Connecticut Permanent Long-Term Bridge Monitoring Network Volume 2: Monitoring of Curved Post-Tensioned Concrete Box-Girder Bridge – I-384 WB Over I-84 in East Hartford (Bridge #5686)

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16. Abstract
This report describes the instrumentation and data acquisition for a three-span continuous, curved post-tensioned box-girder bridge in Connecticut. The computer-based remote monitoring system was developed to collect information on the deformations, accelerations and temperature distributions to evaluate the long-term behavior and performance of the bridge. The monitoring system was installed in 1999, as the first system in a long-term research project to evaluate a variety of bridges in Connecticut. The data collected over multi-year periods from normal vehicular traffic has been used to learn about long-term performance of this bridge, resulting in a series of papers. The initial study developed an approach using histograms to better define natural frequencies from the extensive field data. The second study explored the influence of temperature distributions on the overall behavior of the bridge, including evaluation of the cause of cracking in both the box girders and the interior column supports. The third study looked at the influence of temperature variations on the baseline data, needed to remove the effect of temperature variations from data generated for long-term structural health monitoring. The final study has used the information previously developed for both this bridge and others to establish a baseline for long-term structural health monitoring and performance evaluation to determine if changes in the structural integrity are developing over time. 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)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		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)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		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)

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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 bridges 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 addition of video collection, and development of techniques for long-term structural health monitoring. Specifically for this bridge, during the current project 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 the I-384 Overpass in East Hartford (Inventory Number 5686), located at the intersection of this highway with I-84. The bridge was constructed in 1985. It is a curved, post-tensioned, five-celled, box-girder bridge with three unequal spans. An aerial view is shown in Figure 1. There are two curved box-girder bridges in the interchange, located between the two transverse expansion joints that appear as white lines in the photo. The monitored bridge is the one on the right, i.e. the longer of the two bridges. Figure 2 shows a view taken below the bridge from one of the two abutments.

Figure 1. Aerial View of Post-Tensioned Box-Girder Bridge

Figure 2. Underside of Post-Tensioned Box-Girder Bridge

The bridge plan and elevation and cross-sections are shown in Figures 3 and 4, respectively. The bent caps are integral at the interior supports, which creates a total of 15 separate interior cells in the bridge. Each is accessed from a hatch on the underside of the bridge. The two interior round column supports are connected integrally with the superstructure. The ends are partially restrained against longitudinal displacements, so that they are neither pinned nor fixed.

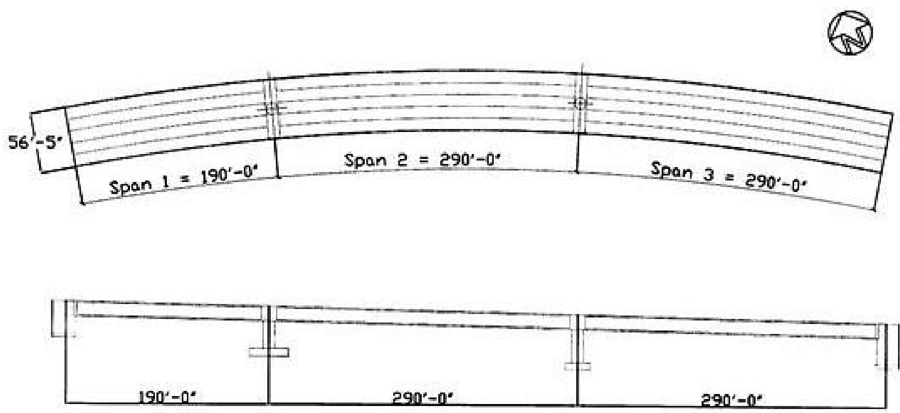

Figure 3. Plan and Elevation of Post-Tensioned Box-Girder Bridge

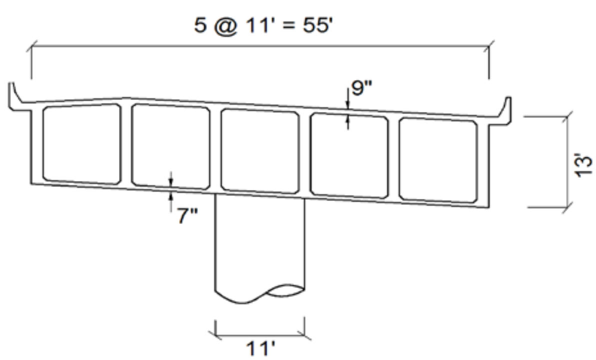

Figure 4. Cross-Section of Post-Tensioned Box-Girder Bridge

Immediately following construction of the bridge, cracks developed in the box sections at the interior supports, columns and decks. The cracks in the columns were in the spiral direction, indicating torsion cracking. The cracks in the box girders were visible in the interior of the boxes, with major components in the diagonal directions. The deck cracks are no longer visible because of the wearing surface. The cracks were injected with an epoxy compound shortly after they occurred, and this repair continued during the next few years as the cracks continued to propagate.

In 1998, following an extensive review, the engineers responsible for the bridge decided that renovations were needed. This involved (1) further epoxy injection; (2) the addition of post-tensioning in the box girders over the interior supports in the transverse direction; and (3) the use of FRP column wrapping on the two interior columns.

OBJECTIVES AND SCOPE OF STUDY

This bridge was the first in the research project to implement long-term monitoring systems on a network of bridges in Connecticut, using different bridge types and sensor combinations. As the first bridge in this research project, the initial goal was to develop a monitoring system that would be operable over a multi-year period and then to use what was learned with this system to develop additional bridge monitoring systems for other bridges. Additionally because this was to be the first monitoring system, it would have four sensor types and a larger number of sensors than envisioned for other bridges in the network.

