November 2016

Detailed Study of Integral Abutment Bridges and Performance of Bridge Joints in Traditional Bridges

Brooke H. Quinn

DETAILED STUDY OF INTEGRAL ABUTMENT BRIDGES AND PERFORMANCE OF BRIDGE JOINTS IN TRADITIONAL BRIDGES

A Dissertation Presented

by

BROOKE QUINN

Submitted to the Graduate School of the
University of Massachusetts Amherst in partial fulfillment
of the requirements for the degree of

DOCTOR OF PHILOSOPHY

September 2016

Civil and Environmental Engineering

DETAILED STUDY OF INTEGRAL ABUTMENT BRIDGES AND PERFORMANCE OF BRIDGE JOINTS IN TRADITIONAL BRIDGES

A Dissertation Presented

by

BROOKE QUINN

Approved as to style and content by:

____

Scott A. Civjan, Chair

____

Sergio F. Breña, Member

____

Peggi L. Clouston, Member

____

Richard N. Palmer, Department Head

Civil and Environmental Engineering

ABSTRACT

DETAILED STUDY OF INTEGRAL ABUTMENT BRIDGES AND PERFORMANCE OF BRIDGE JOINTS IN TRADITIONAL BRIDGES

SEPTEMBER 2016

BROOKE QUINN, B.S., UNIVERSITY OF MASSACHUSETTS AMHERST

M.S.C.E., UNIVERSITY OF MASSACHUSETTS AMHERST

Ph.D., UNIVERSITY OF MASSACHUSETTS AMHERST

Directed by: Professor Scott A. Civjan

Integral Abutment Bridges (IABs) are jointless bridges in which the superstructure is cast monolithically with its substructure. Eliminating expansion joints from the superstructure reduces corrosion of bridge elements that typically result from leaking joints in traditional bridges. IABs have proven to be cost effective for both construction and life-cycle analysis. As a result, they are the standard single span highway bridge of choice by the majority of State Departments of Transportation (DOTs) across the country. Despite the widespread use of these bridges, there are no uniform guidelines in place. Factors such as pile orientation, design assumptions, maximum span length, skew angle, and curvature vary widely. A study of expansion joint performance was done to investigate typical problems with joints through information collected from meetings with Massachusetts DOT as well as survey results collected from DOT personnel from nine states in and around New England. Results highlight the many issues associated with expansion joints which have resulted in the preference to construct IABs whenever possible. The Vermont Agency of Transportation (VTrans) instrumented three IABs of increasing complexity for long term monitoring and analysis of their performance. The bridges include a straight bridge with 141 ft (43 m) span, a 15 degree

skew bridge with 121 ft (37 m) span, and a two-span continuous curved structure with 11.25 degrees of curvature and 221 ft (68 m) total bridge length. This dissertation presents over five years of field data. Results are compared with three-dimensional finite element model predictions. Variations in response due to skew, curvature, and field conditions are addressed. The finite element models were the basis for a parametric study investigating the effect of pile orientation on IABs of varying length and skew angle. Results highlight the factors that affect optimal pile orientation to avoid pile yielding.

TABLE OF CONTENTS

LIST OF TABLES

Page

LIST OF FIGURES

Page

CHAPTER 1

INTRODUCTION

Integral Abutment Bridges (IABs) are jointless bridges in which the superstructure deck is cast monolithically with the substructure. IABs have been constructed by State Departments of Transportation (DOTs) for years, serving as a cost-effective alternative to traditional jointed bridges. Jointed bridges are known to have expensive repairs due to water seepage through expansion joints which leads to corrosion of the joint itself as well as other superstructure and substructure elements. The annual direct cost of corrosion damage in the United States was estimated to be $8.3 billion in 1998, with indirect costs of this damage estimated to be up to ten times greater (Koch et al. 2001). IABs have numerous attributes and few limitations, offering lower construction and maintenance costs while remaining in service for longer periods of time with only occasional repairs and minimal maintenance required (Arsoy and Duncan 1999) (Mistry 2005). As a result, they have become increasingly popular and are now the standard single span choice in the majority of states. However, common design guidelines for these bridges are lacking and design provisions are often based on individual state's experiences (Kunin 2000).

A detailed survey (Maruri and Petro 2005) reported the variations in design standards of IABs through responses compiled from thirty-nine states. For steel girder fully-IABs: the allowable maximum span length ranged from 65 ft (19.8 m) to 300 ft (91.4 m), maximum skew angle ranged from 15 to 70 degrees, and maximum curvature ranged from 0 to 10 degrees. The survey also highlighted differences in how forces are accounted for. Of the responding states, 28 percent did not account for temperature related forces, 41 percent did not account for passive earth pressure, and 59 percent did not account for additional forces due to skew angle. There was no consensus on preferred orientation of piles; 33 percent of states reported orienting the piles with the strong axis parallel to the centerline of bearing, 46 percent orient the piles with the weak axis parallel to the centerline of bearing, 8 percent leave it to the discretion of the Engineer, and the remaining 13 percent did not comment or noted use of symmetric piles (Maruri and Petro 2005).

The design of the substructure components of IABs is generally governed by long term cyclic thermal loading and the resulting frame action of the structure. However, soil properties are complex and non-linear making it difficult to predict soil-structure interaction. The hysteretic response of soil under cyclic thermal load is not clearly defined for mixed soil types typical of backfill material; this non-linear response of soil is further complicated by dependencies on load history and rate of loading.

Finite Element Modeling is a useful tool for analysis of these bridges and gives insight into their behavior under varying conditions of both load and geometry, however analyzing field data of existing IABs is essential in understanding their behavior, and can be used to validate accuracy of models and address variations in response that may not be captured in typical analysis.

The objectives of this dissertation are to: highlight the associated issues with bridge expansions joints and provide recommendations on improving performance of bridge joints when they are needed, provide a detailed analysis of the performance on

IABs through use of field data and FEMs, and use the results to provide recommendations on design of IABs.

A study on bridge expansion joints is presented in Chapter CHAPTER 2. This study investigates the problems associated with expansion joints using information collected from meetings with Massachusetts DOT personnel, as well as survey responses from DOT personnel from nine states in and around New England. The many issues, and associated costs, related to expansion joints have resulted in the majority of DOTs preference to eliminate joints and construct IABs whenever possible. However, the change from traditional jointed bridges to IABs takes time, and IAB construction is not always possible due to limitations in design that vary across the country. Therefore, recommendations are provided for improving joint performance.

The Vermont Agency of Transportation (VTrans) initiated a program of field instrumentation and analysis to evaluate the performance of three IABs beginning at construction, and monitored for over five years. The bridges are of increasing complexity, a straight girder non-skew bridge, a straight girder 15 degree skew bridge, and a curved girder two-span continuous structure with 11.25 degrees of curvature. Details of the three bridges are presented in Chapter CHAPTER 3. Three-dimensional finite element models (FEMs) of the three bridges were created using SAP2000, details are presented in Chapter CHAPTER 4. The long-term response of these bridges, including girder stresses, longitudinal and transverse bridge movement, abutment rotations, earth pressures, substructure displacement and pile bending moments are presented in Chapter CHAPTER 5. Results are compared to FEM predictions and highlight variations in response that are not captured using the nominal FEMs. The FEMs are calibrated to field data, and the results provide insight into the soil response to cyclic thermal loading. A further in-depth analysis of the straight and skewed bridge is presented to highlight similarities and differences in response, and discuss bridge behavior that is affected by skew.

A parametric study investigating the effects of pile orientation in single-span IABs of varying length and skew is presented in Chapter CHAPTER 6. FEMs of three bridge lengths, and four skew angles, are analyzed with piles oriented about the weak axis and strong axis to determine which pile orientation is optimal to avoid pile yielding. Results discuss the factors that should be considered in choosing pile orientation, and how they affect bridge response.

CHAPTER 2

STUDY ON PERFORMANCE OF BRIDGE EXPANSION JOINTS

Poor performance is associated with bridge expansion joints. These problems have resulted in the majority of DOTs preferring IABs to traditional jointed bridges. However, constructing IABs is not always possible due to the design limitations, and furthermore changing from traditional jointed bridges to IABs is a long process. Therefore, it is important to understand the problems associated with bridge joints, and causes of these problems, in order to improve the performance and lifespan of bridge expansion joints moving forward.

This chapter presents research conducted in response to a request by the Massachusetts Department of Transportation (MassDOT) regarding the performance of bridge expansion joints. The research included understanding how joints and headers are used and maintained in Massachusetts and several states in and around New England, and what factors and practices have had a positive or negative impact on joint and header performance. This research investigated current practice with bridge expansion joints in Massachusetts and other states in the northeast through a literature review, personal meetings with each of the districts of the MassDOT, compilation of existing data on joints and jointed bridges through the PONTIS and Nation Bridge Inventory (NBI) databases and a survey created and sent to surrounding states. While all DOTs agree that jointless bridges are preferential, the process to converting bridges into IABs could take many years, and some jointed bridges may have skew angles, lengths, or curvature not allowed in current IAB designs, therefore this research was important to investigate steps to extend the life of jointed bridges while they are needed.

2.1 Introduction

Expansion joints play an important role in bridges, allowing the superstructure to expand and contract as it undergoes cyclic thermal changes without generating significant stresses. Bridge joints notoriously suffer deterioration as a result of thermal deformations, impact forces induced by traffic, freeze-thaw cycles and weather conditions such as rain and snow. In the Northeast, the winters can bring months of heavy snow which also means the expansion joints are subjected to road salts and other anti-icing materials. Corrosion of steel superstructure and substructure elements including reinforcement within concrete elements is greatly accelerated when exposed to salts. If expansion joints stop performing properly the elements below are exposed to water and salts. Both the superstructure and the substructure can subsequently be damaged leading to costly repairs and replacements. Therefore, it is important to determine best practices in Massachusetts as well as surrounding states to better understand not only the causes of joint failure, but measures that can be taken to prevent failure and extend the service life of joints.

2.2 Literature Review

Bridge joints have been studied extensively in the past. Two comprehensive studies that were examined as part of this research were the NCHRP *Synthesis of Highway Practice 319*, published in 2003 (Purvis 2003), and a more recent report *Survey of Past Experience and State-of-the-Practice in the Design and Maintenance of Small Movement Expansion Joints in the Northeast*, published in 2014 (Milner and Shenton 2014).

Purvis (2003) includes an extensive description of joint types and classifications, common issues of maintenance, and the instances where various bridge joints are used.

Furthermore, the report presents results from a survey with data collected from 34 respondents from state Departments of Transportation and other similar agencies in 10 Canadian provinces.

The report by Milner and Shenton III contains a comprehensive literature review of multiple prior studies and reports with summaries, key findings and conclusions from each source. This report is a great resource of past research done on bridge joints and also contains information on small movement bridge joints and survey results with data that are specific to small movement bridge joints (defined as less than two inches in the report).

Both resources are recommended for thorough information on bridge joint types, general performance issues and details, as well as fairly complete literature review of available resources. This information is therefore not repeated in this chapter. This research differs from previous research in that it focuses on all bridge joint types (small and large movement), focuses on New England and surrounding states that experience similar weather and collects survey data from a wider group than previous surveys. The last point is important as it was found that responses vary widely between districts within state Departments of Transportation. Regional states' joints are subject to the typical problems experienced with bridge joints everywhere, but have the added element of issues involved with road salts/anti-icing materials and plow damage. The data and survey presented in this chapter address performance of joints types accommodating all ranges of expansion, and the issues of deterioration and maintenance with headers and joints specific to weather conditions experienced in and around New England.

2.3 Importance of Expansion Joints

Bridges are subject to thermal variations that generate thermal expansion and contraction with changing temperatures. In recent years, there has been a move towards constructing integral abutment bridges, or eliminating bridge joints in retrofits wherever possible. This shift in preferred design is in large part due to problems experienced with expansion joints, including damage from leaking/failed joints and the fact that joints require frequent maintenance to keep them functioning. While jointless bridges are a desirable alternative to conventional expansion joints, they are not always a viable option. Jointless bridges generally have skew limitations as well as expansion limitations; when these limitations are exceeded, expansion joints are needed. In addition, there are a large number of joints currently in use across the country and even if state departments of transportation decide to eliminate joints where possible, this process will take a long time, during which existing bridge joints need to function and perform as well as possible.

In conventional jointed bridges, as the superstructure expands and contracts, the movement is accommodated by expansion joints that allow these movements to occur without causing damage to the bridge. While the joints open and close with fluctuating temperatures, they also play a critical role of keeping water and salt from flowing onto the superstructure and channel it away from the bridge while also protecting the substructure components. An ideal joint would be able to perform these tasks seamlessly while causing minimal disturbance to drivers (i.e. sitting flush with the roadway and remaining quiet under vehicle traffic). However, when bridge joints are damaged they can lead to numerous problems including causing damage to other bridge elements, often leading to costly and time consuming repairs.

Keeping drains maintained and cleaned is important to prevent them from filling up with debris which inhibits them from performing the task for which they were designed. Similarly, washing out and cleaning joints is important to prevent the build-up of debris. When joints continuously fill up with debris, road salts/sands, and other materials, it not only prevents the joints from expanding/contracting correctly but can result in tearing the joint material and/or corrosion which can both cause leakage. The leakage can prevent the joint from performing correctly and cause serious damage to the substructure which, if left un-treated, could threaten the integrity of the bridge itself.

2.3.1 Deterioration Resulting from Damaged Joints

When joints stop performing correctly and allow salt water and anti-icing products to pass through to the substructure, they can cause expensive and extensive damage to the steel and concrete below. Joint failure can be costly in regards to repairing or replacing the joint, but the cost can extend far beyond the joint itself if it fails to protect the superstructure and substructure elements. The images shown in this section come from MassDOT inspection reports. Figure 2.1 shows substructure damage under a leaking deck joint where there has been section loss and web crippling.

Figure 2.1: Section Loss and Web Crippling at Deck Joint

An example of superstructure damage is shown in Figure 2.2 where a leaking joint resulted in corrosion and other damage to the beam ends. A way to prevent leakage is to continue the deck over the backwall so that the joint does not leak here; beam end corrosion is a major issue from leaking joints. Figure 2.3 shows substructure damage where leaking joints lead to concrete damage beneath the bridge.

Figure 2.2: Section Loss, Rust Holes and Corrosion Cracks at Deck Joint

Figure 2.3: Substructure Damage to Concrete from Leaking Joint

A report compiling cost of corrosion to the United States in 1998 gives insight into direct and indirect costs of corrosion in infrastructure. At the time the information was compiled, over 87,000 (15%) of the 583,000 highway bridges in the United States were classified as structurally deficient as a result of corrosion. The annual direct cost of this damage was estimated to be $8.3 billion. Indirect costs, such as traffic delays and lost productivity, were estimated to be up to ten times greater than the direct costs (Koch et al. 2001). In 2012, further research was done demonstrating the extent of corrosion damage increasing with time; this research noted that not only the degree of damage increases with time, but the rate at which the damage occurs increases with time as well (NACE 2012). Hence, the annual cost of corrosion from highway bridges has increased significantly since the data was collected in 1998. Proper installation and systematic maintenance of joints could dramatically reduce future bridge expenses.

2.4 Categories of Joint Types

There are two main classifications of bridge joints: open joints and closed joints. Open joints allow water to pass through them into a drainage system, typically a trough, that diverts the water away from the bridge. Closed joints are watertight joints that are designed to prevent any water from getting into the joint. Both joint types have associated issues. In open joints, the drainage troughs are known to fill with debris and become clogged, which can then allow salt water and anti-icing chemicals to overflow, reach substructure components of the bridge and cause damage. In closed joints, salt accelerates the corrosion of reinforced concrete and steel while anti-icing chemicals and debris deteriorate the joint. Traffic issues can also lead to rutting of joints and tearing of seals which then cause leakage to substructure components. When joints are damaged or have failed, they can either be repaired or replaced. In a joint repair, the existing joint remains in place and only the damaged component is fixed. In a joint replacement, the entire existing joint is removed, and may be replaced by a new joint of the same type or by a different type of joint.

The following sections will present a description of joint types that will be addressed in this chapter, including schematic images and examples of damaged or failing joints where applicable. The organization of joint descriptions will include all closed joints, followed by all open joints, and finally a separate category of a “jointless option” as an alternative to expansion joints that could replace the joints in a retrofit or be used in new construction. Joints in each category will be presented in order of increasing expansion accommodations.

2.5 Description of Closed Joints

The following joint types are classified as closed joints. These joints are designed to prevent any water from getting through the joint.

2.5.1 Saw and Seal

Saw and seal joints are intended to accommodate small movements of up to ½”. The joint details are fairly simple; a saw cut is created in the riding surface, and sealant is poured into the opening and allowed to cure. A schematic of the saw and seal joint is shown in Figure 2.4. They are designed using the saw cut detail for minimum to no movement from the bridge manual (MassDOT 2013). The saw cut generally goes 2” into the deck/wearing surface. While saw and seal joints are low maintenance, the most important step during construction is to properly locate where the saw cut needs to be made. Otherwise, cracking occurs adjacent to the joint. One possible remedy is to mark the curb where the deck ends prior to putting down wearing surface so the installer knows where the joint should be. While specifically marking the curb is not in the item for sawing and sealing joints (MassDOT 2013), the special provisions for this item emphasize that the contractor must accurately locate joints by referencing the existing joints before they are covered with hot mix asphalt overlay.

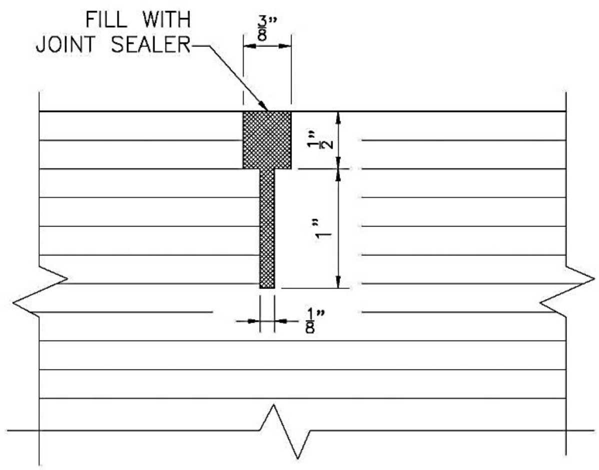


Figure 2.4: Schematic of Saw and Seal (MassDOT 2013)

2.5.2 Asphalt Plug Joints

Asphalt plug joints are generally used for expansion 2” or less and for skew angles less than 30°. Manufacturers do not recommend asphalt plug joints for use under high traffic volume. The general details of asphalt plug joints include a backer rod under a block out connected to a steel gap plate that sits over the block-out (typically by galvanized nails 16d (3.5 inches) or larger). The block-out is then covered in binder and filled with the asphalt plug joint mix. The benefits of these joints include the ease of installation, low cost of installation, low cost of repair and low instance of snow plow damage. They can be installed segmentally so they are not as disruptive to traffic flow as other joint types that require entire installation at one time. There are also multiple problems associated with asphalt plug joints. They do not perform well where they meet curbs, barriers, or parapets because the backer rod is not easily maneuvered at up-turns and they end up with leakage at these locations. When used in heavy traffic, they experience rutting. Plow damage occurs when they heave and rise above grade. If installed in hot weather, the material can soften. There has been debonding at the interface of joint and pavement, and cracking in cold weather. An alternative to a conventional asphalt plug joint (referred to as a “modified asphalt plug joint”) has recently been used in some states. The modified asphalt plug joint uses EM-SEAL (which will be discussed in another section) underneath the asphalt to help create a more watertight joint. A typical asphalt plug joint is shown in Figure 2.5.

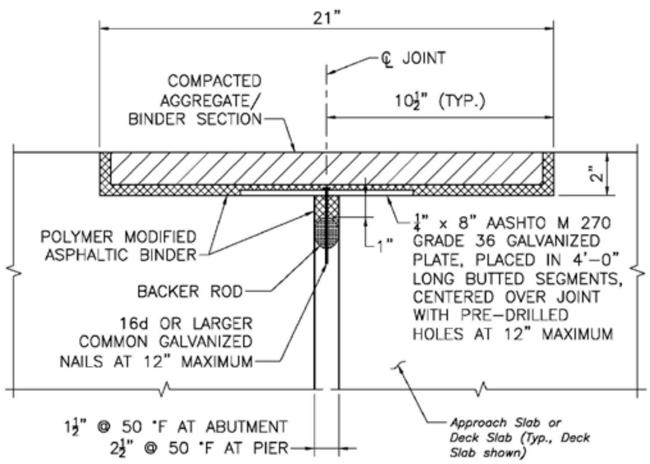


Figure 2.5: Asphalt Plug Joint Details (MassDOT 2013)

2.5.3 Compression Seals

Compression seals can accommodate movement from 0.25” to 2.5”. They rely on continuous pre-formed neoprene elastomeric rectangular section that is installed by squeezing and inserting seal into joint opening. An illustration of a cross-section of an open cell compression seal is shown in Figure 2.6 which demonstrates that compression seals can be installed with or without metal facing. The seal must always stay in compression in order to remain in place and stay watertight.

