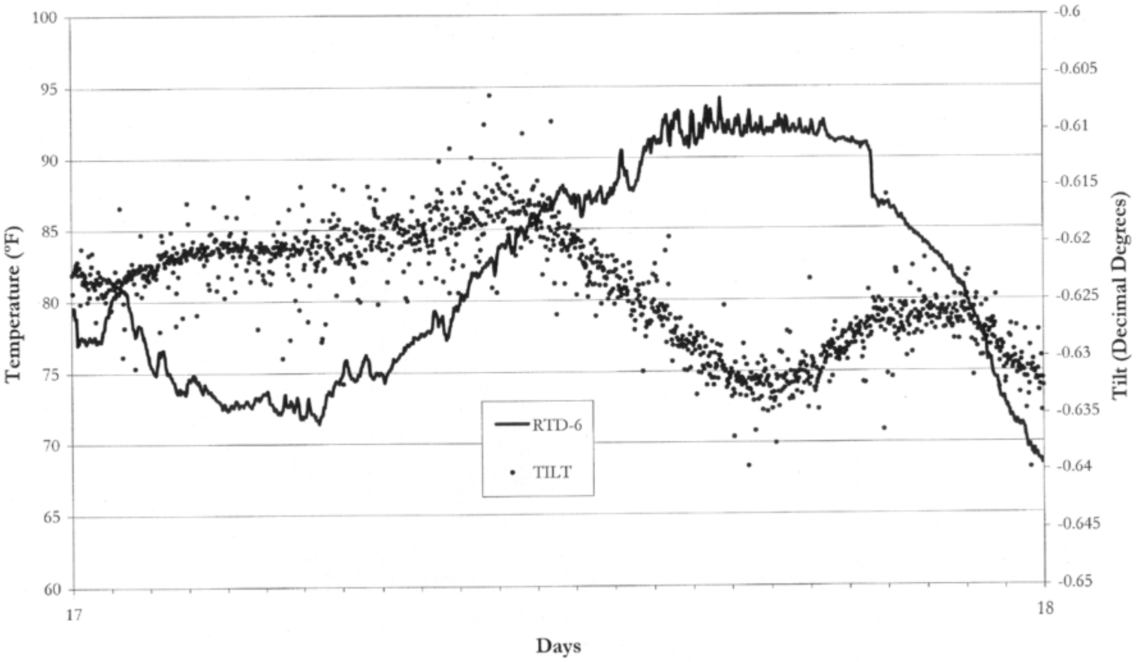


Figure 7. Comparison of Temperature with Tilt

Additional analyses of the data by Virkler and DeWolf involved looking at changes in temperature along the bridge axis. They concluded that the cause of the column cracking is due to longitudinal temperature variations over time. Temperature increases create longitudinal forces as a result of the constraints at piers 3 and 6. This leads to changes in the horizontal curvature. This in turn causes transverse displacements that place the tall column in bending, leading to the cracking noted in the field inspections.

The acceleration data was processed to provide natural frequencies and basic information needed to establish mode shapes. Following the approach developed by Lengyel (2001) and Lengyel and DeWolf (2003), histograms were used to establish natural frequencies. Figure 8 shows a typical histogram example, showing the number of times there were peaks in the Fast

Fourier Transform (FFT), associated with potential natural frequencies. This figure is for the month of November, and it is typical of other months.

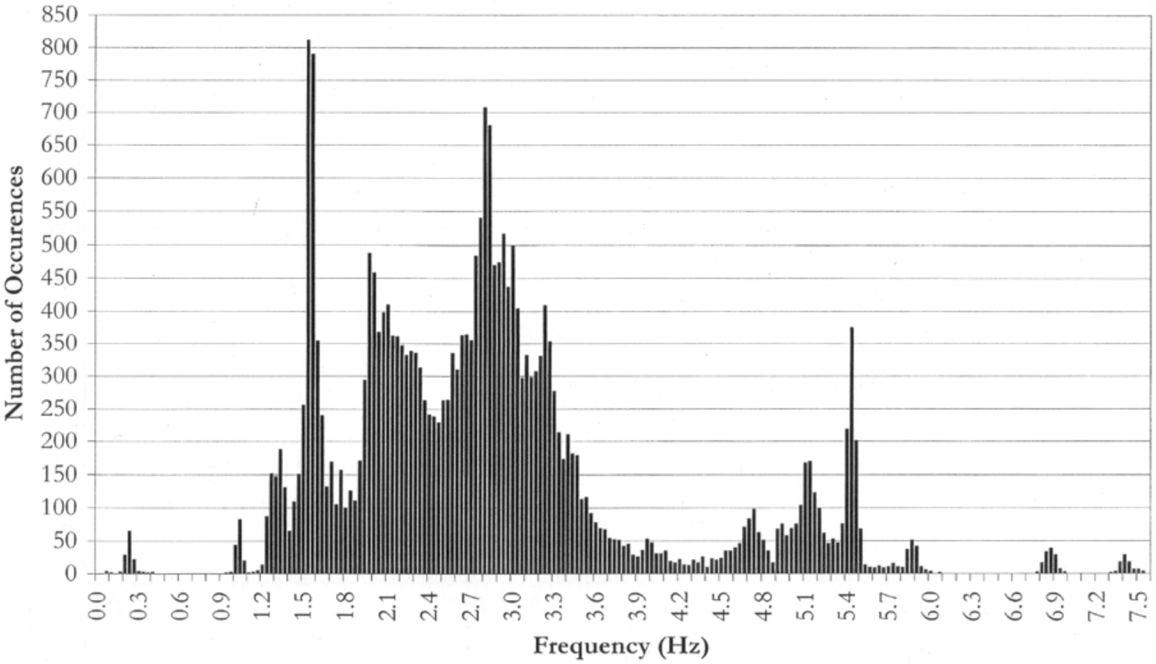


Figure 8. Typical Histogram of Natural Frequencies

In this figure, the peaks correspond to natural frequencies, with the lowest at 1.55 Hz. The next natural frequencies are at approximately 2.05, 2.85 and 3.25 Hz. A review of the long-term data demonstrated these natural frequencies do not show significant shifts over time, either due to changes in temperature or due to changes in loads.