One of the main interests in this system 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 data would be collected over multiple years in order to provide information on how the temperature differences influence the cracking behavior and to use it to explore structural health monitoring techniques for use in the long-term evaluation of the bridge's structural integrity.

INSTRUMENTATION AND DATA ACQUISITION

After development of a detailed specification, the monitoring system was put out to bid. The system chosen has twelve thermocouples, sixteen accelerometers, six tilt meters, and sixteen strain gages. Three additional thermocouples were added in the summer of 1999. The sensors are connected to an HP Computer, with two scanning A/D Converters, one capable of handling 32 channels and one 64 channels. The strain gages are connected to the 32 channel converter, and the accelerometers, tilt meters, and thermocouples are connected to the 64 channel converter. An external power source provides excitation, and an onsite laptop computer is connected to the monitoring instrument. The computer controls the instrument using a monitoring program written in HP VEE. The system has been designed to be remotely accessed for data retrieval and system control.

The strain gages were problematic using the original monitoring system, prior to its upgrade in the current phase of this research. The strain data collected was clearly not correct. It was found that the strain data typically did not change with time, and where small changes were noted, the changes essentially followed similar patterns. The main problem was that strain gages lacked the resolution to measure dynamic strains, i.e. those associated with live loads. Thus, all data collection prior to the recent upgrade of the monitoring system has been based on data from the thermocouples, accelerometers and tilt meters placed inside the box girders. They are distributed over the three spans and across the cross-section. The plan layout is shown in Figure 5.

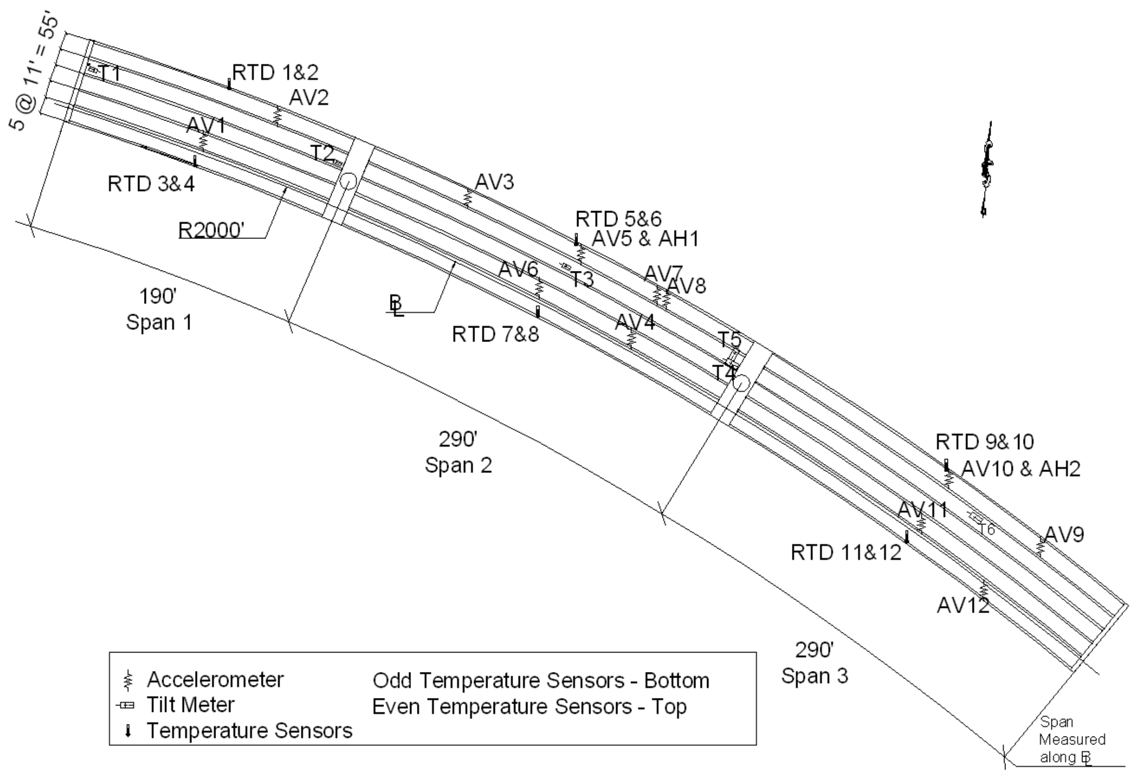

Figure 5. Locations of Sensors for Post-Tensioned Box-Girder Bridge

The monitoring system was designed to collect two general types of data. The tilt and temperatures do not change dynamically, and thus they only need to be collected at pre-specified intervals, normally every fifteen minutes. The acceleration data changes with the dynamic vehicular loading and is collected using a trigger-based approach. Whenever the acceleration of a specific accelerometer is above a certain value, the data collection system is triggered. It records the activity on all the accelerometers for several seconds before and after the trigger. Thus, each data set records the full crossing of the vehicle, typically a truck of some type. Both these monitoring tasks are carried out simultaneously by the monitoring software.