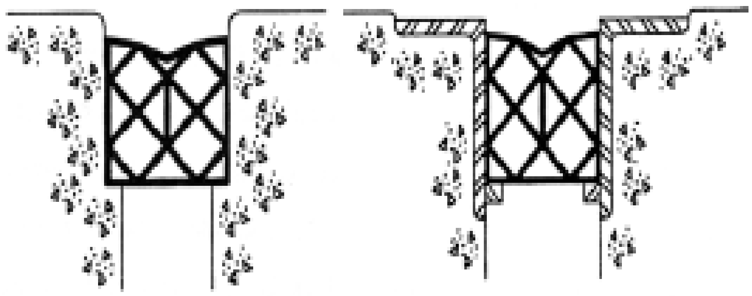


Figure 2.6: Schematic of Compression Seal with and without Facing (Purvis 2003)

If the seal is improperly sized, or if the joint opening is not a uniform width, early failure can occur. Over time, compression seals have been reported to have decreased ability to stay in compression as a result of loss of resilience. This is even more prevalent if the movement is large. Compression seals should not be spliced. Compression seals can be made of closed cell foam or open cell foam. Figure 2.7 shows a failed compression seal from a MassDOT inspection report where 90% of the seal is displaced. The right side of the figure shows the picture of the same joint from below where the seal has fallen through and heavy debris build-up surrounds it.

Figure 2.7: Damaged Compression Seal

2.5.4 Strip Seals

Strip seals can accommodate up to 4” of movement, and can be used on skew angles greater than 30° as opposed to asphalt plug joints. The material of a typical neoprene strip seal is pre-molded into a V-shape that opens and closes with expansion and contraction of the joint. The neoprene is attached to metal facing on either side of the joint and anchored into the edges of the deck slabs. While strip seals are watertight when properly installed, can be used under high traffic volume, and can achieve service lives longer than other joint seals under ideal conditions, they are also subject to a number of issues. They are difficult to replace, as the seal should be completely replaced and not spliced. These joints also have issues where there are sharp changes in geometry. Strip seals experience plow damage often, and have occasionally pulled out of the metal facing. One of the most common problems is that non-compressible debris builds up in the expanded V-shape, and then under joint contraction the debris can tear the seal causing rupture and leakage to the substructure. Typical strip seal details are shown in Figure 2.8.

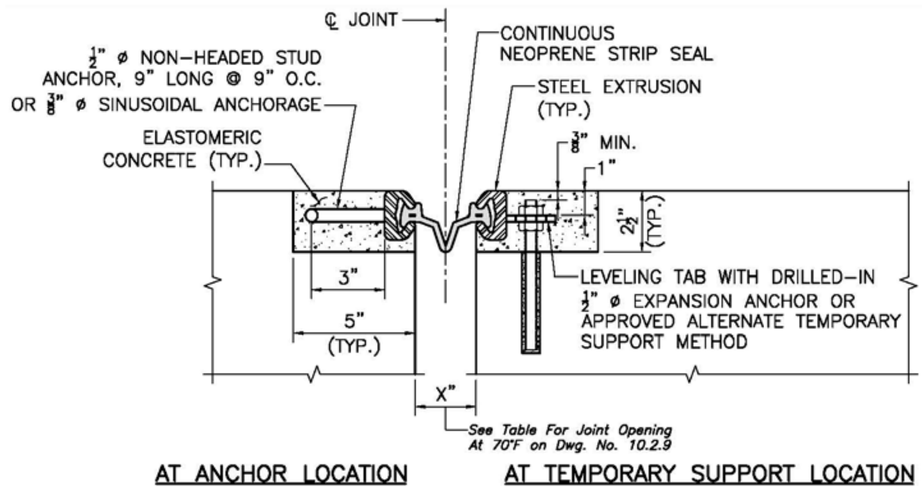


Figure 2.8: Strip Seal Details (MassDOT 2013)

New installation of strip seals specifies the use with elastomeric concrete headers. A picture of the failure of a strip seal from a MassDOT inspection report is shown in Figure 2.9. The figure shows where debris has built up and the seal has torn through in some locations allowing water to leak through.

Figure 2.9: Damaged Strip Seal

2.5.5 EM-SEAL

EM-SEAL is a pre-compressed, watertight, tensionless silicone seal. EM-SEAL can be used in new joint construction, as a replacement for failed strip seals, as part of “modified asphalt joints”, or in other instances where a typical seal joint is used. One of the main benefits of EM-SEAL is that it comes with pre-fabricated corner and transition pieces that can be easily maneuvered up and over curbs and parapets. This results in a continuous watertight seal that is nearly impossible to achieve with a typical backer rod. EM-SEAL comes supplied on a reel for sizes ½” to 1 ¼” and as a straight run stick for sizes 1 ½” to 4”. In Figure 2.10, an EM-SEAL schematic is shown in typical concrete substrates (new or retrofit). Figure 2.11 shows the EM-SEAL being installed by MassDOT. The image also shows the vertical pieces that come for ease of maneuvering over changes in geometry.

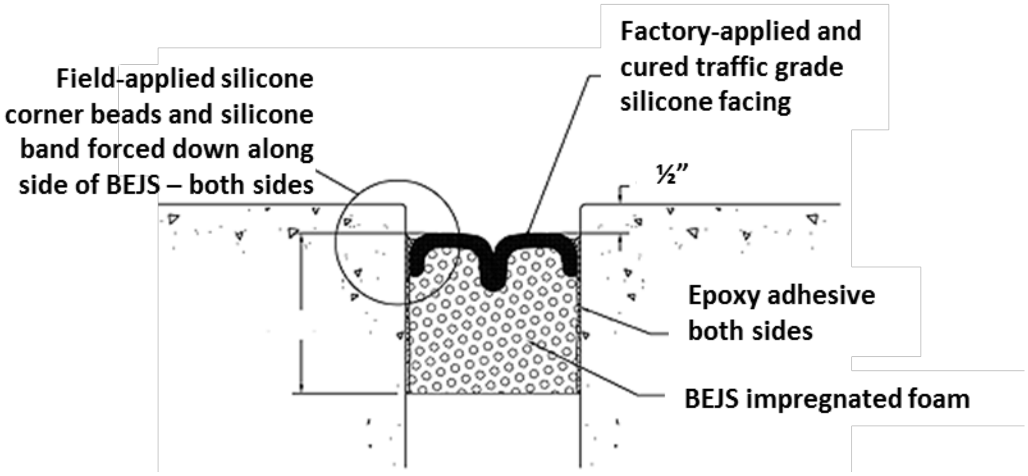


Figure 2.10: Schematic of EM-SEAL (EM-SEAL 2015)

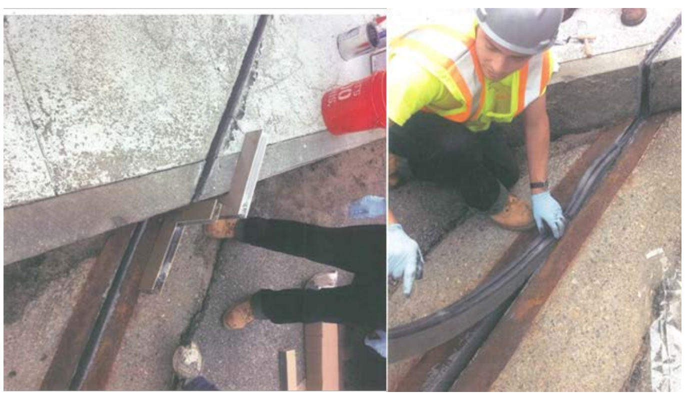


Figure 2.11: Installation of EM-SEAL on MassDOT Bridge

2.5.6 Pourable Seals

Pourable seals accommodate movement up to 4". A simplified cross-section of a pourable seal is shown in Figure 2.12. Generally, silicone is used as a pourable sealant over a backer rod which prevents the sealant from flowing through the joint. Once the sealant molds to the joint opening, and bonds to the sides, it remains flexible and is able to expand and contract. Pourable seals are typically used with elastomeric headers. The joint edges should be clean and sound to ensure proper, tight, bonding.

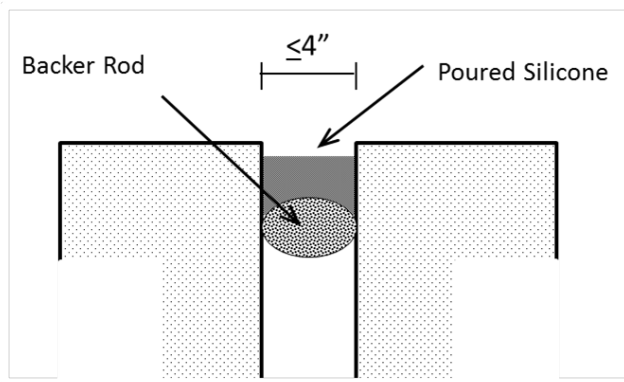


Figure 2.12: Simplified Schematic of a Pourable Seal

One benefit of pourable seals is that the performance is not affected if the joint walls are not perfectly parallel since it will mold into the shape of the irregular opening. The joint is generally easy to repair since just a portion of the seal can be repaired if needed which also minimizes traffic impact. Problems include: de-bonding, splitting, and damage from debris build-up. A picture of a damaged pourable seal from a MassDOT inspection report is presented in Figure 2.13. The seal has debonded and is missing from many areas of the joint, and is now filled up with debris.

Figure 2.13: Damaged Pourable Seal

2.5.7 Modular Joints

Modular joints, like finger joints, are large movement expansion joints that can accommodate movements greater than 4”. Generally, modular joints are made up of multiple strip seals. The system consists of three main components: sealers, separator beams and support bars. The separator beams allow joining of strip seals in series, while the separator beams and sealers create a watertight joint (Purvis 2003). Modular joints are expensive to install, and can have multiple issues. They can experience fatigue cracking of welds, damage to seals, damage from plows, and once the seals are damaged they leak and cause damage to the substructure.

A schematic of a modular joint is presented in Figure 2.14. An image of a damaged modular joint from a MassDOT inspection report is presented in Figure 2.15. The inspection report noted that several support beams were cracked, loose, deflected upward (up to ½”) and the joints were excessively moving under live load. Some steel beams were also missing, and others were broken.

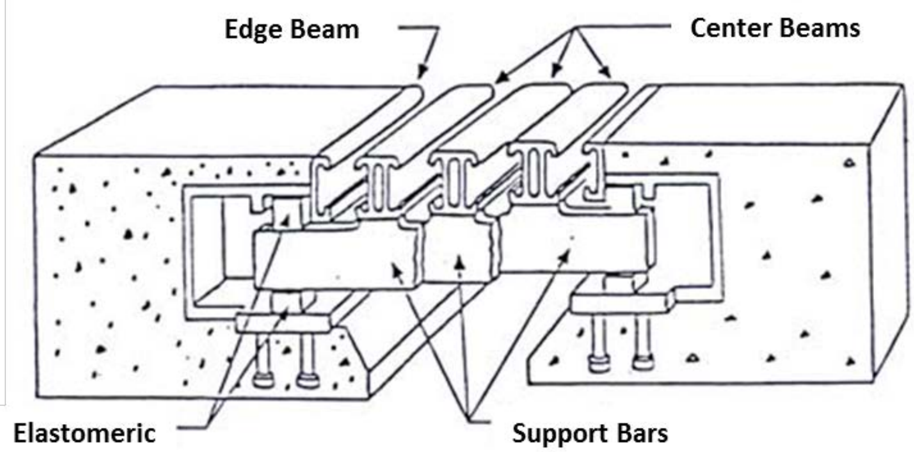


Figure 2.14: Schematic of Modular Joint (Purvis 2003)

Figure 2.15: Damaged Modular Joint

2.6 Description of Open Joints

The following joint types are classified as open joints; these joints allow water to pass through the joint into a drainage system beneath it that is designed to channel the water away from the bridge. While “open joints” are defined categorically, this term will be used in the following chapters to define a basic joint type. Open joints are simply headers with no seal or a joint where instead of replacing a seal that has fallen through it is left as an “open joint” over a backer-rod. The joints described in the following sections, which are categorically open joints, will be referred to by their joint name in proceeding chapters.

2.6.1 Sliding Plate Joints

Sliding plate joints are used to accommodate movement of 1-3”. A simplified schematic is presented in Figure 2.16. In this joint, a steel plate is attached to one side of the joint and extends over the joint opening. The side of the plate that is unattached rests in a slot opposite the attached plate. The joint is anchored into the concrete with welded steel bolts or studs. Common failures include the plates loosening over time and becoming noisy under traffic. There can be loss of support and poor anchorage of the plates. At the slot end of the plate, build-up of debris can occur and pry the plate up over time. Plates and anchors are subject to plow damage. Anchors can corrode and fail from fatigue under traffic.

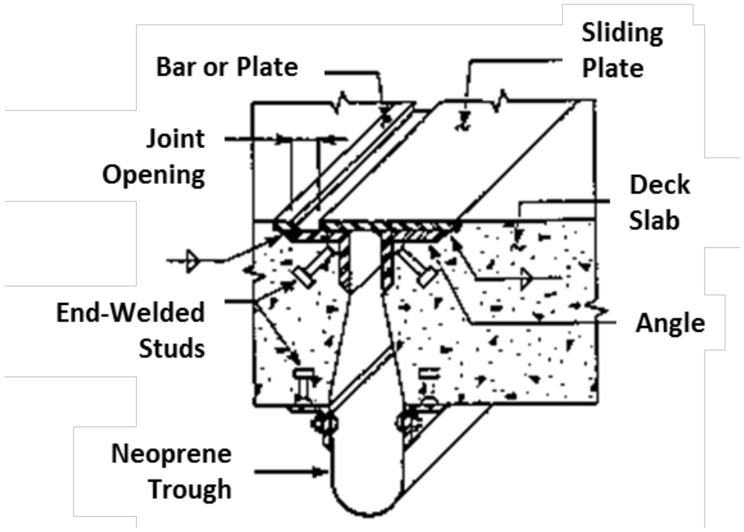


Figure 2.16: Simplified Schematic of Sliding Plate Joint (Purvis 2003)

2.6.2 Finger Joint

Finger joints accommodate movements greater than 4”. They are classified as an open joint since the steel plate fingers that mesh together also move apart and allow water to pass through the joint, typically into a drainage trough. While finger joints typically have longer service lives than most joints, they also have numerous associated problems. Concrete around the joint tends to deteriorate, there can also be anchorage issues and fingers can bend upward when impacted by a plow. These issues can result in increased noise and a rough riding surface. Plows can also catch them and cause damage. The most problematic area of finger joints tends to be the drainage troughs. These troughs are the only barrier between water and the substructure and they are very difficult to maintain which leads to them building up with debris and failing, resulting in corrosion damage to substructure elements. A schematic of a finger joint is presented in Figure 2.17. An example of a finger joint that is not functioning correctly is presented in Figure 2.18. This picture was taken on a Massachusetts bridge where the joint is completely closed at 75°F (well below the maximum temperature in Massachusetts).

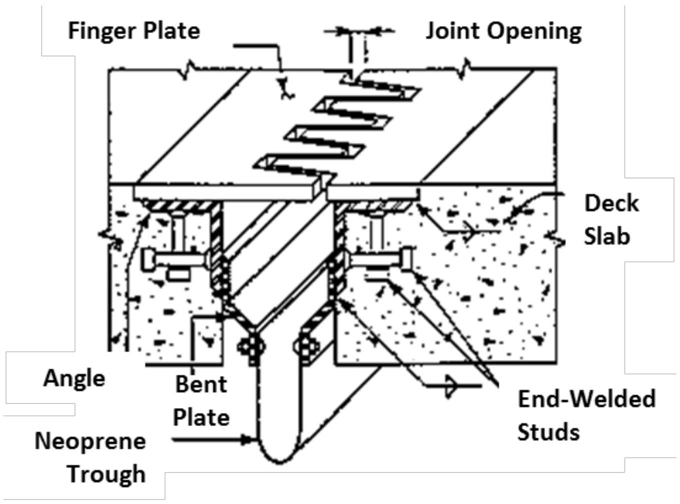


Figure 2.17: Schematic of Finger Joint (Purvis 2003)

Figure 2.18: Finger Joint Completely Closed at 75°F

2.7 Description of a Jointless Option

A preference in new bridge construction is to minimize expansion joints wherever possible. There are varying options for jointless bridges. One option is to create an integral abutment bridge, where the girders are cast monolithically with the abutment and thermal movement is accommodated by the substructure. Most jointless options require complete bridge replacement. The following section details a straightforward method that can be used on existing jointed bridges to make them jointless.

2.7.1 Link-slabs

Link slabs are generally created to connect simply supported spans over piers where each span is supported on elastomeric bearing without anchor bolts. While link-slabs are not necessarily an expansion "joint", they are a good alternative to joints and can be used to replace joints in retrofits, and during deck replacement. The process of connecting the spans in a retrofit would include cutting back concrete to a specified distance to either side of where the girders are supported on their corresponding bearings. In most cases, the concrete decks are connected to the supporting girders by shear studs to make the superstructure composite. Conversely, where the link slab is installed the shear studs are ground down to allow for more freedom of movement over the connecting region. The concrete is cast between the two adjoining decks with reinforcing bars. It is ideal to spray a waterproof membrane on top to prevent water from getting through the small cracks that may occur. The reinforcement is designed to resist bending moment induced by end rotations under service live loads. A simplified schematic of a link-slab is shown in Figure 2.19.

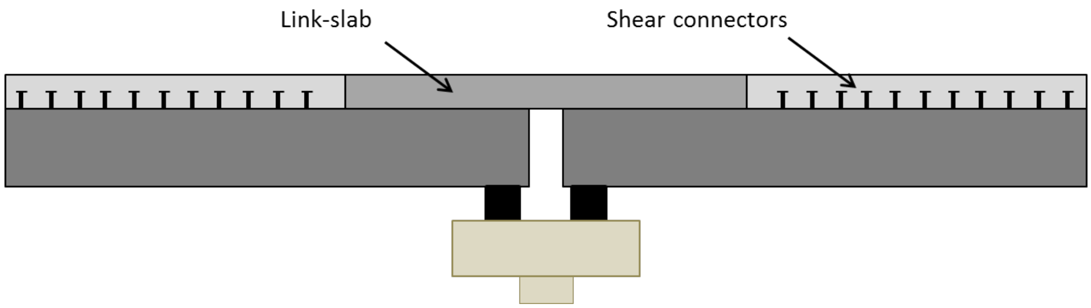


Figure 2.19: Simplified Schematic of Link-Slab

2.8 Bridge Inventory and PONTIS Database

The first part of this research is focused on bridges in Massachusetts that use expansion joints to accommodate thermal movement, which will be referred to as “jointed bridges.” Some of the information presented in this chapter comes from the PONTIS database. MassDOT uses PONTIS to catalog bridge inventory which includes basic information about the bridges, inspection reports, condition ratings, and joint types. Massachusetts has a total of 5,062 bridges, over half of which are classified as jointed bridges. It is important to note that these jointed bridges are classified by PONTIS, which only classifies bridge joints in bridges greater than 20 ft. in length. Therefore, expansion joints for minimal movement such as saw and seal joints are not included in the PONTIS data presented in this report. These joint types will be addressed in other sections of the chapter as these joints are used on many bridges in Massachusetts.