The accelerations associated with the natural frequencies, obtained from an FFT, can be used to plot mode shapes. Since only eight accelerometers were used, the field data could be used only to establish the lowest mode shape. This mode shape is associated with simple bending in the three-span segment. To obtain additional mode shapes from field data would require additional accelerometers, with additional expense. Since one of the key goals in this research has been to establish a monitoring program using economical field systems, the use of additional accelerometers was not considered feasible.

An alternative approach to obtain additional mode shapes is to use the field data to develop a finite element analysis model, correlated with the field data. Virkler (2003) developed a model with 26,000 eight-node quadratic shell elements with six degrees-of-freedom at each node for the steel box girder and the composite concrete deck. Axial elements were used for the stiffeners. The model confirmed the lowest mode shape as modeling simple bending and provided additional mode shapes. This information was used to establish a basis for long-term monitoring.

Benchmark Parameters for Structural Health Monitoring

Olund (2007) and Olund and DeWolf (2007) used the extensive field data collected over a multi-year period, along with a comprehensive finite element analysis, to evaluate options that can be used for structural health monitoring. The goal was to develop techniques that can be used to determine if there are changes in the structural integrity that would be cause for alarm. The study has used the data to study natural frequency based methods, the modal assurance criterion, the signature assurance criterion, sensitivity coefficients of natural frequencies, and tilt meter data. These methods have been correlated with the known small structural changes in the support columns during the multi-year monitoring period.

Structural health monitoring should provide those responsible for overseeing the bridge infrastructure with a diagnosis of the state of the monitored structure, with a snapshot of the health or strength of the structure. When monitoring is performed over longer periods of time, it can provide health status updates between biennial inspections, as well as deterioration rates which can lead to predictions of remaining service life. Information from structural health monitoring can be used to maintain the safety of the structure and lead to reduced costs when repairs are needed. Most importantly, it can provide timely warning when the structural integrity is being significantly compromised.

There are two general monitoring approaches, active and passive. Active monitoring uses known input load information along with the data from sensors to define the structural behavior. For a bridge, it is necessary to close the bridge and use known loads to get a clear picture of the structure. Passive monitoring involves collecting information while the bridge is open to traffic, and thus it can be carried out on a continuous basis. The drawback is that data analysis becomes more difficult because the actual loads are not fully defined and vary over time. The solution is to review data collected in passive monitoring statistically, using higher level analytical approaches to determine if significant changes have occurred. The goal of this phase of the research has been to develop and refine approaches previously proposed, primarily based on use of vibration data.

Virkler (2004) noted that in the second and third winter there were changes in the tilt at one of the supporting columns. Olund (2006) used these changes as a basis to explore the different structural health monitoring approaches. Some of the approaches considered for detecting damage in this study were proposed in previous research studies (Alampalli, 1995; Zhao and DeWolf, 1999; Carden and Fanning, 2004; Olund, 2007).

The following briefly summarizes some of comparisons made using different approaches to determine if there are changes in the structural integrity. This material is summarized from the thesis by Olund (2007) and the paper by Olund and DeWolf (2007).

To make comparisons, it is first necessary to account for variability in the parameters through the use of thermal regression models and statistical comparisons. Not only does this approach account for variability in unknown loading of the structure, noise in the data signal, and thermal influences, but it also avoids assumptions and lengthy calculations and procedures that are required in some of the analytical methods. It is anticipated that when a structure's integrity changes, a parameter of interest will change from that of the healthy data. A "t-test" is used to statistically determine if the mean of a benchmark parameter has changed (Ott, 2001). These tests determine the difference between a sample mean from data collected over a specific time period and the benchmark mean with a specified confidence level.

As noted earlier, there were changes in the tilt at specific locations during two winters in the three-span segment. These changes offer the opportunity to look at different approaches in terms of structural health monitoring. Since the tilt changes correspond to changes in the structure, a basis of exploring the different structural health monitoring approaches is to see if they work with the data collected during the tilt changes.

The changes in the tilt are shown in Figure 9 for tilt meter number 5 at Pier 5. This figure shows that a permanent rotation occurred in the winter of 2001-2002, primarily during December, 2001. Further permanent rotation occurred during the 2004-2005 winter months. In addition, there appeared to be another small permanent rotation during the winter of 2005-2006, although a sufficient amount of data was not collected to draw conclusions. Of the remaining five tilt meters, a similar permanent rotation was only observed in tilt meter number 6 (same span, but opposite end).