All the data is stored in comma delimited data files that are easily read by Excel. The data files were transferred over a modem connection from the field computer to a computer located at the University of Connecticut. The data processing is carried out by post-processing software developed at the University of Connecticut. The acceleration files are processed with Fast Fourier Transform (FFT) software to extract the fundamental vibration frequencies. A peak finding routine developed in this research was then utilized to determine natural frequency values, along with the accelerations at these peaks (Lengyel and DeWolf, 2003). The data is then saved for analyses at the University of Connecticut.

DATA ANALYSIS

There has been a series of studies using the extensive data collected over multi-year periods from this bridge. The initial task was to reduce the extensive vibration information into a form that would be useful for long-term structural health monitoring. An approach based on using histograms developed from accelerations in the frequency domain was used to define the lowest seven natural frequencies. In the next study, the data, along with an extensive finite element analysis, was used to look at the deformations in the bridge due to temperature variations across the width and through the depth. This was used to determine the cause of cracking in the box girders and interior columns. This was followed with a study to determine how external temperature variations due to climate influence the vibration behavior. This information is necessary to develop a basis for long-term structural health monitoring, so that the influence of temperature variations can be removed from structural changes. The most recent work has involved development of a structural health monitoring approach that can be used to check for major changes in structural integrity of the bridge.

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 more complete information.

Basic Vibration Information

The initial installation of the monitoring system, development of software to manage the data acquisition, storage and retrieval remotely is described by Lengyel (2001), Lengyel et al. (2000), and DeWolf et al. (2002). Use of vibration information for structural health monitoring at a minimum requires natural frequencies and associated acceleration levels.

Lengyel (2000) and Lengyel and DeWolf (2003) used FFTs to extract the natural frequencies from events associated with larger vehicles during the first year and a half of monitoring. They used a peak finding routine to determine the coordinates of each peak in the FFT for each accelerometer, and they analyzed the occurrences with histograms. This approach was based on the fact that true natural frequencies should occur significantly more often than other supposed frequencies. Figures 6 and 7 show histograms for data collected in the months of January and May, respectively.

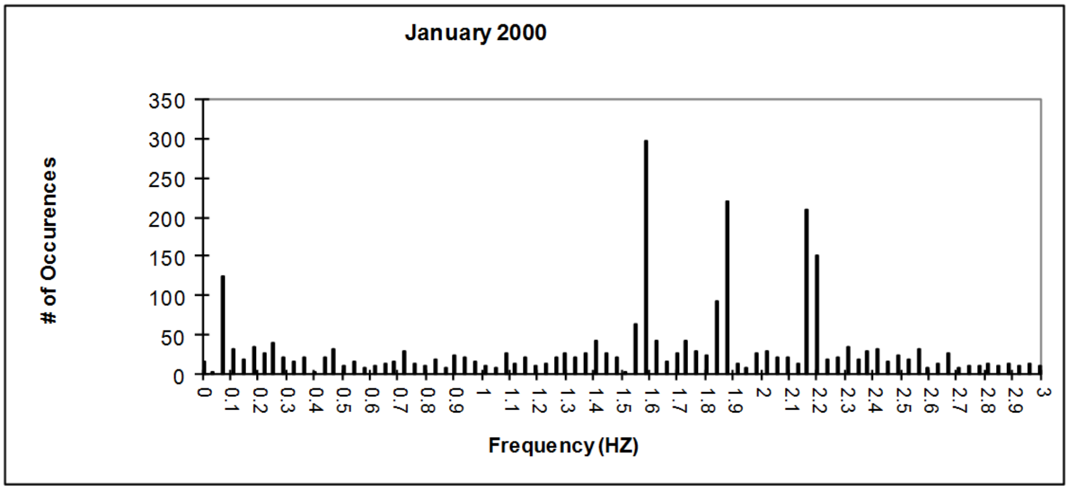

Figure 6. Histogram for January 2000

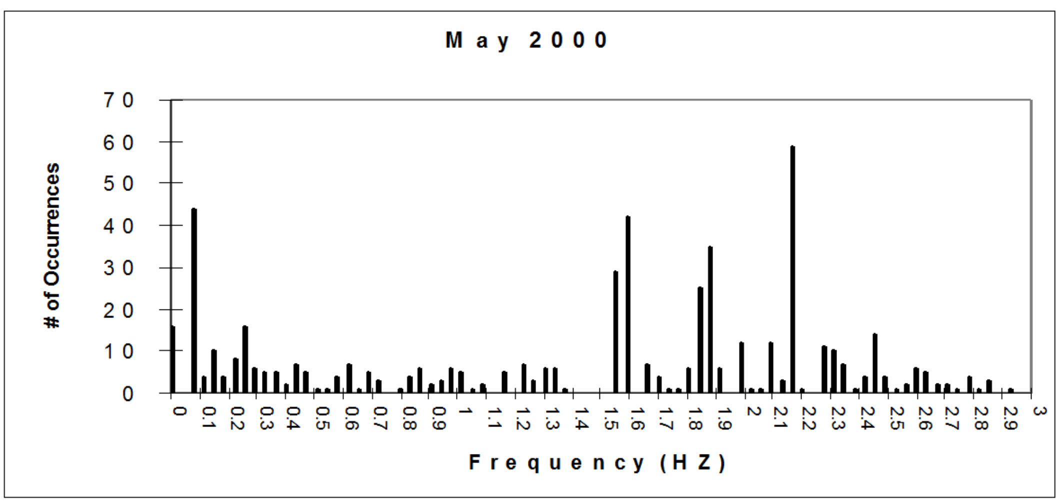

Figure 7. Histogram for May 2000

These figures show slight shifts in the natural frequencies. Ultimately the natural frequencies were confirmed with an extensive finite element analysis by Fu (1999), reported in the next section of this report.

