A TEMPERATURE DRIVEN METHOD FOR STRUCTURAL HEALTH MONITORING

John Patrick Reilly

A DISSERTATION PRESENTED TO THE FACULTY OF PRINCETON UNIVERSITY IN CANDIDACY FOR THE DEGREE OF DOCTOR OF PHILOSOPHY

RECOMMENDED FOR ACCEPTANCE BY THE DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING

ADVISER: PROFESSOR BRANKO GLIŠIĆ

June 2019
Abstract

Structural Health Monitoring (SHM) is the process of equipping a structure with sensors in order to assess the condition of that system using a combination of structural and analytic techniques. In every application, SHM involves the comparison of an unknown structural state to a baseline of healthy behavior. SHM as a field has come to prominence as a solution to aging infrastructure but still lacks the ability to consistently monitor and assess different types of structures. One obstacle is that the uniqueness of bridges and buildings necessitates individualized monitoring strategies. Another challenge is the influence of environmental effects on structures, especially temperature. In bridges, daily temperature changes can influence the strain on the structure more than daily traffic loads. Many SHM techniques struggle to compare a baseline, healthy structural state to a new unknown state because of differences in the environmental conditions.

Temperature Driven-Structural Health Monitoring (TD-SHM) considers temperature as the driving force in structural behavior and therefore the driving force in monitoring. TD-SHM compares an input temperature to an output strain and displacement to form three-dimensional signatures for the structure. These signatures describe the behavior for the structure across all monitored temperatures. Changes in these signatures indicate unusual behavior or damage in the structure. Two immediate benefits of this method are the universal nature of temperature effects on structures and the ability to measure temperature as an input on a structure for the formulation of a complete input-output model.
This thesis develops TD-SHM using strain, temperature, and displacement data measured on the Streicker Bridge at Princeton University. The main conclusions are the following: 1) thermal gradients on structures can obscure the temperature-strain relationship, but it is possible to filter out time periods where these effects are present; 2) the coefficient of thermal expansion (CTE) of concrete structures varies throughout seasons and throughout structures but are consistent through the years, and 3) three-dimensional temperature signatures can provide insight into the thermal behavior of the structure and highlight unusual behavior based on changes to that signature.
This research is primarily focused on the development of TD-SHM using data taken from Streicker Bridge. This dissertation would not be possible without the hard work of past undergraduate and graduate students in installing and maintaining the Streicker monitoring system. These students include Chienchuan Chen, Jeremy Chen, Jessica Hsu, George Lederman, Kenneth Liew, Maryanne Wachter, Konstantinos Bakis, Allison Halpern, David Hubbell, Morgan Neal, Daniel Reynolds, and Daniel Schiffner. Past graduate students Dorotea Sigurdardottir and Hiba Abdel-Jaber in particular were instrumental in maintaining the monitoring system and data for use by future students such as myself. The Streicker Bridge monitoring project has benefitted from the hard work of many faculty, staff, and students and Princeton University and will provide knowledge and research opportunity for years to come.

This research has been supported by National Science Foundation Grant CMMI-1434455. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect the views of the National Science Foundation. Additional funding was provided by USDOT Office of the Assistant Secretary for Research and Technology (Grant No. DTRT13-G-UTC28).
Acknowledgements

Dr. Branko Glisic deserves the first acknowledgement for his continuing hard work, patience, and direction he provided me throughout my five years at Princeton. His constant availability and insight to the thermal behavior of concrete structures has helped shaped this thesis, and I was very lucky to have had him as a mentor.

Dr. Matthew Yarnold at Texas A&M University has provided guidance with temperature based SHM since the beginning of my graduate work and has contributed to my work time and time again by serving on research committees, providing academic guidance, and finally by reading my thesis. TD-SHM would not exist in its current form with Dr. Yarnold. Dr. Maria Garlock and Dr. Sigrid Adrianssens have also helped guide me throughout my PhD, continually serving on research committees, generals committees, and finally at my FPO. Their continued dedication to students beyond their own lab significantly contributes to the strength of our department. Joe Vocaturo was extremely helpful during the installation of the displacement sensors on Streicker Bridge, making sure that the sensors stayed in place and active for years rather than days.

I’ve been fortunate to work with some incredibly smart and dedicated people in the SHM lab at Princeton. Dorotea, Hiba, Xi, and Vivek have been extraordinarily helpful with research ideas and problem solving, as well as more real-world problems in graduate school. My greatest success at Princeton has been making friends with some wonderful people. Isabel, Link, and Victor pushed me over the finish line with my thesis, and I’d gladly do it all again to spend more time with them. Kelsey, thank you for standing with
me, helping me with work on late nights, sharing in my success and failure, and playing soccer with me. I’ve loved all the time we’ve spent together, and I can’t wait to spend even more.

Finally, the only reason I’ve ever had a chance at achieving anything in life is because of my family. I’m grateful for my parents, supporting me in whichever field I choose, and raising me to be the person I am today. To my siblings, for making sure I stay grounded and helping to save the world from infectious disease one boardgame at a time. To all my cousins: Elysia, John, Peggy, Ned, Bridget, Joe, Matt, Finn, Michael, Corinne, Abigail, Jack, Harry, and Thomas, thank you for your support, even though you might not have known it.
Contents

Abstract iii

Acknowledgements vi

1 Introduction.................................................................................................................. 1

1.1 Motivation .................................................................................................................. 2

1.2 Temperature Influences on Structures................................................................. 2

1.3 Aims and Objectives ............................................................................................... 4

1.4 Dissertation Organization......................................................................................... 5

2 Literature Review ...................................................................................................... 7

2.1 Structural Health Monitoring Methods ................................................................. 7

2.2 Thermal Action ......................................................................................................... 8

2.3 Coefficient of Thermal Expansion of Concrete ................................................... 10

2.4 Elastomeric Bridge Bearings.................................................................................... 12

3 Streicker Bridge ......................................................................................................... 14

3.1 Overview .................................................................................................................. 14

3.1.1 Southeast Leg Abutment ..................................................................................... 16

3.2 Monitoring System .................................................................................................. 17
3.2.1 Fiber Bragg Gratings ................................................................. 19
3.2.2 Displacement Sensors ............................................................... 20

4 Time Periods of Minimal Thermal Gradient ........................................ 22
4.1 Introduction .................................................................................. 22
4.2 Temperature Distribution and TD-SHM ....................................... 26
4.3 Methods for Determination of Minimal Thermal Gradients ............ 33
  4.3.1 Maximum Range ................................................................. 35
  4.3.2 Local Gradient ................................................................. 36
  4.3.3 Process for Identifying Time Periods of Minimal Thermal Gradient .. 38
4.4 Evaluation of Methods ................................................................. 39
  4.4.1 Evaluation of Maximum Range Method .................................. 40
  4.4.2 Evaluation of Local Gradient Methods .................................. 47
  4.4.3 MR and LG Comparison ..................................................... 52
4.5 Summary .................................................................................... 56

5 Evaluating the Coefficient of Thermal Expansion ............................. 58
5.1 Introduction ............................................................................... 58
5.2 Streicker Bridge ........................................................................ 60
5.3 Temperature and Strain in Concrete ............................................. 60
  5.3.1 Sources of Strain ............................................................... 60
  5.3.2 Thermal Expansion .......................................................... 63
7.1 Conclusions and Contributions ................................................................. 104

7.2 Future Work .............................................................................................. 106

Appendix A Sensor Validation and Regeneration ........................................... 108

Appendix B CTE Stiffness Calculations and Sensitivity Studies ....................... 116

8 Bibliography .................................................................................................. 119
List of Tables

Table 4.1 Number of time periods identified by Maximum Range (MR). 42
Table 4.2 Best fit line statistics for MR method. 47
Table 4.3 Number of time points identified by using Local Gradient (LG) metric. 48
Table 4.4 Best fit line statistics for LG methods, bounds in °C/m. 52
Table 4.5 Method comparison by number of points identified for different seasons, 2010. 55
Table 5.1 Fall CTE evaluations, µε/°C 75
Table 5.2 Summer CTE evaluations, µε/°C 77
Table 5.3 Winter CTE evaluations (µε/°C) 79
Table 5.4 Season average CTE evaluations, µε/°C 80
Table 6.1 Best fit line statistics for three-dimensional signatures. 91
Table 6.2 Best fit line statistics using a MR 6 gradient filter 92
Table 6.3 Best fit surface statistics for construction of signature 101
Table 6.4 Polynomial surface fit for P12h13 temperature, strain, and tilt signatures 101
Table B.1 Summer IQR values 117
Table B.2 Fall IQR values 117
Table B.3 Winter IQR Values 117
List of Figures

**Figure 3.1** Streicker Bridge plan view (“Streicker Bridge; Google Earth” n.d.)  
**Figure 3.2** Southeast leg (left) and main span (right) of Streicker Bridge  
**Figure 3.3** Southeast and Northeast legs intersect at junction with main span  
**Figure 3.4** Streicker Bridge Cross Section for main span (varies) and legs (constant)  
**Figure 3.5** SE leg abutment (left) and close up of neoprene bearing at SE abutment (right)  
**Figure 3.6** Elastomeric bearing schematic  
**Figure 3.7** Streicker Bridge plane view with sensor locations (Sigurdardottir and Glisic 2015)  
**Figure 3.8** Streicker Bridge elevation with sensor locations  
**Figure 3.9** FBG strain and temperature sensor (Glisic 2015)  
**Figure 3.10** Displacement sensor orientations at end of SE leg  
**Figure 3.11** Displacement gauges installed at SE leg abutment  
**Figure 4.1** Example of a beam-like bridge instrumented with a chain of parallel sensors along the deck and displacement sensors at extremity (Streicker Bridge).  
**Figure 4.2** Example of a bridge cross-section with linear temperature distribution.  
**Figure 4.3** Example of a bridge cross-section with non-linear temperature distribution (causing non-linear thermal gradient).  
**Figure 4.4** Example of a three-dimensional signature of temperature, strain, and longitudinal displacement with best fit line, taking into account data resulting from non-linear temperature distributions.  
**Figure 4.5** 2D projections of three-dimensional signature, keeping temperature as the vertical axis.  
**Figure 4.6** Example of temperature variation over two days in two cross-sections of Streicker Bridge (top and bottom sensors in cross-sections at P11 and P12).  
**Figure 4.7** Flowchart describing process of using methods for identifying time periods of minimal thermal gradient.
Figure 4.8 Range of temperature measurements from Spring 2016.
Figure 4.9 Points considered as having minimal gradients, observed using MR methods.
Figure 4.10 Three-dimensional signature example with best fit line (BFL) for time points with minimal thermal gradient, identified using MR method.
Figure 4.11 2D projections of three-dimensional signature, keeping temperature as the vertical axis. Red indicates time points of minimal thermal gradient found using MR = 4 °C/m, and red line indicates best fit line for minimal gradient data set, as in Figure 4.10.
Figure 4.12 Maximum vs. mean absolute local thermal gradient.
Figure 4.13 Three-dimensional signature with best fit line (BFL) for time points with minimal thermal gradient, identified using MeLG method.
Figure 4.14 2D projections of three-dimensional signature which significantly reduce the bi-linearity. Red indicates time points of minimal thermal gradient found using MeLG = 4.5 °C/m, and red line indicates best fit line for minimal gradient data set, as in Figure 4.13.
Figure 4.15 MR and MeLG Comparison, Pier 12.
Figure 4.16 Summer 2010 Streicker Bridge minimal thermal gradient methods comparison.
Figure 5.1 Example of one-month relationship between temperature and strain (example taken from Streicker Bridge during month of October 2010 at location P12up).
Figure 5.2 Example of two-month relationship between temperature and strain (example taken from Streicker Bridge from June to August 2010 at location P11h12down).
Figure 5.3 Simplified beam model under restrained expansion.
Figure 5.4 One month of strain and temperature data from P10h11, highlighting two different six-hour windows.
Figure 5.5 Distribution of slopes for P10h11, using six-hour window for the entire distribution in blue, and filtering out R$^2$ under 0.90 in red.
Figure 5.6 Fall CTE evaluations.
Figure 5.7 Summer CTE evaluations.
Figure 5.8 P13 temperature and strain for Summer 2011 to 2014.
Figure 5.9 Winter CTE evaluations

Figure 6.1 Longitudinal displacement from Apr 2016 to June 2017 with average temperature on the SE leg

Figure 6.2 Temperature and Strain at centroid of P13 with longitudinal displacement and best fit line (black)

Figure 6.3 Trapezoid integration of strain with longitudinal displacement, November 2016

Figure 6.4 Comparisons of real displacement with generated displacement

Figure 6.5 Residuals of strain integration with real displacement

Figure 6.6 Three-dimensional signatures with predicted displacement from multiple time periods: two viewpoints

Figure 6.7 Temperature signature for July 2011 with time periods of MR < 6 °C shown in red

Figure 6.8 Planar signature for P12h13 from April 2016 to September 2017

Figure 6.9 P12h13 thermal gradient, elastic curvature and longitudinal displacement with thermal gradient filter of MR = 6.0°C

Figure 6.10 Vertical displacement from April 2016 to June 2017 with average temperature on the SE leg

Figure 6.11 Tilt with average SE leg temperature

Figure 6.12 Tilt with P12h13 temperature and strain, 5 time period shown

Figure 6.13 P12h13 strain and displacement with tilt, and best fit polynomial surface. Data shown from April to November 2016, ending before shift

Figure 6.14 Transverse displacement from April 2016 to June 2017 with average temperature on the SE leg

Figure A.1 RMSE for P10 MS-UP

Figure A.2 Failed Sensor P10MS-UP

Figure A.3 Regeneration of Healthy Signal

Figure A.4 Failed Sensor Regeneration
1 Introduction

Structural Health Monitoring (SHM) is a process that uses a combination of sensor data and analytic techniques to assess the integrity of a structure. Typically monitoring involves comparing measured data to a known or theoretical baseline behavior. SHM is not a new subject of research but is still in its infancy of implementation as a means of monitoring real world structures. SHM has the potential to take a leading role in maintenance of critical infrastructure systems by drastically reducing the cost of this maintenance, as well as providing a basis for deeper understanding and research on the behavior of structures during their lifetime. There are four levels, or questions, that SHM methods attempt to answer in analysis (Rytter 1993):

1. Is there damage to the structure?
2. Where is the damage to the structure?
3. What is the extent of the damage to the structure?
4. How safe is the structure with the damage?

While the current practice of SHM faces many challenges, the main obstacle to overcome is to consistently achieve these four levels of analysis across all different types of structures. Each different structure necessitates a unique monitoring plan in order to maximize sensor utilization while minimizing the costs of the system. One unifying factor across all structures is the extent that daily and seasonal thermal influences can influence the behavior of the structural system. Temperature Driven – Structural Health Monitoring (TD-SHM) aims to capture this fundamental and complex relationship
between thermal influences and structural behavior, achieving the first three levels of SHM, and providing the necessary information for the fourth level.

1.1 Motivation

Civil infrastructure is crucial to the wellbeing of any country. Bridges ensure safe travel, buildings provide safe living spaces, and many other pieces of infrastructure provide water, electricity, and general transportation to the population. Failure in this infrastructure, though rare, can have disastrous results on communities. For example, the collapse of the I-35 Bridge in Minneapolis caused the death of 13 people and injured 145 people, not to mention the economic cost to the city (Economic Impacts of the I-35W Bridge Collapse 2009). With over 600,000 highway bridges in America and an average age of over 40 years, there is a growing need for an efficient method of monitoring and rehabilitation (Infrastructure Report Card: Bridges 2017). As of 2018 47,000 of these American bridges qualify as structurally deficient (Bridge Condition by Functional Classification Count 2018), meaning that the bridge requires significant rehabilitation or maintenance. According to the American Society of Civil Engineers, there is a backlog of current bridges needed rehabilitation, requiring 128 billion dollars (Infrastructure Report Card: Bridges 2017). SHM has the potential to mediate future rehabilitation costs by providing accurate information on the structural integrity of bridges across the country, identifying bridges in need of repair and bridges that are safe for the public.

1.2 Temperature Influences on Structures

Temperature changes substantially affect the day to day behavior of a structure but have a comparably smaller role in many contemporary SHM methods. In some instance these temperature effects are filtered out or ignored as noise, while in many cases daily
temperature changes can produce comparable stresses to daily traffic loads (Glisic and Inaudi 2008; Koo et al. 2013; Kromanis and Kripakaran 2014a; Laory et al. 2011; Ni et al. 2005; Yarnold 2013). Part of the reason these thermal effects are filtered out is the difficulty in accurately modeling or predicting the effects of different temperature distributions across a structure, even though temperature changes themselves are relatively easy to quantify compared to traffic loads on a bridge. In order to perform the most accurate analysis of a structure, temperature effects need to be considered very heavily, if not as one of the central considerations.

Increases in temperature induce expansion in a material. These thermal expansions or contractions can generate large thermal stresses if the material is not free to expand or contract. In many cases, however, a thermal gradient is applied to a structure rather than a uniform temperature change. Thermal gradients refer to the distributions, both local and global, of temperature across a structure. Thermal gradients occur naturally in all structures, inducing curvature in the material. Thermal effects pose a large challenge in monitoring as a structure can display different behavior when experiencing different environmental conditions. Varying environmental conditions can make the comparison between two states of a structure difficult or inconclusive. Thermal gradients are frequently non-linear, causing non-linear strain distributions, deplanations of the cross section, and obscuring the effects of thermal loads, making analysis complex and sometimes inconclusive (Emanuel & Hulsey, 1978). Identifying time periods of minimal thermal gradient on a structure will allow for the establishment of a relationship between temperature change, strains, and displacements. Central to this temperature-strain-displacement relationship is the coefficient of thermal expansion (CTE). The CTE
governs the rate at which a material expands for a given change in temperature. For many materials the CTE is well known and relatively constant. For concrete the CTE can vary widely depending on the concrete mix and moisture content in the material and can be very difficult to assess in situ. Modern methods necessitate taking a core sample of the concrete to evaluate the CTE of an as built structure, which is impractical after initial casting in many cases.

1.3 Aims and Objectives
The primary objective of this work is to create a Temperature Driven method of Structural Health Monitoring. This methodology will consider temperature as the driving force in structural behavior, compelling temperature as the driving force in monitoring and analysis. Temperature changes will be related to relevant strain and displacements to form a three-dimensional, damage sensitive signature for different locations on the structure. The hypothesis for TD-SHM is that if the strain and displacement for a structure are known for a wide range of temperatures, these three-dimensional relationships will accurately describe the behavior (thermal and otherwise) of the structure and be sensitive to unusual behavior or changes in structural condition. One main benefit of TD-SHM is the universality of temperature effects on structures. All structures undergo thermal influences; if TD-SHM proves viable as a SHM technique, this method could apply to all different types of structures. Another benefit is that temperature can be measured on a structure for the completion of a full input-output model. Many other types of monitoring either have an unknown input or require expensive testing to quantify an input for changes in strain or vibration. Formulation of TD-SHM is the overarching goal of this thesis, and there are many intermediate steps
before identifying and accurately analyzing these three-dimensional signatures. To achieve the objective of this thesis, the following tasks are identified:

1. Create a method for the identification of time periods of minimal thermal gradient on a structure
2. Create a method for the accurate evaluation of the CTE of built concrete structures
3. Formulate a means to quantify three-dimensional temperature signatures in order to identify changes to the signature representing damage
   a. Identify the best combination of temperature, strain, and displacement
   b. Assess the sensitivity to damage of the signatures
   c. Explore the ability to interpret changes in the signature into meaningful structural information

1.4 Dissertation Organization
This thesis is organized as follows: Chapter 1 introduces Temperature Driven – Structural Health Monitoring and sets the hypotheses and goals. Chapter 2 presents a review of relevant literature on SHM techniques, temperature effects on structures, and the coefficient of thermal expansion of concrete. Chapter 3 describes the Streicker Bridge and monitoring system, the source of all data used in this thesis. Chapter 4 creates and analyzes several new methods of identifying time periods of minimal thermal gradient on a structure. Chapter 5 creates and presents a novel method for evaluating the coefficient of thermal expansion of a built concrete structure. Chapter 6 explores relationships between strain, temperature, and displacement, as well as their derivatives, in order to formulate three-dimensional temperature signatures and infer their dependence on
unusual structural behavior. Chapter 7 summarizes the main contributions of this thesis and outlines several avenues of future work.
This chapter presents a literature review of SHM methods, focusing on monitoring thermal effects in structures as well as the coefficient of thermal expansion of concrete. A short section on the behavior of elastomeric bridge bearings is also presented, as the bearings play a large role in the temperature-strain-displacement relationship on the Streicker Bridge.