Of the 5062 bridges in Massachusetts, 2814 bridges have joints categorized as element number 300-305. The rest of the bridge inventory is either saw and seal, jointless, or culvert with fill. The five joint classifications in PONTIS are as follows:

- **300:** Strip seals: this element defines only joints which utilize a neoprene waterproof gland with steel extrusion to anchor the gland
- **301:** Pourable seals: this element defines only joints filled with a pourable seal
- **302:** Compression seals: this element defines only joints filled with a pre-formed compression type seal
- **303:** Assembly (modular) joint/seal: This element defines only joints filled with an assembly mechanism that may or may not have a seal. This includes modular assemblies
- **304:** Open: This element defines only joints that are open and not sealed
- **305:** Other joint/seal: This element is used for sliding plate joints and asphalt plug joints

The PONTIS database was used to create spreadsheets containing information for a straightforward comparison of many bridge factors to joint types. These factors include the structure main type, the maximum span length, the structure length, the condition rating (of joint elements), and the bridge age. In order to obtain all data needed for these comparisons, some of the data (structure main type, maximum span length, structure length, and bridge age) were not included in the PONTIS inventory file. However, through the National Bridge Inventory (NBI) this information was obtained for the jointed bridges in the PONTIS database, and was added to the spreadsheet by cross-referencing the bridge structure numbers. For bridge age, the year built (or reconstructed) was subtracted from the year 2015. This information was compiled in spreadsheets with bridges specific to each of the six districts, as well as a summary sheet with all of Massachusetts information, and was provided to MassDOT personnel.

2.9 Overview of Districts

Massachusetts Department of Transportation is divided into six districts. Each district number and their corresponding territory are presented in Figure 2.20. The headquarters of the district offices are located in Lenox (District 1), Northampton (District 2), Worcester (District 3), Arlington (District 4), Taunton (District 5), and Boston (District 6). Table 2.1 shows the distribution of bridges in each district, which includes the total number of bridges and the number of those bridges that have bridge joints classified in PONTIS. This table also shows how many of the total bridges are owned by MassDOT (as opposed to municipal bridges). In November, 2009 ownership of all bridges owned by the Department of Conservation and Recreation (except for pedestrian bridges) was transferred to MassDOT; therefore, these bridges are included in the total.

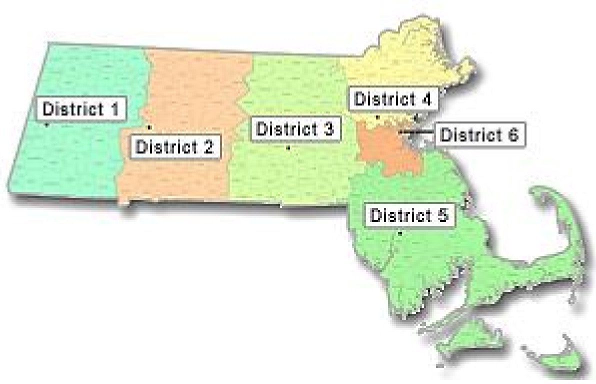


Figure 2.20: Districts of MassDOT

Table 2.1: Distribution of Bridges in MassDOT Districts

No. of Bridges	District Number						TOTAL	TOTAL OWNED BY MASSDOT
	1	2	3	4	5	6		
Total	703	834	1158	827	858	682	5062	3474
"Jointed Bridges"	185	413	624	558	455	579	2814	2557

When there is new construction, bridge joints are restricted to those in the Bridge Design Manual (MassDOT 2013). For joint repair, the individual districts handle repair choices internally; and are not held to the same restrictions as joint options for new construction. When re-decking is performed, the design decisions could be done in-house or by consultants, depending on the district. At least one district has a designated "bridge group." The source documents of special provisions originate from the Boston headquarters, but districts can add their own unique specifications to supplement these.

There is no maintenance manual for Massachusetts, and maintenance decisions are left to the individual districts. While each district receives a portion of the allotted federal money to be used towards bridges, the districts choose where this money would be spent in their district. Additionally, using the federal money requires specifying each job that the money will be used for and therefore the districts noted that it would not be feasible to use the money on general "maintenance". Any maintenance practices employed by a district would come out of the districts funds; given limited resources, none of the districts have an existing maintenance policy. There are no preservation specifications in Massachusetts. One district pointed out that this is the biggest issue affecting joint performance.

Each of the individual districts has unique needs based on many factors in their district. Districts have different construction time restraints, traffic volume, types of roads, etc. which dictate what joint types they use and what construction methods are available. As a result, joint preferences, as well as joint performance, vary from district to district. A breakdown of the joint types, per PONTIS classification, per district is presented in Figure 2.21.

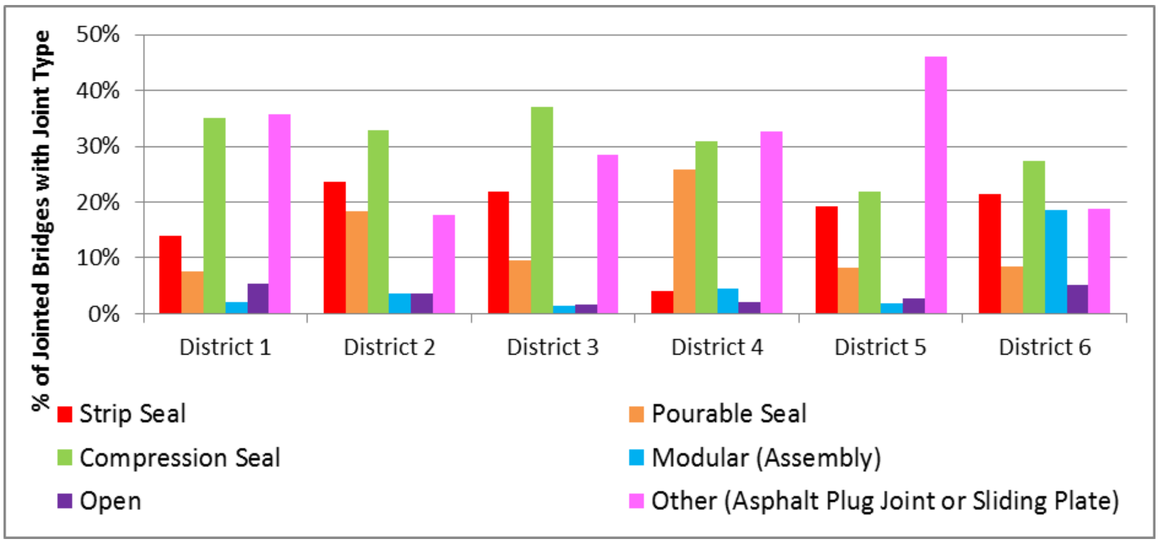


Figure 2.21: Percent of Jointed Bridges with Each PONTIS Joint Type

A map of the major roadways in Massachusetts is presented in Figure 2.22. Table 2.2 shows the distribution of turnpike bridges as well as other major interstate bridges for each of the districts in Massachusetts.

Figure 2.22: Major roadways in Massachusetts (MassDOT)

Table 2.2: Distribution of Turnpike and Interstate Bridges in Districts

No. of Bridges		**District Number**						**TOTAL**
		1	**2**	**3**	**4**	**5**	**6**	
I-90 (Turnpike)	**All Bridges**	39	73	93	0	0	51	256
	"Jointed Bridges"	35	65	77	0	0	36	213
Major Interstates	**All Bridges**	0	131	240	198	136	187	892
	"Jointed Bridges"	0	131	227	183	123	187	851
	Interstate #	*N/A*	91, 291, 391	84, 190, 290, 395, 495	93, 95, 495	95, 195, 295, 495	93, 95	

District 1 is located at the western end of Massachusetts. This area has the most short- span bridges of any district, of which many use saw and seal joints but are not in PONTIS because of their short spans. This is why the 185 “jointed bridges” for District 1 appears to be far less than any other district. District 1 also has significantly lower traffic volume than the other districts. This lower traffic volume allows them to do lane closures for repairs, have extra time for installations, and they do not face the same time constraints as other districts. This district has some of the Massachusetts Turnpike (Turnpike) bridges, as shown in Table 2.2.

District 2 is also located in western Massachusetts, with 413 jointed bridges within the district. This district has many medium to shorter span bridges, with the majority of bridges being less than 200 ft. in total length. This district also has some turnpike bridges, as well as some bridges on Interstates 91, 291 and 391.

District 3 has many Turnpike bridges, with 624 total jointed bridges. The majority of bridges in this district have a total length ranging from 60 ft to 200 ft. District 3 has some bridges on Interstates 84, 190, 290, 395, and 495.

District 4 has 558 jointed bridges, the majority of which (similarly to District 3) range from 60 ft to 200 ft. District 4 does not have Turnpike bridges, but has some on Interstates 93, 95, and 495.

District 5 has 455 jointed bridges. However, many of the bridges in this district are on limited access highways (including Rt.3) with high traffic volumes. District 5 does not have Turnpike bridges, but has some on Interstates 95, 195, 295, 495. The high traffic volumes result in shortened construction time availability, which in turn limits the joint types and materials they are generally able to use. Unless the bridge work is classified as "new construction", any repairs or replacements on any bridges in these high traffic volume areas must be completed between 8pm and 5am Monday through Friday.

Additionally, there are seasonal restrictions for Cape Cod where no work can be done between Memorial Day and Labor Day.

District 6 is the Boston district which has turnpike bridges, some on Interstates 95 and 93 and city bridges, with a total of 579 jointed bridges. Essentially all bridges in this district are subject to extremely high traffic volumes and strict time constraints for completing bridge work. Any bridge joint repairs or replacements must be completed in the 8pm-5am time period, which limits joint options.

For all districts except for District 5, part of the districts responsibility is the Massachusetts Turnpike bridges. The Turnpike is unique in that the wearing surface is significantly thinner than any other roadways in Massachusetts. This thin wearing surface limits joint options because many call for a wearing surface thicker than that of the Turnpike. Furthermore, bridge joints tend to deteriorate and fail sooner as a result of both the thin wearing surface and high traffic volume.

All districts inspect town-owned bridges as well as the state-owned bridges, with the majority of inspections being done in-house. Large or complex bridges that take multiple days to inspect are contracted out to consultants. For town-owned bridges, the condition tends to be slightly better due to the lower traffic volume. Although the districts inspect the town bridges, they are not in charge of maintaining them or repairing them.

2.10 PONTIS and NBI Inventory Compilation

A number of tables and comparisons were created to correlate factors related to Massachusetts Bridges. This section presents some of the tables compiled which sort bridges within each of the Massachusetts districts and present joint types used compared with other factors. The number of bridges per district was presented in 2.8. Joint number classification is presented in Table 2.3.

Figure 2.23 shows the number of bridges in each district by total bridge length. Table 2.4 through Table 2.9 show the percent of bridges in each district with corresponding joint types (joint type 300 through 305, respectively). Next, Table 2.10 shows the number of steel stringer/multi-beam bridges in each district by joint classification. This is the classification of the majority of bridges in Massachusetts (over 50%).

The number of bridges in each district is sorted by bridge age in Figure 2.24. The percentage of bridges (per district) is presented for bridges in the age ranges that contain each joint type (joint 300 through 305) in Table 2.11 through Table 2.16.

Finally, the total length (linear meters) of each joint type per district is shown in Figure 2.25. Element condition state can be rated a 1, 2, or 3 for good, fair, or poor. The percent of joint types with each condition rating are presented for joints 300 through 305 in

Table 2.17 through Table 2.22.

Table 2.3: Joint Number Classification

Joint Number	Joint Type
300	Strip Seal
301	Pourable Seal
302	Compression Seal
303	Modular/Other
304	Open
305	Other

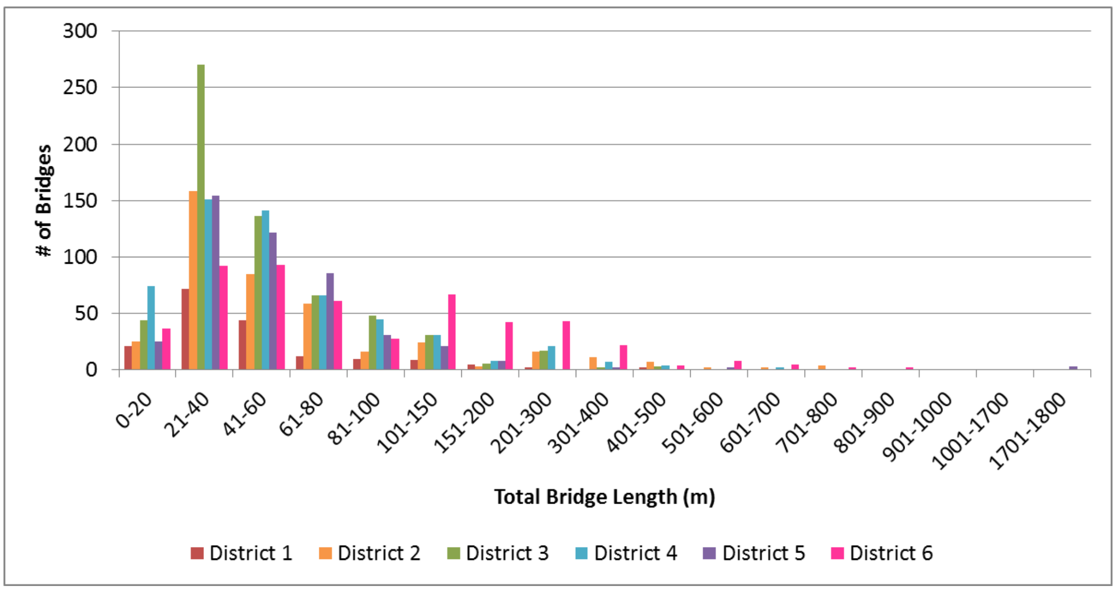


Figure 2.23: Number of Bridges in each Massachusetts District by Total Length (m)

Table 2.4: Percent of Bridges (by total length) in each District with Strip Seal Joints

Bridge Total Length		Strip Seal					
ft	m	District 1	District 2	District 3	District 4	District 5	District 6
0-20	**0-65.6**	5%	40%	9%	1%	24%	16%
21-40	**65.7-131.2**	11%	13%	21%	3%	20%	24%
41-60	**131.3-196.9**	18%	29%	24%	1%	16%	16%
61-80	**197-262.5**	8%	20%	18%	2%	16%	21%
81-100	**262.6-328.1**	20%	25%	19%	9%	13%	25%
101-150	**328.2-492.1**	11%	33%	39%	10%	43%	16%
151-200	**492.2-656.2**	40%	67%	33%	25%	13%	24%
201-300	**656.3-984.3**	50%	31%	41%	14%	100%	9%
301-400	**984.4-1312.3**	0%	45%	0%	29%	50%	23%
401-500	**1312.4-1640.4**	0%	43%	67%	0%	0%	75%
501-600	**1640.5-1968.5**	0%	0%	0%	0%	50%	38%
601-700	**1968.6-2296.6**	0%	50%	0%	50%	0%	40%
701-800	**2296.7-2624.7**	0%	25%	0%	0%	0%	0%
801-900	**2624.8-2952.8**	0%	0%	0%	0%	0%	50%
901-1000	**2952.9-3280.8**	0%	100%	0%	0%	0%	0%
1001-1700	**3280.9-5577.4**	0%	0%	0%	0%	0%	0%
1701-1800	**5577.5-5905.5**	0%	0%	0%	0%	33%	0%

Table 2.5: Percent of Bridges (by total length) in each District with Pourable Seal Joints

Bridge Total Length		Pourable Seal					
ft	m	District 1	District 2	District 3	District 4	District 5	District 6
0-20	**0-65.6**	14%	8%	23%	46%	0%	38%
21-40	**65.7-131.2**	10%	20%	11%	28%	8%	11%
41-60	**131.3-196.9**	7%	18%	10%	27%	5%	11%
61-80	**197-262.5**	0%	24%	8%	20%	12%	15%
81-100	**262.6-328.1**	10%	19%	4%	9%	13%	7%
101-150	**328.2-492.1**	0%	21%	0%	23%	14%	0%
151-200	**492.2-656.2**	0%	0%	0%	0%	25%	0%
201-300	**656.3-984.3**	0%	19%	0%	10%	0%	0%
301-400	**984.4-1312.3**	0%	9%	0%	14%	0%	5%
401-500	**1312.4-1640.4**	0%	0%	0%	0%	0%	0%
501-600	**1640.5-1968.5**	0%	50%	0%	0%	0%	0%

Table 2.6: Percent of Bridges (by total length) in each District with Compression Seal Joints

Bridge Total Length		Compression Seal					
ft	m	District 1	District 2	District 3	District 4	District 5	District 6
0-20	**0-65.6**	19%	40%	48%	15%	28%	16%
21-40	**65.7-131.2**	43%	42%	36%	34%	18%	36%
41-60	**131.3-196.9**	30%	28%	35%	31%	24%	38%
61-80	**197-262.5**	58%	31%	41%	44%	27%	28%
81-100	**262.6-328.1**	30%	38%	50%	38%	26%	32%
101-150	**328.2-492.1**	44%	25%	39%	29%	14%	19%
151-200	**492.2-656.2**	20%	0%	0%	25%	0%	10%
201-300	**656.3-984.3**	0%	19%	12%	14%	0%	14%
301-400	**984.4-1312.3**	0%	18%	0%	0%	50%	23%
401-500	**1312.4-1640.4**	50%	0%	0%	50%	0%	0%
501-600	**1640.5-1968.5**	0%	0%	0%	0%	0%	13%
601-700	**1968.6-2296.6**	0%	0%	0%	0%	0%	0%
701-800	**2296.7-2624.7**	0%	25%	0%	100%	0%	100%
801-900	**2624.8-2952.8**	0%	0%	0%	0%	0%	50%
901-1000	**2952.9-3280.8**	0%	0%	0%	0%	0%	0%
1001-1700	**3280.9-5577.4**	0%	0%	0%	0%	0%	0%
1701-1800	**5577.5-5905.5**	0%	0%	0%	0%	33%	0%

Table 2.7: Percent of Bridges (by total length) in each District with Modular Joints

Bridge Total Length		Assembly (Modular) Joint/Seal					
ft	m	District 1	District 2	District 3	District 4	District 5	District 6
0-20	**0-65.6**	0%	0%	0%	7%	0%	5%
21-40	**65.7-131.2**	4%	1%	1%	2%	0%	7%
41-60	**131.3-196.9**	0%	2%	1%	4%	3%	6%
61-80	**197-262.5**	0%	2%	0%	2%	1%	8%
81-100	**262.6-328.1**	0%	6%	0%	4%	0%	25%
101-150	**328.2-492.1**	0%	8%	3%	3%	0%	34%
151-200	**492.2-656.2**	0%	33%	0%	25%	13%	45%
201-300	**656.3-984.3**	0%	6%	6%	5%	0%	51%
301-400	**984.4-1312.3**	0%	9%	100%	43%	0%	27%
401-500	**1312.4-1640.4**	0%	29%	0%	25%	0%	25%
501-600	**1640.5-1968.5**	0%	0%	0%	0%	50%	38%
601-700	**1968.6-2296.6**	0%	50%	0%	50%	0%	40%

701-800	**2296.7-2624.7**	0%	25%	0%	0%	0%	0%
1001-1700	**3280.9-5577.4**	0%	0%	0%	0%	0%	0%
1701-1800	**5577.5-5905.5**	0%	0%	0%	0%	33%	0%

Table 2.8: Percent of Bridges (by total length) in each District with Open Joints

Bridge Total Length		**Open**					
ft	**m**	**District 1**	**District 2**	**District 3**	**District 4**	**District 5**	**District 6**
0-20	**0-65.6**	0%	0%	0%	1%	4%	0%
21-40	**65.7-131.2**	4%	3%	0%	0%	3%	1%
41-60	**131.3-196.9**	2%	2%	1%	1%	2%	0%
61-80	**197-262.5**	0%	2%	0%	0%	2%	0%
81-100	**262.6-328.1**	20%	0%	0%	0%	3%	7%
101-150	**328.2-492.1**	11%	0%	0%	3%	0%	10%
151-200	**492.2-656.2**	20%	0%	67%	0%	13%	10%
201-300	**656.3-984.3**	0%	19%	24%	38%	0%	7%
301-400	**984.4-1312.3**	0%	9%	0%	0%	0%	9%
401-500	**1312.4-1640.4**	50%	29%	0%	0%	0%	0%
501-600	**1640.5-1968.5**	0%	50%	0%	0%	0%	13%

Table 2.9: Percent of Bridges (by total length) in each District with Other Joints