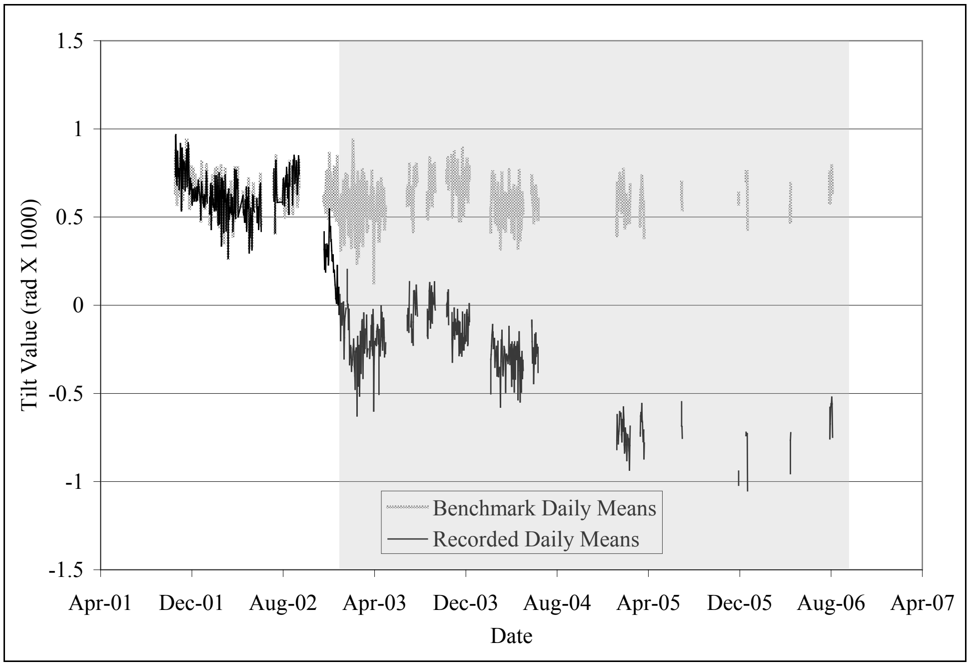


Figure 9. Tilt Meter Measurements for Tilt Meter 5

As a comparison, Figure 10 shows tilt measured by tilt meter number 2 (located at Pier 3). Note that this sensor also shows unexpected rotations during the winter months beginning in

2002-2003. This sensor, unlike tilt meter number 5, rebounds to its anticipated values during the summer months, and it does not show substantial additional rotations in subsequent years.

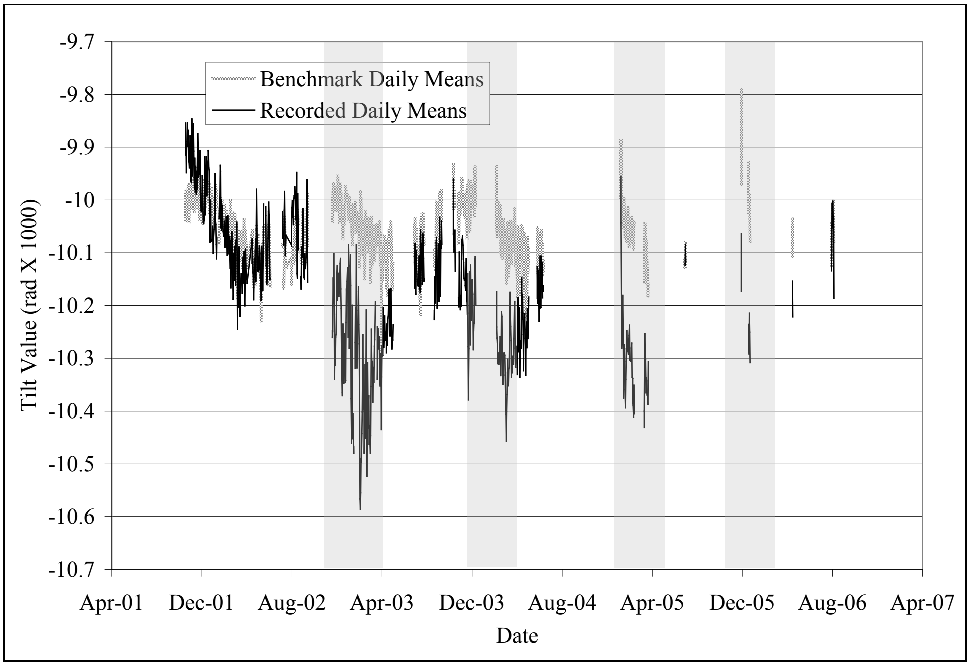


Figure 10. Tilt Meter Measurements for Tilt Meter 2

Another potential approach for showing changes in the structure is to look for shifts in natural frequencies. As shown by Olund (2007) and Olund and DeWolf (2007), three natural frequencies that reliably appear in FFTs and thus serve as benchmark values are at approximately 1.52 Hz, 2.02 Hz and 2.85 Hz. They also have developed equations to use to account for thermal variations due to changes in climate. Figure 11 shows a plot comparing the lowest natural frequency, determined from monthly averages, with its benchmark value for accelerometer number 2. These benchmark values have been adjusted to account for temperature variations. Comparisons for other sensors produce similar comparisons.