2.1 Structural Health Monitoring Methods

SHM has seen an increase in research as the need for reliable monitoring of infrastructure grows. One popular area of research involves vibration analysis of structures, which employs accelerometers and various numerical or dynamic models to monitor the structure. These vibration methods often use a form of modal analysis or time series analysis to detect damage in a structure (Buren et al. 2017; Figueiredo et al. 2011; Mei and Gül 2016; Sohn and Farrar 2001). Environmental conditions pose a significant challenge in vibration-based methods and have attracted substantial research into novel methods of either filtering or accounting for thermal effects (Cross et al. 2012; Chengyin and DeWolf. 2007; Mata et al. 2013; Ni et al. 2005). Other methods of SHM focus on strain measurements. One study tracked the location of the neutral axis of a composite beam as a damage sensitive feature (Sigurdardottir and Glisic 2013), while another explored the comparison of curvatures at different locations of a structure as a damage sensitive feature (Kliewer and Glisic 2017). Distributed fiber optic sensors offer one avenue for assessing strain over large distance and were used to monitor concrete piles.
and columns in high rise buildings (de Battista et al. 2017; Glisic et al. 2013). Another important aspect of SHM is development of a framework for economic decision making involved with infrastructure. One study presented a methodology based on Bayesian logic, decision theory, and an estimation of the expected revenue lost due to action or inaction for decisions on whether to perform maintenance or not on a structure (Zonta et al. 2014). Another work developed a method using expected utility theory for decision making based on SHM by providing probabilities of damage and expected financial costs for each action (C. Cappello et al. 2016).

2.2 Thermal Action

Temperature effects in a healthy structure can affect the monitoring features (strain, modal parameters, etc.) as much as or more than damage on a structure (Peeters and De Roeck 2001; Sohn et al. 1999; Yarnold and Moon 2015). Current design codes for bridges have considerations for temperature changes with both linear and non-linear thermal gradients (AASHTO LRFD Bridge Design Specifications 8th Edition. 2017; Comité Européen de Normalisation 2010), but these specifications are not exhaustive for thermal behavior in bridges. For example, the sun can heat parts of a bridge to much more than ambient temperatures, an event usually ignored in design codes (Peiretti et al. 2014; Xu et al. 2010), and the thermal response of structures can be highly variable depending on climate, structural system, and other factors (Hedegaard et al. 2013). Many areas of SHM research filter out temperature effects as they can affect the principal parameters being monitored, such as mechanical strain or modal parameters. For example, Laory et al. examined the effects of filtering out seasonal variations of temperature on damage detectability in a numerical bridge model (Laory et al. 2011),
while Worden et al. employed a variety of sophisticated analytic techniques including robust regression, cointegration, and the mahalanobis distance squared to remove environmental influences from modal parameters in free vibration analysis (Cross et al. 2011; Deraemaeker and Worden 2018; Dervilis et al. 2015; Worden et al. 2002). Current SHM sometimes treats thermal influences as an unavoidable environmental side effect that is to be filtered out. However, the effects of temperature, while sometimes necessary to filter out to isolate other parameters, may contain important information and reveal critical aspects of structural behavior (Yarnold 2013).

Temperature has been a central consideration in several notable works. Thermal effects have been used within the paradigm of structural identification, where real data from a structure is used to tailor a finite element model to real world behavior. This temperature-based structural identification has been implemented on real structures and successfully predicted damage in multiple cases (Murphy and Yarnold 2018; Yarnold and Moon 2015). One work focused specifically on thermal responses of an experimental bridge segment, and was able to assess the normal thermal behavior of the structure and detect damage in the form of change in that behavior (Kulprapha and Warnitchai 2012). Xia Qi has performed several studies on long-span suspension bridges, focusing on identifying thermal stresses, and noting that peak strains in the deck are often delayed by one hour from peak temperatures (Xia et al. 2017, 2018). SHM has realized the importance of temperature in monitoring, as almost every notable work has some mention of temperature effects on the monitoring methods.
2.3 Coefficient of Thermal Expansion of Concrete

For many materials, such as steel, the CTE has some small variation within typical seasonal temperatures (Schneider et al. 1982) but is consistent throughout the lifetime of the material. While typical design practices consider the CTE of both steel and concrete to be constant, concrete has a much more variable CTE in practice. A typical value for the CTE of hardened concrete is often assumed as 10-12 µε/°C, but several studies found values as low as 8 µε/°C (Glisic and Inaudi 2008; Ndon and Bergeson 1995). In reinforced concrete design, thermal expansion is governed by the CTE. Incorrect design with regards to the thermal expansion can lead to the development of large thermal stresses and premature cracking in concrete (Bofang 2014). The CTE of concrete depends on several factors, including the composition of the concrete mix (primarily the type of aggregate and water to cement ratio), the Relative Humidity (RH) in the material, and the age of the concrete (Lim et al. 2016; Mallela et al. 2005; Sellevold and Bjøntegaard 2006). The mix of the concrete is constant, but the RH changes during the lifespan of the concrete depending on the environmental conditions and can change the value of the CTE by 10-12% (Yeon et al. 2009). Additionally, there is some small change in the CTE of concrete with temperature and small differences in the rates of thermal expansion and thermal contraction (Islam and Tarefder 2015; Johnson and others 2005).

To measure the CTE of concrete in a laboratory setting, there are several standardized testing procedures. Most common in the United States is the AASHTO T336 testing method. This test submerges a concrete core fixed in a metal frame into a water bath and measures the change in length of the concrete for changes in temperature between 10°C and 50°C (Tanesi et al. 2013). Despite the standardized testing method, one study found
that CTE values found during the test had a standard deviation of 0.125 με/°C within a laboratory, and 0.852 between laboratories (Crawford et al. 2010). The submersion of the concrete in a water bath during testing saturates the concrete, allowing for a better agreement between tests, but distancing the results of the test procedure from the behavior of concrete in a structure, where there is a variable water content in the material. In a built structure, the construction and environment can often force the concrete into a state of non-perfect homogeneity (e.g., due to segregation of aggregate, leakage of water, etc.), which in turn may result in variability in physical properties depending on physical location on the structure. Research using ultrasonic surface waves as a form of non-destructive evaluation have shown large variations in the modulus of elasticity in concrete bridge decks (Gucunski et al. 2015; Kim Jinyoung et al. 2017). These variations in concrete properties within a structural member are difficult to predict and are often overlooked or not considered in design.

The above described challenges led to research in predicting or evaluating the CTE of concrete. Viviani et al. used fiber optic sensors to determine CTE of concrete at early and very early age in laboratory settings; however, this method cannot be applied on-site as it requires curing of concrete under very controlled conditions (Viviani et al. 2006). Zhutovsky et al. used ultrasonic pulse velocity testing to measure the CTE of early age concrete, and while successful in very early age, was not able to predict the effects of self-desiccation (Zhutovsky and Kovler 2016). Zhou et al. created a micro-mechanical model to predict the development of the concrete pore structure and water content in order to predict the CTE (Zhou et al. 2013). Another study found a theoretical model for predicting the CTE of asphalt concrete, comparing with lab tests of the actual asphalt
(Hou et al. 2016). While these models provide some accuracy, they still ignore the structural effects on concrete thermal expansion. The Federal Highway Administration (FHWA) specifies typical values for Portland cement concrete at 7.4 -13 µε/°C, while the American Association of State Highway and Transportation Officials (AASHTO) values typical concrete between 5.4 and 14.4 µε/°C. To determine the CTE of concrete FHWA recommends a laboratory test of the material using the relevant AASHTO testing method. Alternatively, the weighted average of the CTE’s of the concrete components based on their relative volumes provides an estimate of the CTE (“Index - Portland Cement Concrete Pavements Research, -” n.d.). These studies show the challenges in evaluating or predicting the CTE of concrete in a controlled laboratory setting; however, the challenges are even greater on-site as conditions cannot be controlled (especially temperature and relative humidity, but also mechanical and rheological straining), which introduces uncertainties.

2.4 Elastomeric Bridge Bearings

Elastomeric bridge bearings are designed to allow a structure to expand freely but, in practice, are not an ideal roller support. Neoprene is a widely used material, with extensive literature and design specifications available (AASHTO LRFD Bridge Design Specifications 8th Edition. 2017). Neoprene bearings are designed to have low shear stiffness to allow expansion but high axial stiffness to prevent vertical deflection at the support. They often include layered steel plates in the bearing to increase axial stiffness and decrease bulging (Aria and Akbari 2013). In many cases, the bearings offer some level of restraint to the structure, sometimes even adding a negative moment to the section and increasing the moment capacity (Bakht and Jaeger 1988; Sen and Spillett
Temperature plays a large role in the stiffness of the bearings, with changes in stiffness starting to occur near -10°C and significant changes in stiffness at -45°C (Roeder et al. 1989; Yakut and Yura 2002; Yazdani et al. 2000). Age and fatigue can also affect bearing stiffness (Cook and Allen n.d.), possibly causing debonding in the steel and neoprene layers (Belivanis et al. 2015). Nguyen and Tassoulas successfully modeled the effects of combined directions of shear on the bearings in static conditions using an ABAQUS model (Nguyen and Tassoulas 2009) while other work has focused more on the effects of loading rate on the bearings, showing that faster load rates experience a larger stiffness in the bearings (Yakut and Yura 2002).

2.5 Summary

This literature review demonstrates the need for a comprehensive monitoring strategy that is applicable to all structures, while accounting for environmental changes. Currently there exists no method to use temperature as the main input in SHM, relating to an output strain and displacement. Work related to thermal effects on structures and SHM has shown the feasibility of a temperature driven method of SHM, signifying the viability of this work.
The methods proposed in this thesis are validated using data taken from the Streicker Bridge on the campus of Princeton University from 2010 to 2017.

3.1 Overview

The Streicker Bridge was built in 2009 as a pedestrian bridge over Washington Road at Princeton University. Conceptual design is by Christian Menn and detailed design is by Theodor Zoli and Ryan Woodward. The 35-meter main span of the bridge is a deck-stiffened arch with a weathered steel arch supporting a concrete deck. The bridge has four continuous girder approaches, called ‘legs’ throughout this work. The legs and the main span of the bridge form two horizontal arches, acting as a means of lateral stiffness for the structure. A plan view in Figure 3.1 shows the curved ‘X’ shape of the bridge, while Figure 3.2 and Figure 3.3 show the main span, Southeast (SE) leg, and the junction of the Northeast and Southeast legs.

Figure 3.1 Streicker Bridge plan view ("Streicker Bridge; Google Earth" n.d.)
Figure 3.2. Southeast leg (left) and main span (right) of Streicker Bridge

Figure 3.3 Southeast and Northeast legs intersect at junction with main span

The deck is made of reinforced high-performance concrete with a design 28-day compressive strength of 42MPa. The cross section of the main span varies in depth while the legs have a constant cross-section, visible in Figure 3.4. The bridge was designed with voids in the deck, to minimize material and weight. The concrete for the deck was poured in two phases: the main span, Northeast, Northwest, and Southwest legs were poured on August 15, 2009, and the Southeast leg was poured on October 23, 2009. Despite having the same concrete mix design, the concrete in the SE leg developed differently from the concrete in the main span. The Young’s modulus at 28 days found from cylinder testing for the main span concrete was estimated using ACI318-08 8.5.1 to be 33 GPa, while the SE leg concrete had a Young’s modulus of 36 GPa (Abdel-Jaber 2017; Sigurdardottir 2015). The main span and legs are both prestressed, with
prestressing on the main span performed on August 21, 2009 and on the SE leg on November 2 & 3, 2009.

![Figure 3.4 Streicker Bridge Cross Section for main span (varies) and legs (constant)](image)

3.1.1 Southeast Leg Abutment

Each leg of the Streicker Bridge sits on neoprene expansion bearings at the abutment, shown in Figure 3.5. These bearings are designed to allow free expansion of the structure, due to thermal or other effects. These bearings have a shore ‘A’ durometer hardness between 50 and 60 as well as a corresponding shear modulus of 1.38 MPa (*AASHTO LRFD Bridge Design Specifications 8th Edition*. 2017). The bearings consist of alternating layers of steel and neoprene, shown in Figure 3.6. These steel inserts help reduce bulging in the material, increasing the axial stiffness while maintaining a low shear stiffness to allow free movement of the bridge (Belivanis et al. 2015).

![Figure 3.5 SE leg abutment (left) and close up of neoprene bearing at SE abutment (right)](image)
3.2 Monitoring System

The Streicker Bridge is equipped with a fiber optic monitoring system and has been monitored since construction in 2009 for research and educational purposes. Strain and temperature sensors are embedded in the concrete deck in half the main span and in the SE leg. This limited coverage is a result of the symmetry of the structure as well as the academic purpose of the monitoring rather than safety. Fiber optic sensors were chosen for their long-term accuracy and resistance to corrosion and electromagnetic interference. Fiber optic sensors also have the option of long gauge or distributed sensing. Figure 3.7 and Figure 3.8 show the monitoring plan for Streicker Bridge.
Sensors are arrayed in a parallel topology, where at each monitoring location one sensor is near the top of the section and the other sensor is near the bottom. This topology, shown in Figure 3.4, captures both the axial strain in the section and the curvature in the section. Sensors are installed along the centerline of the bridge at locations of maximum or minimum moment based on a cursory structural analysis. These points include mid-spans and column connections, as well as additional pairings at the connection at Pier 10, and the long span between Piers 10 and 11. Measurements are recorded every five minutes as an average temperature and strain over that five-minute period. Other projects
often require the use of the fiber optic interrogator, causing large gaps in the data despite the long-term nature of Streicker monitoring. Each strain sensor is accompanied by a temperature sensor for temperature compensation. Displacement sensors were installed at the abutment of the SE leg in 2016.

3.2.1 Fiber Bragg Gratings

The strain and temperature sensors are based on Fiber Bragg Grating (FBG) technology. An example of a FBG is shown in Figure 3.9. As light moves through the optical fiber, specific wavelengths are reflected by the gratings in the fiber. The reflected wavelength changes based on the change in strain and temperature in the gratings. Each sensor uses two FBG’s. One FBG is on a loose portion of the fiber to experience only thermal strain to assess the temperature. The second FBG is on a tensioned portion of the fiber and is sensitive to changes in temperature and changes in strain due to the strain in the material. The temperature FBG is used to compensate the tensioned FBG to read only mechanical strain in the fiber.

![Figure 3.9 FBG strain and temperature sensor (Glisic 2015)](image)

The FBG sensors on Streicker Bridge are all long-gauge sensors with a gauge length of 600mm. The temperature readings have a measurement uncertainty of 0.14°C, and the strain readings have an uncertainty of 3.1µε. Long gauge sensors measure the average strain in the material over the gauge length. Non-homogeneous materials such as concrete can have unpredictable strains at a point due to local defects in the material such as
cracks, pockets of moisture or air, or aggregate. These strains at the micro-level are not useful for monitoring or understanding structural behavior. Long gauge sensors filter out these non-homogeneities to capture the strain at the meso-level of the material. Very long gauges can lose accuracy and read values detached from a specific section. A balanced gauge length needs to be long enough to be insensitive to local non-homogeneities but short enough to still be accurate for a given section. For the Streicker Bridge, this length was determined to be 600mm.

3.2.2 Displacement Sensors
In April 2016 five displacement sensors were installed at the end of the SE leg at the abutment of Streicker Bridge, shown in Figure 3.10 and Figure 3.11. The displacement sensors measure uniaxial displacement within a range of 50 mm with a long-term accuracy of 0.5 mm. The sensors do not require thermal compensation and use two internal FBG’s to measure displacement. Two sensors measure longitudinal displacement: one on the north side of the leg and one on the south. Two sensors measure vertical displacement of the leg, one on the North side of the leg and one on the South. The final sensor records transverse displacement. Each sensor employs a universal joint between the displacement gauge and the attachment to the abutment; this joint allows for some displacement perpendicular to the gauge without effecting the displacement in the desired direction.
Figure 3.10. Displacement sensor orientations at end of SE leg

Figure 3.11 Displacement gauges installed at SE leg abutment
4 Time Periods of Minimal Thermal Gradient

The following chapter is adapted from the following published paper:


4.1 Introduction

As mentioned previously, temperature poses a challenge in SHM. Changing environmental conditions can induce similar levels of strain to traffic loading and can make the comparison between two states of a structure difficult. Uniform temperature changes cause expansion in the material, or generate large thermal stresses if not free to expand. Linear thermal gradients can cause curvature in the material in a structure, though thermal gradients are frequently non-linear due the non-instantaneous nature of heat transfer and uneven heating through sunlight or other environmental factors. Non-linear gradients can cause bending and deplanation of the cross-section, generating thermal stresses in even isostatic structures and making structural analysis difficult or inconclusive. SHM techniques cannot ignore or filter out environmental effects on a structure without excluding a substantial portion of the structure’s daily, seasonal, and yearly behavior.

Contrary to traditional SHM techniques, TD-SHM focuses on temperature as the principal driving force in structural behavior and analysis, treating generalized temperature changes as an input affecting a generalized strain and generalized displacement output. The term “generalized temperature” encompasses temperature
change at a point and any algebraic combination of temperature changes at different points (e.g., linear thermal gradient in a cross-section). Similarly, the term “generalized strain” encompasses strain value at a point and any algebraic combination of strain values at different points (e.g., curvature in cross-section). Finally, “generalized displacement” encompasses displacement at a point, rotation at a point, and algebraic combinations of these parameters. To simplify presentation in further text, the word “generalized” will be omitted and used only when needed to clarify the subject. These three measurable quantities—temperature, strain, and displacement—will form three-dimensional signatures for different parts of the structure with the expectation of sensitivity to changes in structural behavior (Yarnold 2013; Yarnold and Moon 2015). An important challenge in TD-SHM is the fact that temperature change in one part of the structure can generate mechanical strain and displacement in another part of the structure, and thus the latter can falsely be interpreted as unusual structural behavior (e.g., damage). Numerical modelling based on all possible thermal inputs is impractical. A complete model needs to include the thermo-mechanical properties of the structure, the large scale and geometrical complexity of the structure, as well as site-specific environmental effects such as partial shadow due to trees, variable proximity to land or water, etc. By deriving and analyzing the expressions of structural behavior under thermal actions, it is proven in this thesis that an appropriate approach could be to use only the data sets collected during the times when thermal gradients in the structure were minimal. Thus, this work seeks to address the above challenge by identifying time points when the thermal gradients on a monitored structure are minimal and use data collected at these time points to analyze structural behavior. Two simple yet practical classes of methods are proposed, showing that they
identify significantly different sets of time points with different values characterizing parameters describing structural behavior (i.e., three-dimensional signatures). This is an important insight that demonstrates the complexity of the presented problem and calls for future research. To achieve TD-SHM, first the influence of thermal gradients has to be minimized.

Contemporary research on thermal gradient influences on structures has produced a variety of results. Few works are cited here to emphasize the challenge of interpretation of data collected while thermal gradients were present on the structure. Hedegaard et al. created a two-dimension finite element mesh and were able to tailor the model to predict structural response to thermal gradients based on temperature and strain readings from the I-35W St. Anthony Falls Bridge (Hedegaard et al. 2013). Despite this ability to create an accurate model, Hedegaard and other researchers noted that the thermal response of a structure is highly variable for each bridge due to dependence on climate, location, orientation, and the properties of the structure (Hedegaard et al. 2013; Potgieter and Gamble 1989). Non-linear thermal gradients can induce stress in even cases of free expansion or rotation, contrary to normal assumptions (Emanuel and Hulsey 1978). These non-linear thermal gradients can be difficult to clearly relate to structural response. For example, Kromanis employed a data-driven strategy (as opposed to model-based) on distributed temperature measurements to accurately predict the response of the structure after a set amount of training data (Kromanis and Kripakaran 2014b).