Bridge Total Length		Other					
ft	**m**	**District 1**	**District 2**	**District 3**	**District 4**	**District 5**	**District 6**
0-20	**0-65.6**	62%	12%	20%	30%	44%	24%
21-40	**65.7-131.2**	28%	20%	31%	33%	51%	22%
41-60	**131.3-196.9**	43%	20%	29%	35%	51%	29%
61-80	**197-262.5**	33%	22%	33%	33%	42%	28%
81-100	**262.6-328.1**	20%	13%	27%	40%	45%	4%
101-150	**328.2-492.1**	33%	13%	19%	32%	29%	19%
151-200	**492.2-656.2**	20%	0%	0%	25%	38%	12%
201-300	**656.3-984.3**	50%	6%	18%	19%	0%	19%
301-400	**984.4-1312.3**	0%	9%	0%	14%	0%	14%
401-500	**1312.4-1640.4**	0%	0%	33%	25%	0%	0%
501-600	**1640.5-1968.5**	0%	0%	0%	0%	0%	0%
601-700	**1968.6-2296.6**	0%	0%	0%	0%	0%	20%
701-800	**2296.7-2624.7**	0%	25%	0%	0%	0%	0%

Table 2.10: Number of Steel Stringer/Multi-beam Bridges with each Joint Type

	District Number					
Joint Type	**1**	**2**	**3**	**4**	**5**	**6**
300	9	63	71	8	45	27
301	6	58	47	77	29	30
302	40	94	125	96	73	99
303	2	4	5	12	2	2
304	2	4	3	1	7	6
305	26	51	99	110	132	57
Total	*85*	*274*	*350*	*304*	*288*	*221*

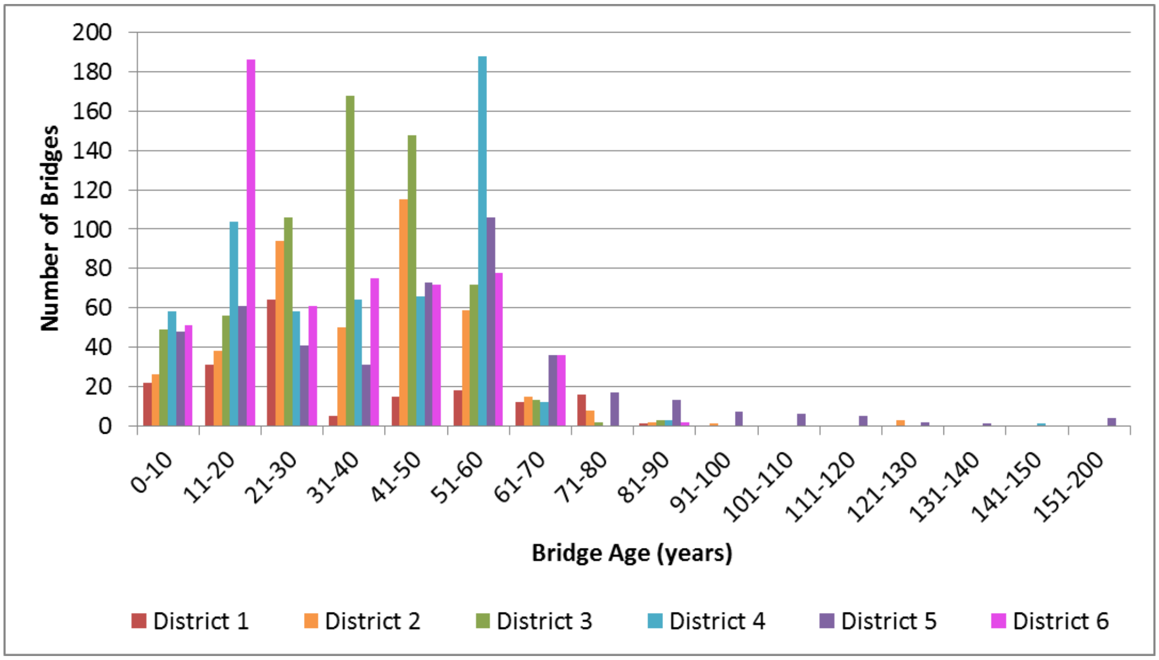


Figure 2.24: Number of Bridges in each District by Bridge Age

Table 2.11: Percent of Bridge in each District (by bridge age) with Strip Seals

Bridge Age (years)	Strip Seal					
	District					
	District 1	District 2	District 3	District 4	District 5	District 6
0-10	18%	35%	18%	9%	21%	22%
11-20	13%	26%	27%	5%	18%	30%
21-30	14%	40%	22%	10%	20%	13%
31-40	0%	18%	35%	6%	16%	15%
41-50	13%	15%	16%	0%	15%	19%
51-60	11%	14%	8%	2%	23%	10%
61-70	0%	27%	0%	0%	11%	28%
71-80	31%	13%	50%	0%	24%	0%
81-90	0%	50%	0%	0%	38%	0%
91-100	0%	0%	0%	0%	14%	0%
101-110	0%	0%	0%	0%	0%	0%
111-120	0%	0%	0%	0%	20%	0%
121-130	0%	0%	0%	0%	100%	0%
131-140	0%	0%	0%	0%	100%	0%
141-150	0%	0%	0%	0%	0%	0%
151-200	0%	0%	0%	0%	25%	0%

Table 2.12: Percent of Bridge in each District (by bridge age) with Pourable Seals

Bridge Age (years)	Pourable Seal					
	District					
	District 1	District 2	District 3	District 4	District 5	District 6
0-10	18%	15%	12%	33%	8%	12%
11-20	6%	11%	2%	33%	13%	9%
21-30	3%	7%	1%	19%	5%	16%
31-40	0%	8%	1%	11%	3%	4%
41-50	7%	28%	19%	24%	12%	3%
51-60	22%	34%	22%	28%	8%	14%
61-70	0%	13%	8%	25%	3%	3%
71-80	6%	25%	50%	0%	6%	0%
81-90	0%	0%	33%	33%	0%	0%
91-100	0%	0%	0%	0%	14%	0%
101-110	0%	0%	0%	0%	17%	0%
111-120	0%	0%	0%	0%	20%	0%

Table 2.13: Percent of Bridge in each District (by bridge age) with Compression Seals

Bridge Age (years)	Compression Seal					
	District					
	District 1	District 2	District 3	District 4	District 5	District 6
0-10	0%	0%	4%	14%	21%	4%
11-20	16%	32%	13%	6%	21%	8%
21-30	66%	41%	58%	57%	24%	51%
31-40	80%	52%	55%	53%	29%	65%
41-50	60%	31%	30%	55%	21%	33%
51-60	11%	25%	22%	26%	22%	37%
61-70	8%	20%	31%	17%	19%	11%
71-80	13%	38%	0%	0%	24%	0%
81-90	0%	50%	33%	67%	23%	50%
91-100	0%	100%	0%	0%	29%	0%
101-110	0%	0%	0%	0%	33%	0%
111-120	0%	0%	0%	0%	20%	0%
121-130	0%	33%	0%	0%	0%	0%

Table 2.14: Percent of Bridge in each District (by bridge age) with Modular Joints

Bridge Age (years)	Assembly (Modular) Joint/Seal					
	District					
	District 1	District 2	District 3	District 4	District 5	District 6
0-10	0%	15%	0%	3%	0%	16%
11-20	3%	0%	4%	4%	0%	46%
21-30	0%	3%	0%	2%	2%	3%
31-40	0%	10%	3%	6%	6%	0%
41-50	0%	1%	1%	3%	3%	3%
51-60	11%	2%	1%	3%	0%	0%
61-70	8%	7%	0%	42%	8%	28%

Table 2.15: Percent of Bridge in each District (by bridge age) with Open Joints

Bridge Age (years)	Open Joint					
	District					
	District 1	District 2	District 3	District 4	District 5	District 6
0-10	0%	4%	2%	5%	4%	6%
11-20	0%	8%	7%	0%	3%	1%
21-30	3%	1%	0%	2%	2%	2%
31-40	0%	0%	0%	2%	3%	0%
41-50	0%	2%	2%	2%	0%	24%
51-60	11%	3%	0%	3%	5%	4%
61-70	25%	13%	15%	8%	3%	8%
71-80	19%	25%	0%	0%	0%	0%
81-90	0%	0%	0%	0%	0%	0%
91-100	0%	0%	0%	0%	0%	0%
101-110	0%	0%	0%	0%	0%	0%
111-120	0%	0%	0%	0%	0%	0%
121-130	0%	67%	0%	0%	0%	0%

Table 2.16: Percent of Bridge in each District (by bridge age) with “Other” Joints

Bridge Age (years)	Other					
	District					
	District 1	District 2	District 3	District 4	District 5	District 6
0-10	64%	31%	63%	36%	46%	41%
11-20	61%	24%	48%	53%	44%	7%
21-30	14%	6%	19%	10%	46%	15%
31-40	20%	12%	6%	22%	42%	16%
41-50	20%	23%	32%	17%	49%	18%
51-60	33%	22%	46%	39%	43%	35%
61-70	58%	20%	46%	8%	56%	22%
71-80	31%	0%	0%	0%	47%	0%
81-90	100%	0%	33%	0%	38%	50%
91-100	0%	0%	0%	0%	43%	0%
101-110	0%	0%	0%	100%	50%	0%
111-120	0%	0%	0%	0%	40%	0%
121-130	0%	0%	0%	0%	0%	0%
131-140	0%	0%	0%	0%	0%	0%
141-150	0%	0%	0%	0%	0%	0%

151-200	0%	0%	0%	0%	75%	0%

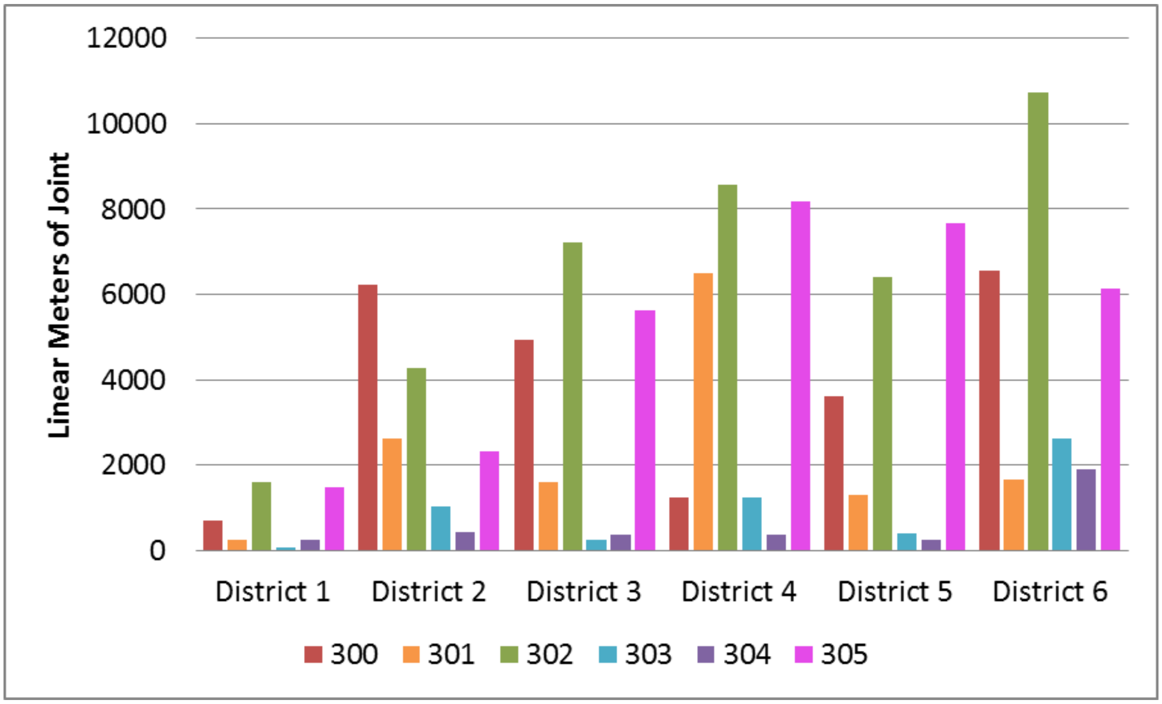


Figure 2.25: Total Linear Meters of each Joint Type per District

Table 2.17: Percent of Joint Type in each Condition State: Strip Seal

Condition State	Strip Seal					
	District 1	**District 2**	**District 3**	**District 4**	**District 5**	**District 6**
1- Good	63%	58%	39%	53%	31%	44%
2- Fair	35%	37%	44%	30%	49%	41%
3- Poor	2%	6%	17%	18%	20%	14%

Table 2.18: Percent of Joint Type in each Condition State: Pourable Seal

Condition State	Pourable Seal					
	District 1	**District 2**	**District 3**	**District 4**	**District 5**	**District 6**
1- Good	46%	38%	22%	52%	35%	47%
2- Fair	30%	25%	25%	31%	33%	36%
3- Poor	24%	37%	53%	17%	32%	17%

Table 2.19: Percent of Joint Type in each Condition State: Compression Seal

Condition State	Compression Seal					
	District 1	District 2	District 3	District 4	District 5	District 6
1- Good	32%	23%	17%	29%	51%	22%
2- Fair	61%	62%	50%	46%	35%	60%
3- Poor	6%	15%	34%	25%	15%	18%

Table 2.20: Percent of Joint Type in each Condition State: Modular Joint

Condition State	Assembly (Modular) Joint/Seal					
	District 1	District 2	District 3	District 4	District 5	District 6
1- Good	0%	29%	31%	36%	72%	73%
2- Fair	76%	54%	14%	37%	18%	24%
3- Poor	24%	18%	54%	27%	11%	3%

Table 2.21: Percent of Joint Type in each Condition State: Open Joint

Condition State	Open					
	District 1	District 2	District 3	District 4	District 5	District 6
1- Good	63%	51%	50%	46%	36%	70%
2- Fair	37%	31%	19%	51%	32%	30%
3- Poor	0%	18%	31%	3%	32%	1%

Table 2.22: Percent of Joint Type in each Condition State: Other

Condition State	Other					
	District 1	District 2	District 3	District 4	District 5	District 6
1- Good	62%	51%	37%	53%	47%	47%
2- Fair	28%	35%	38%	29%	38%	42%
3- Poor	10%	14%	25%	18%	14%	11%

2.11 Massachusetts Joints

Information was collected from Massachusetts by attending meetings with each of the six districts and discussing their joint and header practices. After each of these meetings, the districts also completed the survey that was created and sent to other state Departments of Transportation. This chapter will present all of the information collected from Massachusetts from both the individual meetings and survey responses.

2.11.1 Saw and Seal

Five of the six districts have saw and seal joints currently in service, and four districts report using them in new construction. The majority of districts also use this joint type as a replacement joint both for overnight construction and when there is no time constraint. Saw and seal joints are believed to perform adequately with routine repair and maintenance, and the reported service life ranges between districts with some reporting 5-12 years, and one reporting more than 16 years. No districts plan on phasing these joints out. It was noted that these have been used to replace existing joints that were oversized for the actual movement experienced by the bridge.

Saw and seal joints are used for very short span bridges, accommodating movement of ¼” to ½”, and they are not designated as a joint in the PONTIS database. While there is not a joint classification for saw and seal joints, it is unclear as to whether some inspectors would put them in the “305: Other” category if the saw and seal replaces another joint type. Many districts use them over fixed joints at the abutments or over piers with fixed bearings on both sides.

The benefits of saw and seal joints are that they are easy to maintain and provide a smooth driving experience. The majority of districts classify the performance of these joints as very successful when used as deck over backwall detail, and slightly less successful when used over existing joints.

Some problems with the saw and seal joint can come from the contractor inaccurately locating the joint location, such as the ends of beams on a skew. This may be due to the contractor and field engineer perception that location on these small joints is not critical. However, incorrect location of the joint leads to leakage when a secondary crack occurs at the location of rigidity where the saw should have been placed. Using saw and seal joints on the Massachusetts Turnpike has been a problem, as with other joints which require 2-3" of overlay; the Massachusetts Turnpike only has 1 ½" wearing surface and this thin surface leads to early failure.

Some steps that could be taken to improve the performance of saw and seal joints are to mark the curb where the deck ends before placing the wearing surface so that the joint location can still be clearly seen (District 1). While this is not stated in the item for "sawing and sealing joints in asphalt at bridges", it is the contractor's responsibility to install the proposed joint at the proper location. Typically, when these joints are installed everything is excavated and exposed and locating the deck end is a straightforward task with no additional cost. However, sometimes this task is so simple that it is overlooked and the contractor ends up guessing where the deck end is after the paving is done (District 1).

Another critical piece of the saw and seal joint that is often not perceived as important is placement of the bond breaker tape; this detail is often skipped on site during installation. Bond breaker tape allows free expansion and contraction of the joint sealant with fluctuating temperatures. When the bond breaker tape is skipped the sealant can bond to the wrong areas of the joint, which can result in tearing or stretching to the point of failure.

Overall, saw and seal joints have been successful for very small movements, and with close attention to specifications and construction details can provide very good performance.

2.11.2 Asphalt Plug Joints

Asphalt plug joints are currently in use in all districts, and are being used for new construction. They are also being used as replacement joints with the majority of districts using them as an option in all situations. The asphalt plug material can set quickly enough to use in overnight construction. While almost all districts agree that asphalt plug joints could perform well if routine maintenance and repair was performed, the service life reported for them varies throughout Massachusetts. Three of the seven respondents (43%) believed asphalt plug joints have a typical service life of five to eight years, followed by two respondents (29%) noting the service life to be zero to four years, one respondent (14%) choosing nine to 12 years, and one choosing 13-16 years. This wide range of typical service life is likely due in part to what the individual defines as failure of the joint. Satisfaction or lack thereof, with asphalt plug joints tends to depend heavily on the expectation of service life for joints and perceptions of acceptable levels of deterioration before replacement. Both of these factors also rely heavily on traffic volume and detailing.

Asphalt plug joints are generally used to accommodate movement of ½” to 1 ½” (District 4 will use them for up to 2”), with bridge skews less than 30°. Three districts provided a typical thickness of asphalt plug joints. District 2 and District 3 reported a minimum thickness of 2”, while District 1 reported using a thickness of at least 2 ½” to 3”. A few of the districts have started using “modified asphalt plug joints” by incorporating EM-SEAL into the joint and then covering the EM-SEAL with asphalt to create the asphalt plug joint. Asphalt plug joint details at parapet or barrier can be difficult due to vertical projection over curb. EM-SEAL comes with vertical pieces to help solve this problem and create a watertight seal (better than typical backer rod detail which is difficult to maneuver up and over curb). So far, the districts that have tried using the modified asphalt plug joints have been happy with the performance and have not had problems, though this is still a fairly new practice.

District 5 is currently trying to phase out asphalt plug joints due to the poor performance with the asphalt surface course above the joint, and instead use strip seals with elastomeric headers. In this district, asphalt plug joints will no longer be used as replacement joints and moving forward they will be replaced with other joint types. District 6 is also unhappy with the performance of asphalt plug joints saying that they are not a preferable joint. This district did note that the poor performance may be, in part, due to the use of Duracel quick-set concrete headers with the joint. These districts have both noted that they are limited in time for joint work and almost all work needs to be done between 8 pm and 5 am, which limits the ability to use normal setting concrete and the amount of preparation time. These districts also have a significant amount of high traffic volume, a condition under which asphalt plug joints are reported to perform poorly.

The majority of districts state that asphalt plug joints do not perform well in high traffic volume roads, and note that the manufacturers often state that they are not recommended for use with high traffic volume. Asphalt plug joints also don't perform well with skewed bridges. They are not used for skews greater than 30° but can also experience problems with lower skew angles. On the Massachusetts Turnpike, the wearing surface is thinner than other roadways which limits the thickness of joints that can be used; while asphalt plug joints should have a minimum thickness of 2", asphalt plug joints have to be installed with 1 ¾" thickness which has been noted to result in early failure (along with the high traffic volume). Mix type has also been noted to impact asphalt plug joints; it was suggested that change in mix compositing causes poor performance.

Typical failure modes of asphalt plug joints include rutting, heaving, tire damage where the tires shove the joint out over time, cracking, and dislodged sections coming out of the joint. If there is too much movement, cohesion induced cracks can form through the middle of the joint. Districts noted different definitions of "failure" of the joint; some reporting failure at the onset of rutting, while others state that the joint is only considered failed once binder is lost and water begins leaking through the joint.

The majority of districts agree that when asphalt plug joints are first installed, they have great ride-ability and a smooth transition over joint. They are also generally easy to repair since they can be repaired segmentally. The general consensus of the districts is that quality control of asphalt plug joints during preparation and installation is critical to their success, and this is an area that needs improvement. This includes proper preparation before joint installation, including sandblasting and cleaning of the cut prior to joint placement. Neglecting this step can significantly decrease the service life and overall performance of the joint. Some districts have noted variability in the materials used in asphalt plug joints and suspect that specific materials or mixing practices affect performance. District 3 has added specifications that eliminate contractor interaction with the component materials by requiring pre-mixed bagged materials and use a qualified installer; since these specifications have been implemented the quality of the joints has improved.