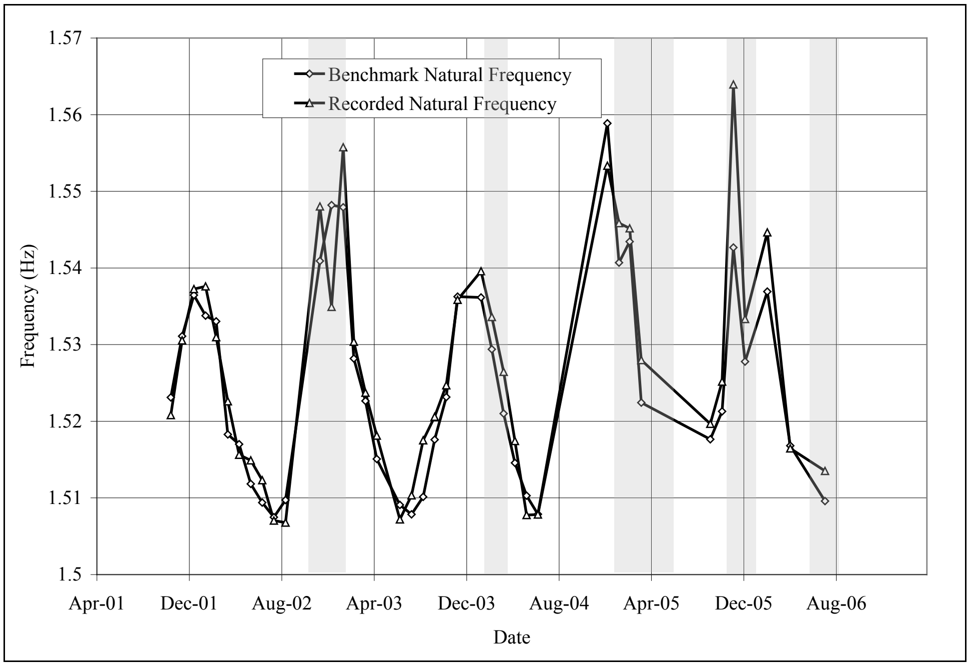


Figure 11. Comparison of Lowest Natural Frequency Benchmark Value with Recorded Value for Accelerometer AV2.

As shown in Figure 11, there do not appear to be significant observable variations between the benchmark values and the recorded values. Olund and DeWolf have found that the t-test approach shows changes during the winter months when there were changes in the tilt. While the use of statistics with natural frequencies may be used to detect some changes in the structural integrity, Olund and DeWolf concluded that using natural frequencies was not sufficient for structural health monitoring. However, they can be used to supplement other approaches.

Another approach proposed by Olund and DeWolf is to compare the acceleration levels obtained from the FFTs at specific natural frequencies. While these accelerations are required for the more sophisticated methods, some of which follow, the idea is that they can be used directly to determine if structural changes have occurred. A plot comparing acceleration magnitudes at the lowest natural frequency, averaged over individual months and corrected for temperature, compared to the benchmark values that were also corrected for temperature is shown in Figure 12. A significant increase in the magnitude is observed beginning in the winter of 2004-2005 when the second permanent rotation occurred. This is one indication that there could have been a change in the structural integrity.

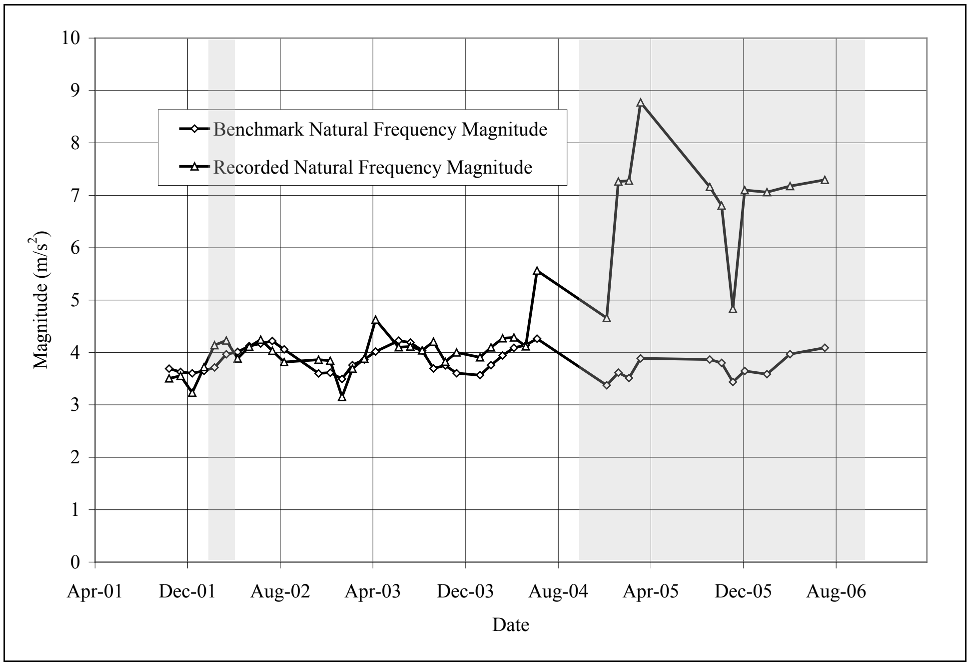


Figure 12. Comparison of Average Acceleration Level Associated for the Lowest Natural Frequency with Recorded Value for Accelerometer AV2.

A more sophisticated approach is to look at two assurance criterions, which use the acceleration data to make comparisons. There are two approaches, the modal assurance criterion (MAC) and the signature assurance criterion (SAC). These two criterions are vector comparisons of a system’s mode shapes. The first is based on phase angles, while the second does not use this information. Either of these criterions can be used to determine if a structures mode shape has changed, indicating a change in the structural integrity. While there are only a limited number of accelerometers and thus insufficient information to fully define mode shapes from the experimental data, Olund and DeWolf found that it was still possible to use these approaches in this study.

Figure 13 shows a plot of percent differences from respective benchmark values of monthly averaged MAC values for the three recordable modes from the beginning of monitoring, November 2001 through July 2006. Figure 14 shows a plot of percent differences from respective benchmark values of monthly averaged SAC values for the three recordable modes for the same time period. As shown, significant differences from their respective benchmark parameters develop over time, primarily beginning in approximately 2002.