For the purposes of this work, thermal effects to the beam-like structure, its parts, or its cross-sections could be defined in terms of global thermal change and local thermal gradients (see details in the next section). Thermal gradients in a structure are time-
dependent, occurring while heat enters or leaves the material. Due to the non-instantaneous nature of heat transfer, depending on the thermal inertia of the material, thermal gradients are frequently non-linear. These non-linear gradients can cause non-linear thermal strain distributions, which can result in bending and non-linear stress and strain distribution within the cross-sections and obscure the effects of thermal change and make data analysis complex and in many cases even inconclusive (Abdel-Jaber and Glisic 2018; Emanuel and Hulsey 1978; J. Reilly et al. 2017). Thus, identifying time periods of minimal thermal gradients will allow for the establishment of a relationship (signature) between the temperature change and resulting strains and displacement, and enable direct comparison between the relationships occurring at different times. Hence, this chapter focuses on an initial component of TD-SHM: the creation of methods for the identification of time periods of minimal thermal gradients in a structure.

The main challenge in the creation of these methods is that due to the large scale of many structures and the varying exposure to temperature, real structures are practically never in a state of zero thermal gradients. Thus, a certain level of thermal gradients has to be tolerated, which raises a trade-off challenge. If a large range of thermal gradients is tolerated, a substantial amount of data will fall into that range, but the uncertainty in data analysis will be significant. On the other hand, if a small range of thermal gradients is tolerated, the data that falls into that range might be very small and insufficient for effective data analysis. While several methods were considered in the course of this research, such as the standard deviation of temperatures and median local gradient (J. Reilly et al. 2016), the two approaches that were identified as most promising are presented in this chapter: evaluating the range of temperature measurements on the
structure and the distribution of local gradients on the structure. Their performance is evaluated using data from a real structure, the Streicker Bridge, on the Princeton University campus.

4.2 Temperature Distribution and TD-SHM
For any temperature distribution within a beam-like structure under linear-theory assumptions, there is a unique deformation and displacement distribution. Thus, theoretically, if a very dense network of temperature, strain, and displacement sensors were applied to a structure, it would be possible to accurately determine the relationship between (generalized) temperature, strain, and displacement, and create their three-dimensional diagrams, i.e., TD-SHM signatures.

However, in practical applications the available sensor networks might not be sufficiently dense. Frequently, few sensors are installed within the cross-section of a beam member, and only a few cross-sections are instrumented along the beam member. A typical example is given in Figure 4.1, where a chain of parallel strain and temperature sensors is installed on the structure, and displacement sensors are installed only at the expansion bearing at Pier 13 (P13 denotes the bridge pier or supports).
In Figure 4.1, each instrumented cross-section contains two sensors, one at the top and the other at the bottom of the cross-section, with approximately five meters between instrumented cross-sections. This limitation in sensor coverage results in increased uncertainty in determining TD-SHM signatures. This research aims to reduce the uncertainty by creating methods to conveniently choose data to be used in analysis; namely, choosing time periods of minimal thermal gradient on the structure in order to increase the linearity in the temperature–strain–displacement relationship and thus reduce the uncertainty in the TD-SHM signatures. In the following text, planar structures are considered for simplicity of presentation. The planar case can be expanded to the spatial case by adjusting equations to account for bending moments and thermal gradients in the horizontal transverse direction.

Thermal gradients in a beam-like structure induce curvatures in cross-sections and non-uniform elongation along the length and may or may not generate stresses depending on structural system and boundary conditions. The largest effects from these thermal gradients appear from vertical thermal gradients acting across the depth of the cross-section of a beam. In the case of steady temperature conditions, the thermal gradients are linear. Linear temperature distributions in a cross-section can be decomposed into temperature change and vertical thermal gradient, as shown in Figure 4.2.
The linear temperature distribution shown in Figure 4.2 deconstructs into the combination of a uniform temperature change ($\Delta T_{CS}$), corresponding to temperature change at the centroid of stiffness of the cross-section, and a linear thermal gradient, shown in Equation 4.1:

$$G_{\Delta T} = (\Delta T_{bottom} - \Delta T_{top})/h$$

(4.1)

Linear thermal gradient ($G_{\Delta T}$) is equal to the difference between temperature variations at the bottom and the top of the cross-section divided by the depth of the cross-section $h$.

Linear temperature distributions induce elongation along the beam and curvature in cross-sections, but not deplanations of the cross-section. Assuming that a linear temperature distribution $\Delta T(x,y)$ is present on a beam-like structure, the change in strain $\Delta \varepsilon_{total}(x,y)$ at a point with coordinates $x$ along the beam and $y$ from the centroid of the section, due to the linear temperature distribution, can be expressed in Equation 4.2 as follows for a plane-stress case:

$$\Delta \varepsilon_{total}(x,y) = \Delta \varepsilon_{mech.}(x,y) + \Delta \varepsilon_T(x,y) = \frac{N_{\Delta T}(x)}{EA} + \frac{M_{\Delta T}(x)}{EI}y + \alpha_T \Delta T_{CS}(x) + \alpha_y G_{\Delta T}(x)y$$

(4.2)

where $\Delta \varepsilon_{mech.}(x,y)$, $N_{\Delta T}$, and $M_{\Delta T}$ represent thermally generated mechanical strain, normal force, and bending moment, respectively; $\alpha_T$ and $E$ represent the thermal expansion coefficient and Young’s modulus of the material; $A$ and $I$ represent area and moment of
inertia of the cross-section; and $\Delta T_{CS}$ and $G_{\Delta T}$ represent temperature change at the centroid of stiffness and thermal gradient in the cross-section with coordinate $x$.

Note that $N_{\Delta T}$ and $M_{\Delta T}$ in a cross-section depend not only on $\Delta T_{CS}$ and $G_{\Delta T}$ at that cross-section but also on $\Delta T_{CS}$ and $G_{\Delta T}$ at other cross-sections, the structural system, and boundary conditions. Thus, the relationship between temperature and strain at a point is not linear in the general case. However, if no longitudinal and transverse (vertical) thermal gradients are present, i.e., $\Delta T_{CS}(x) = \Delta T = \text{constant}$ and $G_{\Delta T}(x) = 0$, then the relationship between strain $\Delta \varepsilon_{\text{total}}(x,y)$ and uniform temperature change in the structure $\Delta T$ becomes linear. In that case, the coefficient of linearity depends on the thermal expansion coefficient $\alpha_T$, the Young’s modulus $E$, the structural system (boundary conditions), and the geometrical properties of the structure. Similar statements can be derived for any generalized deformation parameter (e.g., curvature).

As shown in Equation 4.3, the change in generalized displacement $\Delta \delta(x_0)$ at a point with coordinate $(x_0)$ along the structure can be determined using the principle of virtual work by applying the corresponding generalized unit force, determining the virtual normal force and virtual bending moment distributions, and applying the formula (influence of shear force is neglected):

$$
\Delta \delta(x_0) = \int_l \frac{N^v(x)N_{\Delta T}(x)}{EA(x)} \, dx + \int_l \frac{M^v(x)M_{\Delta T}(x)}{EI(x)} \, dx + \int_l N^v(x)\alpha_T \Delta T_{CS}(x) \, dx + \int_l M^v(x)\alpha_T G_{\Delta T}(x) \, dx
$$

(4.3)

where the superscript $v$ indicates influences due to virtual unit generalized force, and $l$ indicates integration over all beams in the structure.
Note that in structures consisting of several beams, $N^r$ is constant along each beam, and $M^r$ is linear or bi-linear along each beam. Analysis similar to the strain–temperature relationship demonstrates that in the absence of gradients the relationship between generalized displacement $\Delta \delta(x_t)$ and uniform temperature change in the structure $\Delta T_{CS}$ becomes linear. Based on the above considerations, the relationship between uniform temperature change, generalized strain, and generalized displacement is linear, and the graph of this relationship, i.e., its TD-SHM signature, is a segment of a line or planar figure (i.e., three-dimensional linear).

In reality, temperature distributions are non-linear (causing non-linear gradients) because of the non-instantaneous nature of the heat transfer through the material of the section. This is a consequence of thermal inertia and low diffusivity, particularly in the case of concrete. In addition to the global non-linearity of TD-SHM signatures introduced by linear thermal gradients (see Equation (4.2)), non-linear thermal gradients cause non-linear strain distributions within the cross-section and additional internal stresses (Kong et al. 2014). This is schematically shown in Figure 4.3.

![Figure 4.3 Example of a bridge cross-section with non-linear temperature distribution (causing non-linear thermal gradient).](image)

This non-linear thermal gradient causes an error when creating TD-SHM signatures. The error is directly correlated with the amplitude of non-linearity $\Delta T_{n-l}(y)$ (see Figure 4.3),
which is in turn correlated to the difference in temperature between the top and bottom of the cross-section and to whether the bridge is cooling or heating (AASHTO LRFD Bridge Design Specifications 8th Edition. 2017; Comité Européen de Normalisation 2010). An example TD-SHM signature created for Streicker Bridge (see Figure 4.1) using a full set of data collected in Spring 2016, i.e., containing some data resulting from non-linear temperature distribution, is given in Figure 4.4. The figure shows a signature using strain and temperature at the centroid of the section located at Pier 12 and displacement readings taken at the abutment of the SE leg (see Figure 4.1). This signature is used as an example in this text to validate the methods of identifying time periods of minimal thermal gradients. Final formulation of TD-SHM will evaluate three-dimensional signatures at multiple locations in the structure. The signature is shown from different angles to emphasize its non-linear shape and noise related to non-linear temperature gradients.

Figure 4.4 Example of a three-dimensional signature of temperature, strain, and longitudinal displacement with best fit line, taking into account data resulting from non-linear temperature distributions.
Figure 4.4 shows the three-dimensional signature of temperature, strain, and longitudinal displacement, along with best fit line, and Figure 4.5 shows two projections of the signature to help present the three-dimensional shape. While there is a definite trend and linearity to the signature, it is obscured by noise due to large, non-linear thermal gradients in the structure. There is also some bi-linear behavior, seen in Figure 4.5 in the temperature and strain plot. This bi-linearity could be caused by thermal gradients in the structure, or by some other, currently unknown effect. The best fit line provides a method to compare the effectiveness of the different minimum gradient filters by examining the vector \( \mathbf{n} \), intersection point \( A \) of the line, the coefficient of determination \( R^2 \), and the standard error \( \sigma \) of the best fit line. The vector of the best fit line and intersection point provide a description of the linear relationship between temperature, strain, as well as displacement and are also sensitive changes in structural behavior. Higher \( R^2 \) and lower \( \sigma \) would show a more accurate estimation of this behavior, increasing the expected ability to determine smaller changes in this structural behavior (Yarnold 2013; Yarnold and Moon 2015). The vector of the best fit line of the entire data set (including non-linear thermal gradients) in Figure 4.4, is \( \mathbf{n} = [0.084, 0.990, 0.126] \), relating to [displacement, strain, temperature]. This vector describes the linear relationship between the three parameters: for every 0.126 °C change in temperature, there is a corresponding 0.99 µε change in strain and 0.084 mm change in displacement. Boundary conditions, geometry of the structure, and material restraints all have a strong influence on these three-dimensional signatures; the linear relationship identified captures these effects in the vector of the line. The \( R^2 \) value of the best fit line in Figure 4.4 is 0.989, and the standard error is 1.351. To simplify presentation, the intersection point \( A \) of the best fit line is
not analyzed (the focus of this chapter is minimization of thermal gradients and thus further development of TD-SHM is discussed in Chapter 6).

Figure 4.5 2D projections of three-dimensional signature, keeping temperature as the vertical axis.

A change in structural behavior is expected to result in a change in the vector \( \mathbf{n} \) (and intersection point A); however, this change can be unnoticed if the standard error is big or vector \( \mathbf{n} \) is not well defined. For example, it is unclear whether the bi-linear signature in Figure 4.4 and Figure 4.5 indicates damage, or is simply the result of non-linear thermal gradients, and thus should be discarded. Identifying time periods of minimal thermal gradient in the structure is thus expected to provide the data that shapes TD-SHM signatures into linear or planar figures in three-dimensional space and at the same time minimize the possible error (from non-linear thermal gradients) in identification of unusual structural behaviors.

4.3 Methods for Determination of Minimal Thermal Gradients

Using the data when an approximately uniform temperature is present across the structure will minimize the influence of non-linear thermal gradients, allowing for planar TD-SHM
signatures. However, moments in time when the temperature is approximately uniform across the bridge are scarce. As an illustration, Figure 4.6 shows temperature fluctuations at the top and bottom of two cross-sections of the Streicker Bridge (see Figure 4.1) over two days.

![Figure 4.6 Example of temperature variation over two days in two cross-sections of Streicker Bridge (top and bottom sensors in cross-sections at P11 and P12).](image)

Figure 4.6 shows the temperatures recording at the top and bottom of two cross-sections (P11 and P12, see Figure 4.1) on the Streicker Bridge over two days. While each cross-section individually experiences uniform temperatures at two instances per day (red circle corresponds to times with minimal thermal gradient at P11, and blue for P12), Figure 4.6 shows that there is no moment in time over the given two days when the temperature was perfectly uniform at both observed cross-sections. The sections come closest to achieving zero thermal gradient at the end of the second day (green circle) but still do not reach a uniform temperature. This difficulty in achieving zero thermal gradient over the two cross-sections in the bridge highlights the improbability of the entire bridge having a uniform temperature. These moments in time are extremely scarce and significantly
reduce the data available for creating TD-SHM signatures. Because of the unlikelihood of finding an absolutely uniform temperature across an entire bridge, some level of non-uniformity must be tolerated. Higher levels of non-uniformity imply more data available for analysis but also more error in the analysis, and thus an acceptable balance between these two opposite objectives must be found. In this chapter, two classes of methods are proposed to identify time periods of minimal thermal gradient: one using the range of temperature measurements on the structure and the other using the distribution of local thermal gradients. The methods are presented in detail in the next subsections. Their advantages and limitations are discussed.

4.3.1 Maximum Range

The Maximum Range (MR) method stipulates that the total range of temperatures on the structure at an observed moment in time needs to be below some chosen value for the MR, as shown in Equation (4.3). $T_{\text{Max}}$ and $T_{\text{Min}}$ represent the maximum and minimum temperature on the structure with all sensor measurements considered.

$$T_{\text{Max}} - T_{\text{Min}} < \text{MR}$$

(4.3)

Choosing the value of the MR needs to balance the amount of time points identified (larger MR needed) while still minimizing the tolerance of thermal gradients (lower MR needed).

An important challenge of the MR method is its susceptibility to a single or few outlying temperature measurements. If the entire structure rests at a uniform temperature (within MR) with the exception of one or few locations outside the MR, then the structure would not be identified as having a minimal thermal gradient. However, given that the remainder of the structure is within the MR, these outlying temperatures cannot be
excessively far from MR under usual daily temperature variations. For example, if most of the temperatures are within MR = 5 °C, then it is not expected for outlying temperature at one point to be out of range for much larger value, e.g., 10 °C. Thus including them in the analysis would not result in an excessive error. In addition, based on Equations (4.1) and (4.2), if one or few locations have temperatures different from the remainder of the bridge, this difference will influence local behavior around these points rather than global behavior, which in turn will result in a small error in distant TD-SHM signatures. Hence, to address the challenge of small data sets with truly uniform temperature, a modified MR method is created where there is a tolerance of certain percentage (e.g., 5%) of outliers outside of the MR. The acceptable value of MR and the percentage of acceptable outliers depend on the structural system, the scale of the structure, and the number of sensors installed on the structure. These values can be determined based on a sensitivity study as shown in the next section with results.

4.3.2 Local Gradient

The local gradient (LG) methods examines the local thermal gradients (thermal gradients through the cross-section) at several cross-sections. For this application (Streicker Bridge with horizontal deck), vertical thermal gradients through the cross sections are considered as they have the largest effect on the structure, though other structures may have different dominant directions of gradient in the section. The local vertical thermal gradient in a cross section \( G_{\Delta_T}(x) \) is defined in Equation (4.4), where \( h \) is the distance between the measurements \( \Delta T_{\text{bot}} \) and \( \Delta T_{\text{top}} \).

\[
\frac{\Delta T_{\text{bot}} - \Delta T_{\text{top}}}{h} = G_{\Delta_T}(x) \tag{4.4}
\]
Two methods were identified using this metric. First the Maximum Local Gradient (MxLG) method identifies time periods where every cross-section on the structure has an absolute local gradient under some chosen MxLG bound. If the absolute value of every local thermal gradient on the structure is less than the chosen MxLG, then the temperature state of the structure is deemed suitable for TD-SHM method. This method is fundamentally different from MR. Beyond using gradients rather than raw temperatures, the MxLG method only compares each location on the bridge to its corresponding sensor at its cross-section only (e.g., bottom sensor with top sensor) and not with every other sensor. Thus, while the temperature gradient is minimized across the cross-section, there could be some temperature change along the centroid line of the structure, i.e., $\Delta T_{CS}(x) \neq \text{constant}$.

Next, the Mean Local Gradient (MeLG) method identifies time periods where the mean absolute value of all local thermal gradients on the structure is under some chosen bound. A mean local gradient closer to zero directly corresponds to the minimization of all cross-sectional thermal gradients on the structure. This method is affected by each local gradient on the structure while MxLG is only concerned by the individual maximum local gradients on the structure.

Minimization of local gradients, i.e., $G_{\Delta T}(x) \approx 0$, removes the last terms from Equations (4.1) and (4.2). In addition, it minimizes the values of $N_{\Delta T}$ and $M_{\Delta T}$ since they are result of only axial thermal elongations of the beams (no thermal curvature due to gradients). Thermal elongations are in turn a consequence of temperature change along the beam, and non-linear effects will arise only if the change is abrupt at some points, which is not expected to happen (similar comment applies as in case of MR). Thus, the TD-SHM
signature is expected to be only slightly “distorted” from the linearity. Both MxLG and MeLG parameters can be evaluated based on a sensitivity study as shown in the next section.

4.3.3 Process for Identifying Time Periods of Minimal Thermal Gradient

The entire process of identifying time periods of minimal thermal gradient is shown in Figure 4.7

![Flowchart](image)

Figure 4.7 Flowchart describing process of using methods for identifying time periods of minimal thermal gradient.

Starting with the total temperature data set, the metric and method for identifying minimal thermal gradients needs to be chosen. This choice can depend on the individual monitoring system and the monitored structure. Next, the chosen method is applied to the data set using a variety of bounds. It can be difficult to predict the bounds that will maintain the minimization of thermal gradients, while producing a large enough basis of
time points. To choose a bound and validate the method, the three-dimensional signature of the total data set is compared to the three-dimensional signature of the time points of minimal thermal gradient. If there is an improvement in the linearity (coefficient of determination and standard error) of the three-dimensional signature, the method is deemed valid. The final bound is chosen based on the stabilization of the vector of the best fit line or plane, as discussed later in Section 4.4. If the three-dimensional signature does not improve with the application of the chosen method, a different metric or method can be chosen to repeat the process.

4.4 Evaluation of Methods

Data collected from the Streicker Bridge is used for exploration and assessment of the two classes of methods proposed in previous section. Streicker Bridge is described in detail in Chapter 3, but a simplified monitoring plan is shown in Figure 4.1. Temperature and strain sensors are arranged in a parallel topology at critical cross sections: column beam intersections and mid spans. This topology with one sensor at the top and one at the bottom of the sections captures the largest change in strain in the sections, while providing information on the thermal gradients and curvature in the section. Having more sensors across the cross-section would be more beneficial for full understanding of thermal behavior; however, economic limitations imposed the installation of only two sensors per cross-section. Additional displacement sensors were installed in 2016 at the abutment of the SE leg. The instrumented length of the bridge is approximately 58 m and uncertainty in temperature monitoring is estimated to be ±0.1.4 °C (Sigurdardottir and Glisic 2013). Sensor drift is a concern with all long-term monitoring projects and could significantly affect the results of this work. Sensor drift due to manufacturing error has
been previously identified and addressed in earlier work on the Streicker Bridge (Abdel-Jaber and Glisic 2016). Temperature changes are influenced by a variety of environmental factors: ambient weather and exposure to wind, rain, snow, and sun. Strain changes are primarily influenced by temperature (through thermal expansion and thermally generated mechanical strain). The structure was built in 2009 and is beyond the majority of rheological strains (creep and shrinkage are mostly stabilized). The influence of transient mechanical loads is minimized by recording strain as the average value of ten measurements taken every five minutes (J. Reilly et al. 2017). Each measurement session included all sensors. More detailed information on the Streicker Bridge and monitoring system can be found in (Sigurdardottir 2015).