Overall, the majority of districts agree that asphaltic plug joints require routine maintenance due to their shorter design life and that current funding levels make this difficult. Additionally, District 5 states that a lack of preservation specifications is a major problem. It was noted by several districts that quality control would improve performance, as well as research to develop a more resilient/flexible material. Improper installation is thought to be a leading source of failure in asphalt plug joints which could be improved with better training of inspectors and whoever the district sends to oversee the installation. Problems with installation, reported by the districts, include: improper setting of plate, backer rod not being installed over curb correctly, lack of proper cleaning after cut is made and wrong box-out dimensions. Successes in preventing leakage, especially at the detail over the curb and parapet, have been seen with modified asphalt plug joints using EM-SEAL. Asphalt plug joints are not recommended to be used with high traffic volume, or high skew angles (maximum 30°).

2.11.3 Compression Seals

Compression seals are currently in service in all districts, but only two districts reported using compression seals in new construction. They are also being used by a few districts as replacement joints for existing compression seals, both for overnight construction and when there are no time constraints. Five of the six districts reported that if routine repair and maintenance were performed, compression seals would perform adequately. The typical service life varies between districts with District 1 reporting five to eight years, District 4 reporting nine to twelve years, and Districts 2, 5 and 6 reporting thirteen to sixteen years. Almost all districts have started phasing out the use of compression joints, or would like to phase them out moving forward.

Compression seals generally perform decently when they are first installed. District 6 noted that the steel armoring with compression seals usually gets damaged by plows and since the armored headers are difficult to repair, they usually remove the entire compression seal joint resulting in an open joint until a full repair can be completed. One of the problems with compression seals is that the neoprene seals suffer from compression set (the loss of ability to self-expand after cyclic loading) and tear/fall out as a result. Districts have reported compression seals failing at attachment with bonding noted as the main problem with this joint type. One district stated that a new type of foam compression seal bonds to sidewall well, and has had good performance so far.

Overall, districts are choosing to replace compression seals with other joint types. Some districts have replaced compression seals with EM-SEAL, noting that if the steel armor is in good shape it is straightforward to install EM-SEAL and can be done quickly. Another retrofit method is to take out steel armor, replace with new concrete headers and a poured silicone seal with bituminous overlay. Some districts noted using saw and seal joints to replace failed compression joints, where District 6 noted that they replace the compression joint then do a saw cut overlay. It was also stated that Closed Cell Foam is not suitable and is likely to fall out, so compression seals should be made of neoprene. The wide range of materials that have been classified as compression seals over the years makes it difficult to distinguish performance problems specific to the joint type versus material used.

2.11.4 Strip Seals

Strip seals are currently in service in all districts, and five of the six districts are using them in new construction. All districts are using them as replacement joints when there are no time constraints on construction and four of the six districts use them as replacement joints when overnight construction is needed. Of all the joint types, the strip seal is the only joint type that all six districts listed as performing well when routine repair and maintenance is performed. The service life reported for strip seals range from five to eight years to greater than sixteen years, with the majority of districts reporting between nine and sixteen years. The performance of strip seals is rated as fairly successful by all districts. Only District 4 reported that they would like to phase out strip seals moving forward.

Strip seals are used to accommodate movement of up to 4”, and can be used for 1½” to 4” if skew is greater than 30°. Strip seals, unlike asphalt plug joints, are approved for high skew angles. Therefore, strip seals may be used for smaller expansion needs in a place where an asphalt plug joint would typically be used if the skew exceeds the 30° limitation that asphalt plug joints are limited to. Strip seal joints have broad application, being used for longer spans, skewed bridges and high traffic volume areas.

Strip seals have been used to replace compression seals and pourable seals in some districts. One district stated that while strip seals are a little more expensive than pourable seals (approximately $350/linear ft. for strip seal installation, everything included, while pourable seals are approximately $300/linear ft.), the strip seals are more durable so they tend to be worth the extra cost and time to install. If an existing strip seal joint is being replaced and the headers are still in good condition, the strip seal can be installed within the existing headers.

Strip seals can be used with armored headers, elastomeric headers, or normal setting concrete headers. The headers of strip seals have impacted the performance, with districts noting problems with the various types of headers and how they affect the joints. District 2 noted that elastomeric headers have issues when used with joint replacements if the substrate is in poor condition and unsound concrete is not completely removed, but these headers can work well in new construction. District 1 stated that they have had numerous issues with elastomeric headers performing poorly and falling apart shortly after installation, so they have switched to 4000 psi concrete headers. District 6 has experimented with using open cell foam with the strip seal and so far it has been a success. Some districts noted that anchorage is not great for elastomeric headers; they rely heavily on bonding and when substrate is in poor condition or if improperly installed then the headers debond.

There are some problems associated with strip seals. Replacing strip seals is not a quick process because the seal cannot be spliced, which requires the whole seal to be replaced at once. Failure of seals and missing seals were reported. Debris and sand can build up on seal, tearing the seal and causing damage over time if not cleaned.

Overall, the consensus from all districts is that strip seals perform very well even with minimal maintenance. The details for strip seal installation are simple, and districts stated that the current specifications are sufficient, but that implementation of the specifications could be improved. One of the areas that needs improvement is preparation before installing the joint; this is true for many joint types, where improper cleaning prior to installation and not following all manufacturer specifications leads to problems. If strip seals were properly maintained and construction followed the specifications closely it is believed that the joint could last approximately 20 years.

2.11.5 EM-SEAL

EM-SEAL is currently being used in four districts, while no districts are using them as an option for new construction (EM-SEAL is not a joint option in the bridge design manual for Massachusetts). Two districts report using them for joint replacements. Another use of EM-SEAL is being applied in tandem with other joint types such as the Modified Asphalt Plug Joint. The service life is still not known since these are fairly new joints. The consensus of the performance of EM-SEAL is that with routine repair and maintenance, this joint type is close to an absolute success. This product is still fairly new in comparison to the other joint types in Massachusetts, so time will be the ultimate test for its performance, but so far the feedback of its performance has been positive.

There are many benefits of EM-SEAL. The joint is watertight and although it is still a fairly new joint type, districts report that EM-SEAL in service for four years still shows no leaking. EM-SEAL has been used to replace many joint types such as silicone seals, compression seals, and plug joints. When armored headers are in good condition, they can be left in place with EM-SEAL replacing the damaged joint. Perhaps the biggest benefit of EM-SEAL is that it comes with transition pieces, which allows for easily constructing the joint up and over curbs and parapets while maintaining a watertight seal, something that has proven difficult when using backer-rods in joint construction.

EM-SEAL is described as a simplified method to keep joints watertight, especially around the curb. It has had successful performance on limited access highways, demonstrating that it can stand up to high traffic volume. Across the districts it is described as a preferable joint. While still being a fairly new joint, EM-SEAL's performance has been promising.

2.11.6 Pourable Seals

Pourable seals are currently in service in four of the six districts; however they are not used in new construction. District 1 uses pourable seals as a replacement when overnight construction is required, District 2 uses them as a replacement when there is no time constraint, and District 5 uses them in both instances. Half of the districts report that if routine repair and maintenance are performed then pourable seals would perform adequately. Only three districts reported a typical service life of pourable seals, with all of them selecting the shortest service life of zero to four.

Pourable seals are economical and quick to install and therefore good for short term fixes. However, the problems seem to outweigh the benefits. Some problems with pourable seals include adherence issues, holes and tears in the joint seal, low durability and problems with backer rods. Many of these problems appear to be related to installation and inspection issues so performance may be significantly improved with proper training. Problems noted during construction include use of improper backer rod diameter, inconsistent depth of backer rod along the joint and material being installed too thinly. Once installed, debris can build up and push the sealant and backer rod through the joint creating an open joint. Districts are moving away from pourable seals, noting that they are perceived to be more vulnerable than other joint types.

2.11.7 Modular Joints

Modular joints are currently in service in five of the six districts, and half of the districts are using them in new construction. District 1 does not have any modular joints, and they are also a district with many short-span bridges where there is not a high demand for joints that can accommodate large expansions. District 2 reported that they would like to phase modular joints out in the future, however they have not been able to do this yet because there is not currently a better alternative to accommodate the large movement demands. Finger joints are the alternative for large movement demands, but they are unhappy with the performance of these joints as well.

The majority of districts state that when routine repair and maintenance is performed, these joints perform adequately, with the expected service life ranging from thirteen years to greater than sixteen years. There is a modular joint in District 4 that has been in service since the late 1970’s and is still performing well. The overall performance of modular joints varies between districts from average to highly successful.

There are a few problems described with modular joints. One district states that support bars can be a problem with support pins eventually falling out and leading to failure. If the joints are not regularly cleaned, debris builds up and causes problems. In some cases where the salt builds up in the joint, it can thicken into a cake-like substance and would need to be brushed to loosen debris before washing.

While modular joints have some problems, it is difficult for districts to stop using them at this time. Districts report that modular joints and finger joints are typically the only options for large expansion needs, so with no alternatives they have to use one or the other.

2.11.8 Sliding Plate Joints

Sliding plate joints are currently in use in all districts, but only one district uses them in new construction. Half of the districts state that sliding plate joints would perform adequately if routine repair and maintenance were performed. Of the districts that reported an average service life of these joints, most stated thirteen to sixteen years while one district stated greater than sixteen years.

Four districts would like to phase out or have phased out sliding plate joints. Districts generally believe that sliding plate joints are difficult to repair and are very susceptible to plow damage. Accordingly, over the years they are being phased out rather than repaired. The reported performance of these joints is highly variable throughout the state, with three districts reporting that they are close to an absolute failure, and two districts reporting that they are an absolute success. When these districts defined success of a joint, the three that rated the sliding plate joint close to an absolute failure all defined success as including water-tightness. The two districts rating the joint an absolute success put the emphasis of a successful joint in a long service life, while one of the two also mentioned preventing water from passing through but only as a secondary factor of success. These ratings show that joint preferences vary throughout the state based on what each district wants in a successful joint.

2.11.9 Finger Joints

Finger joints are currently in service in five of the six districts. District 4 and District 2 report using them in new construction; however, District 2 did state that they would like to phase them out and are only using them because they do not have a better option to accommodate large movements. Finger joints could be used as replacement joints if there were no time constraints, however they would be difficult to replace in the short term. Four of the districts report that finger joints perform well if routine repair and maintenance were performed; all districts state that the expected service life of these joints range from thirteen years to greater than sixteen years. The consensus of the performance of finger joints varies, with two districts reporting they are highly successful and two districts rating their performance on the lower end of the spectrum.

Finger joints are used to accommodate large expansion movements. One district reported that these used to be the large expansion joint of choice, but they have since moved to modular joints. Some finger joints that were installed in the 1960's are still holding up. However, the main problem with these joints seems to stem from the lack of maintenance. These are expensive joints to replace, so proper maintenance would be cost effective in the long term.

When the troughs under the finger joints are not regularly cleaned they can get clogged, which can lead to failure. In one case, a failed trough caused leakage to the girder underneath and resulted in an extremely costly repair of the girder. One district has started using fiber reinforced polymer (FRP) troughs in place of metal troughs, which they say will last a lot longer but are also more expensive. These FRP troughs could also be used to replace traditional neoprene troughs. Most districts try to get the troughs cleaned out at least every two years to keep them maintained, and some districts say if the inspector notes that they are in bad condition they will try to get them cleaned sooner. District 6 noted that the lower levels of multi-level bridges do not see much rainfall. Therefore, the troughs don't get cleaned out by rainwater flowing through them and tend to build up debris. For this reason, cleaning the troughs on lower decks should be done much more often.

Overall, finger joints can be expected to have a long service life compared to most other joint types, but they need to be maintained in order to keep them in service and functioning correctly. Removing these joints for replacement takes a lot of money and time, and damage to troughs can lead to damage of other elements of the superstructure. It is therefore important to routinely clean the debris from them and maintain the joint to avoid other expenses.

2.11.10 Open Joints

Basic open joints (either headers with no joint or headers where the joint has come through and only a backer rod or something similar remains) are currently in service in three of the six districts. No districts are using open joints in new construction, and no districts are using them as a repair joint. Districts report unfavorable performance of these joints, even with routine maintenance and repair. Most districts have already phased them out, or would like to phase them out in the future. When ranking the performance of open joints on a scale of absolute failure to absolute success, three districts ranked them an absolute failure while two districts ranked them neutral. District

6 did note that a go-to repair, in the past, has been to create an open joint, which was a quick fix in a district that requires extremely timely repairs. Nevertheless, it does appear that all districts have decided that the performance of open joints is not worthy of keeping them in their inventory, even if they provide a quick fix.

2.11.11 Link-Slabs

Link-slabs are currently in service in all districts. The majority of districts are using them in new construction, and all districts use them as an option for replacing existing joints with this jointless alternative where possible. Five of the six districts report that link-slabs perform well if routine repair and maintenance is performed. The average service life of link-slabs ranges from 13 to greater than 16 years, with the majority of districts reporting greater than 16 years. In ranking the performance of link-slabs, the consensus is that the performance of this joint type is highly successful.

Link-slabs have typically been used in fixed-fixed locations (between bearings that allow rotation but do not allow lateral or transverse translation), but can also be used with neoprene bearings. They have been used for retrofit of deck sections or complete re-decking. If there is an open joint and a fixed-fixed bearing (or neoprene), a link-slab can be created by putting in a continuous deck with rebar. The design manual has a simplified design for link-slabs, with design tables provided.

One factor preventing districts from using link-slabs is the cost. District 1 stated that they would prefer to use them wherever possible if they had the money to do it; they also noted that they used a link-slab in a retrofit in 1999 to eliminate a pier joint and it has been performing well with no issues. Another problem is that in districts where construction has to be done extremely quickly to avoid lane closures, link-slabs are not a practical option. When link-slabs can be used as an option, districts agree that they are a desirable choice to accommodate expansion needs and have been extremely successful from a maintenance point of view.

2.12 Massachusetts Headers

Armored headers, normal set concrete headers, and elastomeric concrete headers are types of joint headers that have been used in all districts. Five of the six districts have also used quick setting concrete. The only district that does not use quick setting concrete is District 1, which does not have high traffic volume and time constraints that the other districts face. Currently, three districts still use armored headers in some cases, all districts use normal set concrete and elastomeric concrete, and quick setting concrete is still used by the five districts with high traffic volume. Elastomeric concrete is specified as the standard to be used with strip seals (MassDOT 2013). Header type for other joints can vary slightly between districts where the decision can be based on both district preference as well as time and weather factors.

Construction issues (including improper preparation work), weather at time of installation, snow plows, and quality control are all factors that can impact header performance. One example of improper preparation was referenced when discussing elastomeric headers installed in replacement projects; if substrate is saturated in chloride it can corrode reinforcing bars and result in spalling and joint failure. When replacing headers, a saw cut is recommended to be made two feet to either side of the joint and then remove the concrete until sound concrete is reached. However, the definition of “sound” concrete is subjective, with districts noting that concrete removal is often insufficient, resulting in poor substrate conditions. Improper preparation can also lead to bond issues. When debond occurs, debris can build up and tear at the joint. Another construction issue noted by District 4 is that it is very difficult to get good concrete consolidation and to completely fill the voids, especially under the horizontal leg of the embedded steel angles. Weather conditions can affect bonding of header and setting of concrete; manufacturers typically provide specific weather conditions for installation and if these are not met then early failure can occur. Plows hitting headers is an issue especially with armored headers, where the plow can hit the steel plate and pull it out. Finally, quality control can impact header performance. The quality of headers mixed on site can be affected if specifications are not followed closely.

Armored headers that currently exist in districts are generally left in place if in good condition and just the seal of the joint will be replaced (with EM-SEAL, compression seal, etc.). However, armored joints have had many issues resulting from plow damage, which rips them out and/or causes them to protrude from the surface of the road. The risk of plow damage is increased where armored headers are used on skewed bridges; as the skew of the bridge gets closer to the angle of the plow, there is more surface area of the joint that is likely to be caught by the plow. When armored headers start becoming dislodged from impact of plows and traffic, they become noisy for drivers as they move up and down as cars drive over. Due to the plow damage, difficulty to repair quickly, and banging of these headers (when anchorage gets pulled out or failed and plates move loudly under traffic), they are not a preferred header type. Armored headers are not typically repaired because repairs tend to not remain watertight and leakage occurs where welds break; districts will tend to wait until there are funds to completely replace the armored headers with another joint type.

Normal set concrete is a header option for all six districts when there are no time constraints for construction. District 1 reported having bad experiences with elastomeric headers and therefore has changed to using 4000 psi (3/8" thick) concrete for headers instead. This district does not have high traffic volume so they are able to use normal set concrete with more flexibility in time and lane closures and they do not use quick setting concrete. These normal setting concrete headers have only been in place one to two years but are still holding up well, although minor cracking can occur. District 5 noted that their standard is to use normal set concrete headers for exposed concrete decks with deck over backwall, as well as with plug joints. District 6 stated that they would consider using normal set concrete with any joint type, however this is a district with strict traffic demands and allowing for the curing time is not typically an option unless it is on new construction.

While elastomeric headers are generally the header of choice among districts, two districts note that the use of elastomeric concrete as a replacement header can be compromised by the condition of the substrate. Elastomeric concrete header performance relies heavily on proper installation. Multiple districts point out that elastomeric concrete requires understanding of the material and mixing requirements, noting that failure to adhere to these requirements will result in premature failure. Furthermore, elastomeric concrete is highly weather sensitive and the conditions need to be perfect (based on the Watson Bowman Acme instructions) for proper adherence to the substrate. Other weather conditions can lead to premature failure. One district pointed out that some states stay away from elastomeric headers because high strength concrete becomes brittle and needs proper curing time. It was also noted that varying temperature extremes in other states may impact choice of header type. Elastomeric headers have the potential to perform well, but care should be taken to understand how to properly install them in both new construction and header replacement projects, taking special care to clean the substrate, reach sound concrete, and prepare the surface well for bonding. Some elastomeric concrete brands being used are WaboCrete (with strip seals) and Delcrete (D.S. Brown), with Delcrete being more common among the districts but both being reported as high quality when installed properly.

Quick-setting concrete is used in many districts, and is good for timely repairs that need to be done in one night. While the convenience of being able to install headers overnight is a benefit, quick setting concrete headers were reported to fail within two to three years due to the limitation of the materials and direct exposure to traffic. For districts with high traffic volume, such as District 5 on the limited access highways, they are forced to use quick-setting concrete in order to meet the public demands for speedy road work and minimize impact on traffic flow, even if this is not the ideal header material in terms of long-term durability. Duracel is one brand that has been used for quick overnight fixes, however it has had mixed reviews. District 6 reported problems with freeze-thaw performance when using Duracel and has switched to Thoroc1060 BASF as their quick-set concrete of choice. Meanwhile, District 5 was using Thoroc1060 and reports that this, too, failed freeze-thaw testing and they have now switched to CTS

Low Permeability Cement which, when mixed on site with water and aggregates, can be ready to open to traffic in 1 to 3 hours from pouring.

2.13 Massachusetts Practices: Installation

Installation workmanship is ranked by all districts as the most important factor influencing success of joint performance. Issues during construction and installation are cited as being a likely leading cause of decreased service life of joints. For new joint installation, joint type and general specifications are provided through the MassDOT Load and Resistance Factor Design (LRFD) Bridge Manual (2013). While the majority of districts agree that the specifications for installation of joints are likely adequate, they often noted that inadequate implementation of the specifications often decreases the service life and overall performance of joints.

Districts all have an engineer on site during installation. The on-site engineer could be a maintenance engineer, bridge engineer, or construction engineer depending on the district. While manufacturer specifications often state that a manufacturer representative should be on-site during installation, this does not happen for the majority of cases unless it is an early implementation of a product or material. Most districts agree that unless they are trying a new joint type, the manufacturer representative is not on-site. One district did note that they will typically have a manufacturer on site for large expansion joints, such as modular joints and finger joints, since these are more complex to install. Another district stated that company representatives have come out to projects when there have been problems with a specific joint type. It is generally the contractor’s responsibility to bring a company representative to the site, but this was noted to rarely occur unless specifically required by the field engineer.