Table 1 presents the lowest natural frequencies using both the finite element analysis by Fu and the field data. As shown, Lengyel was able to extract a total of 7 natural frequencies from the data using histograms, ranging between 1 and 5 Hz.

Table 1. Natural Frequency Determined from Analytical and Experimental Data

Mode Shape	Natural Frequency Finite Element Analysis (Hz)	Natural Frequency from Histogram (Hz)
Torsional	1.55	1.5509 – 1.6228
Flexural	1.85	1.8391 – 1.8752
Torsional	2.3	2.1275 – 2.2237
Flexural-Torsional	2.65	
Flexural	3.12	3.029 – 3.0651
Flexural	3.34	
Torsional	3.72	3.642 – 3.7141
Flexural-Torsional	4.4	4.1469 – 4.2911
Bending	4.65	4.43 – 4.50

Structural Behavior and Cause of Cracking

The monitoring data has been used to study the overall behavior and to explain the initial causes of cracking. Fu (1999) and Fu and DeWolf (2001; 2004) developed an extensive finite element model using the data to determine how temperature influences the bridge behavior.

Studies of the static and dynamic behavior of box-girder bridges have included both laboratory models and full-scale structures (Fu, 1999). Thermal behavior of box-girder bridges has been studied by many researchers, including Churchward and Sokal (1981), Shiu and Tabatabai (1994), Hunt and Cooke (1975), Elbadry and Ghali (1983), Branco and Mendes (1993), Mirambell and Aguado (1990), Shushkewich (1998), Hoffman et al. (1980), and Imbsen et al. (1995).

Fu (1999) developed a finite element model to explore the influence of differential temperatures on the overall behavior. The box girder flanges, webs and diaphragms are modeled with shell elements, with six degrees of freedom per node. The top flange, at the deck level, is modeled with the shell elements in two layers, one for the bituminous concrete and one for the reinforced concrete box girder flanges. The bottom box girder flange is modeled with the shell elements in a single layer. The webs and diaphragms are also modeled with shell elements, connected to the top and bottom flanges. The two reinforced concrete columns are modeled with beam elements. The bottom of the columns is fixed against all displacements, based on the rigidity of the spread footings that support the columns. The top of the columns is rigidly connected to the box sections, consistent with the design, as well as changes introduced during the renovation of the bridge prior to installation of the monitoring system. The finite element model was calibrated using the natural frequencies determined from the experimental data, along with the tilt data. The bearings at the ends of the bridge were designed to limit the expansion in the longitudinal direction, providing partial restraint. The finite element used boundary beam elements to model this restraint, with these elements acting as axial springs. The natural frequencies, along with further verification from the tilt data was then used to adjust the axial springs. The calibrated model was first used to develop the natural frequencies given in Table 1.

The finite element model was also used to evaluate the influence of differential temperatures on the deformations and behavior, with the goal of determining the cause of cracking in the interior support columns and box girders at the interior supports (Fu, 1999; Fu and DeWolf, 2004). Two studies were carried out. The first involved looking at changes in the mean interior temperature. Using field data, it was determined that a mean temperature increase of 28°F could be used to reflect the change over a 24-hour period. Applying this change to the finite element model, with its end constraints as previously explained, the bridge superstructure deforms horizontally as shown in Figure 8. This figure shows the torsion deformations at the interior column locations. These rotations correspond to the column spiral cracks that were found during field inspections prior to the renovations.

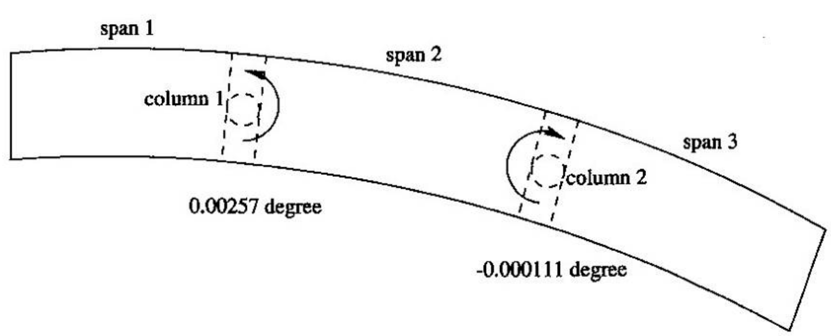

Figure 8. Rotations of Interior Columns Due to Temperature Increase

The second part of the study involved looking at the influence of the temperature distribution across the bridge width. This modeled behavior that occurs when the sun warms the southern side of the bridge. A review of the extensive data determined that a maximum difference horizontally across the top of the deck is 1.4ºF and across the bottom of the deck is 6.4°F. The maximum difference vertically is 4.8ºF, on the side opposite to the side warmed by the sun. When these values were applied to the finite element model, it was shown that the temperature differential increased the transverse rotation, resulting in additional torsion deformations. The changes in the deformations due to this temperature change explained that torsion cracks in the box girder sections near the interior supports. The transverse temperature differential was shown to have a greater effect on the bridge than the mean temperature change studied in the first part of this study.

The use of the finite element model along with temperature data collected in the field has shown qualitatively that the cracking in the interior support columns and box girders is a result of differential temperatures due to the sun and that it is not due to live loads.

Influence of Temperature on Vibration Information

Liu and DeWolf (2007) and Liu, DeWolf and Kim (2009) have used the data to determine how changes in temperature influence the modal information. There is a decrease in natural frequencies with increasing temperature. This information is needed to establish a basis for the long-term structural health monitoring of this bridge.