Both presented classes of methods MR and LG demonstrated the capability of identifying time periods of minimal thermal gradients on the Streicker Bridge, but they are different in their results as shown in the following subsections.

4.4.1 Evaluation of Maximum Range Method

Figure 4.8 shows the temperature ranges recorded from 18 April to 15 June 2016 on the Streicker Bridge with temperature sensors in the SE leg.
Each point in Figure 4.8 represents the range of temperatures within each time point of measurement taken over twenty sensors in the concrete deck of the SE leg. A lower range in temperature indicates a more uniform temperature across the bridge. The minimum range shown in the figure is approximately 2.9 °C and the maximum is 16.9 °C. While the lowest ranges were under 3.0 °C, taking this value as the MR bound would not be useful—a very small number of time points would be found (see Table 4.1). Too small a number of points will skew the three-dimensional signature, allowing noise or random effects to dominate rather than the thermal behavior of the structure and preventing any definitive estimation for the vector of the best fit line. Thus, a parametric study was made to establish the MR value which would be optimal, i.e., as small as possible to identify state of the bridge with temperature as close to uniform as possible yet providing sufficiently large data sets for TD-SHM analysis. Table 4.1 summarizes the number of
time points identified based on the given range of temperature measurements on the
structure during Spring 2016 (see Figure 4.8).

Table 4.1 Number of time periods identified by Maximum Range (MR).

<table>
<thead>
<tr>
<th>Temperature Range (°C)</th>
<th>2.5</th>
<th>3</th>
<th>3.5</th>
<th>4</th>
<th>4.5</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td># Time Points in MR</td>
<td>0</td>
<td>15</td>
<td>438</td>
<td>2,748</td>
<td>5,967</td>
<td>8,616</td>
</tr>
<tr>
<td># Time Points 95% in MR</td>
<td>0</td>
<td>152</td>
<td>2,773</td>
<td>6,703</td>
<td>9,779</td>
<td>11,341</td>
</tr>
<tr>
<td># Time Points 90% in MR</td>
<td>4</td>
<td>1,576</td>
<td>6,989</td>
<td>10,104</td>
<td>11,519</td>
<td>12,228</td>
</tr>
</tbody>
</table>

The Spring 2016 data set contains 16,641 total time points of temperature distributions. MR95 seeks to retain 95% of the sensors within the given range, while MR90 retains 90%. The amount of outliers to allow was chosen based on the number of sensors available (95% excludes one out of 20 sensors, while 90% excludes two out of 20 sensors in Streicker Bridge). Table 4.1 shows that in many cases the range of temperatures is dominated by two or fewer outliers (10% of sensors or less), as allowing these outliers greatly increases the number of time points for each bound. An interesting pattern arising from the data set shown in Table 4.1 is that an increase of MR for 0.5 °C adds a similar number of time points as adding a tolerance of 5% of outliers. For example, starting at a MR of 3.5 °C (438 time points) and either increasing the range by 0.5°C (MR = 4 °C, 2748 time points) or adding a 5% tolerance of outliers (MR95 = 3.5 °C, 2773 time points) results in a similar number of time points of minimal thermal gradient. To illustrate the effects of increasing the tolerance of outliers, Figure 4.9 shows the three levels of outlier tolerance for two weeks in the spring of 2016.
Figure 4.9 Points considered as having minimal gradients, observed using MR methods.

Figure 4.9 shows the temperature sensors at Pier 12 at the top and bottom of the cross-section as a reference while highlighting the time points identified as having minimal thermal gradient. Only two weeks of data are shown in order to illustrate the effect of allowing outliers with the MR method, while Table 4.1 refers to the entire data set stretching from 18 April to 15 June 2016. Each point with a black “X” shows a time point where the range of all sensors (not just the two sensors shown above) is below 3 °C. The MR method prioritizes the entire structure over any individual section, as can be seen above on 16 May in the black circle. There is a time period with very small thermal gradients at Pier 12, but that time period has no time points identified as having minimal thermal gradient due to a more global non-minimized state of the thermal gradients on the structure. Allowing 5% outliers (MR95) will keep all of the time points without outliers (MR) but identify a larger set of time points with minimal thermal gradient. This tradeoff between identifying a larger set of time points with minimal gradients and compromising...
the minimization of the thermal gradients can be seen in the best fit line of the three-dimensional signature for the structure shown in Figure 4.10.

Figure 4.10 Three-dimensional signature example with best fit line (BFL) for time points with minimal thermal gradient, identified using MR method.

Figure 4.11 2D projections of three-dimensional signature, keeping temperature as the vertical axis. Red indicates time points of minimal thermal gradient found using MR = 4 °C/m, and red line indicates best fit line for minimal gradient data set, as in Figure 4.10.

Figure 4.10 shows the same three-dimensional signature in Figure 4.4 but with time points of minimal thermal gradient in red. For clarity, Figure 4.11 shows two 2D representations of the same three-dimensional signature as in Figure 4.10. Only one
method is shown in Figure 4.10, but Table 4.2 summarizes some statistics of the best fit lines for the different methods including the vector of the best fit line ($\mathbf{n}$), the coefficient of determination ($R^2$), and the standard error ($\sigma$). The vector of the line describes the relationship between temperature, strain, and displacement, and this vector is expected to behave as a damage sensitive feature (in addition to the coefficient of determination and the standard error). Thus, an accurate evaluation of the vector $\mathbf{n}$ would enable sensitive detection of unusual behaviors. Results in Table 4.2 lead to several conclusions:

1. Each metric above shows a better linear fit in terms of coefficient of determination and standard error than the total data set (without minimum gradient filters), showing an improvement in the accuracy of the best fit line.

2. The first component of the vector $\mathbf{n}$ is significantly different for MR with bound 3.0 °C (0.097) from any other MR bound shown in Table 4.2; this shows that there is an insufficient number of points used in calculating the vector $\mathbf{n}$ (only 15 points).

3. For all other methods, the first component of the vector is a stable value at 0.080 ± 0.001 using the minimal gradient filters, the second component is very stable at 0.989, and the third component has some variation around 0.127 ± 0.003. These values for the vector of the best fit line fluctuate when there is a low number of time points forming the signature but stabilize as the number of time points identified increases. This consistency in the vector can be seen in MR with bounds 4.0 °C and 4.5 °C where the values are the same for the second and third component and 1% different in the first component, making confidence that any significant variation in any component of vector $\mathbf{n}$ from these values can be interpreted as unusual structural behavior in TD-SHM.
4. The stabilized vector of the best fit lines for minimal thermal gradient discussed in Point 3 (0.080, 0.989, 0.127) shows a more restrained section than with the vector of the total data set (0.084, 0.990, 0.126). For a larger change in temperature (0.127 °C), there are smaller resulting changes in strain (0.989 με) and displacement (0.080 mm). This systematic difference, especially in the first component, of the vector of the best fit line shows that filtering out thermal gradients from the structure identifies a difference in the temperature–strain–displacement relationship.

The MR 3.0 °C bound only identified 15 time points of minimal thermal gradient; so few points compared to the other metrics results in a less reliable best fit line. Allowing some outliers (MR95 and MR90) greatly increases the number of time points identified but does not significantly reduce the quality of the best fit line. The standard error increases with the allowance of some outliers, but the $R^2$ increases slightly. At the 3.5 °C bound, again the fit does not deteriorate with the allowance of outliers when comparing MR to MR95. This provides validation for the allowance of some small amount of outliers when identifying time periods of minimal thermal gradient. The exact number of points needed to provide an accurate description of the temperature–strain–displacement relationship will depend on the structure and specific application of the minimal gradient filters but can be identified by employing a range of methods (as above) and looking for a stabilization of the vector of the best fit line while keeping standard error low and coefficient of coefficient of determination high.
### Table 4.2 Best fit line statistics for MR method.

<table>
<thead>
<tr>
<th>Bound</th>
<th>Metric</th>
<th>n</th>
<th>$R^2$</th>
<th>$\sigma$</th>
<th># Points</th>
</tr>
</thead>
<tbody>
<tr>
<td>MR</td>
<td>3.0 °C</td>
<td>0.097 0.982 0.164</td>
<td>0.993</td>
<td>0.546</td>
<td>15</td>
</tr>
<tr>
<td>MR95</td>
<td>3.0 °C</td>
<td>0.079 0.989 0.123</td>
<td>0.994</td>
<td>0.810</td>
<td>152</td>
</tr>
<tr>
<td>MR90</td>
<td>3.0 °C</td>
<td>0.081 0.989 0.126</td>
<td>0.995</td>
<td>0.816</td>
<td>1576</td>
</tr>
<tr>
<td>MR</td>
<td>3.5 °C</td>
<td>0.079 0.989 0.128</td>
<td>0.993</td>
<td>1.003</td>
<td>438</td>
</tr>
<tr>
<td>MR95</td>
<td>3.5 °C</td>
<td>0.080 0.989 0.125</td>
<td>0.993</td>
<td>0.965</td>
<td>2773</td>
</tr>
<tr>
<td>MR</td>
<td>4.0 °C</td>
<td>0.081 0.988 0.130</td>
<td>0.994</td>
<td>0.968</td>
<td>2748</td>
</tr>
<tr>
<td>MR95</td>
<td>4.0 °C</td>
<td>0.082 0.988 0.130</td>
<td>0.996</td>
<td>1.042</td>
<td>5967</td>
</tr>
<tr>
<td>MR</td>
<td>4.5 °C</td>
<td>0.082 0.988 0.130</td>
<td>0.996</td>
<td>1.042</td>
<td>5967</td>
</tr>
<tr>
<td>Total Set</td>
<td></td>
<td><strong>0.084 0.990 0.126</strong></td>
<td><strong>0.989</strong></td>
<td><strong>1.351</strong></td>
<td><strong>16641</strong></td>
</tr>
</tbody>
</table>

#### 4.4.2 Evaluation of Local Gradient Methods

Figure 4.12 shows the maximum vs. mean absolute local gradient found at each time step of recorded data in the spring of 2016. Each point in Figure 4.12 represents a single time point during the spring of 2016. The two methods identify different sets of time points—time points with a maximum gradient on the bridge under 10 °C/m could have a mean gradient from near 3.5 °C/m to more than 6 °C/m. To illustrate the magnitude of the thermal gradients in Streicker Bridge, a difference between the bottom and top sensors of −5 °C (if top is hotter, difference is negative) divided by a distance between sensors of approximately 350 mm would give a local thermal gradient of approximately 14 °C/m. Figure 4.12 shows that there is frequently at least this level of gradient on the bridge.
Table 4.3 summarizes the number of time points identified for each level of MxLG and MeLG.

<table>
<thead>
<tr>
<th>Maximum Local Gradient (°C/m)</th>
<th>7.00</th>
<th>7.50</th>
<th>8.00</th>
<th>8.50</th>
<th>9.00</th>
<th>9.50</th>
<th>10.00</th>
</tr>
</thead>
<tbody>
<tr>
<td># Time Points under MxLG</td>
<td>1</td>
<td>7</td>
<td>112</td>
<td>324</td>
<td>890</td>
<td>1,919</td>
<td>3,176</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Mean Local Gradient (°C/m)</th>
<th>3.50</th>
<th>3.75</th>
<th>4.00</th>
<th>4.25</th>
<th>4.50</th>
<th>4.75</th>
<th>5.00</th>
</tr>
</thead>
<tbody>
<tr>
<td># Time Points under MeLG</td>
<td>0</td>
<td>26</td>
<td>380</td>
<td>1,516</td>
<td>2,871</td>
<td>4,377</td>
<td>5,844</td>
</tr>
</tbody>
</table>

The two local gradient methods cannot be compared by their bounds, as they look at different metrics of the local gradients. Table 4.3 gives a sense of the number of time points identified by different bounds for each method. A bound of 4.5 °C/m MeLG identifies a similar number of time points as a bound of 10 MxLG (2,871 to 3,176). Table 4.3 shows the persistence of thermal gradients on the structure. Out of the entire data set...
of 16,641 time points of measurement, there was only one time point where the maximum local gradient was under 7 °C/m, and there were no time points when the mean local gradient was under 3.5 °C/m. Figure 4.13 shows the same three-dimensional signature as in Figure 4.4, with time points of mean local gradient under 4.5 °C/m in red and the corresponding best fit line (BFL).

Figure 4.13 Three-dimensional signature with best fit line (BFL) for time points with minimal thermal gradient, identified using MeLG method.

The following conclusions can be carried out from the results shown in Table 4.4:

1. Each method above shows a better linear fit in terms of coefficient of determination and standard error than the total data set (without minimum gradient filters), for a number of points close to 1000 (i.e., 890) or more. This emphasizes the need of having a relatively large number of points to use, in which case there is an improvement in the accuracy of the best fit line.
2. The first component of the vector \( \mathbf{n} \) is significantly different for MxLG with bound 8.0 °C/m (0.078) from any other MxLG bound shown in Table 4.3; this shows that there is an insufficient number of points used in calculating the vector \( \mathbf{n} \) (only 112 points).

3. For all other MxLG methods, the first component of the vector is a stable value at 0.083–0.084, the second component is very stable at 0.988, and the third component is also stable at 0.132 ± 0.001. These values are very stable and provide confidence that any significant variation in any component of vector \( \mathbf{n} \) from these values can be interpreted as unusual structural behavior in TD-SHM.

4. The value of the third component of vector \( \mathbf{n} \) as determined from the entire data set is about 5% lower than if calculated using MxLG (0.126 vs. 0.132). Comparing the vector of MxLG with bound 9.5 °C/m to the total data set, a larger change in temperature (0.132 to 0.126) would result in a smaller change in strain (0.988 and 0.990) and the same change in displacement. This systematic difference in the third component of the vector of the best fit line shows that filtering out thermal gradients from the structure identifies a difference in the temperature–strain–displacement relationship.

5. The third component of the vector \( \mathbf{n} \) is significantly different for MeLG with bound 4.00 °C/m (0.108) from every other MeLG bound shown in Table 4.4; this is to a lesser extent true for the first component too. These differences show that there is an insufficient number of points used in calculating the vector \( \mathbf{n} \) using MeLG with bound 4.00 °C (380 points).

6. For MeLG with bounds 4.25 °C/m to 5.00 °C/m, the vector \( \mathbf{n} \) of the BFL does not reach a consistent set of values. The first component has an upward trend from 0.077 to 0.080, increasing with each increase in bound, as does the third component. The vector begins
to approach the vector of the total data set, showing that the larger bounds do not filter out thermal gradient effects.

8. For MeLG with bound 4.50 °C/m, the first component of the vector is approximately 8% lower from the same component from entire data set (0.078 vs. 0.084). The third component is also lower (for about 5%), but the second component is the exact same. This result is interesting, as the MeLG method requires a lower change in temperature to produce the same strain as from the normal vector. This difference highlights the difference in behavior identified by removing the influence of thermal gradients.

One strong result can be seen in Figure 4.14, where the MeLG metric filters out the bilinear behavior in the signature as the thermal gradients are filtered out. To emphasize this effect, 2D projections of the signature are given in Figure 4.14. The higher bounds of MxLG and MeLG show a slightly higher $R^2$ than the total data set, possibly because as more points are identified the signature begins to stabilize, whereas smaller numbers of points can skew results. The two metrics have very similar goodness of fit statistics but vary more in the vector of the best fit line, describing different relationships between temperature, strain, and displacement. Both MeLG and MxLG show the capability to improve on the total data set. Future research will be able to show which method can be more useful for specific applications.
Figure 4.14 2D projections of three-dimensional signature which significantly reduce the bi-linearity. Red indicates time points of minimal thermal gradient found using MeLG = 4.5 °C/m, and red line indicates best fit line for minimal gradient data set, as in Figure 4.13.

<table>
<thead>
<tr>
<th>Metric</th>
<th>Bound</th>
<th>n</th>
<th>R²</th>
<th>σ</th>
<th># Points</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.00</td>
<td>0.078</td>
<td>0.990</td>
<td>0.125</td>
<td>0.983</td>
<td>0.564</td>
</tr>
<tr>
<td>8.50</td>
<td>0.083</td>
<td>0.988</td>
<td>0.133</td>
<td>0.989</td>
<td>0.561</td>
</tr>
<tr>
<td>MxLG</td>
<td>9.00</td>
<td>0.083</td>
<td>0.988</td>
<td>0.132</td>
<td>0.992</td>
</tr>
<tr>
<td></td>
<td>9.50</td>
<td>0.084</td>
<td>0.988</td>
<td>0.132</td>
<td>0.994</td>
</tr>
<tr>
<td></td>
<td>10.00</td>
<td>0.084</td>
<td>0.988</td>
<td>0.131</td>
<td>0.995</td>
</tr>
<tr>
<td>4.00</td>
<td>0.074</td>
<td>0.992</td>
<td>0.108</td>
<td>0.980</td>
<td>0.587</td>
</tr>
<tr>
<td>4.25</td>
<td>0.077</td>
<td>0.990</td>
<td>0.119</td>
<td>0.992</td>
<td>0.584</td>
</tr>
<tr>
<td>MeLG</td>
<td>4.50</td>
<td>0.078</td>
<td>0.990</td>
<td>0.121</td>
<td>0.994</td>
</tr>
<tr>
<td></td>
<td>4.75</td>
<td>0.079</td>
<td>0.989</td>
<td>0.123</td>
<td>0.994</td>
</tr>
<tr>
<td></td>
<td>5.00</td>
<td>0.080</td>
<td>0.989</td>
<td>0.126</td>
<td>0.994</td>
</tr>
<tr>
<td>Total Set</td>
<td>0.084</td>
<td>0.990</td>
<td>0.126</td>
<td>0.989</td>
<td>1.351</td>
</tr>
</tbody>
</table>

4.4.3 MR and LG Comparison

MR and LG classes of methods filter time periods based on different metrics of minimizing thermal gradient and identify different sets of time periods. To illustrate this,
Figure 4.15 shows the time periods identified using both methods for five days in April 2016.

Figure 4.15 shows the time points identified as having a minimal thermal gradient over the entire structure, shown with two sensors for context. These are two of twenty sensors comprising the total data set on the structure. The bounds of each method in Figure 4.15 return a comparable number of points over the entire time period (2748 for MR = 4 °C and 2871 for MeLG = 4.5 °C/m) but identify different sets of time points. While this is not a strict rule, the MR tends to identify time points as the bridge is cooling down while the MeLG tends to identify more time periods during heating. The Streicker Bridge often heats up unevenly due to the shading on different parts of the structure. These differences along the span of the structure will cause time periods to not pass an MR bound, while each individual LG is still minimal.

Figure 4.15 also shows some time periods where there is zero thermal gradient at Pier 12 but are not identified as time periods of minimal gradients over the structure as other
bridge sections remain with thermal gradients. This again shows the uneven heating of the structure.

Temperature data recorded on the Streicker Bridge in the summer of 2010 provides another example of the different sets of time points identified by the LG and MR methods. Due to incomplete data sets in the summer of 2016 (monitoring system disconnected for use in other projects), Figure 4.16 shows the summer of 2010 time points of minimal thermal gradient with the average temperature on the structure.

![Figure 4.16 Summer 2010 Streicker Bridge minimal thermal gradient methods comparison](image)

The MxLG identifies many more time periods in the summer than the MR method. This could be because of sunlight heating certain parts of the bridge much more than others. The local gradients would not be overly affected as the top and bottom both heat, but the overall temperature difference between different parts of the bridge longitudinally would exceed the maximum range. Table 4.5 shows a comparison of the MR and MxLG methods by means of examining the number of time points identified during different seasons in 2010.
Table 4.5 Method comparison by number of points identified for different seasons, 2010.