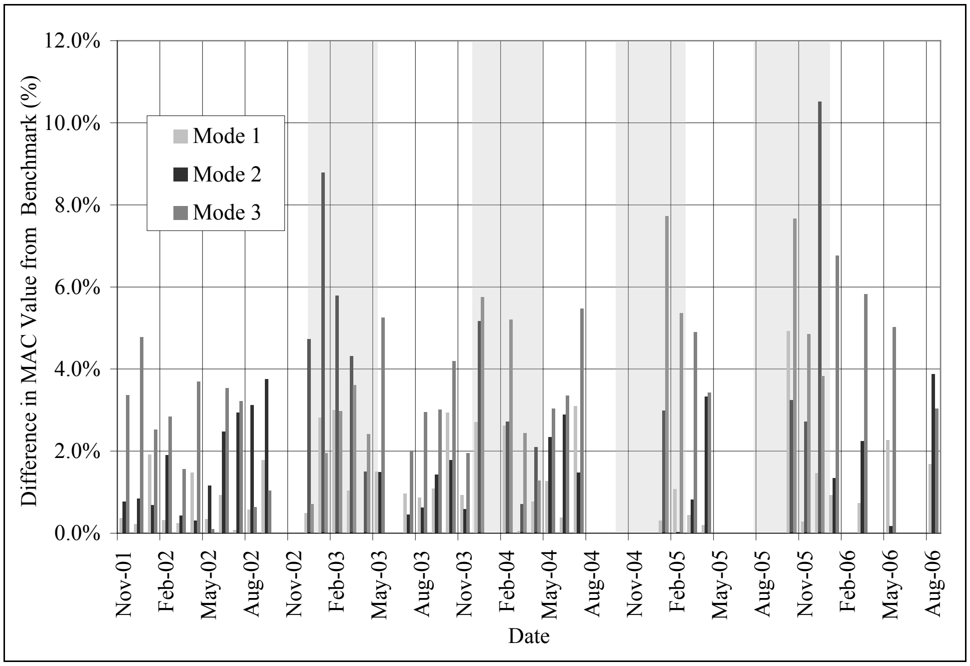


Figure 13. Comparison of Percent Differences of Monthly Averaged Modal Assurance Criterion (MAC) Values with Benchmark MAC Values for Lowest Three Modes

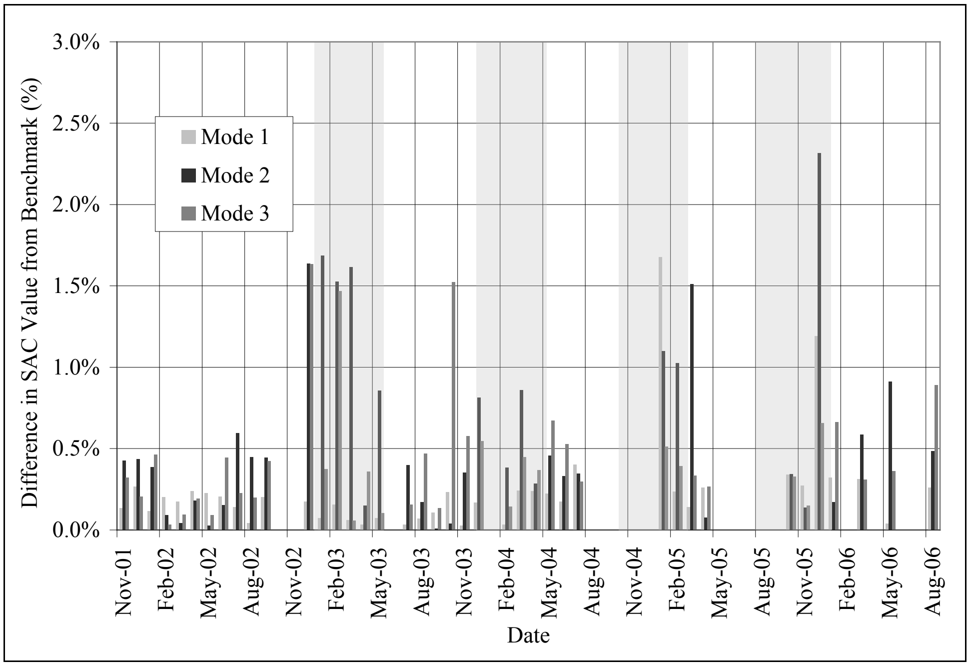


Figure 14. Comparisons of Percent Differences of Monthly Averaged Signal Assurance Criterion (SAC) Values with Benchmark SAC Values for Lowest Three Modes

Both the SAC and MAC values exhibit the greatest differences during the same winter time periods. There is larger variability in the MAC values, most likely because they are based on phase angles. It has been concluded, therefore, that the MAC values are not useful for structural health monitoring. Although the SAC values are more stable than the MAC values, the change in these values is less than 2.5% during the winters in which there were known changes in structural integrity. This is a relatively small change, considering the amount of variability in the collected data. Olund and DeWolf concluded that SAC comparisons should only be used to supplement other, more reliable structural health monitoring approaches.

Another approach that uses acceleration data is based on determination of sensitivity coefficients which are derived from the natural frequencies. The sensitivity coefficients are based on the diagonal terms in the structural stiffness matrix, modified by the natural frequencies. Figure 15 shows a plot of the percent difference between the benchmark values and the monthly averaged values for three sensitivity coefficients, based on data collected from November 2001 through July 2006. These sensors were chosen for comparison because accelerometer number 2 has been used for the previous methods, accelerometer number 5 is in the adjacent box girder, and accelerometer number 8 is at a quarter point. Each of the sensitivity coefficients show a significant change, ranging from 95% to 114%, during the winter months, most notably during December 2002 and March 2003.