The basic concept underlying the use of linear, vibration-based damage detection is that global modal parameters (notably natural frequencies, mode shapes, and modal damping) are functions of the physical properties of the structure (mass, damping, and stiffness). Changes in the physical properties will cause changes in the modal properties and the measured responses of the structure. Environmental changes, primarily due to temperature changes, can have a significant effect on the modal properties. Thus it is necessary to quantify changes due to temperature so that that the changes in vibration response resulting from any damage can be discriminated from changes resulting from environmental variability.

A number of researchers have investigated the influence of environmental effects on modal variability in different types of bridges. DeWolf, Conn and O'Leary (1995) and Fu and DeWolf (2001) looked at a two-span composite steel-girder bridge. Farrar et al. (1997) and Sohn and Dzwonczyk et al. (1999) looked at a seven-span composite bridge. Wahab and Roeck et al. (1997) conducted dynamic tests on a skewed three-span box-girder bridge. Rohrmann et al. (2000) looked at a three-cell box-girder concrete bridge. Peeters and Roeck (2001) and Sohn et al. (2002) and Ko et al. (2003) looked at analytical techniques to determine the correlation between temperature and modal frequencies.

A new finite element model was developed to fully define the model behavior and explore the overall dynamic behavior of the bridge following the deployment of monitoring system. The finite element model allowed for better determination of the natural frequencies and mode shapes, necessary because there are not sufficient accelerometers on the bridge to fully identify the mode shapes. The properties of the finite element model were based on the design requirements, and the model was based on the same approach used by Fu in the preceding section, including development of the partial restraint at the ends of the bridge. Four-node shell elements with six degrees of freedom per node were used to model the superstructure of the bridge. The analysis was based on study of the lowest eight vibration modes in the frequency range.

A total of 932 events during 2002 data were selected to establish the baseline. The procedure used for extraction and selection of natural frequencies ensured the high resolution of post-processed data.

Figures 9, 10 and 11 show the first three measured natural frequencies versus the in-situ concrete temperatures. The scatter in these plots is mainly due to the noise resulting from the data acquisition system. This is a normal problem with field data. It is necessary to look at large sets of data to determine the general trend. Thus, the field data shows, from a statistical view, that the temperature-natural frequency relationship is basically linear. As shown, in all modes the natural frequencies decrease as the temperature rises. This needs to be incorporated into the baseline for long-term monitoring.

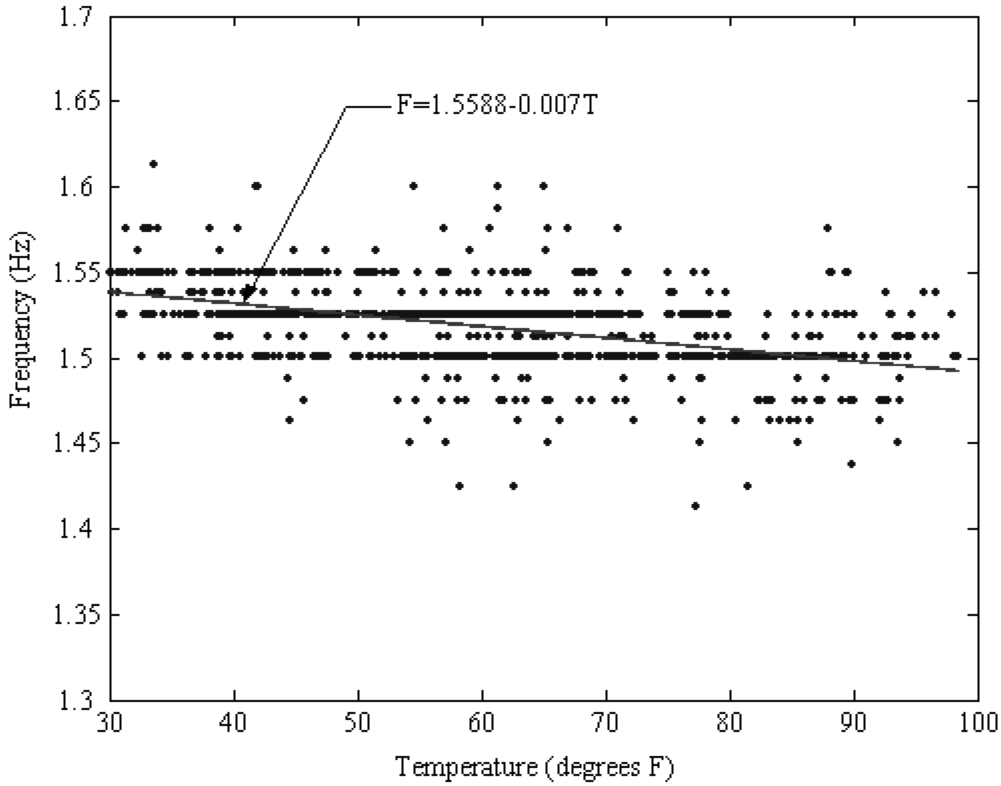

Figure 9. Natural Frequency Variation with Temperature Change for Mode 1 (1.52 Hz)