<table>
<thead>
<tr>
<th></th>
<th># Time points identified, 2010</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MR</td>
</tr>
<tr>
<td>Bound</td>
<td>3.0  3.5</td>
</tr>
<tr>
<td>°C</td>
<td>°C/m</td>
</tr>
<tr>
<td>Year</td>
<td>1895  4582</td>
</tr>
<tr>
<td>Summer</td>
<td>5     178</td>
</tr>
<tr>
<td>Fall</td>
<td>1875  4261</td>
</tr>
</tbody>
</table>

The MR identifies more time periods of uniform temperature in the fall than the MxLG. An MR of 3 °C identifies over a thousand less time points of minimal thermal gradient over the entire year compared to MxLG of 5 °C/m but more time points in the fall. The MR identifies very few time points of minimal gradient in the summer, while the MxLG is fairly even in identifying time points over the entire year.

The differences in these methods can be seen when examining the relationships between the maximum range of temperatures on a structure and possible local gradients. For example, under an MR of 4 °C, the maximum possible local gradient would occur if that difference (4 °C) was present at one cross-section, creating a local gradient of 11.4 °C/m (using Equation (4.4) with an average h of 0.35 m). This level of thermal gradient would only occur if one cross-section was controlling the time period, with the maximum and minimum temperatures occurring at that cross section. A thermal gradient of 11.4 °C/m exceeds any bound set by the local gradient methods in Table 4.3. Conversely, using an MxLG of 9 °C/m would have a temperature difference of 3.15 °C at that cross section.

While this value is lower than any MR bound used, the actual temperature values of different cross-sections are not compared. The difference in the sets of data each method identifies as time points with minimum thermal gradients will provide distinct options for
the basis of a model relating temperature change to strain and displacement, as can be seen in their different best fit line vectors.

The comparison shows that the two classes of methods are simply different rather than one being clearly superior. The two methods identify different subsets of the overall data with the maximum local gradient method identifying many more times in the summer and maximum range in the fall, demonstrating the difference in the identification of minimal thermal gradients. The bounds chosen to be used within each method need to balance two competing necessities: the need for a large enough basis of time periods for the formulation of the temperature–strain–displacement model, and the need to minimize the thermal gradients in the identified time periods. The final choice of the method, or combination of methods, will depend on specific project application.

4.5 Summary

TD-SHM considers temperature as the measurable input in SHM analysis. In order to clearly characterize the relationship between temperature as an input and strain and displacement as output, this chapter proposes two classes of methods for identifying time periods of minimal thermal gradient across the structure. The MR method searches for instances where the range of temperature values on the structure is within some chosen bound. The LG methods compute the local vertical thermal gradient at each instrumented location in the structure ensuring that each gradient is under some maximum gradient or that the mean absolute gradient is under the chosen bound. Each method employs different minimization criteria of thermal gradients in the structure, resulting in different subsets of time periods. These methods are validated quantitatively on a set of data taken from Streicker Bridge.
The MR methods showed a consistent improvement in the linear fit in terms of coefficient of determination and standard error of the three-dimensional signature as well as a consistent vector of the line, different from the total data set, when there was a sufficient number of time points identified. Also, allowing a small amount of outlying temperatures (MR90, MR95) showed no degradation in the goodness of fit of the line but allowed for an increase in the number of time points identified.

The LG methods showed an improvement on the linear fit of the three-dimensional signature when identifying a substantial enough basis of time points (890 or over) and showed unique vectors of the BFL for each MxLG and MeLG. The LG methods also filtered out the bi-linearity shown in Figure 4.4 as the method removes time points of thermal gradient on the structure.

Each metric, the range of temperatures on the structure and the local gradients on the structure, found a linear temperature–strain–displacement relationship that is different from the total data set and has a stronger linear fit than the total data set. This vector of the line relating temperature, strain, and displacement will be sensitive to unusual structural behavior once thermal gradients are filtered out. The influence of errors in strain, temperature, and displacement monitoring to three-dimensional signatures was considered out of scope and is not evaluated in this chapter. One class of methods is not necessarily better in general than the other, and it will depend on the specific application which method is used.
5 Evaluating the Coefficient of Thermal Expansion

This section is based on the following paper, soon to be submitted

_Evaluating the Coefficient of Thermal Expansion of a Built Concrete Structure. Reilly, J. Glisic, B. 2019._

5.1 Introduction

TD-SHM considers temperature as the driving force in structural behavior, relating temperature as an input to a strain and displacement output (J. Reilly et al. 2017). Fundamental to the development of TD-SHM is an accurate representation of the Coefficient of Thermal Expansion (CTE).

Temperature changes in the environment may pose a significant concern in the both the design and monitoring of structures. These challenges arise from the fact that temperature can cause changes in material size and sometimes material properties. The CTE governs the rate of change in strain that a material will experience for a given change of temperature, expressed commonly in the form of the coefficient of linear thermal expansion. While the CTE is typically held as a constant value, concrete structures in practice often exhibit a different thermal behavior than predicted by laboratory experiments or models. The CTE of concrete depends primarily on the formulation of the concrete mix and the moisture content in the concrete and can be difficult to consistently assess even in laboratory settings. Typical values for the CTE of concrete range from 8 to 12 $\mu\varepsilon/^{\circ}\text{C}$ but can be difficult to assess in a built concrete structure. The environment of the concrete structure, including the temperature and relative humidity, affect the
concrete behavior. Even within the concrete of one structure, some studies have found variation in the Young’s modulus and other concrete properties (Gucunski et al. 2015; Kim Jinyoung et al. 2017).

This work seeks to evaluate the relationship between temperature and strain in built reinforced and/or prestressed concrete structures. The CTE of concrete is the driving factor in thermal behavior of the structure but not the only factor. The reinforcement and/or prestressing in the concrete, as well as other various restraints exerted on the material (e.g., by supports), all affect how the structure behaves under thermal influences. The challenge addressed in this research is evaluation of an ‘effective CTE’ at different cross-sections of a structure. This effective CTE is predominantly influenced by the material CTE of the concrete but also incorporates effects of internal restraint on the section, changes in humidity of the concrete, and changes in temperature of other sections of the structure. The effective CTE describes the thermal behavior of a section of a structure, inseparably from its location and history on the structure. This effective CTE is different from the “pure” concrete CTE but is still useful when analyzing a structure as it describes the thermo-mechanical behavior of material at macro-scale. Hence the methods presented in this work seek to isolate the relationship between changes in temperature and strain in order to evaluate the effective CTE (simply termed ‘CTE’ for the remainder of this work) at different locations of a complex structure over time. The method is developed and evaluated using data from a real structure, Streicker Bridge, on the Princeton University campus.
5.2 Streicker Bridge

These methods are explored using data from the Streicker Bridge. Details on the Streicker bridge can be found in Chapter 3, as well as in previous work on the structure (Abdel-Jaber and Glisic 2018; Reilly and Glisic 2018; Sigurdardottir and Glisic 2015; Sigurdardottir 2015).

5.3 Temperature and Strain in Concrete

5.3.1 Sources of Strain

Strain in an undamaged concrete can be attributed to three sources, shown in Equation 5.1.

\[ \varepsilon_{\text{Total}} = \varepsilon_{\text{Mech}} + \varepsilon_{\text{Thermal}} + \varepsilon_{\text{Rheo}} \]  

The total strain (\(\varepsilon_{\text{Total}}\)) is the summation of the mechanical strain (\(\varepsilon_{\text{Mech}}\)), the thermal strain (\(\varepsilon_{\text{Thermal}}\)) and the rheological strain (\(\varepsilon_{\text{Rheo}}\)). Rheological strain in concrete is the combination of the creep and shrinkage. Changes in creep and shrinkage are largest in the early age of concrete and are typically complete after several years, depending on the size of the concrete and structure (Abdel-Jaber and Glisic 2018; Glisic et al. 2013).

Mechanical strain is the effect of loading on the structure, including dead loads, live loads, and any prestressing effects. Mechanical strain can also occur as a response to a restrained thermal expansion or contraction and restrained rheological strain. Thermal strains occur when there is either a change in uniform temperature or a thermal gradient present in the material. Thermal gradients occur when there are varying temperatures throughout the material and can cause curvature, moment, or de-planation of the cross-
section, limiting linear static analysis and making it less accurate, complex, and in some cases inconclusive (Abdel-Jaber and Glisic 2018; Emanuel and Hulsey 1978; Reilly and Glisic 2018). In a laboratory setting, it is often possible to isolate a uniform temperature change as the only source of strain, resulting in Equation 5.2, where there is a linear correlation between changes in temperature and changes in total strain on the material.

\[
\varepsilon_{\text{Total}} = \varepsilon_{\text{Thermal}} = \alpha \Delta T
\]  

Equation 5.2 shows the linear relationship between changes in temperature (\(\Delta T\)) and strain characterized by the CTE (\(\alpha\)). Equation 5.2 assumes linear, free expansion of the material with no mechanical load on the material, no creep or shrinkage, and no thermal gradients effects. However, these conditions are rarely met in practice, and the linear relationship presented in Equation 5.2 frequently cannot be used to determine CTE. Two examples of temperature and strain data taken from a real structure are shown in Figure 5.1 and Figure 5.2 and illustrate the complexity of the problem.
Figure 5.1. Example of one-month relationship between temperature and strain (example taken from Streicker Bridge during month of October 2010 at location P12up).

Figure 5.2. Example of two-month relationship between temperature and strain (example taken from Streicker Bridge from June to August 2010 at location P11h12down).

Figure 5.1 shows one month of temperature and strain data taken from the concrete slab of the Streicker Bridge. The plot shows a temperature and strain relationship close to that described in Equation 5.2. There are minimal rheological strain changes and small
influence of thermal gradients, resulting in a near linear relationship (although mechanical strain might not have been accounted properly). In this case, the slope of the best fit line of the data (7.7 \( \mu \varepsilon/\degree C \)) provides a reasonable estimate of the CTE. Figure 5.2 shows data taken at different location of the same pedestrian bridge, including two months in the summer of 2010. This summer data shows a complex relationship between temperature and strain. Rheological strain changes are more emphasized, and large thermal gradients are present. Figure 5.2 provides little insight to the CTE. Although Figure 5.2 provides more insight, it is not perfect for evaluation. There are still some influences of rheological strains, thermal gradients, and some mechanical response to thermal expansion. A more detailed description of the strain experienced by the concrete can be seen in Equation 5.3,

\[
\varepsilon_{\text{Total}} = \varepsilon_{\text{M_Load}} + \varepsilon_{\text{M_Thermal_Response}} + \varepsilon_{\text{M_Rheo_Response}} + \alpha \Delta T + \alpha \Delta G_T y + \varepsilon_{\text{Rheo}}
\]

(5.3)

where \( \varepsilon_{\text{M_Load}} \) is mechanical strain due to loads, \( \varepsilon_{\text{M_Thermal_Response}} \) is mechanical strain due to temperature change, \( \varepsilon_{\text{M_Rheo_Response}} \) is mechanical strain due to rheological changes, \( \Delta G_T \) is change in thermal gradients, and \( y \) is the location within the depth of the cross-section. Thus, to reduce Equation 5.3 to a form as close as possible to Equation 5.2, it is necessary to minimize the effects of mechanical responses, thermal gradients, and rheological effects.

5.3.2 Thermal Expansion

In practice, materials often experience some restraint even in directions of designed free expansion. For example, neoprene expansion bearings are often used to allow for
expansion, but AASHTO design codes and some other studies allow for the occurrence of restraint in these bearings (AASHTO LRFD Bridge Design Specifications 8th Edition. 2017; Cook and Allen n.d.); studies have shown that typical Neoprene bearings can provide a restraint force equal to 1% of the live loading with larger restraint in colder weather typically below 5 °C. (Cook and Allen n.d.; Yakut and Yura 2002; Yazdani et al. 2000). To show the effect that restraint can have on expansion can have, Figure 5.3 shows a simplified, one-dimensional beam model with a fixed end and a partially restrained end.

![Figure 5.3. Simplified beam model under restrained expansion](image)

Under a uniform temperature change, the total strain on the material is the sum of the linear thermal strain and the mechanical response, \( \varepsilon_{M,\text{Thermal_Response}} \), (provided by the partial restraint at the support) to the thermal expansion, shown in Equation 5.4.

\[
\varepsilon_{Total} = \varepsilon_{Thermal} + \varepsilon_{M,\text{Thermal_Response}}
\] (5.4)

Solving for \( \varepsilon_{Total} \) (shown in Appendix B) provides Equation 5.5, where the strain is linearly related to the change in temperature but by a combination of the CTE, the stiffnesses of the beam (\( K_B \)), and the stiffness of the support (\( K_s \))

\[
\varepsilon_{Total} = \frac{\alpha}{1 + \frac{K_s}{K_B}} \Delta T
\] (5.5)
Equation 5.5 shows that the mechanical response to thermal strain is still dependent on the temperature change and provides insight to the relationship between the temperature and strain under partial restraint. As the stiffness of the spring reduces to zero, the equation simplifies back to Equation 5.2. Complementarily, if the stiffness of the spring increases to infinity as in a fixed support, the total strain approaches zero as there can be no expansion. More realistically, there will be a condition where the support adds some stiffness that is much smaller than the stiffness of the beam. For example, on the Streicker Bridge each leg is supported at the abutment by two neoprene bearings. The restraint on each leg can be approximated using Figure 5.3 and Equation 5.5. The symmetry of the structure and expansion acts as a fixed support at the main span, and the legs expand out towards the abutments. Equation 5.6 evaluates the axial stiffness of the continuous girder legs ($K_B$) as a combination of the Young’s modulus of the concrete (E), the cross-sectional area of the leg (A) and the length of the leg (L).

$$K_B = \frac{E A}{L} = \frac{37 \text{ GPa} \times 1.14 \text{ m}^2}{39 \text{ m}} = 1.07 \times 10^9 \text{ N/m}$$  (5.6)

Each bearing shear stiffness can be approximated using Equation 5.7, where the spring stiffness is equivalent to the combined shear stiffness of the two bearings and evaluated as the product of the shear modulus (G) and the area (A) of the bearing divided by the depth (t) of the bearing.

$$K_S = 2 \times \frac{G A}{t} = 2 \times \frac{1.38 \text{ MPa} \times 0.041 \text{ m}^2}{0.076 \text{ m}} = 1.49 \times 10^6 \text{ N/m}$$  (5.7)

The axial stiffness of the bridge is several orders of magnitude larger than the shear stiffness of the bearings ($\frac{K_S}{K_B} = 0.0014$). If the spring provides 0.14% (the case of
Streicker Bridge) the stiffness of the beam, the relationship between temperature change and strain will remain linear, but the slope of the best fit line will be \( \frac{\alpha}{1.0014} \), or 99.86% of the CTE rather than the true CTE of the section. The columns on the legs may also provide some additional restraint to expansion.

\[
K_{P11} = \frac{EI}{1.2h} = \frac{200 \text{ GPa} \times 88.22 \times 10^6 \text{mm}^4}{1.2 \times 3.96 \text{ m}} = 3.71 \times 10^6 \text{N/m}
\]  

(5.8)

Equation 5.8 assumes a fixed-fixed column model (Salmon et al. 2009). The bending stiffness of Pier 11 (\( K_{P11} \)) provides a restraint to longitudinal expansion of the deck. The stiffness of Pier 12 (\( K_{P12} \), 5.07 \( \times \) \( 10^6 \) N/m) also provides some restraint and has the same moment of inertia (I) but a different height (h) than Pier 11. The bridge sections between Piers 10 and 11 experience the restraint from Pier 11, Pier 12, and the neoprene bearings at the abutment. The sum of these stiffnesses is 1.03\( \times \)\( 10^7 \) N/m, or 0.97% of the axial stiffness of the bridge. This restraint will provide less than 1% error in the evaluation of the CTE and can be neglected. Thus, for the Streicker Bridge, the neoprene bearings and columns are assumed to allow free expansion of the structure. This assumption introduces error of approximately 0.99%, but this error could increase depending on condition of the bearings. A more detailed stiffness calculation can be found in Appendix B.

5.4 Methods for Evaluating Coefficient of Thermal Expansion

The methods presented in this work focus on isolating linear thermal expansion from other sources of strain, as in Equation 5.2. Under ideal conditions, strain and temperature will behave linearly, as in Figure 5.1, and a linear regression will provide a practical estimate of the CTE. More commonly there are non-ideal conditions due to environmental or structural effects and thermal inertial of concrete, and there is no clear
linear fit, as shown in Figure 5.2. Isolating linear thermal strain requires three main assumptions:

1. Negligible changes in rheological strain
2. Negligible changes in mechanical strain
3. Minimal effect from thermal gradients

These assumptions are not always valid in a built concrete structure as there is always a degree of creep or shrinkage, loading, thermal gradient, and restraint on the material. Careful consideration of each assumption and proper analytic techniques could validate these assumptions and lead to accurate on-site evaluation of the CTE of concrete.

5.4.1 Rheological Strain
Creep and shrinkage affect concrete throughout the entire life of the material but have the largest effect and changes in the first few years of the life of the material (Abdel-Jaber and Glisic 2018; American Concrete Institute and ACI Committee 209--Creep and Shrinkage 2008; Glisic et al. 2013). After this initial maturation of the concrete, small time increments (hours or even days) are negligibly affected by changes in creep and shrinkage as these rheological strains develop over longer time scales and their rate slows down over time. In order to consider changes in rheological strain negligible (i.e., ranged between -1 and 0 \( \mu \varepsilon \)), time periods that are either sufficiently far from the casting of the concrete or small enough that there are negligible amounts of rheological strain need to be considered.

On the Streicker Bridge, previous work has identified the stabilization of the majority of rheological strains within four years of the casting of the concrete (Abdel-Jaber and
Glisic 2018). To include data taken prior to the stabilization of the rheological strain, small time periods of measurement were analyzed, where rheological strain is negligible. Previous work has estimated the rheological strain over three months in the summer of 2010 in Streicker Bridge to be approximately $-130 \, \mu \varepsilon$ (Abdel-Jaber and Glisic 2018). The earliest data considered in this work is from the summer of 2010. Over 90 days in the three-month time period, this change in strain would become $-1.44 \, \mu \varepsilon$ per day on average. Thus, during a period of six hours, there was no significant development of rheological strain and could be neglected. The change in rheological strain is expected to decrease over time, so any time periods after summer 2010 would also be unaffected by rheological strain when using a six-hour time period. A sensitivity study, shown in Appendix B, has shown that six-hour windows of the Streicker Bridge data provide the optimal size to capture the heating and cooling of the bridge. Larger periods often capture the transition period between heating and cooling. Smaller periods often capture secondary thermal effects, such as temperature changes at another section affecting the focused section.

5.4.2 Mechanical Strain
Mechanical strain in concrete structures can originate from dead or live loads, transient loads, or prestressing and can occur in response to thermal and rheological effects. When monitoring a structure, the monitoring strategy can filter out changes in mechanical strain due to loading. In long term static monitoring, measurements are typically recorded at a rate ranging from once every five minutes to once every hour to once every month, depending on project requirements (e.g., (Glisic et al. 2013; Sigurdardottir and Glisic 2015)). To minimize changes in strain from transient and live loading, an average
measurement over a short time period is recorded (e.g., over few seconds) rather than a single measurement. Depending on the traffic or use of the structure, this strategy can minimize most changes in dynamic mechanical loading during the time period.