Discussions with manufacturer representatives emphasized the importance of a manufacturer representative on site in order to ensure proper preparation work prior to installation, especially with regard to cleaning and sandblasting of the opening after the cut has been made. However, they stated that time and financial restrictions on projects often preclude them from being called to the site. Alternatively, to increase service life and joint performance it may be beneficial for manufacturer representatives to hold training sessions with each of the districts and contractors to teach the proper techniques for ensuring adequate installation practices. District 1 specifically noted that they have had manufacturers' representatives to their district office in the past, and this is a practice that appears to be worthwhile for the other districts.

The temperature at installation of joints is considered by all districts by adjusting the opening width of the joint gap. In other cases, standard tables may be provided by the manufacturer for installation, tabulating joint gaps that correspond to applicable temperature (at installation). For small expansion joints, adjusting for temperature isn't as important. Saw and seal joints and others designed for minimal movement do not need any special accommodations for installation temperatures. Most specifications for joint openings assume an installation temperature of 50°F and these are then adjusted for different temperatures. Only emergency repairs are done in the winter months, with most joint installations being performed during the summer and fall.

All districts agree that most water leakage starts where the joint meets the face of the barrier because it is difficult to ensure continuity of the seal at the sharp change of plane and direction. This construction detail can be difficult to perform with a backer rod, however care should be taken to ensure this step of installation is performed correctly to prevent this common area of leakage.

After new joint installation, MassDOT contract specifications require a “Watertight Integrity Test” for strip seals but this is called out in special specifications for other joints too according to most districts. The watertight integrity test states that at least five workdays after the joint system has been fully installed the contractor shall test the full length of the system for watertight integrity to the satisfaction of the engineer on site. The entire joint system shall be covered with water (either ponded or flowing) for a minimum of fifteen minutes. During these fifteen minutes, and for a minimum of forty five minutes after the water supply has stopped, the concrete surface under the joint shall be inspected for any evidence of penetrating water or moisture. Water tightness shall be defined as no dripping water on any surface on or outside the joint. If there is any evidence of leaking, the contractor must determine the location(s) of leaking and take all measures necessary (which must be approved by the engineer) to stop the leak. This work shall be done at the contractor’s expense and a subsequent test must be performed to the same conditions as the original test, with the same steps taken if there is still leakage, until the joint is fully water tight.

While the watertight integrity test is outlined in the specifications, one of the districts noted that it is not always done. Furthermore, this is generally only required for new construction of strip seals with no requirements for repaired or replaced joints.

For joint repair there is no warranty on the work, while for joint replacement there is a warranty of 2 years. Districts report that getting a contractor to come back out to do the work under warranty is generally difficult so they are rarely able to enforce it. There is typically no recourse once construction ends.

Districts provided information on installation practices that they believe positively influence overall joint performance. The majority of districts noted that proper workmanship during installation greatly benefits joint performance.

One district noted that in skewed bridges, concrete is poured continuously but can only be rake finished by machine over the straight portion of the bridge. Therefore, the parts of the bridge near the skew need to be hand-raked, which can lead to problems if done poorly. This is something contractors, inspectors, and resident engineers should be aware of to look out for during skewed bridge construction.

Surface and substrate preparation and cleanliness is another influencing factor; sandblasting is in the specifications for most joint installations and yet it is rarely done due to either time or money constraints. This step can make a major difference in the success of a joint, as noted by districts and manufacturers alike.

2.14 Massachusetts Practices: Maintenance

Maintenance is critical to joint performance, but all districts noted a lack of funding to adequately perform preventive maintenance. District 1 used to have a maintenance crew (until 1995) that cleaned bridges annually. The cleaning included blowing out or washing out joints in the spring, cleaning decks, and cleaning the bridge substructure and underneath bridges in the summer. The district noted significant improvement in the joint performance and overall bridge condition when this maintenance was routinely performed. The majority of districts pointed out that routine cleaning of joints would be very worthwhile and would have a positive impact on joint performance.

District 5 came up with a “Bridge Maintenance Policy” a few years ago that received very positive feedback from the Federal Highway Administration (FHWA). Unfortunately funds were not forthcoming to implement the proposed program. The ideal maintenance routine for the district would be systematic maintenance through a corridor style contract where a contractor would work their way down the highways systematically cleaning joints. A contract to power wash bridges was expected to cost approximately $300,000 in its entirety.

While preventive maintenance is not routinely done, inspectors do typically flag bad joints to prioritize work according to safety and traffic volume concerns.

District 2 reports that bridge deck and joint washing/cleaning, re-sealing of joints, and cleaning of bins/troughs under finger joints is currently being performed. District 6 does do some cleaning with maintenance crews, which focuses on blowing out/scratching out debris from joints then running a sweeper through. In general, the funds are not available in any of the districts to do the level of routine, systematic, or preventive maintenance that they would like. Life cycle costing that includes the cost of repairing and replacing joints, as well as the damage that can occur as a result of failing joint systems, was noted to show clear benefits of a full maintenance program. However, the maintenance and construction budgets are separated such that maintenance budgets do not benefit from savings in construction costs. Additionally, substructure repair resulting from joint failure is handled separately from joint maintenance personnel and within separate budgets. An overall perspective that considers the cause and effect of failing joints and looks at overall life-cycle costs would be worth implementing. This would need to include budgetary reward between construction, repair and maintenance budgets, including both superstructure components and elements such as joints.

2.15 Massachusetts Practices: General

2.15.1 Design, Provisions, Specifications

Districts agree that existing general specifications are generally very good, but the implementation in the field by the contractor or inspector needs improvement. One suggestion of a possible approach to fixing this is to have statewide training for inspectors of all districts, as well as resident engineers that will be on-site. The training should have everyone go through the steps of what is expected during construction per the specifications.

While there are special provisions that source from Boston headquarters, at least one district noted that they have provisions unique to the district now where they have added onto the original specifications with practices they have had success with.

When it comes to new joint installation, the districts are limited to joint types in the MassDOT LRFD Bridge Manual (2013). Joint replacements also have guidelines in the Bridge Manual. There are no standard repair details through the Bridge Manual; this is generally left up to the individual districts to decide on the best approach for repair. District 5 noted that joint performance could likely be improved if preservation specifications were detailed. These would include steps to take after construction to keep the joints functioning properly instead of relying on reactive maintenance once problems have already occurred.

2.15.2 Joint Repairs

District 3 noted that the range of expansion is sometimes overestimated in existing bridges. Therefore, they re-evaluate and replace with the most relevant joint type, not necessarily one similar to the existing one. When replacing joints (not the entire deck) the district can decide on the replacement joint; although there are replacement guidelines in the Bridge Manual, they have the ability to determine the best method they want to use within the individual district.

Repairs are more difficult in the districts with high traffic volume, often requiring an 8pm to 5am (Monday through Friday) time period, which leads to restrictions on what methods, materials and joint types are applicable.

Field splicing is only allowed on certain joint types. In general, all districts agree on the materials that can be field spliced. Neoprene seals (both strip seals and compression seals) shall be continuous and may not be spliced under any circumstance. EM-SEAL type seals (silicone) are the only seals that can be field spliced. Asphalt plug joints may be spliced in the field. For non-seal materials, field splicing is allowed by welding of steel strip seal rails and in some cases steel headers.

Deck replacements vary by district. While some districts reported typical deck life of 25 years, this does not necessarily mean that re-decking is done this often. District 6 reports that re-decking is done every 25-35 years maximum (this district is the Boston area district with extremely high traffic volume which likely wears down surfaces more quickly). Decks can be in service over 40 years in other districts, and District 4 reports decks are still in service from the 1960’s (50+ years). District 5 reported that re-decking is rare, and it generally turns into full structure or superstructure replacement. When deck replacement is done, the design choice could be done in-house or by consultants, depending on the district. Joints are replaced during re-decking, but this does not necessarily mean changing joint types. When deck is replaced it is classified as new construction, therefore joint types need to come from MassDOT LRFD Bridge Manual.

2.16 Survey on Bridge Joints

2.16.1 Overview of Survey and State Responses

A survey was sent to state Departments of Transportation (DOTs) in and around New England in order to collect responses regarding other state's bridge joints, construction, repair, and maintenance practices. The survey was sent to DOTs in Connecticut, Maine, Massachusetts, New Hampshire, New Jersey, New York State, Pennsylvania, Rhode Island, and Vermont. The survey was also sent to and completed by the MassDOT Districts in order to have direct comparisons to other states' responses. The survey was sent to 58 people (district engineers and chief engineers); two weeks after the survey was emailed, follow-up emails and/or phone calls were sent to gather as many responses as possible. There were a total of 26 responses to the survey (45% response rate). For DOTs in Maine and Rhode Island, the survey was sent to the Chief Engineer's assistant to distribute. The objective of this survey was to give insight into performance of joints in selected states in the northeast.

The survey was organized into five topics: joints, headers, new installation and repair, maintenance, and overall practice, with a total of 44 questions. Due to the large number of responses, all tables and figures were not included in this chapter, but they have instead been summarized in the following sections of individual state responses, with an overall summary of all state responses at the end of this chapter.

For some states, there were multiple respondents to the survey. However, not all respondents provided answers to all questions (or for all joint types). Therefore, some of the tables and figures will show multiple responses to some questions, and no responses or one response to another. The results presented show all answers provided. In the plots where an average value is shown, the average may not be the midpoint of the maximum and minimum rating because in many cases multiple respondents selected a single point or selected a value between the maximum and minimum, so the average shown is weighted.

2.16.2 Connecticut

Connecticut currently has six joint types in use: asphalt plug joints, compression seals, sliding plate joints, finger joints, modular joints, and saw and seal: deck over backwall. For new construction and joint replacement, however, only asphalt plug joints and modular joints are being used (both for overnight construction and construction without time constraints).

Asphalt plug joints and modular joints are the only joints that they believe perform adequately if routine repair and maintenance are performed. The typical service life experienced with these, however, is quite short: for asphalt plug joints (zero to four years), while modular joints have a longer service life of thirteen to sixteen years. These service lives are presented in Table 2.23.

Table 2.23: Typical Service Life of Joints (Connecticut Survey Respondent Answers)

Connecticut: Typical Service Life of Joints		
Years	**Asphalt Plug Joint**	**Modular Joint**
0-4	1	
5-8		
9-12		
13-16		1
>16		

In Connecticut, a successful joint is defined as one that provides good ride-ability and water tightness. Failure of a joint is defined as one that leaks or has poor ride-ability. Connecticut rated the performance of both asphalt plug joints and modular joints as neutral. These ratings are shown in Figure 2.26. The importance of multiple factors to joint performance was rated and is shown in Figure 2.27 (note that maintenance practices were not rated).

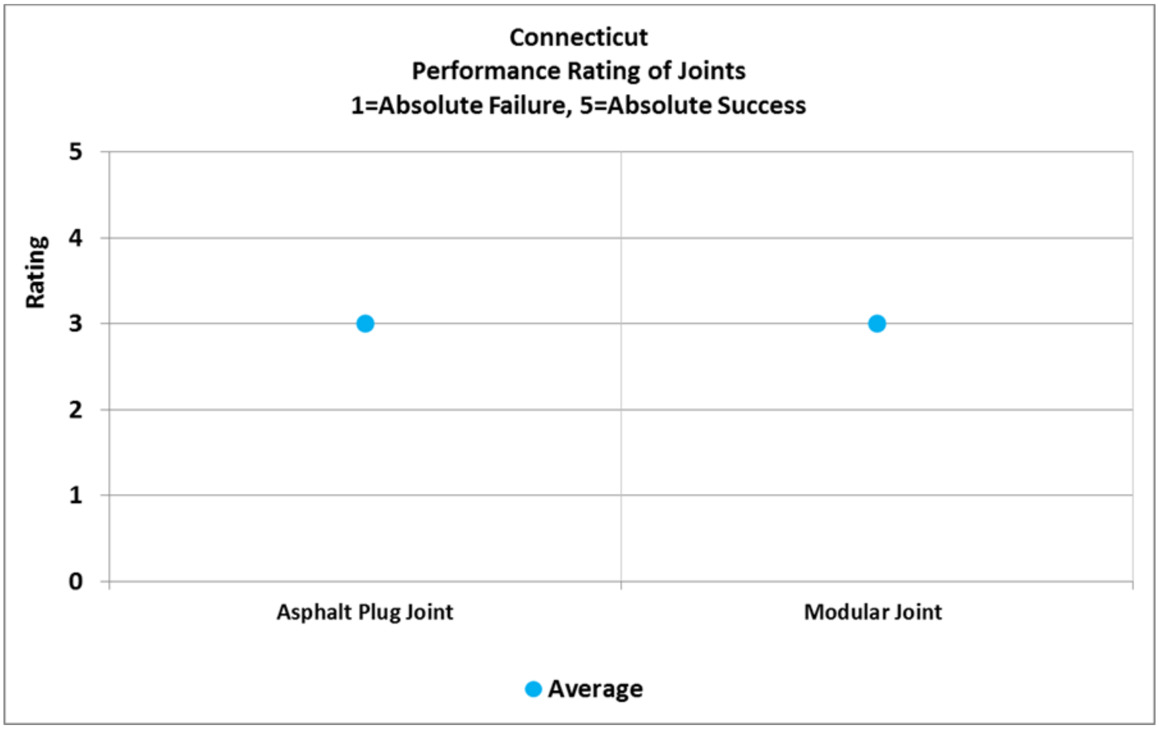


Figure 2.26: Performance Rating of Joints (Connecticut Survey Respondent Answers)

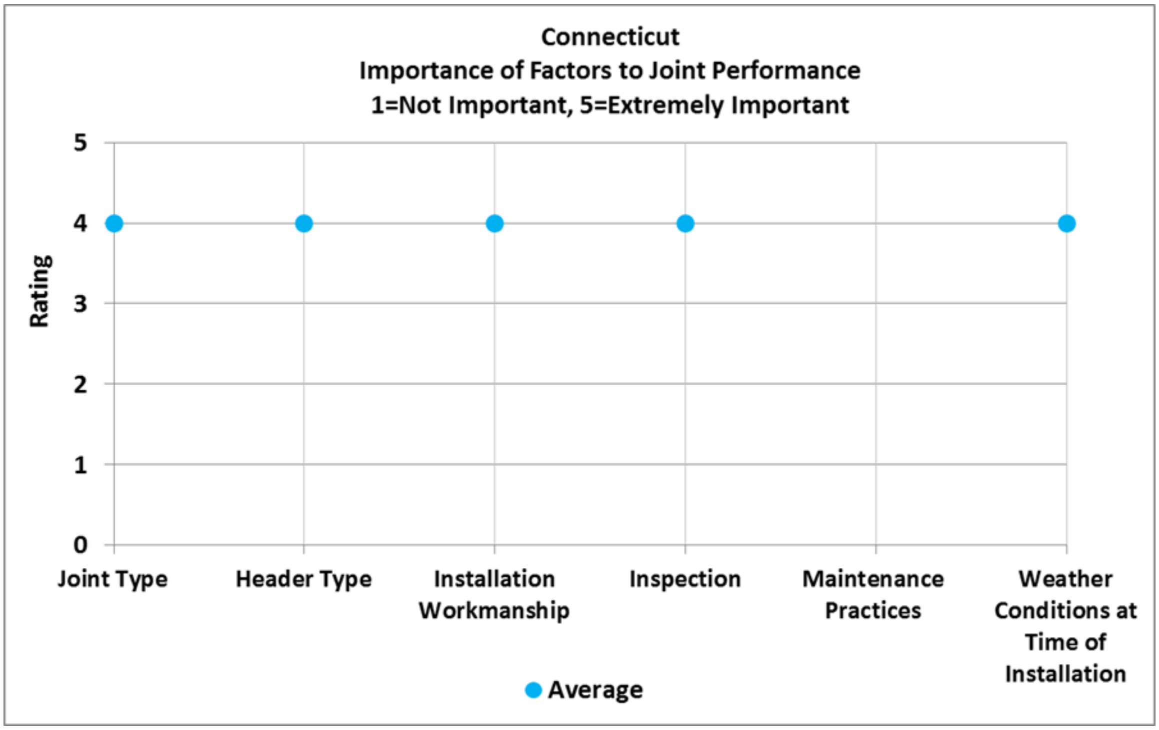


Figure 2.27: Importance of Factors to Joint Performance (Connecticut Survey Respondent Answers)

For headers, currently either normal setting concrete or elastomeric concrete are used when there are no time constraints, and quick setting concrete is used when overnight construction is required. Armored headers are not currently used, and were not used in the past according to the respondent.

Temperature is considered for joint installation. No testing is done (such as watertight testing) to verify proper installation or repair. There are also no required weather conditions for installation of joints or headers. It was noted that lack of attention to proper surface preparation prior to joint installation or dampness of the base concrete can both negatively impact the performance of joints. Field splicing is not allowed on repairs of any joint type. The manufacturer's representatives are sometimes on site to oversee joint work at time of construction. There are standard joint and header replacement details for Connecticut, but no standard repair details. The respondent did not comment on anti-icing chemicals, preventive maintenance, or whether there is a bridge maintenance manual separate from the design manual.

2.16.3 Maine

Maine is one of the states with the broadest range of joint types in use. The current joints in service in Maine include asphalt plug joints, strip seals, compression seals, pourable seals, EM-SEAL, sliding plate joints, finger joints, modular joints, link-slabs, open joints, saw and seal: deck over backwall, and Silicoflex. In new construction, asphalt plug joints, strip seals, compression seals, EM-SEAL, finger joints, modular joints and saw and seal: deck over backwall are used. For replacement projects, asphalt plug joints, strip seals, compression seals, EM-SEAL, finger joints and saw and seal: deck over backwall are used when there are no time constraints. If overnight replacement is required, asphalt plug joints, strip seals, compression seals, EM-SEAL or finger joints are used.

With routine repair and maintenance, all respondents stated that the joints that perform adequately are asphalt plug joints, compression seals, EM-SEAL, and finger joints. Half of the respondents also added strip seals, sliding plate joints and saw and seal: deck over backwall to this list. The typical service life of joints was rated, and the joint with the shortest service life is the pourable seal, followed by asphalt plug joints. The greatest variability in responses was for the service life of modular joints. Many of the joints have typical service lives greater than sixteen years. All responses are presented in Table 2.24.

Table 2.24: Typical Service Life of Joints (Maine Survey Respondent Answers)

Maine: Typical Service Life of Joints											
Years	**Asphalt Plug Joint**	**Strip Seal**	**Compression Seal**	**Pourable Seal**	**EM-SEAL**	**Sliding Plate Joint**	**Finger Joint**	**Modular Joint**	**Open Joint**	**Saw and Seal: Deck Over Backwall**
0-4				1						
5-8	1							1		
9-12		1	1		1					
13-16										
>16			1		1	1	2	1	1	1

All respondents from Maine stated that they would like to phase out modular joints, while half of the respondents would like to phase out all joints. It was noted that joints leak, which leads to multiple problems when not properly maintained, including issues with bearings, beams, abutment/pier concrete, etc. Joints are difficult to install properly to provide a smooth ride, particularly when being replaced as part of rehabilitation efforts.

In Maine, success of a joint is defined as one that does not leak and can remain in place for 20 years without having to do any major work on it. Failure of a joint is defined as one that leaks, falls apart shortly after it is installed requiring emergency measures to repair, or one that leads to other issues when not properly maintained. Joint performance was rated and is shown in Figure 2.28. None of the joints were rated an absolute success, however the joints rated close to an absolute success are EM-SEAL, sliding plate joints, finger joints, and saw and seal: deck over backwall. Open joints, modular joints, and pourable seal joints have the lowest performance rating. When rating the importance of multiple factors to joint performance, the most important factors were installation workmanship and inspection. These ratings are all shown in Figure 2.29. The most promising new products are Silicoflex and EM-SEAL. MaineDOT is also looking at/using on a trial basis joints using heavy steel angles for joint armor and steel plates/rebar hoops for anchorages. They are very heavy duty joints but use readily available materials and the welding details are relatively simple.