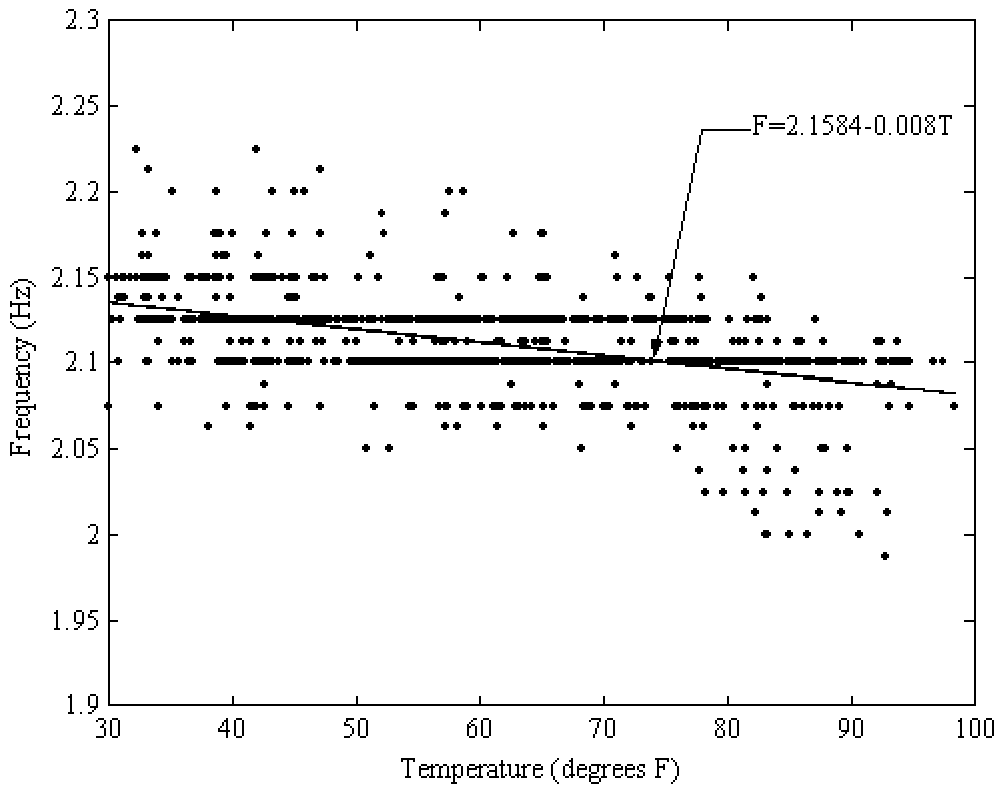

Figure 10. Natural Frequency Variation with Temperature Change for Mode 1 (2.11 Hz)

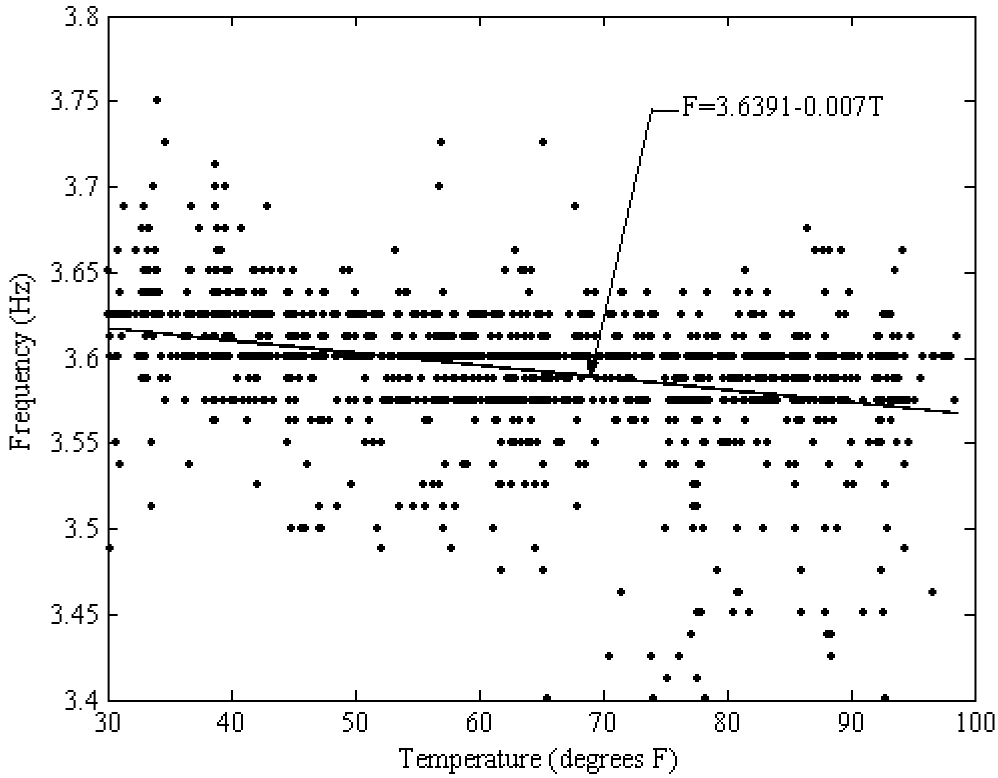

Figure 11. Natural Frequency Variation with Temperature Change for Mode 1 (3.59 Hz)

A linear regression model has been developed to estimate the three natural frequencies as a function of temperature:

F_1=1.5588-0.007T *(First Natural Frequency: 1.52 Hz),*

F_2=2.1584-0.008T *(Second Natural Frequency: 2.11 Hz),*

F_3=3.6391-0.007T *(Third Natural Frequency: 3.59 Hz)*

The resulting equations are shown on the figures. These three linear regression equations, developed with a full-year cycle data from 2002, were also used for comparison with data collected in 2005. There were gaps in the data collection during 2005, but there is sufficient data collected in both winter and summer months for comparison. The results demonstrated that equations developed for the 2002 data work well for the 2005 data.