Mechanical strain as a response to the thermal change in a structure depends on the restraint imposed on the material. In a case of free expansion, there is no mechanical response to a uniform temperature change. Partial restraint, shown in Figure 5.3, causes a change in strain dependent on the change in temperature. The mechanical response to thermal change will affect the rate of change of total strain for change in temperature. On the Streicker Bridge, the viscoelastic bearing at the end of the SE leg ideally provides for free expansion of the structure. The viscoelastic bearings can provide a small measure of restraint, depending on the temperature and the rate of expansion. The column supports can also impose some restraint to expansion. These effects can induce some mechanical response to thermal expansion but at a scale orders of magnitude smaller than the total change in strain due to the low shear stiffness of the bearings and low bending stiffness of the columns relative to the axial stiffness of the structure, shown with Equations 5.5-5.8. The presence of this restraint of expansion for the Streicker Bridge legs may add up to 1% error in the evaluation of the CTE. This level of error will not significantly alter the evaluation of the CTE. Discussion related to mechanical strain as a response to rheological strain is similar to that regarding thermal changes; however, given that the observed time periods of six hours practically cancel the effects of rheological strain, mechanical strain due to rheological effects can be neglected during the observed periods.
5.4.3 Thermal Gradients

Thermal gradients occur when there is a non-uniform temperature distribution in a material. On bridges, thermal gradients are often caused by sunlight or other sources of temperature change on a structure. In the case of a linear vertical thermal gradient in a material (throughout the cross-section), there will be a resultant bending of the cross-section. To minimize the effects of vertical thermal gradients on the strain and temperature relationships, a linear approximation of strain and temperature at the centroid of the section is taken. Under the conditions of linear theory, bending from linear thermal gradients will have no effect on the strain at the centroid of the section. Equation 5.9 shows the linear approximation for temperature at the centroid of a section, assuming that two sensors are used to monitor temperature at the top and bottom of the cross-section.

\[ T_{\text{Centroid}} = T_{\text{Bot}} + d \frac{(T_{\text{Top}} - T_{\text{Bot}})}{h} \]  

(5.9)

In this equation, \( T_{\text{Centroid}} \) is the temperature at the centroid, \( d \) is the distance from the bottom sensor to the centroid of cross-section, \( h \) is the depth of the cross-section, and \( T_{\text{Top}} \) and \( T_{\text{Bot}} \) are the temperatures at the top and bottom of cross-section, respectively. \( T_{\text{Centroid}} \) has a measurement uncertainty of 0.3 °C. While this centroid approximation will remove effects from linear thermal gradients, there will still be some consequence of non-linear thermal gradients in the form of an induced error in Equation 5.2. This error introduced by non-linear thermal gradients will be related to the amplitude of the non-linear portion of the gradient, which is governed by the difference in temperature between the top and bottom of the section, and the rate of heating or cooling (AASHTO LRFD Bridge Design Specifications 8th Edition. 2017; Comité Européen de Normalisation 2010). Previous work by the authors proposed several methods to reduce error in temperature at the
centroid due to non-linear thermal by identifying time periods of minimal thermal gradient for analysis (Reilly and Glisic 2018). Those methods are not applied to this application as they would depopulate the data set, not allowing for a linear regression over a fully populated six-hour window.

Strain at the centroid of the section is similarly evaluated, using Equation 5.10.

\[ \varepsilon_{\text{Centroid}} = \varepsilon_{\text{Bot}} + d \frac{(\varepsilon_{\text{Top}} - \varepsilon_{\text{Bot}})}{h} \]  \hspace{1cm} (5.10)

\( \varepsilon_{\text{Centroid}} \) has a measurement uncertainty of 3.13 \( \mu \varepsilon \) and experiences some error due to the non-linear thermal gradients in the section.

### 5.4.4 Development of the Method for Evaluating CTE

The method for evaluating the CTE combines the approaches described in above Subsections 5.4.1-5.4.3, to isolate a linear temperature strain relationship with data analytic techniques to identify the CTE. First, selection of six-hour time periods for analysis filters out changes in rheological strain. Next, the use of short-term dynamic measurements to capture static strain filters out any transient or live loads causing mechanical strain. Finally, approximating the strain and temperature at the centroid of the section minimizes the effects from thermal gradients.

The analysis of data consists of three layers of statistical analysis applied to all possible six-hour periods of collected strain and temperature measurements within observed periods of time. For each of these six-hour windows linear regression is performed and the coefficient of determination (\( R^2 \)) calculated. The combination of linear regressions for a data set forms a distribution of the thermal behavior of the structure. Filtering this distribution by the coefficient of determination removes times when there is a poor linear
fit between temperature and strain. These poor fits can occur either when there is very small change in temperature and the noise overshadows the signal, or when irregularities occur in the environment such as a sudden rain on the structure. Figure 5.4 shows a month of strain and temperature data from July of 2010 on the Streicker Bridge, highlighting two different six-hour windows.

![Graph showing strain vs temperature with R² and slope annotations](image)

Figure 5.4. One month of strain and temperature data from P10h11, highlighting two different six-hour windows.

Window 1 shows a change in temperature of 3°C and has a strong linear fit between temperature and strain. Window 2 has a small change in temperature and almost no change in strain. The best fit line for Window 2 fits more sensor noise than the thermal behavior. Each six-hour window provides a line a best fit with an accompanying slope and R². A sensitivity study (shown in Appendix B) has shown that filtering out windows when R² is less than 0.90 removes times when there is very little change in temperature or when there are irregularities occurring while also leaving enough points in the distribution for analysis. Figure 5.5 shows the distribution made by the slopes of the regression line of every six-hour window taken in July 2010 at P10h11.
The distribution of regression windows with a $R^2$ had close to half the number (N) of CTE evaluations, reduced the IQR, and increased the median by 8.6%. The filtered distribution (red) loses the peak centered around 9 $\mu e/°C$, and coalesces around the larger peak near 12.5 $\mu e/°C$. Filtering by $R^2$ identifies the times when the temperature-strain relationship is behaving the most linear, which is when the necessary assumptions regarding mechanical strain, rheological strain, and thermal gradients are the most valid. A $\chi^2$ test was applied to all of the distributions of CTE evaluations to test for normality, and an insignificant amount were found to have a normal distribution. Ideally, the resulting distribution would be normal, as the variations around the mean value are cause by noise. The non-normality of the distributions suggests the presence of other thermal effects unaccounted for in analysis. In many cases there was not a substantial difference between the mean and median of the distributions, but in some cases the median was more useful for being less sensitive to outliers. The median of this distribution of slopes
provides the evaluation for the CTE, while the interquartile range (IQR) provides a quantification of the uncertainty associated with the evaluation. The entire process of CTE evaluation is summarized below and was repeated for all sensor locations on the SE leg of Streicker Bridge for time periods from 2010 to 2017.

1. Design monitoring strategy to minimize transient mechanical strain
2. Approximate values of strain and temperature at centroid of section
3. Apply sliding window linear regression
4. Form a distribution of slopes of best fit line
5. Filter by $R^2$
6. Find median and interquartile range of distribution

5.5 Results
There are in total eight cross-sections along the SE leg of the Streicker Bridge where the CTE evaluations were performed: at Piers 10, 11, 12, and 13 (P10, P12, P12, P13), at the mid-spans (P10h11, P11h12, 12h13), and at the quarter span between P10h11 and P11 (P10qqq11), see Figure 3.8. Each sensor location has its own unique thermal behavior, resulting in correspondingly different CTE values. Table 5.1, Table 5.2, and Table 5.3 show the CTE evaluations using the method described with a six-hour window and filtering for the $R^2$ values over 0.90. Temperature and strain data were collected from 2010 to 2017, with multiple missing data due to periodical malfunction of the monitoring system and frequent necessity of use on other projects for the reading unit. Because of the large timespan of monitoring, the results are condensed to a single time period of monitoring from the winter, summer, and fall of each year. Spring was not included
because of the large amount of rain in the season, frequently shifting the temperature on the structure, obstructing the linear increases in temperature and strain. Due to missing data, the periods used for each season are not identical, but rather adapted to the available data. In addition, for the same reasons, in some years it was not possible to calculate CTE, in which case the corresponding field in table is filled with “NaN” (not a number).

A helpful frame of reference for assessing the CTE evaluations are the FHWA guidelines that Portland cement concrete typically has a CTE from 7.4-13 με/° and the bridge design value of 10.8 με/°C.

5.5.1 Fall CTE Evaluations

<table>
<thead>
<tr>
<th>Date</th>
<th>P10</th>
<th>P10h11</th>
<th>P10qqq11</th>
<th>P11</th>
<th>P11h12</th>
<th>P12</th>
<th>P12h13</th>
<th>P13</th>
<th>T (°C)</th>
<th>RH (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oct 8-20, 2010</td>
<td>11.4</td>
<td>11.3</td>
<td>12.8</td>
<td>12.3</td>
<td>11.9</td>
<td>8.3</td>
<td>8.5</td>
<td>6.6</td>
<td>13.7</td>
<td>63.1</td>
</tr>
<tr>
<td>Oct 5-28, 2011</td>
<td>10.0</td>
<td>10.7</td>
<td>11.6</td>
<td>11.6</td>
<td>10.9</td>
<td>8.7</td>
<td>9.6</td>
<td>NaN</td>
<td>15.3</td>
<td>66.0</td>
</tr>
<tr>
<td>Sept 26-Oct 8, 2012</td>
<td>10.0</td>
<td>10.2</td>
<td>11.4</td>
<td>10.3</td>
<td>NaN</td>
<td>8.9</td>
<td>7.1</td>
<td>NaN</td>
<td>16.7</td>
<td>80.0</td>
</tr>
<tr>
<td>Oct 9-19, 2013</td>
<td>10.2</td>
<td>10.5</td>
<td>11.0</td>
<td>9.9</td>
<td>10.2</td>
<td>8.0</td>
<td>9.1</td>
<td>9.2</td>
<td>15.4</td>
<td>73.6</td>
</tr>
<tr>
<td>Oct 19-Nov 6, 2016</td>
<td>9.9</td>
<td>10.5</td>
<td>NaN</td>
<td>NaN</td>
<td>12.3</td>
<td>10.0</td>
<td>11.3</td>
<td>10.4</td>
<td>12.7</td>
<td>68.4</td>
</tr>
</tbody>
</table>

Table 5.1 shows the evaluations of the CTE for five time periods consisting of one to three weeks in the fall from 2010 to 2016. The IQR (included in Appendix B) provides a quantification of uncertainty with these evaluations. The last two columns provide the ambient temperature and RH over the given time period. Figure 5.6 graphically represents the information in Table 5.1.
After the first year (2010) many of the sensor locations stabilize. P10 and P10h11 show very stable evaluations at 10 $\mu$e/°C and 10.5 $\mu$e/°C. Other locations display differing levels of stability, though P10qqq11 is consistently the largest evaluation, and P12 and P12h13 are consistently the lowest. Evaluations are fairly consistent from 2011 to 2013, with 2010 and 2016 presenting as different values still within typical values. All evaluations fall within or very close to typical values for reinforced concrete CTE except for Pier 13. The sensors at Pier 13 experience a unique temperature strain relationship that is highly dependent the condition of the bearings and temperature. Pier 13 behavior is explored with the summer evaluations.

5.5.2 Summer CTE Evaluations

Table 5.2 and Figure 5.7 show the evaluation taken over two-four weeks during the summer. The summer has higher temperatures, more sunlight, and higher RH than other
seasons. These differences in environment cause unique thermal behavior in the structure and result in consistent evaluations that are different from the fall periods.

Table 5.2. Summer CTE evaluations, με/°C

<table>
<thead>
<tr>
<th>Date</th>
<th>P10</th>
<th>P10h11</th>
<th>P10qqq11</th>
<th>P11</th>
<th>P11h12</th>
<th>P12</th>
<th>P12h13</th>
<th>P13</th>
<th>T (°C)</th>
<th>RH (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>July 8-Aug 5, 2010</td>
<td>9.1</td>
<td>12.6</td>
<td>10.3</td>
<td>9.8</td>
<td>12.6</td>
<td>9.2</td>
<td>8.3</td>
<td>10.6</td>
<td>26.5</td>
<td>66.0</td>
</tr>
<tr>
<td>June 30 -July 21, 2011</td>
<td>NaN</td>
<td>11.8</td>
<td>10.6</td>
<td>8.8</td>
<td>12.2</td>
<td>10.2</td>
<td>8.4</td>
<td>10.7</td>
<td>26.0</td>
<td>63.0</td>
</tr>
<tr>
<td>June 7-18, 2012</td>
<td>9.1</td>
<td>11.5</td>
<td>10.4</td>
<td>8.7</td>
<td>11.3</td>
<td>8.4</td>
<td>8.6</td>
<td>5.2</td>
<td>20.7</td>
<td>69.3</td>
</tr>
<tr>
<td>July 8-21, 2013</td>
<td>9.7</td>
<td>11.6</td>
<td>10.4</td>
<td>9.6</td>
<td>12.5</td>
<td>9.2</td>
<td>7.6</td>
<td>13.5</td>
<td>27.2</td>
<td>71.7</td>
</tr>
<tr>
<td>July 2-25, 2014</td>
<td>9.3</td>
<td>11.0</td>
<td>9.5</td>
<td>9.1</td>
<td>12.1</td>
<td>8.0</td>
<td>6.6</td>
<td>9.3</td>
<td>23.9</td>
<td>65.9</td>
</tr>
<tr>
<td>Aug 8-Aug 29, 2016</td>
<td>9.3</td>
<td>9.9</td>
<td>NaN</td>
<td>NaN</td>
<td>12.3</td>
<td>9.3</td>
<td>10.1</td>
<td>8.0</td>
<td>26.1</td>
<td>70.4</td>
</tr>
<tr>
<td>June 8-July 7, 2017</td>
<td>8.8</td>
<td>10.1</td>
<td>10.5</td>
<td>NaN</td>
<td>NaN</td>
<td>9.2</td>
<td>10.5</td>
<td>10.3</td>
<td>23.9</td>
<td>73.0</td>
</tr>
</tbody>
</table>

Figure 5.7 Summer CTE evaluations

P10, P10qqq11, P11, and P11h12 all have consistent evaluations from 2010 to 2016 with a range of evaluations of less than 2 με/°C. This consistency in evaluation over time is in contrast to the consistency across the structure: each different location has a unique, consistent thermal behavior. Again, P13 has the largest spread in evaluations with both the lowest and highest evaluations. Figure 5.8 helps illustrate the source of the variable behavior at P13.
Figure 5.8 P13 temperature and strain for Summer 2011 to 2014

Figure 5.8 shows temperature and strain taken at the centroid of the cross-section at Pier 13, from the summers of 2011 through 2014. P13 sits on two neoprene expansion bearings. At approximately 30 °C, there is a change in the thermal behavior of the section. Above 30 °C there is a linear thermal behavior at rate of approximately 13.5 με/°C. In 2011 and 2012, below 30 °C there was a reasonable linear thermal behavior, but at a much lower rate than above 30 °C. The year 2011 experienced a large range of temperatures, but in 2012 the temperature did not rise above 30 °C leading to the very low CTE approximation. In 2013 the temperature rose above 30 °C, but when returning below 30 °C the section exhibited almost no change in strain for change in temperature. This indicates that the bridge section was somehow restrained from expanding. This data exhibits poor R² values in linear regression windows and is filtered out of the distribution of slopes leaving 13.5 με/°C as the evaluation. In 2014, only a small portion of the time period reaches over 30 °C and exhibits a linear behavior. The CTE evaluation reaches a
middle ground at 9.3με/°C but has a very large IQR of 8.97 (found in Appendix B), showing the increased uncertainty in the evaluation. This behavior at P13 is explored more fully with the addition of displacement in Chapter 6.

5.5.3 Winter CTE Evaluations

Winter has the coldest temperatures and lowest RH during the year, with the possibility of snow and ice on the structure. Table 5.3 and Figure 5.9 show the winter CTE evaluations.

Table 5.3. Winter CTE evaluations (με/°C)

<table>
<thead>
<tr>
<th>Date</th>
<th>P10</th>
<th>P10h11</th>
<th>P10qqq11</th>
<th>P11</th>
<th>P11h12</th>
<th>P12</th>
<th>P12h13</th>
<th>P13</th>
<th>T (°C)</th>
<th>RH (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dec 20, 2010-Jan 15, 2011</td>
<td>8.2</td>
<td>10.5</td>
<td>9.8</td>
<td>9.6</td>
<td>10.6</td>
<td>7.8</td>
<td>10.0</td>
<td>8.8</td>
<td>-0.7</td>
<td>59.6</td>
</tr>
<tr>
<td>Jan 26-Feb 22, 2012</td>
<td>9.7</td>
<td>10.9</td>
<td>11.0</td>
<td>11.1</td>
<td>11.9</td>
<td>7.5</td>
<td>9.5</td>
<td>6.9</td>
<td>4.9</td>
<td>55.7</td>
</tr>
<tr>
<td>Jan 25-Feb 6, 2013</td>
<td>9.2</td>
<td>9.7</td>
<td>9.9</td>
<td>10.5</td>
<td>NaN</td>
<td>8.9</td>
<td>9.5</td>
<td>NaN</td>
<td>-0.4</td>
<td>68.9</td>
</tr>
<tr>
<td>Jan 16-Jan 25, 2014</td>
<td>7.8</td>
<td>8.7</td>
<td>9.4</td>
<td>10.2</td>
<td>10.8</td>
<td>8.3</td>
<td>9.6</td>
<td>7.2</td>
<td>-4.4</td>
<td>60.2</td>
</tr>
<tr>
<td>Jan 5-Jan 27, 2017</td>
<td>8.8</td>
<td>9.1</td>
<td>9.7</td>
<td>NaN</td>
<td>10.2</td>
<td>8.7</td>
<td>9.5</td>
<td>9.0</td>
<td>2.7</td>
<td>78.0</td>
</tr>
</tbody>
</table>

Figure 5.9 Winter CTE evaluations

While all the evaluations (outside of P13) again fall within typical FHWA and AASHTO limits, there is some behavior in winter that is different from fall and summer. In winter, P12 and P12h13 have been the most stable evaluations over the years while P10 and
P10h11 vary the most. In January 2014 the ambient temperature was very cold (average -4.4°C), but very little effect was visible in the CTE evaluations. To compare the CTE evaluations for the different seasons, Table 5.4 lists the average CTE evaluations for each section as well as the average ambient temperature and RH.

<table>
<thead>
<tr>
<th>Season</th>
<th>P10</th>
<th>P10h11</th>
<th>P10qqq11</th>
<th>P11</th>
<th>P11h12</th>
<th>P12</th>
<th>P12h13</th>
<th>P13</th>
<th>T (°C)</th>
<th>RH (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Winter</td>
<td>8.8</td>
<td>9.8</td>
<td>10.0</td>
<td>10.4</td>
<td>10.9</td>
<td>8.2</td>
<td>9.6</td>
<td>8.0</td>
<td>0.4</td>
<td>64.5</td>
</tr>
<tr>
<td>Fall</td>
<td>10.3</td>
<td>10.6</td>
<td>11.7</td>
<td>11.0</td>
<td>11.3</td>
<td>8.8</td>
<td>9.1</td>
<td>8.7</td>
<td>14.7</td>
<td>70.2</td>
</tr>
<tr>
<td>Summer</td>
<td>9.2</td>
<td>11.2</td>
<td>10.3</td>
<td>9.2</td>
<td>12.2</td>
<td>9.1</td>
<td>8.6</td>
<td>9.7</td>
<td>24.9</td>
<td>68.5</td>
</tr>
</tbody>
</table>

There is a clear difference in thermal behavior of the structure during the different seasons. These differences could stem from a variety of factors: settlement of support with temperature, expansion in the columns, or relaxation of post-tensioning tendons among other possible causes. Ambient temperature and RH most likely play a role in these effects. Ambient temperature and RH both have some effect on the CTE of concrete on a material level, but it is difficult to distinguish those effects in practice. The change in CTE over typical yearly temperatures (-10 to 30 °C) does not have a large effect on CTE (Islam and Tarefder 2015; Johnson and others 2005). Ambient temperature most likely influences the CTE evaluations through secondary thermal effects on the structure, such as expansion in the columns, relaxation of post-tensioning tendon, or thermal action in the ground. Changes in ambient RH will affect changes in the humidity in the concrete (and therefore the CTE) at a delayed rate as the moisture transports in or out of the material (Lim et al. 2016; Mallela et al. 2005; Sellevold and Bjøntegaard 2006) but can have substantial effects on the CTE. It is difficult to assess the total thermal action on a
structure in practice, as there is a large number of possible factors affecting the temperature strain relationship.

Different locations on the structure also display their own unique thermal behavior. In some cases locations on the structure behave similarly, such as P10, P11, and P12 in summer (9.2; 9.2; 9.1 µε/°C) but then behave differently in fall (10.3; 11.0; 8.8 µε/°C). Some of these differences in CTE evaluation can be attributed to the mixing and pouring of concrete. Studies using ultrasonic surface waves have shown large variation in the modulus of elasticity in the concrete bridge decks (Gucunski et al. 2015; Kim Jinyoung et al. 2017). These effects can occur due to poor consolidation or variation in the concrete mix. On the Streicker Bridge, during the setting of the concrete on the SE leg there was some pooling of water as it flowed down the grade of the leg toward the support. This pooling could lead to variation in the moisture in the concrete and variation in the CTE of the material.