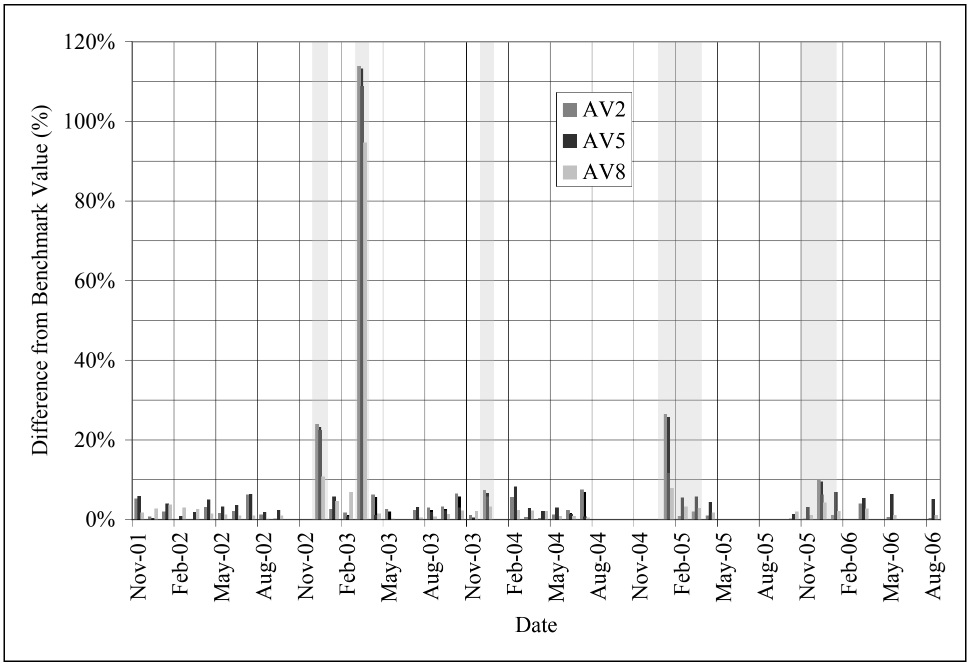


Figure 15. Changes in Sensitivity Coefficients Based on Natural Frequencies from Benchmark Values for Accelerometers AV2, AV5 and AV8

The change in the sensitivity coefficients was greatest for the accelerometers furthest from the identified change in structural integrity. This is consistent with findings from an experimental study in Connecticut using accelerometers to evaluate the vibration information when a crack was introduced into one of the webs in a multi-girder bridge (Lauzon and DeWolf, 2006). The trend shown in Figure 15 is consistent for other months. Also, like other parameters being observed, the sensitivity coefficients generally rebound to values near the benchmark value during the summer months; i.e., the only significant changes were those during the winter periods associated with the support movements. It was concluded that use of sensitivity coefficients shows promise as a tool for structural health monitoring.

Olund (2007) and Olund and DeWolf (2007) also used a finite element model of this bridge to further explore how changes in structural integrity lead to changes shown in the preceding approaches. The box girders, diaphragms, and deck were modeled with shell elements with six degrees of freedom at each of the nodes. The bracing and pier members were modeled with beam elements. The model was correlated to match natural frequencies and mode shapes extracted from the field data. The resulting finite element model generated natural frequencies that were within 1.5% of the field values and MAC values that ranged between 0.96 and 0.99. The finite element model was then used with different damage scenarios based on cracking at the diaphragm connection to the box-girder interface, rotations at the piers consistent with the actual changes during earlier winters, and stiffness reductions in supporting columns. The following conclusions were drawn from the introduction to damage into the finite element model:

- The largest change in natural frequency was just over 2%, based on rotation of Pier 5. Since this is a relatively small change compared to daily variations observed with field data, this supports the earlier conclusion that comparing natural frequencies directly should be used only to supplement other methods.

- The largest change in the SAC values was 4.1%, based on rotation of Pier 4. This is also a relatively small change for detecting a change in structural integrity, and it was concluded that this method should be used only to supplement other methods.

- The largest changes in sensitivity coefficients were those associated with rotations of the piers, with values up to approximately 90%. It was found that changes in pier stiffness and diaphragm integrity resulted in lesser changes, typically up to approximately 10%. With its larger value changes, sensitivity coefficients were judged as appropriate for damage detection.

- The largest percent change in the acceleration magnitudes at the natural frequencies were associated with pier rotations, with values up to approximately 37%. The changes in this parameter were small for cases not associated with a global change in structural integrity. Nevertheless, looking at acceleration magnitudes is appropriate for some types of damage detection.

DESIGN OF NEW MONITORING SYSTEM

Consistent with efforts to upgrade the monitoring systems and capabilities on other bridges in the project, the monitoring system was replaced in 2010. This included removal of the previous data acquisition system and replacement with National Instruments CompactDAQ hardware connected to a Small Form Factor PC. This CompactDAQ has four modules installed that provide power to the sensors and collect data measurements from the sensors previously installed on the bridge. These modules not only support the input of RTDs, but they can measure resistance, voltage, and current as well. This combined with the remaining four expansion slots on the CompactDAQ will enable researchers to add a wider variety of sensors on the bridge for the purposes of structural health monitoring. The updated bridge monitoring system at the Flyover bridge provides:

- improved resolution of the sensor measurements with the 24-bit system;
- connectivity to the Connecticut Department of Transportation computer network over the internet, allowing for full access to the bridge monitoring computers;
- potential for real-time remote viewing of the bridge monitoring data from any PC on the CTDOT network using a java-based Real-Time Data Viewer (RDV);
- capability for automated data archival to an offsite FTP server; and
- flexibility to expand the current system to new sensors.