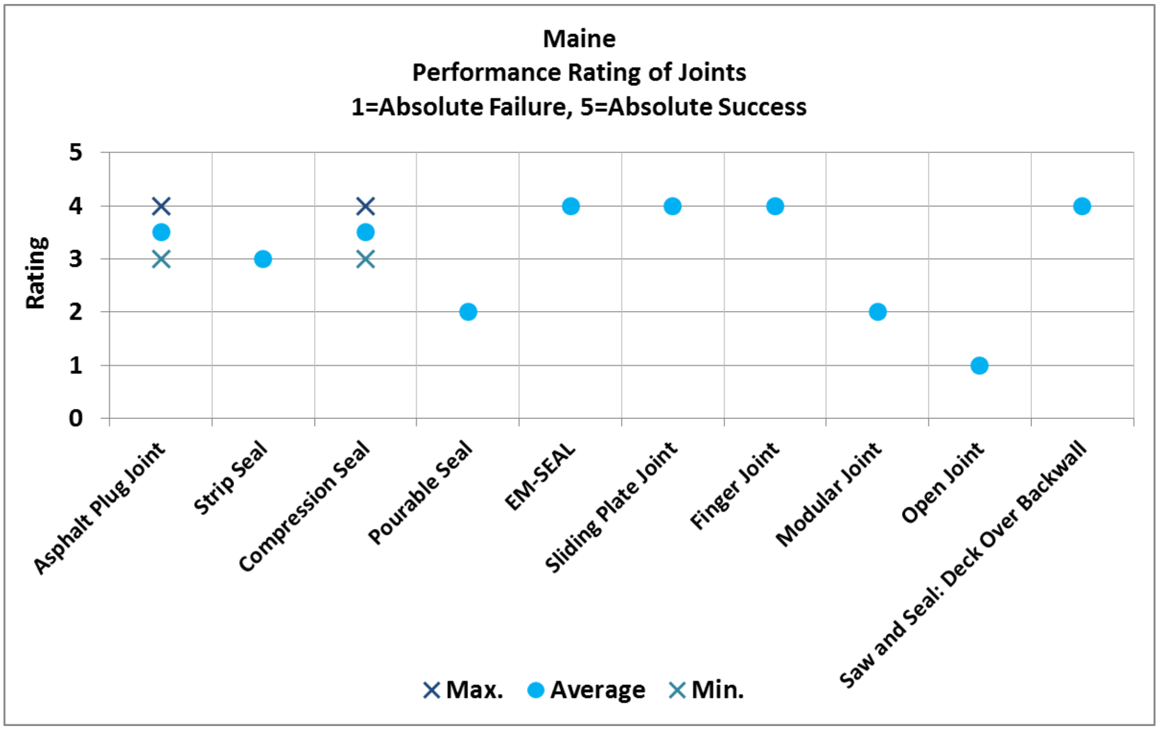


Figure 2.28: Performance Rating of Joints (Maine Survey Respondent Answers)

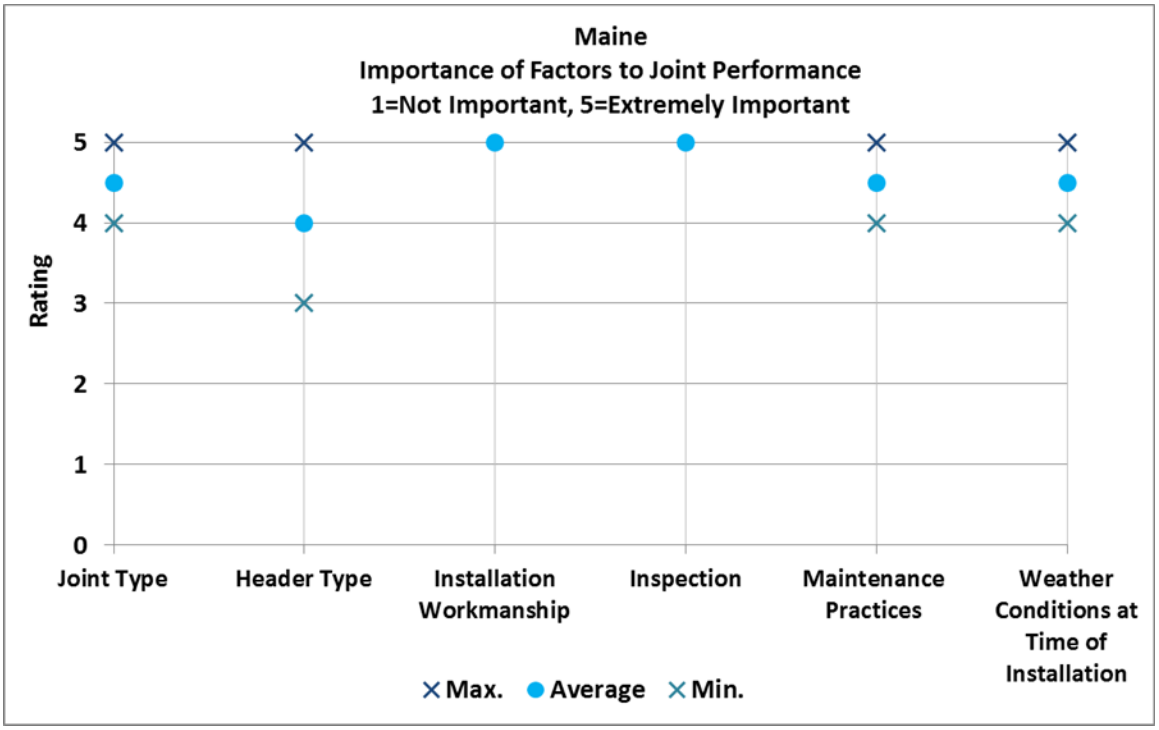


Figure 2.29: Importance of Factors to Joint Performance (Maine Survey Respondent Answers)

Headers currently in use in Maine are elastomeric concrete and quick setting concrete (for both overnight construction and when there are no time constraints), and normal setting concrete and armored headers when there are no time constraints. There is not a specific material used for extreme cold weather, but according to one respondent any material used must meet the requirements of the specifications for maintaining temperature, or must be installed per manufacturer requirements. Elastomeric headers have had some problems in Maine. They can have poor adhesion to substrate, difficulty in providing proper grade, expansion of material once placed, and some material failures after it is in place for a short time. While the reasons for these issues are not known, respondents stated that it may be due to improper mixing of components, moisture on substrate, or environmental conditions.

Temperature is considered during installation of joints by adjusting the opening. However, one practice that has negatively impacted joint performance is installing them when the temperature is too warm. Another factor with a negative influence is inadequate cleaning prior to installation. There is no testing done to ensure proper installation of joints. One respondent noted that joints should be set in the fall, and this has been found to positively influence joint performance. Field splicing is allowed on some joints, although joint types were not specified. The manufacturer's representative is sometimes on site to oversee joint installation. Some practices that negatively impact joint performance include inadequate cleaning prior to installation. Furthermore, some standard joint details show use of steel studs to anchor joint armor into concrete where the studs can be installed with a stud welding machine or by "stick" welding. Preventive maintenance is performed by doing annual cleaning. For anti-icing of roads, Maine is a salt priority state.

In Maine, they have found that the use of studs may not be the most reliable means of anchoring the steel, due to the quality of the weld. They suggest using other options such as steel straps or rebar welded to the joint armor. In addition, they found that their standard details had not been changed or been reviewed for many years so there have not been design checks for the number/spacing of studs for some time. When this was reviewed, they found that in some cases the number and spacing of studs was inadequate for the loads being applied. There are no standard replacement details or repair details for joints and headers. There is a bridge maintenance manual for Maine.

2.16.4 Massachusetts

Massachusetts currently has asphalt plug joints, strip seals, compression seals, pourable seals, EM-SEAL, sliding plate joints, finger joints, modular joints, link-slabs, open joints, saw and seal: deck over backwall, and saw and seal: over existing joint. In new construction, all but pourable seals, EM-SEAL, and open joints are currently used. Not all districts use the same joint types for new construction, however. All but open joints would be used in replacement projects if there is no time constraint, while for overnight construction the majority of districts would use asphalt plug joints and strip seals, with fewer districts also noting they would use saw and seal: deck over backwall and over existing joint, compression seals, pourable seals, EM-SEAL, and link-slabs.

With routine repair and maintenance, all respondents believe strip seals perform adequately while the majority of respondents also believe link-slabs, saw and seal: deck over backwall, compression seals and asphalt plug joints also perform adequately under these conditions. The typical service life of joints is presented in Table 2.25. Asphalt plug joints had the most variability in responses, ranging from zero to sixteen years. The shortest service life was assigned to pourable seals and EM-SEAL. Joints to which the most respondents assigned a service life of over sixteen years include finger joints and link-slabs.

Table 2.25: Typical Service Life of Joints (Massachusetts Survey Respondent Answers)

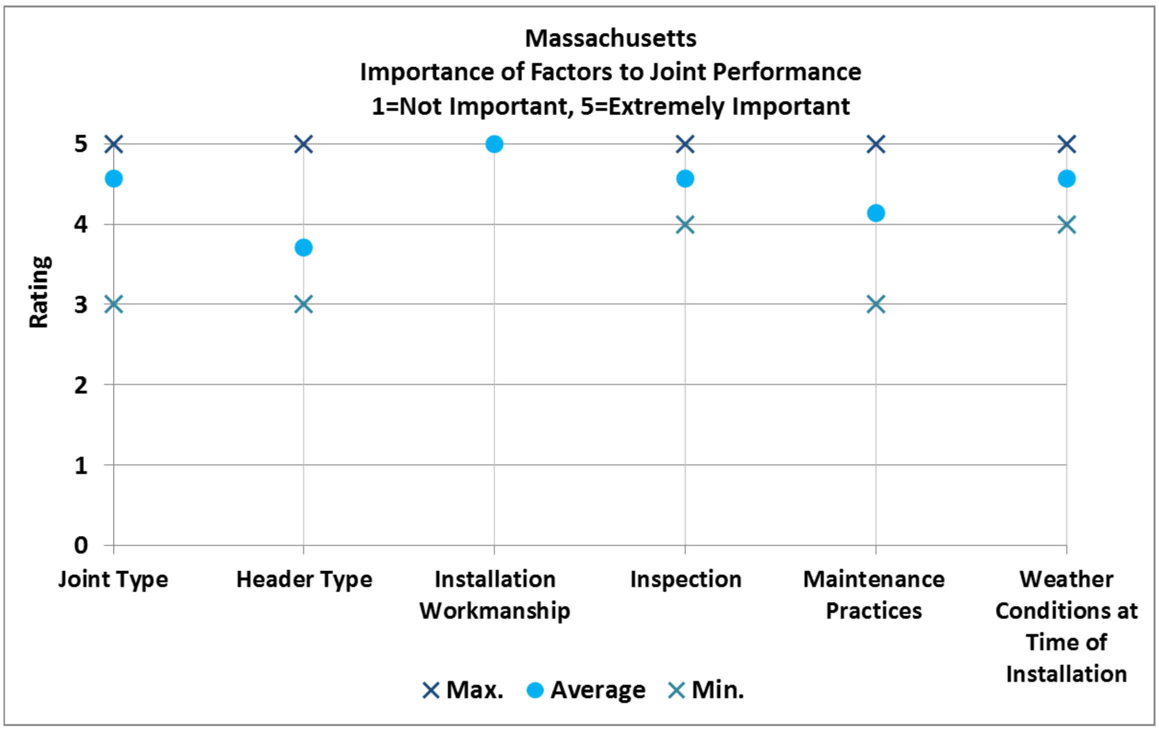
Massachusetts: Typical Service Life of Joints											
Years	**Asphalt Plug Joint**	**Strip Seal**	**Compression Seal**	**Pourable Seal**	**EM-SEAL**	**Sliding Plate Joint**	**Finger Joint**	**Modular Joint**	**Link-Slab**	**Saw and Seal: Deck Over Backwall**	**Saw and Seal: Over Existing Joint**
0-4	2			3	1						
5-8	3		1							1	
9-12	1	2	1							2	1
13-16	1	3	3			3	2	2	2		
>16		1				1	3	1	3	1	

Joints that have been phased out by at least one district include: asphalt plug joints, compression seals, pourable seals, sliding plate joints, finger joints, and open joints. Compression seals had the most respondents that have or want to phase them out. Other joints that at least one respondent would like to phase out moving forward include: asphalt plug joints, strip seals, pourable seals, sliding plate joints, finger joints, modular joints, and open joints. Reasons for wanting to phase out joints types include difficulty to maintain, expensive to repair, rutting of material, or leaking issues.

The definition of success ranges slightly between respondents, with some emphasizing that a long service life is important and others emphasizing that the joint has to be watertight. Other attributes of a successful joint would include smooth ride-ability and requiring minimal maintenance. Failure definitions include a joint that leaks and one that becomes damaged to the point it does not provide a smooth riding surface.

Performance of joint types is presented in Figure 2.30. The highest success ratings were assigned to link-slabs, saw and seal: deck over backwall, strip seal, EM-SEAL, finger joints and modular joints. The lowest rated joints were open joints and pourable seals.

The importance of multiple factors to joint performance is presented in



. The most important factor, unanimously, is joint installation.

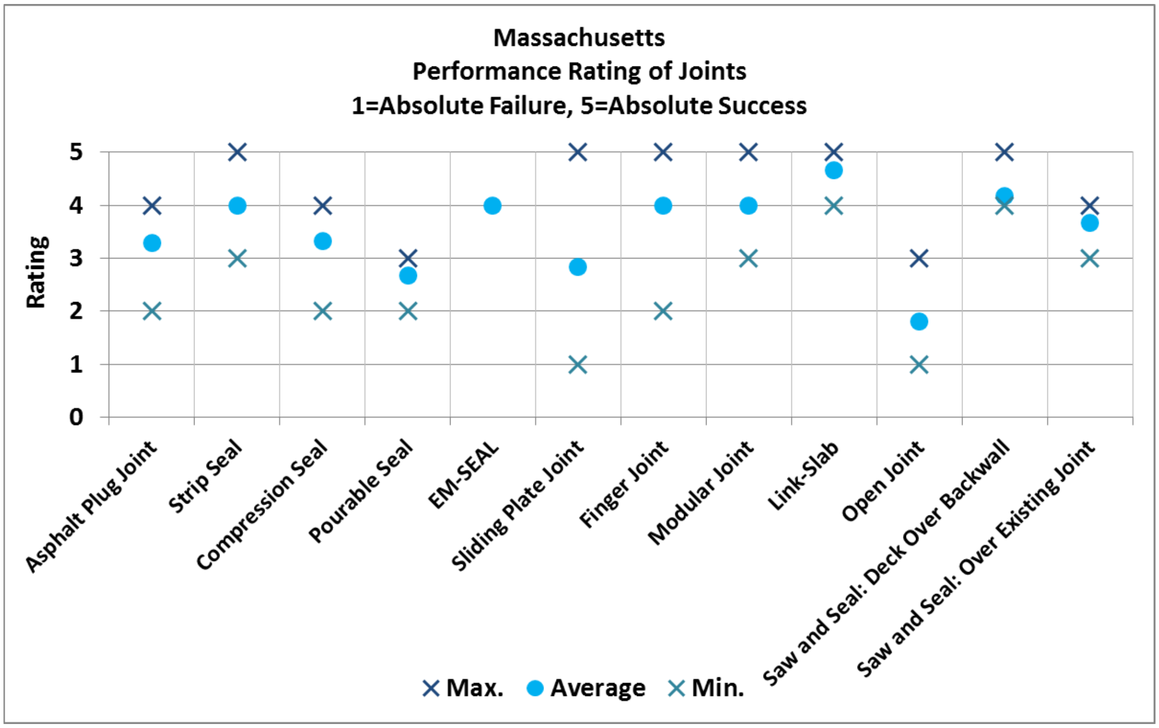


Figure 2.30: Performance Rating of Joints (Massachusetts Survey Respondent Answers)

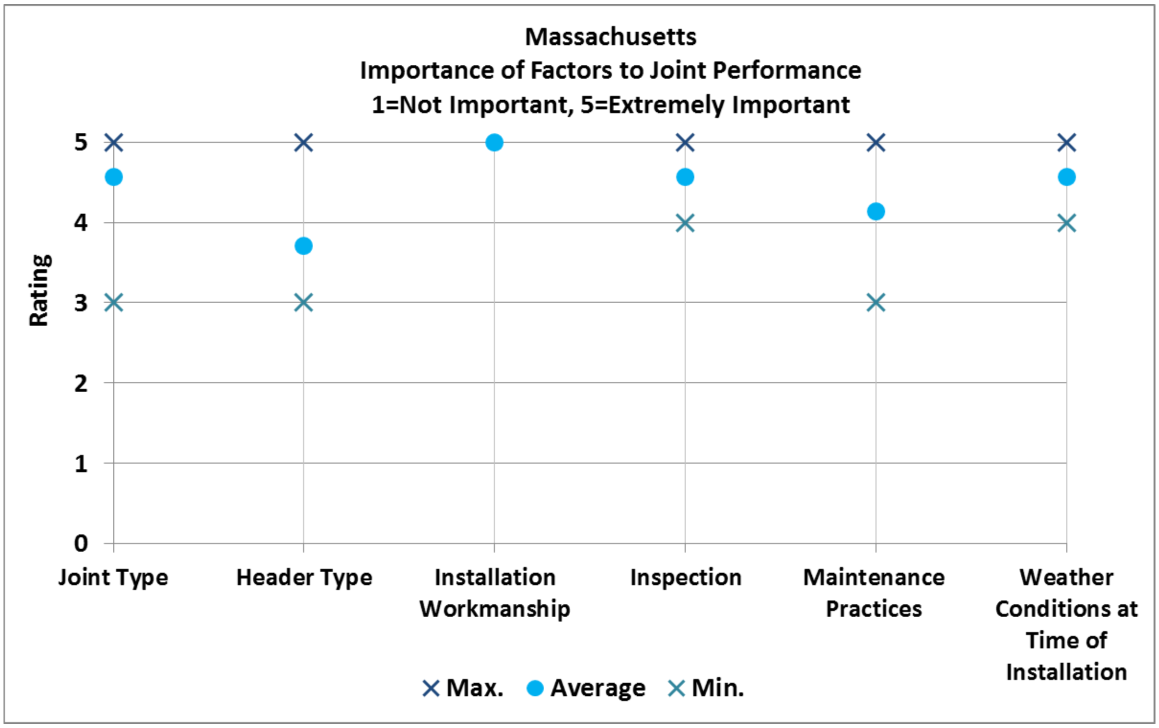


Figure 2.31: Importance of Factors to Joint Performance (Massachusetts Survey Respondent Answers)

Installation practices that have negatively impacted joint performance include lack of inspection at time of construction, not adhering to stated requirements for joint installation, and poor workmanship. All districts and manufacturers agree that lack of surface/substrate preparation and not taking measures to properly clean prior to joint installation are leading causes of early failure of joints. This lack of proper preparation and cleaning leads to many problems (including debonding, corrosion of joint from chloride in substrate, lack of adherence of joint materials, etc.).

Strict quality control is another factor influencing joint success; one district has noted that they have implemented quality control standards in their district and states that it has had a very positive affect on joint performance so far. Using high quality materials is important to the success of joints.

Consideration of weather conditions at time of installation is another important factor, ranked highly in the level of importance by all districts.

Headers currently in use are armored headers, normal set concrete, and elastomeric headers. Quick setting concrete is only used if overnight work is needed, while elastomeric headers may also be used in this case. Armored headers have had issues with plow damage and are not easily repaired, so most districts prefer not to use them anymore. With strip seals, elastomeric concrete is specified in the standard details. Other joint types have headers based on the district's preference.

Officially, there are no standard repair details for joints and headers. However, some districts responded that there are standard details of replacement joints and headers available. Temperature is considered at the time of installation. Recommendations from manufacturer specifications should be followed, as well as making proper adjustments. It is not only important to adhere to temperature recommendations and make proper adjustments, but also not to install the joint in any extreme temperatures (hot or cold), as this affects materials and the opening size for the joint. Watertight testing is typically done after a new strip seal joint installation. There is no watertight testing done on an asphalt plug joint at any time during or after construction. Field splicing is allowed on some joints; these joints vary by district. There is no bridge maintenance manual currently in Massachusetts. Some districts perform bridge deck and joint washing/cleaning as well as cleaning of troughs, but most districts do not have the funds to perform the preventive maintenance that would prolong joint life. For anti-icing, sodium chloride (rock salt) is used, and one district noted sand may also be used.

2.16.5 New Hampshire

New Hampshire has five joint types that are currently in service as well as used for new construction: asphalt plug joints, strip seals, compression seals, finger joints, and modular joints. While all of these are used as replacement joints when there are no time constraints, only asphalt plug joints are used for overnight replacements. All of the joint types used are believed to perform adequately if routine repair and maintenance are performed. Sliding plate joints have been phased out of use in New Hampshire due to issues with leaking.