5.6 Summary
This chapter presents a novel method for the evaluation of the CTE of a built concrete structure. This method uses several analytic techniques to identify a distribution short-term changes in strain with temperature absent of other sources of change in strain. Recording strain and temperature as an average value of five minutes of monitoring removes the effects of transient loading. Measurements interpolated at the centroid of the section minimize the effects of thermal gradients on the strain. A sliding six-hour window minimizes any change in rheological strain and a linear regression taken at each window forms a distribution of CTE estimations for each location on the SE leg of the Streicker
Bridge. The median and IQR of these distributions form an evaluation and uncertainty estimate for the CTE.

The CTE estimates found in this chapter are effective CTE’s, describing the behavior of the entire cross-section with temperature rather than a material CTE assessment. Important findings of this chapter are that there is a different CTE of the structure for each season, various locations along the bridge display different CTE evaluations, and many locations had consistent CTE evaluations throughout years of monitoring. Almost all evaluations outside of P13 fell within typical FHWA values for the CTE of concrete. P13 displayed some unusual behavior, with a different effective CTE above and below 30 °C and in some cases almost no change in strain with temperature. This behavior at P13 is explored more in Chapter 6.
6 Developing Three-Dimensional Temperature Signatures for TD-SHM

This section is based on the following conference presentation


6.1 Introduction

The three-dimensional temperature signatures are central to the process of TD-SHM. Temperature signatures relate a combination of temperature, strain, and displacement on the structure. Temperature and strain alone provide an insight into the day to day behavior, but displacement is a necessary component to explore the restraint on the material. In theoretical analysis, a support is approximated as fixed, free to expand, or some intermediate level of restraint. The true amount of restraint on different parts of a real structure pertaining to thermal expansion can be difficult to assess. Examining the displacement in combination with the strain and temperature allows an exploration of this real-world behavior. Linear combinations of these parameters, such as curvature instead of strain or thermal gradient instead of temperature, show different behavioral aspects of the structure. The hypothesis of this section is that each temperature signature will provide a quantifiable 3-dimensional figure. The numerical fitting of these signatures will provide a means to quantify a baseline behavior for each section of the structure across
the spectrum of typical yearly temperature changes. Changes in the fit or error of these signatures will show damage or unusual behavior in the structure.

6.2 Displacement Sensors
Previous chapters focus on the relationship between temperature and strain. Displacement adds another dimension and insight to the behavior of the structure, showing the effects of the strain on the structure and providing insight to the amount of strain converted to stress. Displacement sensors were installed on the abutment at the SE leg of Streicker Bridge in April 2016. Section 3.2.2 describes the displacement sensors and locations, as well as Streicker Bridge and the monitoring system.

6.3 Longitudinal Displacement
Longitudinal displacement of the SE leg of Streicker bridge correlates very well with temperature. This correlation is a result of the expansion bearings at the abutment, allowing for free expansion of the leg. Changes in temperature directly cause changes in strain and displacement. Figure 6.1 shows this relationship between the average SE leg temperature and the displacement at the abutment.
Figure 6.1 Longitudinal displacement from Apr 2016 to June 2017 with average temperature on the SE leg.

The displacement measurements on the north side of the abutment are very close to the displacement measurements on the south side of the abutment. Three-dimensional signatures for temperature, strain, and displacement were presented briefly in Chapter 4.
Figure 6.2 shows the temperature signature for P13 using temperature and strain (at the centroid) with longitudinal displacement.

![Figure 6.2. Temperature and Strain at centroid of P13 with longitudinal displacement and best fit line (black)](image)

Figure 6.2 shows data from April 2016 to June 2017 using the longitudinal displacement on the south side of the abutment as the north sensor began experiencing issues in 2017. The vector \( \mathbf{n} \), of the best fit line and intersection point provide a description of the linear relationship between temperature, strain, and displacement, and are also sensitive changes in structural behavior. Higher \( R^2 \) and lower root mean square error (RMSE) would show a more accurate estimation of this behavior, increasing the expected ability to determine smaller changes in this structural behavior (Reilly and Glisic 2018; Yarnold 2013; Yarnold and Moon 2015). The vector of the best fit line of the entire data set (including non-linear thermal gradients) in Figure 6.2, is \( \mathbf{n} = [0.13, 0.99, 0.08] \), relating to [strain, temperature, displacement]. This vector describes the linear relationship between the three parameters: for every 0.13 °C change in temperature, there is a corresponding 0.99 \( \mu \varepsilon \) change in strain and 0.08 mm change in displacement. Because of the strong linearity of the best fit line, and the temperatures below 30°C there is not a large need to identify time periods of minimal thermal gradient. Boundary conditions, geometry of the
structure, and material restraints all have a strong influence on these three-dimensional signatures; the linear relationship identified captures these effects in the vector of the line.

6.3.1 Displacement from Strain

Displacement measurements taken on Streicker Bridge only range from April 2016 to September 2017. Unavailability of the monitoring system, followed by malfunction in the monitoring system, reduced the amount of displacement data available. To observe a larger data set for the three-dimensional signatures, Equation 6.1 equates the axial strain in a beam to axial displacement.

\[ \delta = \int_0^L \epsilon(x) \, dx \]  

(6.1)

In ideal condition, the integral of the strain (\( \epsilon(x) \)) over the length (L) of a beam would equal the displacement (\( \delta \)) at the end of the beam. Equation 6.1 can apply to the SE leg of Streicker Bridge if the connection to the main span is assumed to be a stable, fixed boundary condition. Figure 6.3 shows the results of the integration of the strain along the SE leg of Streicker Bridge with the real longitudinal displacement measurements for November 2017.
Because the strain at each point along the leg is unknown, trapezoid integration is used to integrate the strain from point to point at each known value. The resulting changes in displacement correlate well with the real changes in displacement but with a slope near 1.7. This slope means that there is more displacement than Equation 6.1 suggests. Most likely the junction between the SE leg and the main span does not act as a fixed boundary condition, and there is some displacement at Pier 10 as the displacement at the SE abutment corresponds to a much larger integral of strain than over just the SE leg. When combining all the real displacement data, an average slope of 1.76 is obtained. Because of the strong linear fit of each data set, the integration can be multiplied by 1.76 to obtain a good estimate of the true displacement. Figure 6.4 shows November 2017 with a comparison of the real displacement with the strain integration and strain integration multiplied by 1.76. Figure 6.5 shows the residuals associated with each integration.
The residuals are defined as the true value minus the predicted value of the variable. The residuals of integration multiplied by a factor of 1.76 has a mean (μ) further from zero than just the integration but a much lower spread with a standard deviation (σ) of 1.07. This is an acceptable level of error for the exploration of three-dimensional temperature signatures. Equation 6.2 uses this fit to predict displacement for time periods prior to the installation of displacement sensors. The predicted change in displacement (Δδ_P) at time
‘t’ equals the integral of the strain along the SE leg at time ‘t’, multiplied by the factor 1.76.

\[ \Delta \delta_P(t) = \int_L \varepsilon(x, t) \, dx \ast 1.76 \]  

(6.2)

This equation predicts changes in displacement, not absolute displacement. With large gaps in the data set, each time period of monitoring starts at zero and predicts change in displacement for that time period. Shifts in strain data throughout the entirety of monitoring make this change in displacement a better prediction than total displacement.

Using this predicted longitudinal displacement data, several three-dimensional signatures from P13 are shown in Figure 6.6, including the signature using real displacement data from 2016 to 2017. The signatures are spread out because the displacement is predicted as a change within each signature and does not reflect the shift in strain over the years of monitoring. This will affect the intercept of the lines but not the other more important components (n, R², RMSE). Best fit line statistics for each signature are shown in Table 6.1.
Figure 6.6 Three-dimensional signatures with predicted displacement from multiple time periods: two viewpoints

Table 6.1 Best fit line statistics for three-dimensional signatures.

<table>
<thead>
<tr>
<th>Time Period</th>
<th>R²ₜₑ</th>
<th>R²ₜ₅</th>
<th>RMSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>April 2016 - June 2017</td>
<td>0.12</td>
<td>0.99</td>
<td>0.08</td>
</tr>
<tr>
<td>July 2011</td>
<td>0.12</td>
<td>1.18</td>
<td>0.05</td>
</tr>
<tr>
<td>April 2013</td>
<td>0.12</td>
<td>1.14</td>
<td>0.07</td>
</tr>
<tr>
<td>Dec 2013 - Jan 2014</td>
<td>0.12</td>
<td>1.00</td>
<td>0.06</td>
</tr>
<tr>
<td>July 2014</td>
<td>0.12</td>
<td>0.74</td>
<td>0.06</td>
</tr>
</tbody>
</table>

°C με mm

In this scenario, the signature using the real data (green) is held as the baseline behavior to compare with the other signatures. Comparing a baseline to prior time periods to detect unusual behavior is not a typical monitoring strategy but is performed in this instance because of the use of the predicted displacement values. R²ₜₑ refers to the coefficient of determination of the two-dimensional relationship between temperature and strain while R²ₜ₅ represents the temperature and displacement. These two dimensional R² values are more related to the linearity of these signatures than the R² of the three-dimensional plots. The vector of the best fit line (n) in Table 6.1 for the real data is a unit vector while the
other best fit vectors are scaled to have the same temperature component for easier comparison (all best fit line vectors maintain their direction). First, April 2013 and January 2014 match the baseline signature well. Both sets have similar $R^2$ values and low RMSE. The January 2014 data set has a very similar best fit line vector, while April 2013 has larger increases in strain for a given temperature change. The low RMSE helps justify that both time periods are displaying normal behavior. July 2014 has a significantly different vector of best fit line, a larger RMSE value, and much small $R^2_T-\varepsilon$ than the 2017 data. This signifies some type of unusual behavior. July 2011 has a lower $R^2_T-\varepsilon$ and $R^2_T-\delta$ value and a best fit line vector different from the real data but without a substantial difference in any parameter. Applying a filter for minimal thermal gradients, as presented in Chapter 4, clarifies these signatures in the results shown in Table 6.2

### Table 6.2. Best fit line statistics using a MR 6 gradient filter

<table>
<thead>
<tr>
<th>MR &lt; 6.0</th>
<th>n</th>
<th>RMSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Real Data: June 2017</td>
<td>0.12</td>
<td>0.97</td>
</tr>
<tr>
<td>July 2011</td>
<td>0.12</td>
<td>0.94</td>
</tr>
<tr>
<td>April 2013</td>
<td>0.12</td>
<td>1.16</td>
</tr>
<tr>
<td>July 2014</td>
<td>0.12</td>
<td>0.89</td>
</tr>
</tbody>
</table>

An MR of 6.0 °C filtered out time points of high thermal gradient but left a substantial enough basis of points for analysis. The baseline signature improved in RMSE and changed its vector slightly as well. July 2014 improves RMSE significantly but still has a different vector of best fit, showing some unusual behavior not caused by large or nonlinear thermal gradients. This event is some instance of high restraint on the structure where there were changes in temperature but essentially no change in strain. It is possible that there is some type of locking up of the bearing and that the strain integration does not
provide an accurate estimate of the displacement in this case. It appears that there should be much less displacement than normal for this case. The April 2013 signature provides an interesting case. This signature has a very strong fit with low error but a different best fit line than the baseline, suggesting that in April there was much more strain per temperature change than in other time periods. This signature depicts not an unusual behavior but a different normal or baseline behavior. It is possible that the baseline behavior shifts throughout monitoring due to maturation of strain, settlement of supports, or some other unknown factor. The July 2011 signature improved in all parameters with a new best fit line vector much closer to the baseline 2017 signature. Filtering out time periods of high thermal gradient left a signature more compatible with the baseline behavior. Figure 6.7 shows the signature from July 2011 with the time periods of minimal thermal gradient identified in red.

Figure 6.7 Temperature signature for July 2011 with time periods of MR < 6 °C shown in red

In this case the minimal gradient filter makes a substantial difference in the line of best fit, as thermal gradient have a large effect on the P13 July 2011 signature. This suggests
that the July 2011 signature is a normal signature for higher temperature just obscured by the non-linear effects of large thermal gradients. This highlights the importance of monitoring the structure during extreme temperatures. Structures often display a different behavior at very high or low temperatures, and this can be difficult to predict. Three-dimensional signatures can capture this behavior but only if they have been trained on that extreme temperature.

6.3.2 Thermal Gradient and Elastic Curvature

The thermal gradient and elastic curvature describe the distribution of temperature through the cross-section and the mechanical response of the structure to this curvature from temperature. A linear thermal gradient \( G_{\Delta T} \) is approximated in Equation 4.1, while Equations 6.3 and 6.4 describe the elastic curvature \( \kappa_E \).

\[
\kappa_E = \frac{\Delta \varepsilon_{mech,bot} - \Delta \varepsilon_{mech,top}}{h} \quad (6.3)
\]

\[
\varepsilon_{mech} = \varepsilon_{Total} - \varepsilon_{Temp} = \varepsilon_{Total} - \alpha \Delta T \quad (6.4)
\]

The elastic curvature is the mechanical response to a bending in the cross section, defined similarly to the thermal gradient by assuming a linear curvature through the section. The difference between the bottom \( (\Delta \varepsilon_{mech,bot}) \) and top \( (\Delta \varepsilon_{mech,top}) \) mechanical strain divided by the distance between the sensors. To assess the mechanical strain, the CTE \( (\alpha) \) values for Streicker Bridge from Chapter 5 are used to subtract the thermal strain \( (\varepsilon_{Temp}) \) from the total strain \( (\varepsilon_{Total}) \). The thermal gradient and elastic curvature form another three-dimensional signature, shown in Figure 6.8 with longitudinal displacement.
Figure 6.8 Planar signature for P12h13 from April 2016 to September 2017

The signature forms a plane, described by the normal vector \( \mathbf{n} = [0.684 \ 0.096 \ 0.723] \) and goodness of fit metrics RMSE of 2.00 and \( R^2 \) of 0.871. Applying a filter to remove time periods of large thermal gradients removes some ‘fuzzy’ parts of the plot but does not significantly change the normal vector or improve the goodness of fit. For the time periods of minimal gradient (red) \( \mathbf{n} = [0.723 \ 0.095 \ 0.682] \), RMSE = 1.90 and \( R^2 = 0.872 \). There is a small change in \( \mathbf{n} \) and improvements in goodness of fit.
Exploration of the thermal gradient with the displacement generated from strain integration has not yielded satisfying results. There is no consistent fit and generally poor statistics of fit. It is likely that regenerating the displacement as a change in displacement at the start of each time period of monitoring has removed critical information from this type of signature. The value of the absolute displacement of the structure is important rather than the change in displacement during the period. This suggests that the thermal gradient and elastic curvature are related to the longitudinal displacement indirectly rather than directly as with temperature and strain. This is to be expected since in static analysis, linear thermal gradients do not induce any longitudinal displacement in the structure.

6.4 Vertical Displacement

The vertical displacement readings show a different aspect of the thermal behavior than the longitudinal sensors. Figure 6.10 shows the vertical displacement on the north and...
south side of the abutment with the average temperature in the SE leg. Average SE leg temperature is strongly correlated with all displacement at the SE abutment and is a useful tool for viewing displacement plots.

Figure 6.10 Vertical displacement from April 2016 to June 2017 with average temperature on the SE leg. The vertical displacement at the north and south sides of the abutment have different behaviors, with the north side moving over a much wider range. Two important aspects of Figure 6.10 are the small “tail” at -10°C on the south displacement and the shift in the north displacement. At very cold temperatures, the south side of the leg seems to move upwards, while the north side continues to move downwards. The shift in the north data is also related to the cold temperatures. To explain this shift and the interaction of these two displacement sets, Equation 6.6 calculates the tilt of the SE leg at the abutment as the difference between the north (δ_{Vert,N}) and south (δ_{Vert,S}) vertical displacements divided by the length (L) between the sensors. This tilt is shown again with average Streicker temperature in Figure 6.11.

\[ Tilt = \frac{\delta_{Vert,N} - \delta_{Vert,S}}{L} \]  

(6.6)
The tilt generally increases (north has larger displacement) with temperature. This relationship between the average temperature on the SE leg and the tilt is most likely a result of the curve in plan view of the SE leg. The larger side of the curve (north) expands and contracts more than the shorter side of the curve (south). This curve is shown well in Figure 3.7. At cold temperatures, the tilt starts to decrease at a faster rate, and the south side actually starts to rise up. The bridge may be starting to raise the south side off of the bearings, while increasing the axial stress on the north bearing. This increased load on the north bearing occurs at the same time as a stiffening of the bearing due to the cold temperature (Roeder et al. 1989; Yakut and Yura 2002). One possibility for the shift is that the bearing reached near its maximum downward compression, experiencing very small displacement from load caused by the tilting at the abutment. When the temperature increased, the bearing unloaded in the same behavior as before it reached its high compression. This idea is similar to how concrete loaded into its inelastic phase and then unloaded will unload elastically. It is unlikely that there was actual liftoff with the south bearing, as the displacement does not exceed the zero mm displacement (the
displacement when the sensor was installed). Another possibility is that as the angle of the sun shifts throughout the day, some transverse gradients are induced in the structure, causing lateral bending. The shift in the tilt and north data occurs during a two week period where the structure was not being monitored due to other obligations of the monitoring system.

Figure 6.12 shows two views of the temperature signature for P12h13 temperature and strain with vertical displacement.

Figure 6.12 Tilt with P12h13 temperature and strain, 5 time period shown

The colors in Figure 6.12 correspond to data sets of continuous monitoring. The shift in tilt occurs in December of 2016 and is maintained through time periods afterwards. These signatures can be fit using a linear polynomial surface, shown in Equation 6.7.

\[
\delta(T, \varepsilon) = a + bT + c\varepsilon + dT^2 + eT\varepsilon
\]  \hspace{1cm} (6.7)

Displacement (\(\delta\)) is expressed as a function of temperature (T) and strain (\(\varepsilon\)), with a series of coefficients (a,b,c,d,e), solved from minimizing the least square error. The degree of the polynomial is chosen to increase the goodness of fit (determined by \(R^2\)) and reduce
the error (RMSE) without overfitting the data. The lowest degree that achieved significant improvement in $R^2$ and RMSE was second-degree in temperature and first-degree in strain. Figure 6.13 shows the polynomial surface fit on data taken before the shift in tilt measurement.

![Figure 6.13 P12h13 strain and displacement with tilt, and best fit polynomial surface. Data shown from April to November 2016, ending before shift](image)

The shift in tilt measurement occurs in when Streicker Bridge was not monitored in December 2016. The coefficients of the surface, $R^2$, and RMSE each detect some change at this shift. Table 6.3 shows the polynomial surface fit for the accumulation of data throughout 2016 and 2017. Each row corresponds to a continuous time period of monitoring and is combined with all previous (above) rows in finding the current surface at that time. The signature does not immediately find a stable and good surface fit, as the signature is still filling out behavior at different temperatures. By November 6 and November 28 the signature has begun to take form, with a consistent set of coefficients and an $R^2$ of 0.96. Adding the next time period changes coefficients a and b, decreases the $R^2$, and increases the RMSE (highlighted in red).
The shift affects the intercept term (a) the most, and the addition of every subsequent time period maintains this change in a, b, R^2, and RMSE. Table 6.4 compares the fit for the data before and after the shift with the surface fit for the entire data set. The pre-shift and post-shift data show very similar statistics, while the combined data set has an increased error, decreased goodness of fit, and change in coefficients a and b. Table 6.4 shows that before and after the shift the signature is behaving the same way.

The vertical temperature signature has shown to be sensitive to unusual behavior, as the change in statistics of the surface fit of the signature identified the shift in measurement.

These vertical displacement signatures highlight the importance of monitoring the structure during extreme temperature events. Unusual thermal behavior frequently
happens during temperature extremes, and this behavior can significantly affect temperature signatures.

6.5 Transverse Displacement

The transverse displacement sensor recorded the smallest movement at the abutment out of all displacement sensors. Figure 6.14 shows the transverse displacement recorded from 2016 to 2017 with the average temperature on the SE leg of Streicker Bridge.

![Figure 6.14](image)

The transverse displacement does not correlate well with temperature, strain, curvature, thermal gradient, or other displacement readings. Even though the transverse displacement does not incorporate well into a three-dimensional signature, it is useful to know the nature of the transverse movement at the support. The tilting at very cold temperatures corresponds with the greatest transverse movement of the SE leg, corroborating the hypothesis that the curve geometry of the bridge leg is causing the tilt.