DATA ACQUIRED WITH UPGRADED MONITORING SYSTEM

This section describes improvement in the data collected with the upgraded monitoring system for use in SHM as identified in Prusaczyk (2011). The original data acquisition system had a two-pole low-pass filter with a cutoff frequency of 2 Hz used for anti-aliasing. With a sampling rate of 100 Hz, this filter provided sufficient anti-aliasing protection. However, the 2 Hz cutoff frequency of this filter resulted in measured data with distorted frequency content above this 2 Hz. A new data acquisition system on the bridge provides a bandwidth of 500 Hz. Figure 16 shows a plot of the auto-power spectral densities of an accelerometer (A4) located on the fourth span (Span 4) of the bridge from both the original and the current system. It is observed in the auto-power spectral density functions that the original signal was attenuated above 2 Hz. While this was not a problem for the original structural health monitoring analysis that considered only the fundamental frequency of the bridge, at 1.5 Hz, current approaches for this bridge can now use multiple lower frequency modes. This work is ongoing.

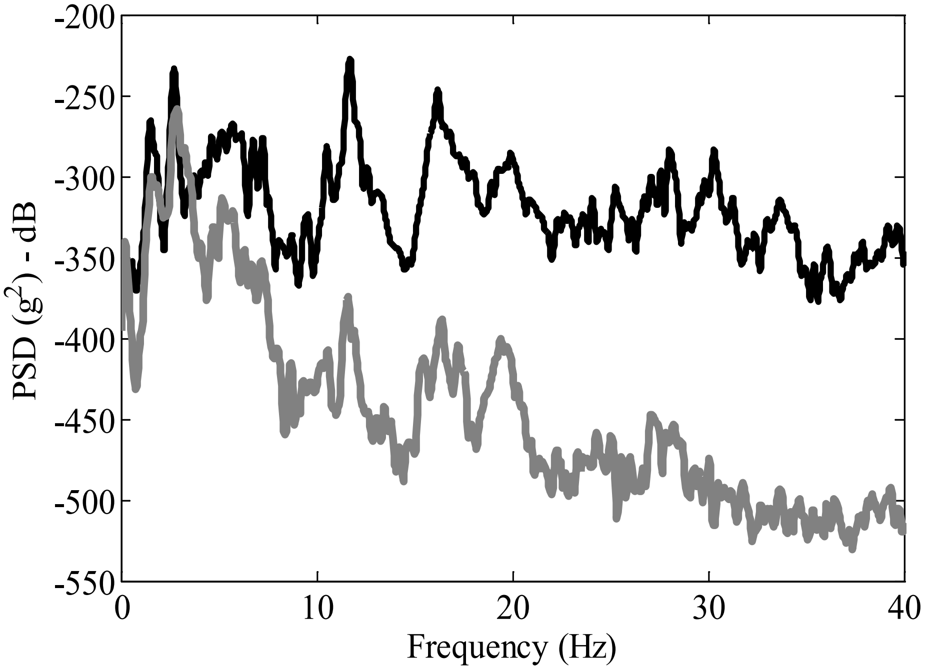
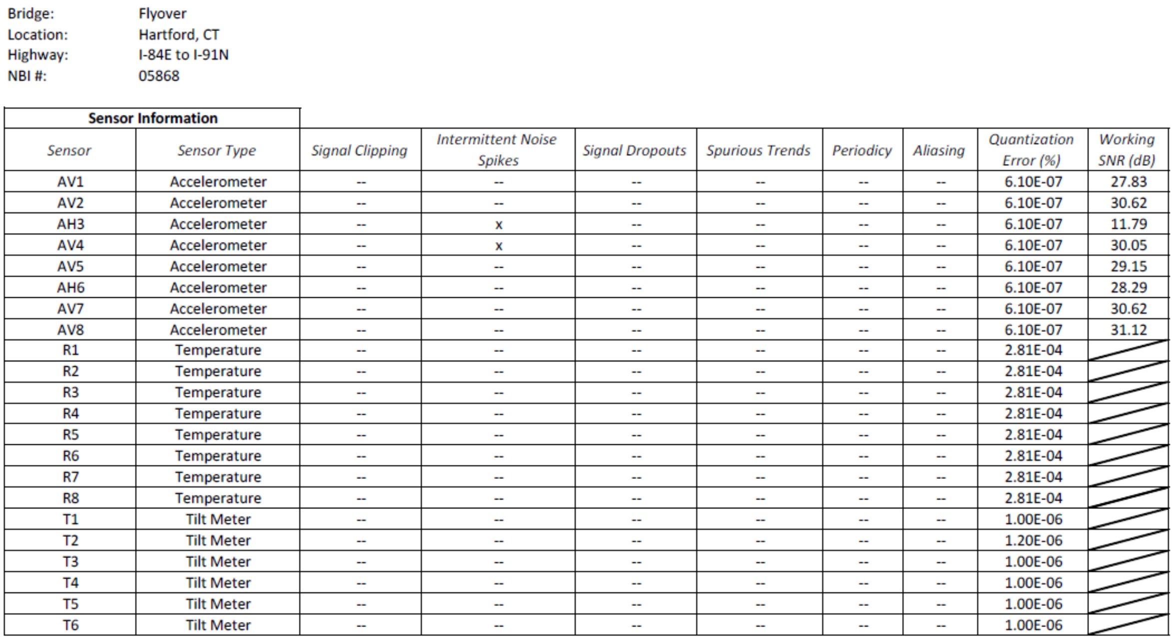


Figure 16. Effect of Filtering on Bandwidth of Measured Data for Original (gray) and Upgraded (black) Monitoring Systems

DATA QUALIFICATION AND QUANTIFICATION

Recent work (Trivedi, 2009; Trivedi and Christenson, 2009; Prusaczyk, et al., 2011; and Prusaczyk, 2011) proposed a data qualification procedure for bridge monitoring and provided data qualification for this bridge. Data qualification is an area that has not previously been addressed in field monitoring studies on bridges. This is one of the key areas addressed as part of the upgrade of the bridge monitoring systems in the current phase of this research. The quality of measured data is of critical importance in drawing reliable conclusions from data analysis in bridge monitoring. Data qualification categorizes the quality of measured data. There is currently no formalized quality certification system in place for data qualification in bridge monitoring. Data qualification, as proposed for bridge monitoring, is divided into identification of data anomalies and error and noise quantification. The results of the data qualification for the upgraded bridge monitoring system on the Flyover highway bridge are shown in Figure 17.