Based on further study, the results from this study demonstrate that use of the natural frequencies alone is not sufficient for damage detection because of their sensitivity to temperature. Nevertheless, monitoring for defects must be based on comparisons of the vibration results obtained at the same temperatures.

Structural Health Monitoring

Using information from earlier studies at the University of Connecticut, Olund (2008) and Olund and DeWolf (2007) established the basis for structural health monitoring for this bridge. The approach is for long-term evaluation of the bridge's structural integrity, to check if there are major changes that require action. This study was part of a combined study to look at two curved bridges in the monitoring program, this post-tensioned concrete bridge and a curved steel box-girder bridge.

There are two general types of monitoring, active and passive. Active monitoring requires knowledge of the input force, i.e. specific information on the actual loads. As noted earlier, the long-term studies in Connecticut are based on a passive monitoring approach. Passive structural health monitoring uses data from a monitoring system where the excitation force is based on random vehicular traffic, i.e. where the actual loads for each test case are not known. The main benefit for passive monitoring is that it allows for continuous monitoring since the bridge does not need to be closed to traffic to conduct the testing. With the high volumes of traffic in Connecticut it would be impractical to close a bridge and conduct active monitoring. It is nearly impossible to close an entire bridge to perform a single test, let alone multiple tests needed for long-term monitoring.

In establishing a structural health monitoring approach, it is necessary to consider how load types, environmental conditions, and sensor placement influence the data. The large amount of data obtained from the long-term monitoring of the bridge necessitates a strong organizational system. Also of importance is the need to eliminate data that is not useful. As noted previously, only data collected from larger vehicles, i.e. trucks, is saved. Truck speed, weight, bridge cross section location, shock absorbency, etc., are all uncontrollable and together will alter one data set from the next. Thus for monitoring, it is necessary to statistically categorize the data sets, using trends for comparisons.

Another major influence is the surrounding environment, namely temperature and thermal gradients caused by solar gain. As Liu and DeWolf demonstrated earlier in this report, temperature inversely affects natural frequencies of bridges. As the temperature increases, the natural frequencies of a bridge generally decrease. It is essential, therefore, that the natural frequencies be adjusted to account for temperature.

The data is a function of the placement of the sensors, as well as the number. The sensors need to be located where they will provide relevant and useful data. With larger bridges as the one of interest here, it is not economical to place sufficient sensors to fully determine mode shapes, and thus the structural health monitoring can't be fully reliant on complete mode shapes. Since mode shapes are normally determined from field data by normalizing the accelerations at specified natural frequencies, it is necessary to combine the information from the field data with a detailed finite element analysis to have a better understanding of what should be included in the structural health monitoring approach.

Because of the forgoing variations, parameters for structural health monitoring should be statistically based on the use of vibration and tilt data (DeWolf, Cardini, Olund and D'Attilio, 2009). The goal is to be able to identify major changes in the structural integrity, ones that would raise concerns that the bridge is undergoing significant structural changes. This approach is based on generating a global view of the bridge, not a localized one. As proposed by Olund and DeWolf, the structural health monitoring of this bridge should evaluate statistically the following items: (1) lowest natural frequencies that occur on a regular basis; (2) acceleration levels associated with the natural frequencies, based on the FFTs; (3) diagonal terms of the sensitivity coefficient matrices for the natural frequencies; and (4) tiltmeter values.

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, with 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 Big Foot 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 ConnDOT network using a java-based Real-Time Data Viewer (RTD);
- Capability for automated data archival to an offsite FTP server; and
- Flexibility to expand the current system to new sensors.

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 this highway bridge are shown in Figure 12.