6.6 Summary

This chapter explores several types of three-dimensional temperature signatures as damage sensitive feature for a temperature driven method of SHM. The signatures
employ combinations of strain, temperature, and displacement to describe some portion of the structural behavior across a range of temperatures. Temperature, strain, and displacement form the most important temperature signature, as lines in three-dimensional space. A three-dimensional signature was formed using real data from the Streicker Bridge and compared to past time periods using real temperature and strain with a displacement generated from strain integration. Good agreement was found with one past signature, as others were identified as having unusual behavior. Longitudinal displacement can also form a signature with thermal gradient and elastic curvature. These signatures form a plane and are quantifiable for real data but not suited for comparison with the generated displacement for past time periods. Vertical displacement was converted into tilt at the abutment and compared with temperature and strain. A three-dimensional polynomial surface described the signature and was able to distinguish a shift in the tilt of the abutment. These temperature signatures have the ability to identify unusual structural behavior, though no damage was detected on the Streicker Bridge.
This work began as an exploration of temperature effects on structures with the idea that temperature could be centrally used in SHM. This work has confirmed the dominant role that temperature plays in structural behavior and proposed a method for incorporating temperature as the driving force in SHM.

7.1 Conclusions and Contributions

This work presents a novel method for monitoring structures using a temperature driven method for SHM. Benefits of this method include the universality of temperature effects on structures, the relative ease of measuring temperature for a complete input-output model, and a baseline structural behavior for a wide range of environmental influences.

First, two methods for identifying time periods of minimal thermal gradient were compared for their ability to improve three-dimensional temperature signatures. Minimizing the effects of non-linear thermal gradients allowed for an accurate static analysis of the structure and can improved the fit of the three-dimensional signatures. Thermal gradients were identified by having either large ranges of temperature on the structure or by having large thermal gradients at each cross section. These methods had variable bounds for declaring a time period as having minimal thermal gradient in order to balance the desire to remove all gradients with the need for a data set of a viable size for analysis.

Next, a novel method for the evaluation of the coefficient of thermal expansion of built concrete structures was developed using temperature and strain data taken from the
Streicker Bridge. Concrete in practice often has some variability in CTE, and this method attempts to identify an effective CTE encapsulating the temperature-strain relationship for the entire section including prestressing tendons, rebar, and voids. This method employed a series of analytic techniques to maintain the validity of the assumptions of negligible rheological strain, negligible changes in mechanical strain, and effects from thermal gradients. By linearly interpolating strain and temperature at the centroid, recording measurements as an average value of five minutes, and assessing the CTE of a sliding six-hour window, the CTE was evaluated over six years of monitoring for cross-sections on the SE leg of Streicker Bridge. Cross-sections near the main span had CTE evaluations within typical values for the CTE of concrete and had a consistent behavior specific to each season (summer, fall, and winter). There was consistency over the years with each location for each season, but variance over the span of the SE leg. This difference in evaluation for various sections could be from actual variability in the concrete CTE or a relic from some unnoticed thermal effect.

Finally, several three-dimensional temperature signatures were created for the Streicker Bridge, employing combinations of temperature, strain, and displacement at the abutment of the SE leg. The quantification of these three-dimensional plots, in the form of a best fit equation, and goodness of fit metrics, provides an avenue for understanding the thermal behavior for the structure and monitoring the structure for damage. One type of signature combined temperature, strain, and longitudinal displacement to form a line in three-dimensional space. This signature was quantified using real data from Streicker Bridge and then compared to previous time periods using strain and temperature from the bridge with a generated displacement from the integral of the strain of the SE leg.
comparisons were able to detect unusual behavior in the abutment by identifying changes in the line of best fit and a large RMSE. Other signatures compared temperature and strain to vertical displacement in the form of the tilt or rotation of the SE leg at the abutment. These signatures were fit using a linear polynomial surface and were able to detect a shift in title measurement. The temperature signatures demonstrated the ability to detect unusual behavior in structure and emphasized the importance of monitoring structures at extreme temperatures where behavior can be difficult to predict.

7.2 Future Work

This work can be expanded in several areas. First, the CTE evaluations do not consider change in temperature and other locations in the structure. An extension of the proposed method that considers the full distributions of temperature on a specific section will be better able to explain the temperature-strain relationship at each section. This type of work could get closer to a material level evaluation of the CTE at each point in the structure. Additionally, some form of validation for the CTE evaluations would greatly increase the impact of the work. Taking a core sample of Streicker or any active bridge is unlikely, but comparison with some form of NDE may be possible in the future.

The temperature signatures are the most promising aspect of this work. Future work can continue to grow these signatures and develop a specific procedure for creating a temperature signature and then assess whether data from a new time period should be added to the signature as healthy behavior or rejected as unusual behavior or damage. The work performed using Streicker Bridge was limited by the amount of displacement data recorded and by only recording displacements at the abutment of the SE leg. Current advances in photogrammetry and precision displacement monitoring using remote
sensing could allow for displacement measurements taken at every point in the Streicker Bridge and enable the creation of temperature signatures using displacement for each specific location. The work presented in this thesis detected unusual behavior at the bearings but did not detect any damage in the structure. It is likely that there is no damage in the structural to detect, but the sensitivity to damage of the temperature signatures has not been rigorously validated. Future work could involve an in-depth sensitivity study, using a finite element model to assess how sensitive these three-dimensional signatures are to changes in stiffness at the abutment or damage in the material. Also, inducing some reversible unusual behavior to simulate damage to Streicker Bridge to be detected using three-dimensional signature would provide a final validation to the TD-SHM method.
Appendix A  Sensor Validation and Regeneration

This section is based on a conference paper presented at the International Workshop on Structural Health Monitoring by the author in 2017. *Long term sensor malfunction detection and data regeneration using autoregressive time series models* (Reilly and Glisic 2017)

A.1 Introduction

Structural Health Monitoring (SHM) uses a combination of installed sensors, data analysis, and knowledge of the structure to assess the integrity of that structure. Critical to any monitoring system is the reliability of the sensors and collected data. A poor integrity of data can lead to incorrect conclusion on the behavior and safety of a structure. Sensor manufacturers rigorously test their products in order to provide the sensor sensitivity, repeatability, and fatigue life to consumers, but it is inevitable that there will be some level of sensor error in any application. Sensor malfunction can cause loss of productivity in monitoring applications by producing inaccurate data that can lead to incorrect assessments and in some cases by causing a pure loss of data. This study uses a series of autoregressive time series models to mitigate any productivity loss in the monitoring process.

Time series models have been used for many applications in SHM. The general procedure involves training the model on data from a healthy structure and then inputting data from an unknown structural state in order to compare the distributions of the residual errors (Sohn and Farrar 2001). More recent research focuses on using more complex models, such as generalized auto regressive conditional heteroscedasticity models (Chen and Yu 2013), examining different methods for choosing the model order (Mei and Gül 2016), or controlling and predicting the levels of environmental and model uncertainty.
(Buren et al. 2017). Similar techniques have been used for temperature sensor data validation and even finding a method for correcting drifting data measurements (Abdel-Jaber and Glisic 2016), though little work exists on regenerating data from failed sensors.

This work presents a method for the detection of sensor malfunction and the regeneration of data for some cases of sensor failure based on autoregressive time series models. First, an autoregressive (AR) model of each individual sensor detects drifting or malfunctioning sensors. Next, a multivariate AR model regenerates data from malfunctioning sensors to produce viable data for analysis. These methods are validated on temperature sensors and data from a real structure: Streicker Bridge on the campus of Princeton University.

### A.2 Detection

A basic overview of the detection of sensor malfunction using AR time series models is presented here. As this work focuses on the more novel aspect of regenerating sensor data and this detection process very closely follows conventional damage detection protocols (Farrar and Worden 2012), this section is kept brief.

An AR time series model, shown in Equation A.1 (Carmona 2014), was trained for each individual sensor over a training period of one year.

\[
\hat{X}_t = \sum_{i=1}^{p} \alpha_i X_{t-i} + \epsilon_t \tag{A.1}
\]
The AR model makes a prediction ($\hat{X}_t$) of the signal ($X_t$) at each time step (t) based on past time steps of the signal, and weights ($\alpha$). The weights are solved by finding the least squares solution to Equation 1 in terms of the residuals ($\varepsilon_t$). The order (P) of the model is identified through examining the effects of different orders on the root mean square error (RMSE) of the residuals of the training data sets. An adequate model order will minimize the RMSE, while still being as low of an order as possible. With a chosen model order, the weighted average of the weights of each training data set form the model for detection. A similar process can be seen in more detail in (Buren et al. 2017).

The RMSE of the residuals also provides a rudimentary damage indicator, though this case pertains to sensor malfunction and not damage. To test for malfunction, each new data set is fed through the trained model. The residuals between the predictions and actual data are sensitive to damage in the initial signal. In particular, the RMSE of the residuals should increase for changes in the signal. Figure A.1, below, shows the RMSE for data sets taken from the Streicker Bridge between 2010 and 2017 for one temperature sensor, P10MS-UP.
The AR model of order ten was trained using data from the first year of measurement, shown in green, where there is assumed to be no sensor malfunction. In 2016, there is a noticeable change in RMSE, shown in red, possibly indicating a damaged or malfunctioning sensor. Figure A.2, below, shows one of the data sets from 2016 where P10MS-UP displays a clear malfunction.

Figure A.2 shows the malfunctioning sensor, P10MS-UP, with five other nearby temperature sensors. P10MS-UP shows a clear malfunction, as it starts to measure 20, 30,
and even 40 °C less than its adjacent sensors. Sensor malfunction can sometimes become clear when individually examining sensors, as in Figure A.2, but this process is not time efficient for a large number of sensors or data sets. In many cases sensor malfunction can be much more subtle than in Figure A.2, requiring more precise identification than visual inspection. The process here is applied to temperature as opposed to strain data. With strain data it can be difficult to decipher whether the changes detected by the model correspond to a change in state of the structure or malfunction of the sensor.

A.3 Regeneration

The data regeneration process involves a modified form of a multivariate AR model, shown in Equation A.2.

\[
\hat{Y}_t = \sum_{i=1}^{P} \alpha_i Y_{t-i} + \sum_{j=0}^{P} \sum_{i=k}^{N} \beta_{j,k} X_{t-j,k} + \epsilon_t
\]  

(A.2)

The signal to be regenerated, \(\hat{Y}_t\), relies on the past P predictions of the signal, \(Y_{t-i}\), as well as the past P values of N nearby sensors \(X_{t-j,k}\). The nearby sensors at the current time, t, also contribute to the prediction. The weights of the model, \(\alpha\) and \(\beta\), are found again by finding the least squares solution to Equation A.2 in terms of the residuals, \(\epsilon_t\). After choosing a model order and training the model for a period of one year, the predictions of the model, \(\hat{Y}_t\), can be used as a regenerated signal for the malfunctioning sensor. To gauge the potential accuracy of this regeneration method, Figure A.3 shows the regeneration of a healthy data set not part of the training data using five nearby sensors with an order of ten.
The solid red line in Figure A.3 represents the predictions from the model, to be compared to the dotted red line representing the actual data. The residuals between the prediction and actual signal have a mean of -0.38 °C and a standard deviation of 0.23 °C. These residuals and a visual inspection of the fit in Figure 3 show a relatively good prediction for sensor P10MS-UP. Different data sets had different levels of accuracy in their prediction, varying based on the season and the extremeness of the swings in temperature. This level of accuracy, though, can be held as a potential for the prediction model. As with the malfunctioned sensors, it can be difficult to numerically assess the validity of the predictions. Figure A.4 shows the regeneration for sensor failure occurring in 2016.
The regeneration of P10MS-UP (black) provides a much more reasonable data set for temperature analysis than the failed sensor (red). The model uses an initial condition from the failed sensor, as can be seen in the way the two lines have the same beginning. The regeneration is quickly pulled into normal sensor behavior by the model, following the adjacent sensors closely. This regeneration has the largest benefits where there is severe sensor malfunction but is still applicable for cases of mild malfunction or sensor drift.

A.4 Summary

This appendix presents a method for the detection of long-term temperature sensor malfunction and data regeneration. Sensor malfunction is detected by a series of AR time series models trained by early, healthy years of measurement. Changes in the RMSE of the residuals shows malfunction in the sensors. To regenerate the sensor data, a modified multivariate AR model relates nearby sensors to the failed sensor trained during healthy years. The predictive model shows strong accuracy during healthy years of service for the failed sensors, making it likely that the model produces an accurate regeneration of the
sensor during the failed years. Future work will examine more complex time series models as well as more refined damage indicators.
Appendix B  CTE Stiffness Calculations and Sensitivity Studies

B.1 Stiffness calculations

Under a uniform temperature change, the total strain on the material is the sum of the linear thermal strain and the mechanical response $\varepsilon_{\text{M, Thermal Response}}$ (provided by the partial restraint at the support) to the thermal expansion, shown in Equation B.1

$$\varepsilon_{\text{Total}} = \varepsilon_{\text{Thermal}} + \varepsilon_{\text{M, Thermal Response}} \quad (B.1)$$

The thermal response related to the force (F) provided by the spring, as a function of the total displacement ($\delta$) and the stiffness of the spring ($K_s$). Assuming a linear Hookean spring provides Equation B.2. The displacement can be related to the total strain by the length (L) of the beam in Equation B.3. Converting the force to strain by dividing by the area and Young’s modulus and substituting in from Equation B.2, B.3, and B.4 produces Equation B.5.

$$F = K_s \delta \quad (B.2)$$

$$\delta = \varepsilon_{\text{Total}} L \quad (B.3)$$

$$K_B = \frac{AE}{L} \quad (B.4)$$

$$\varepsilon_{\text{M, Thermal Response}} = \frac{F}{AE} = \frac{K_s \delta}{AE} = \frac{K_s \varepsilon_{\text{Total}} L}{AE} = \frac{K_s \varepsilon_{\text{Total}} L}{K_B} \quad (B.5)$$

Finally, substituting in yields Equation B.6, which can be simplified as Equation B.7.

$$\varepsilon_{\text{Total}} = \alpha \Delta T + \frac{K_s \varepsilon_{\text{Total}} L}{K_B} \quad (B.6)$$
\[
\varepsilon_{Total} = \frac{\alpha}{\left[1 + \frac{K_S}{K_B}\right]} \Delta T
\]  
(B.7)

B.2 IQR values for CTE evaluations

Table B.1. Summer IQR values

<table>
<thead>
<tr>
<th>Date</th>
<th>P10</th>
<th>P10h11</th>
<th>P10qqq11</th>
<th>P11</th>
<th>P11h12</th>
<th>P12</th>
<th>P12h13</th>
<th>P13</th>
</tr>
</thead>
<tbody>
<tr>
<td>July 8-Aug 5, 2010</td>
<td>1.59</td>
<td>2.28</td>
<td>2.20</td>
<td>2.67</td>
<td>2.17</td>
<td>3.69</td>
<td>2.97</td>
<td>3.41</td>
</tr>
<tr>
<td>June 30 -July 21, 2011</td>
<td>NaN</td>
<td>1.57</td>
<td>2.17</td>
<td>2.17</td>
<td>1.73</td>
<td>3.21</td>
<td>3.02</td>
<td>4.43</td>
</tr>
<tr>
<td>June 7-18, 2012</td>
<td>2.12</td>
<td>1.54</td>
<td>2.34</td>
<td>2.29</td>
<td>1.55</td>
<td>2.68</td>
<td>2.51</td>
<td>3.17</td>
</tr>
<tr>
<td>July 8-21, 2013</td>
<td>2.19</td>
<td>1.60</td>
<td>2.41</td>
<td>2.44</td>
<td>2.04</td>
<td>2.43</td>
<td>1.67</td>
<td>3.80</td>
</tr>
<tr>
<td>July 2-25, 2014</td>
<td>2.21</td>
<td>1.83</td>
<td>1.94</td>
<td>2.51</td>
<td>1.61</td>
<td>3.81</td>
<td>1.93</td>
<td>8.97</td>
</tr>
<tr>
<td>Aug 8-Aug 29, 2016</td>
<td>1.61</td>
<td>2.81</td>
<td>NaN</td>
<td>NaN</td>
<td>1.70</td>
<td>1.84</td>
<td>2.24</td>
<td>3.99</td>
</tr>
<tr>
<td>June 8-July 7, 2017</td>
<td>1.46</td>
<td>3.77</td>
<td>1.49</td>
<td>NaN</td>
<td>NaN</td>
<td>2.05</td>
<td>2.91</td>
<td>3.29</td>
</tr>
</tbody>
</table>

Table B.2. Fall IQR values

<table>
<thead>
<tr>
<th>Date</th>
<th>P10</th>
<th>P10h11</th>
<th>P10qqq11</th>
<th>P11</th>
<th>P11h12</th>
<th>P12</th>
<th>P12h13</th>
<th>P13</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oct 8-20, 2010</td>
<td>3.41</td>
<td>3.21</td>
<td>2.10</td>
<td>2.30</td>
<td>1.31</td>
<td>3.50</td>
<td>4.25</td>
<td>3.36</td>
</tr>
<tr>
<td>Oct 5-28, 2011</td>
<td>1.90</td>
<td>2.12</td>
<td>2.37</td>
<td>2.10</td>
<td>1.23</td>
<td>3.22</td>
<td>2.30</td>
<td>NaN</td>
</tr>
<tr>
<td>Sept 26-Oct 8, 2012</td>
<td>2.26</td>
<td>1.78</td>
<td>1.85</td>
<td>2.32</td>
<td>0.00</td>
<td>2.31</td>
<td>4.44</td>
<td>NaN</td>
</tr>
<tr>
<td>Oct 9-19, 2013</td>
<td>2.24</td>
<td>1.92</td>
<td>2.21</td>
<td>0.78</td>
<td>1.57</td>
<td>1.83</td>
<td>1.53</td>
<td>0.77</td>
</tr>
<tr>
<td>Oct 19-Nov 6, 2016</td>
<td>2.63</td>
<td>3.51</td>
<td>NaN</td>
<td>NaN</td>
<td>2.61</td>
<td>0.95</td>
<td>2.07</td>
<td>1.99</td>
</tr>
</tbody>
</table>

Table B.3. Winter IQR Values

<table>
<thead>
<tr>
<th>Date</th>
<th>P10</th>
<th>P10h11</th>
<th>P10qqq11</th>
<th>P11</th>
<th>P11h12</th>
<th>P12</th>
<th>P12h13</th>
<th>P13</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dec 20, 2010-Jan 15, 2011</td>
<td>1.85</td>
<td>2.12</td>
<td>2.39</td>
<td>2.24</td>
<td>1.78</td>
<td>1.72</td>
<td>1.54</td>
<td>1.27</td>
</tr>
<tr>
<td>Jan 26-Feb 22, 2012</td>
<td>1.74</td>
<td>2.14</td>
<td>2.13</td>
<td>1.65</td>
<td>1.39</td>
<td>4.01</td>
<td>2.48</td>
<td>1.57</td>
</tr>
<tr>
<td>Jan 25-Feb 6, 2013</td>
<td>3.03</td>
<td>3.37</td>
<td>2.28</td>
<td>2.90</td>
<td>NaN</td>
<td>1.40</td>
<td>2.16</td>
<td>NaN</td>
</tr>
<tr>
<td>Jan 16-Jan 25, 2014</td>
<td>2.19</td>
<td>2.49</td>
<td>2.16</td>
<td>2.53</td>
<td>3.55</td>
<td>2.20</td>
<td>2.39</td>
<td>2.57</td>
</tr>
<tr>
<td>Jan 5-Jan 27, 2017</td>
<td>2.59</td>
<td>3.28</td>
<td>2.47</td>
<td>NaN</td>
<td>2.91</td>
<td>1.74</td>
<td>3.17</td>
<td>2.11</td>
</tr>
</tbody>
</table>
8 Bibliography


American Concrete Institute, and ACI Committee 209--Creep and Shrinkage (Eds.). (2008). Guide for modeling and calculating shrinkage and creep in hardened concrete. ACI standard, American Concrete Institute, Farmington Hills, MI.