Bridge:	Flyover
Location:	Hartford, CT
Highway:	I-84E to I-91N
NBI #:	05868

Sensor Information									
Sensor	*Sensor Type*	*Signal Clipping*	*Intermittent Noise Spikes*	*Signal Dropouts*	*Spurious Trends*	*Periodicy*	*Aliasing*	*Quantization Error (%)*	*Working SNR (dB)*
AV1	Accelerometer	--	--	--	--	--	--	6.10E-07	27.83
AV2	Accelerometer	--	--	--	--	--	--	6.10E-07	30.62
AH3	Accelerometer	--	x	--	--	--	--	6.10E-07	11.79
AV4	Accelerometer	--	x	--	--	--	--	6.10E-07	30.05
AV5	Accelerometer	--	--	--	--	--	--	6.10E-07	29.15
AH6	Accelerometer	--	--	--	--	--	--	6.10E-07	28.29
AV7	Accelerometer	--	--	--	--	--	--	6.10E-07	30.62
AV8	Accelerometer	--	--	--	--	--	--	6.10E-07	31.12
R1	Temperature	--	--	--	--	--	--	2.81E-04	
R2	Temperature	--	--	--	--	--	--	2.81E-04	
R3	Temperature	--	--	--	--	--	--	2.81E-04	
R4	Temperature	--	--	--	--	--	--	2.81E-04	
R5	Temperature	--	--	--	--	--	--	2.81E-04	
R6	Temperature	--	--	--	--	--	--	2.81E-04	
R7	Temperature	--	--	--	--	--	--	2.81E-04	
R8	Temperature	--	--	--	--	--	--	2.81E-04	
T1	Tilt Meter	--	--	--	--	--	--	1.00E-06	
T2	Tilt Meter	--	--	--	--	--	--	1.20E-06	
T3	Tilt Meter	--	--	--	--	--	--	1.00E-06	
T4	Tilt Meter	--	--	--	--	--	--	1.00E-06	
T5	Tilt Meter	--	--	--	--	--	--	1.00E-06	
T6	Tilt Meter	--	--	--	--	--	--	1.00E-06	

Figure 17. Results of Data Qualification for Flyover Bridge Monitoring System

There are no data anomalies, including signal clipping, intermittent noise spikes, signal dropouts, spurious trends or periodicity, observed in the measured sensor data. No aliasing is present in the measurements. The quantization error is negligible for all three types of sensors. The working signal-to-noise ratio (SNR) has been determined for the acceleration measurements. The SNRs range around 30 dB (signal is 31.63 times larger than the noise floor) except for one accelerometer with a lower SNR of 11.79 dB (signal is 3.89 times larger than the noise floor). The lower SNR is for an accelerometer that is measuring the horizonal acceleration, which is a lower magnitude signal. The other accelerometers have acceptable SNRs.

CONCLUSIONS

This report is based on the continuous monitoring of a curved, steel box-girder bridge with a composite concrete deck. The monitoring system was installed in 2001. This research is part of a research program to implement long-term monitoring systems on a network of bridges important to Connecticut's highway system.

In the initial phase of this research, data from the temperature sensors, tilt meters and accelerometers were used to show the following:

- The temperature gradients do not greatly impact bridge displacements, and consequently they do not introduce significant stresses into the bridge super structure. However, the global temperature fluctuations create daily movement in the bridge, and these movements are likely responsible for the cracking seen in several of the tall, slender support columns.

- The bridge tilt is a result of external factors acting on the bridge structure. The most important of these is temperature. The primary effect of the tilt is related to the horizontal rotation at the piers due to overall change in bridge length. These rotations are consistent with the cracking in the support columns.

The study to develop approaches for long-term structural health monitoring demonstrated the following:

- Comparisons of the field data for acceleration magnitudes associated with the natural frequencies, sensitivity coefficients based on the natural frequencies, and tilt meter values can be effective in detecting changes in structural integrity.

- Based on the field data, it was demonstrated that the Signature Assurance Criterion values, and natural frequency values, although not effective by themselves, may be used to supplement these three primary methods.

- The subsequent finite element analysis studies confirmed that monitoring methods appropriate for detecting global changes in structural integrity for passively collected data include sensitivity coefficients for the natural frequencies, acceleration magnitudes associated with natural frequencies, and tilt meter data. Supplementary structural health monitoring methods include natural frequency values and SAC values.

An analysis of the bridge monitoring data using the upgraded bridge monitoring system demonstrates that the bandwidth of the acceleration measurements is increased to capture multiple lower modes of vibration of the bridge. Research was initiated under this project and is currently ongoing to utilize this new data to enhance the long-term structural health monitoring of the bridge.

Using the upgraded bridge monitoring system, a data qualification procedure has been developed and applied to the upgraded bridge monitoring system on this bridge. The data anomalies and error quantification are provided in this report. The upgraded bridge monitoring system is shown to be providing good quality sensor data for use in structural health monitoring.

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