Bridge: Bigfoot
Location: East Hartford, CT
Highway: I-384
NBI #: 5686

Sensor Information									
Sensor	*Sensor Type*	*Signal Clipping*	*Intermittent Noise Spikes*	*Signal Dropouts*	*Spurious Trends*	*Periodicy*	*Aliasing*	*Quantization Error (%)*	*Working SNR (dB)*
A1	Accelerometer	--	--	--	--	--	--	6.10E-07	29.73
A2	Accelerometer	--	--	--	--	--	--	6.10E-07	21.50
A3	Accelerometer	--	--	--	--	--	--	6.10E-07	28.52
A4	Accelerometer	--	--	--	--	--	--	6.10E-07	25.91
A5	Accelerometer	--	--	--	--	--	--	6.10E-07	27.20
A6	Accelerometer	--	--	--	--	--	--	6.10E-07	11.64
A7	Accelerometer	--	--	--	--	--	--	6.10E-07	28.80
A8	Accelerometer	--	--	--	--	--	--	6.10E-07	27.80
A9	Accelerometer	--	--	--	--	--	--	6.10E-07	31.79
A10	Accelerometer	--	--	--	--	--	--	6.10E-07	30.29
A11	Accelerometer	--	--	--	--	--	--	6.10E-07	25.41
A12	Accelerometer	--	--	--	--	--	--	6.10E-07	27.75
A13	Accelerometer	--	--	--	--	--	--	6.10E-07	27.66
A14	Accelerometer	--	--	--	--	--	--	6.10E-07	27.04
A15	Accelerometer	--	--	--	--	--	--	6.10E-07	26.49
A16	Accelerometer	--	--	--	--	--	--	6.10E-07	56.28
S1	Strain Gage	--	--	--	--	x	--	7.00E-09	15.31
S2	Strain Gage	--	--	--	--	x	--	7.00E-09	13.68
S3	Strain Gage	--	--	--	--	x	--	7.00E-09	16.03
S4	Strain Gage	--	--	--	--	x	--	7.00E-09	14.06
S6	Strain Gage	--	--	--	--	x	--	7.00E-09	20.08
S7	Strain Gage	--	--	--	--	x	--	7.00E-09	17.03
S8	Strain Gage	--	--	--	--	x	--	7.00E-09	12.78
S9	Strain Gage	--	--	--	--	x	--	7.00E-09	22.26
S10	Strain Gage	--	--	--	--	x	--	7.00E-09	22.27
S11	Strain Gage	--	--	--	--	x	--	7.00E-09	21.79
S12	Strain Gage	--	--	--	--	x	--	7.00E-09	22.34
S13	Strain Gage	--	--	--	--	x	--	7.00E-09	21.81
S14	Strain Gage	--	--	--	--	x	--	7.00E-09	22.62
S15	Strain Gage	--	--	--	--	x	--	7.00E-09	21.94
S16	Strain Gage	--	--	--	--	x	--	7.00E-09	21.53
Temp1	Temperature	--	--	--	--	--	--	2.81E-04	
Temp2	Temperature	--	--	--	--	--	--	2.81E-04	
Temp3	Temperature	--	--	--	--	--	--	2.81E-04	
Temp4	Temperature	--	--	--	--	--	--	2.81E-04	
Temp5	Temperature	--	--	--	--	--	--	2.81E-04	
Temp6	Temperature	--	--	--	--	--	--	2.81E-04	
Temp7	Temperature	--	--	--	--	--	--	2.81E-04	
Temp8	Temperature	--	--	--	--	--	--	2.81E-04	
Temp9	Temperature	--	--	--	--	--	--	2.81E-04	
Temp10	Temperature	--	--	--	--	--	--	2.81E-04	
Temp11	Temperature	--	--	--	--	--	--	2.81E-04	
Temp12	Temperature	--	--	--	--	--	--	2.81E-04	
Tilt1	Tilt Meter	--	--	--	--	--	--	1.20E-06	
Tilt2	Tilt Meter	--	--	--	--	--	--	1.20E-06	
Tilt3	Tilt Meter	--	--	--	--	--	--	1.20E-06	
Tilt4	Tilt Meter	--	--	--	--	--	--	1.20E-06	
Tilt5	Tilt Meter	--	--	--	--	--	--	1.20E-06	
Tilt6	Tilt Meter	--	--	--	--	--	--	1.20E-06	

Figure 12. Results of Data Qualification for Bridge Monitoring System

There are no signal clipping, intermittent noise spikes, signal dropouts or spurious trends observed in the measured sensor data. There is a periodicy observed, a ground loop, at 60 Hz for the strain sensors. This periodicy is well above the sensor's effective bandwidth of 0-20 Hz and is addressed through filtering. No aliasing is present in the measurements. The quantization error is negligable. The working signal-to-noise ratio (SNR) for the accelerometers ranges from 27-30 dB (signal is 22.38-31.63 times larger than the noise floor), while one accelerometer's SNR is 11.64 dB (signal is 3.82 times larger than the noise floor) and at the other extreme another accelerometer's SNR is 56.28 dB (signal is 651.63 times larger than the noise floor). The SNRs for both the accelerometers are considered acceptable for future structural health monitoring work.

The working signal-to-noise ratio (SNR) for the strain sensors is around 20 dB (signal is 10 times larger than the noise floor). With the recent upgrade of the monitoring system it should now be possible to utilize the strain sensors for structural health monitoring.

CONCLUSIONS

This report is based on the monitoring of this curved, post-tensioned box-girder bridge. The field monitoring system was installed in 1999 and has served as a basis for developing other monitoring systems in this continuing research program to implement long-term monitoring systems on a network of bridges important to Connecticut's highway infrastructure.

The computer-based remote monitoring system was developed to collect information on the deformations, accelerations and temperature distributions to evaluate the long-term behavior and performance of the bridge. The report addresses the following areas:

- The initial study involved the development of software to deal with the extensive data generated by the monitoring system and defined key vibration information that could form the basis of continued long-term monitoring. The research developed an approach using histograms to better define natural frequencies from extensive field data.

- The second study used a finite element analysis, calibrated with field data, to explore the influence of temperature distributions on the overall behavior of the bridge. This was developed to study the cause of cracking in both the box girders and the interior column supports.

- The third study determined the influence of temperature variations on the baseline data, needed to remove the effect of temperature variations from data generated for long-term structural health monitoring.

- The final study has used the information previously developed for both this bridge and others to establish a baseline for long-term structural health monitoring and performance evaluation. The goal is to determine if changes in the structural integrity develop over time, ones that are cause for concern.

The monitoring system is now planned for updating so that it uses the same technology as other newer systems in the research project. This will allow for better data collection and generate results that can be automated for the long-term structural health monitoring of this bridge.

A data qualification procedure has been developed and applied to the upgraded bridge monitoring system on this bridge. The data anomalies and error quantification is provided in this report. The upgraded bridge monitoring system is shown to be providing good quality sensor data for use in structural health monitoring. The recent upgrade of the monitoring system now allows for incorporation of the strain sensors for structural health monitoring.

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