The average service lives of joints are presented in Table 2.26. Asphalt plug joints and pourable seals have the shortest service life of zero to four years, while compression seals, finger joints, and modular joints have the longest service lives at more than sixteen years.

Table 2.26: Typical Service Life of Joints (New Hampshire Survey Respondent Answers)

New Hampshire: Typical Service Life of Joint Types						
Years	**Asphalt Plug Joint**	**Strip Seal**	**Compression Seal**	**Pourable Seal**	**Finger Joint**	**Modular Joint**
0-4	1			1		
5-8						
9-12						
13-16		1				
>16			1		1	1

A successful joint in New Hampshire is defined as one that does not leak, requires low maintenance, is repairable, is durable and lasts longer than twenty years. A failed joint would be defined as a joint that leaks, has seals that have fallen out, or has cracks (specifically in asphalt plug joints). Considering these definitions, joint types were rated on their performance. The highest rated joint in New Hampshire is the compression seal, and the lowest rated joints are the pourable seal, sliding plate joint, and saw and seal joints. All ratings are shown in Figure 2.32. The importance of multiple factors to joint performance is presented in Figure 2.33. They rate all factors as highly to extremely important.

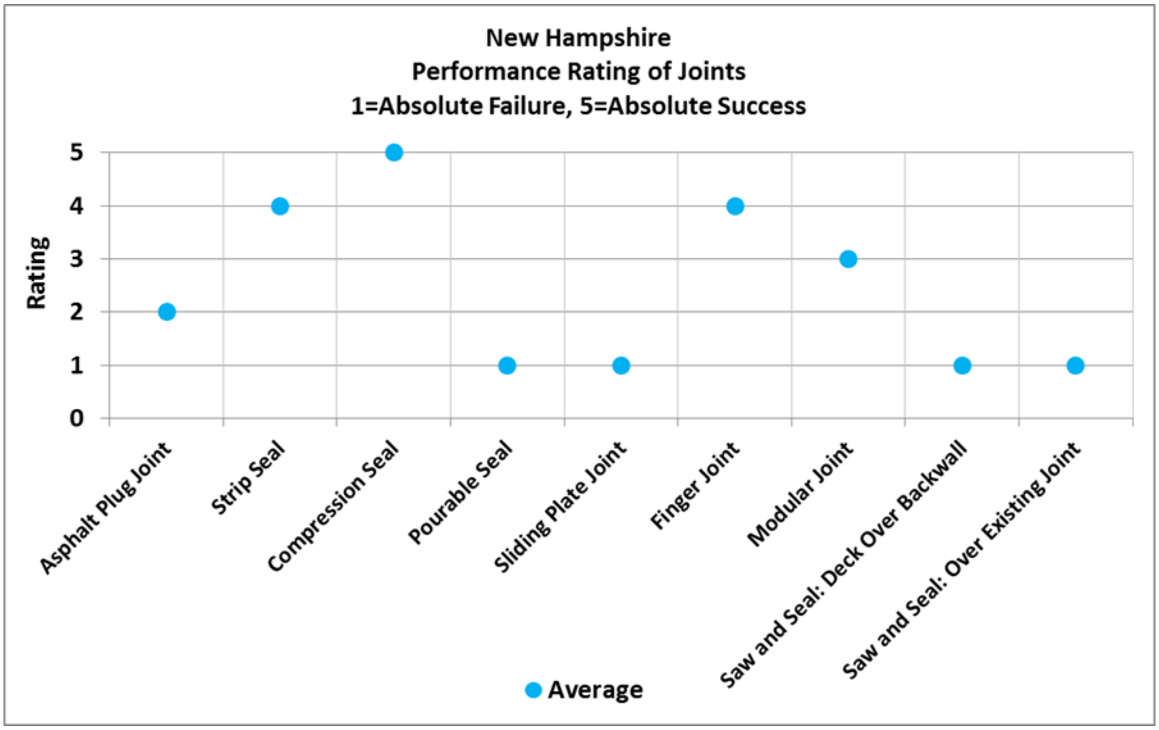


Figure 2.32: Performance Rating of Joints (New Hampshire Survey Respondent Answers)

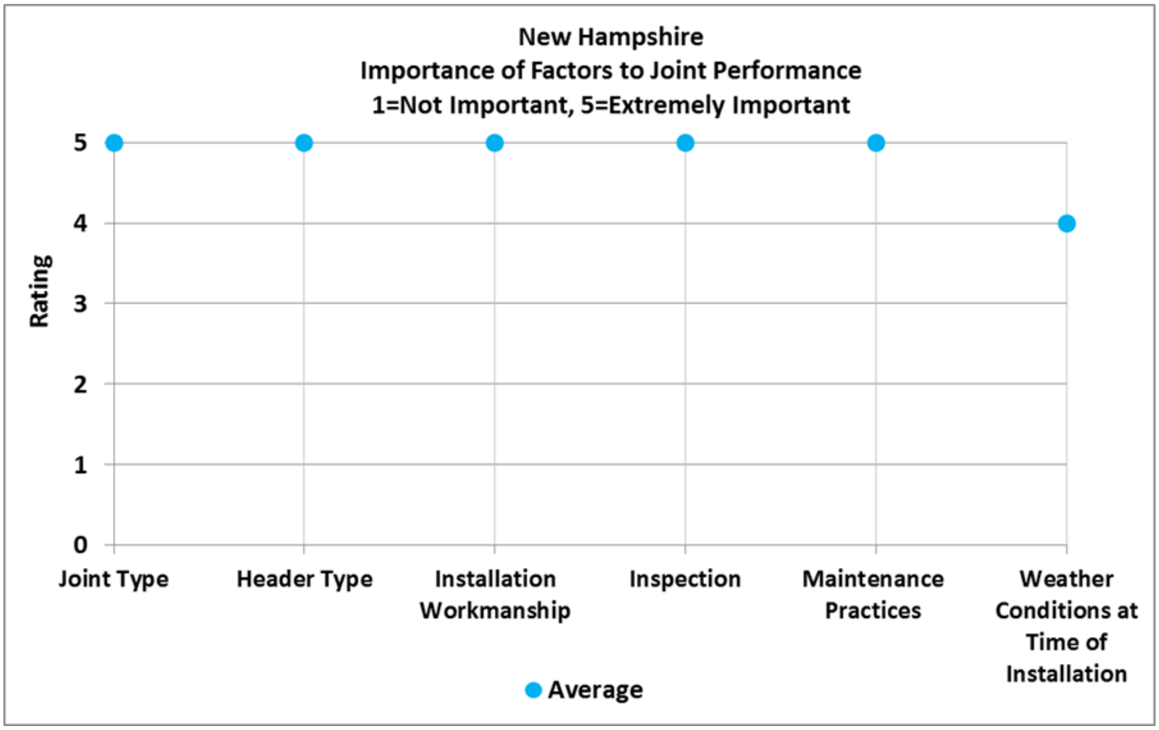


Figure 2.33: Importance of Factors to Joint Performance (New Hampshire Survey Respondent Answers)

Headers currently being used in New Hampshire are armored headers and normal setting concrete when there are no time constraints. Quick setting concrete and elastomeric concrete are used in overnight construction. It was noted that using steel angles and large anchorage is working well, while elastomeric headers have been noted to de-bond and then require replacement.

There are standard new design and replacement details for the state, but no standard repair details. Temperature is considered at the installation of the joint; the joint is sized assuming 150°F temperature change and approximately 65°F at installation. The joint is then installed and set for the current temperature prior to pouring the concrete headers. There are no specific weather condition requirements for installation of joints or headers. Field splicing is permitted on compression seals, strip seals, finger joints and modular joints. It was noted that installation practices that negatively impact joint performance include material and installation not installed according to specification, or not according to design plans. New Hampshire has a preventive maintenance program in which joints are cleaned of debris annually. For anti-icing chemicals, New Hampshire uses rock salt and pre-wetting treatment of calcium chloride.

The State Contract Administrator oversees installation and testing of joints. The watertight integrity test is performed five work days after the joint system has been fully installed. The contractor tests the entire length of the joint system for watertight integrity by employing a method agreed upon by the engineer. After either ponding or pouring flowing water over joints for a minimum of 15 minutes, the concrete surfaces under the joint are inspected. The concrete surfaces are also checked for a minimum of 45 minutes after the water supply has stopped for evidence of dripping water or moisture. Free dripping water on any surface on the underside of the joint is not accepted, while patches of moisture are not cause for non-acceptance.

2.16.6 New Jersey

In New Jersey, the joints currently in service are: asphalt plug joints, strip seals, compression seals, pourable seals, EM-SEAL, and finger joints. For new construction, only strip seals and compression seals are used. These are also the two joint types used for replacements when there are no time constraints. When overnight replacement is required, pourable seals or EM-SEAL are used. Joints that typically perform well with routine repair and maintenance are compression seals and EM-SEAL.

The typical service life of joint types is presented in Table 2.27. Finger joints and open joints have the longest service life of more than 16 years. The shortest service life was assigned to asphalt plug joints and pourable seals with a service life of zero to four years. These are the two joint types that have been or currently are being phased out. Pourable seals exhibit adhesion issues during the winter months when bridges contract, while asphalt plug joints are not able to hold up to truck traffic.

Table 2.27: Typical Service Life of Joints (New Jersey Survey Respondent Answers)

New Jersey: Typical Service Life of Joints							
Years	**Asphalt Plug Joint**	**Strip Seal**	**Compression Seal**	**Pourable Seal**	**EM-SEAL**	**Finger Joint**	**Open Joint**
0-4	1			1			
5-8					1		
9-12		1					
13-16			1				
>16						1	1

Success of a joint in New Jersey is defined as a joint that can form a water-tight seal and expand and contract with the bridge. Failure of a joint is one that does not meet these standards. EM-SEAL and compression seals have the highest rating of an absolute success, while asphalt plug joints received the lowest rating of an absolute failure with pourable seals not performing much better. All ratings are presented in Figure 2.34. In rating the importance of various factors to joint performance, the installation workmanship, inspection, and maintenance practices were rated as the most important. The joint type and header type are not believed to have as significant an impact as these other factors. These ratings are presented in Figure 2.35. Note that “weather conditions at time of installation” was not rated by the respondent, so no data is presented for this factor.

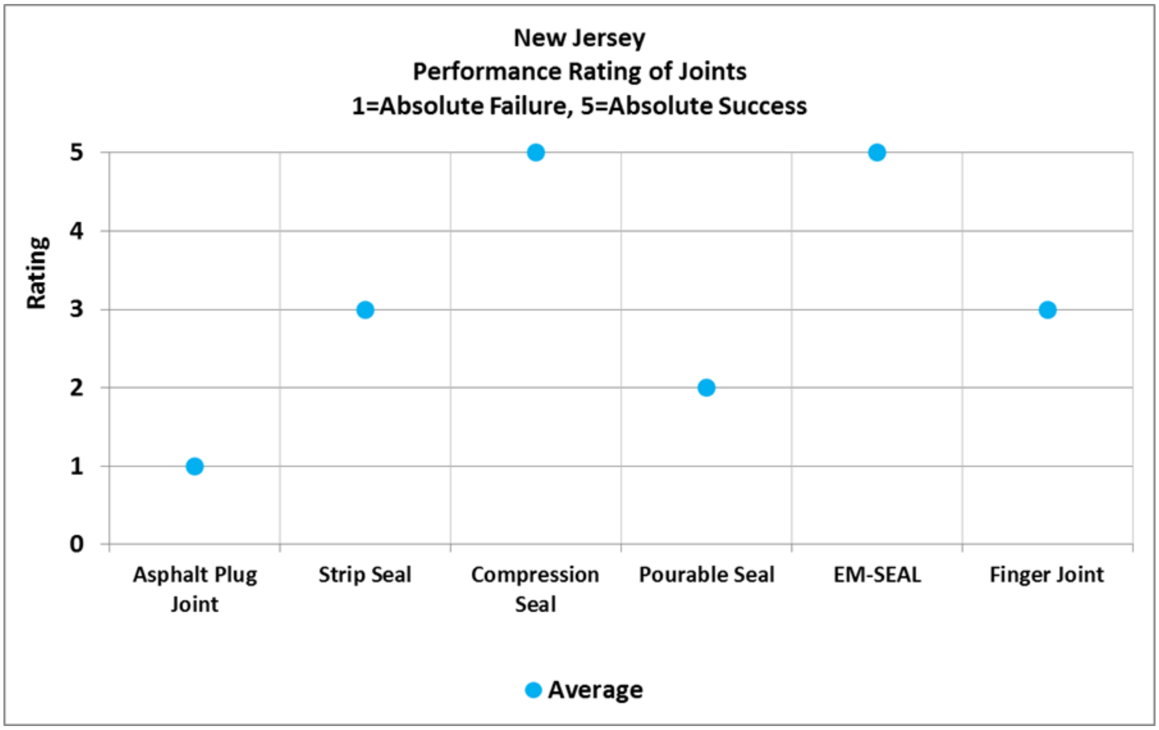


Figure 2.34: Performance Rating of Joints (New Jersey Survey Respondent Answers)

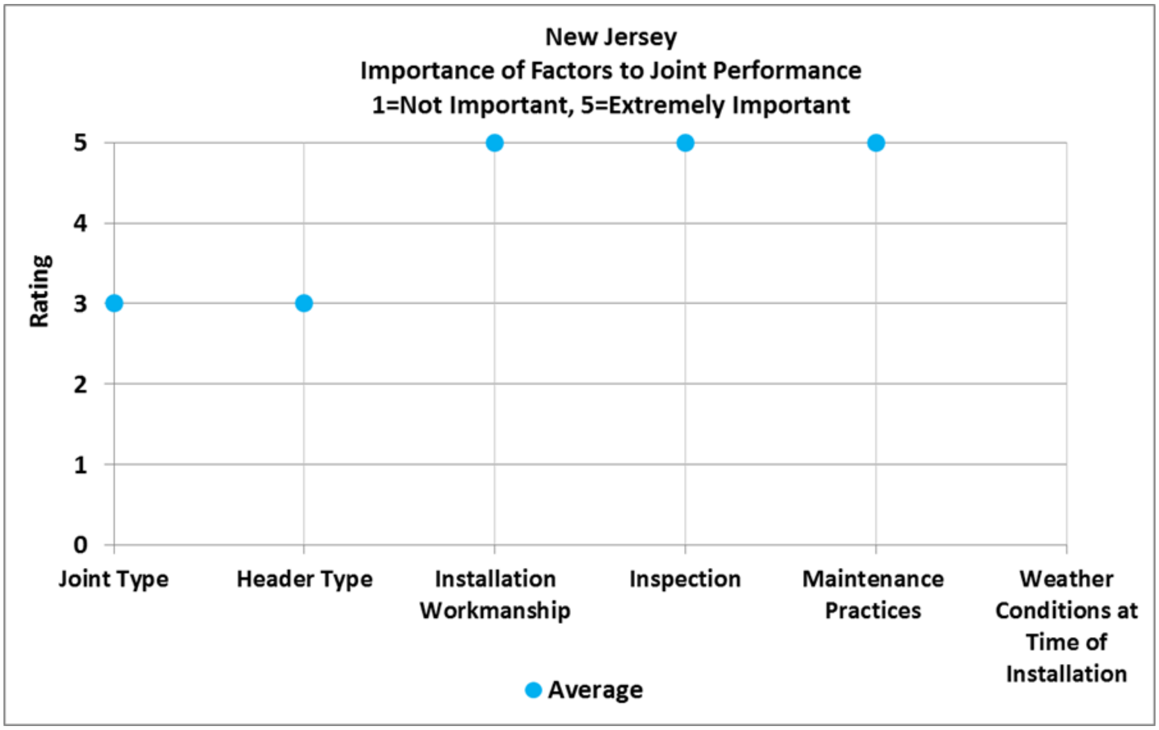


Figure 2.35: Importance of Factors to Joint Performance (New Jersey Survey Respondent Answers)

Headers that are currently being used in New Jersey are armored headers and normal setting concrete when there are no time constraints, while quick setting concrete and elastomeric concrete are used when overnight construction is required. Armored headers were noted to fail over time and potentially create hazardous situations with metal protruding into the roadway.

There are standard joint and header replacement and repair details in New Jersey. Temperature is considered during joint installation by using manufacturer’s recommended install temperatures. While watertight testing is not typically done, one respondent noted that the deck is power washed after joint installation on FHWA maintenance projects and joints are inspected at that time for any failures.

Field splicing is allowed on EM-SEAL and pourable seal. However, splicing pourable joints during winter months has led to failures. Partial replacement of joints has not been found to provide as tight of a seal as complete replacement of joints. A construction practice that positively influences joint behavior is to, where possible, completely remove and reconstruct adjacent concrete then replace joints to provide a new, clean, water-tight seal. Pourable seals do not perform well on vertical re-seals and they tend to pool at the base. EM-SEAL performs best as a vertical joint re-seal.

There are bridge maintenance guidelines in place for FHWA bridge maintenance contracts. There is also a complete NJDOT/FHWA Bridge Preventive Maintenance Program in place. For anti-icing, New Jersey uses sodium chloride (rock salt), liquid calcium chloride, and salt brine. The manufacturer’s representative is sometimes on site for installation of joints. The representatives are usually requested to be on site by the resident engineer during the first installation of any given product by a contractor.

2.16.7 New York State

New York State currently has every joint type in service (based on survey results) as well as armor-less joints with foam seals. The majority of responses stated that for new construction, modular joints, compression seals, link-slabs and pourable seals are used. Some other responses included using asphalt plug joints, strip seals, EM-SEAL, finger joints, saw and seal: deck over backwall, and armor-less joints with foam seals in new construction.

For joint replacement projects, when there are no time constraints, asphalt plug joints, strip seals, compression seals, pourable seals, EM-SEAL, finger joints, modular joints, link-slabs, and saw and seal: deck over backwall are used. When overnight replacement is required, asphalt plug joints, strip seals, compression seals, pourable seals, and EM-SEAL are used.

Strip seals, compression seals, and pourable seals are the joints that 100% of respondents believe perform well with routine repair and maintenance. Other joints that were selected as having good performance with these conditions are asphalt plug joints, EM-SEAL, finger joints, modular joints, open joints, and saw and seal: deck over backwall. Typical service life of joints ranged between respondents. However, all respondents agreed that pourable seals have the shortest service life of zero to four years, and asphalt plug joints and strip seals also have shorter service lives. It was noted that pourable seals and foam compression seals generally last five years or less, sometimes only a couple of years, but they are easy to replace. All responses are presented in Table 2.28.

Table 2.28: Typical Service Life of Joints (New York State Survey Respondent Answers)

New York: Typical Service Life of Joints								
Years	**Asphalt Plug Joint**	**Strip Seal**	**Compre ssion Seal**	**Pourabl e Seal**	**Finger Joint**	**Modular Joint**	**Open Joint**	**Deck Over Backwal**
0-4	1	1	1	3				
5-8	1	1	1					
9-12			1		1	2		1
13-16								
>16		1			2	1	1	1

Strip seals, sliding plate joints, and open joints are currently being phased out. Respondents are phasing out or would like to phase out finger joints. New York would also like to phase out compression seals. It was noted that finger joints last more than 10 years, but that the troughs are difficult/impossible to maintain. For strip seals, it is difficult to replace the seals. The current NYSDOT standard sheets specify closed cell foam joint seals or pourable seals only. Reasons preventing joints from being phased out include difficulty finding a type of joint that will always work. They also noted they would like to use link-slabs more often, but maintaining traffic is problematic and also adds significant cost to the project. Typically, the joints need to be replaced during nightly lane closures so that rush hour traffic can use the lanes daily.

Foam seals are easy to install or replace, but they have had problems with larger sizes (>3") tearing or being punctured under traffic. For reconstructed joints on existing bridges, the header durability appears to be the limiting factor in joint life. They added that installing new headers over old concrete decks is not a good idea. Many joints were noted to suffer from snow plow damage.

A successful joint in New York is one that prevents water penetration for more than 10 years with little to no maintenance, does not cause traffic problems, and does not get damaged by snow plows. Failure of a joint is defined as one that leaks and causes chloride damage to parts of superstructure and substructure, causes traffic problems, a joint that is susceptible to snowplow damage, needs seal replacement before 10 years, and joints that are not continuous at the ends of superstructure and substructure. When joint performance was rated, the highest rating was given to link- slabs and saw and seal: deck over backwall. The lowest rating was given to sliding plate joints and open joints. All ratings are presented in Figure 2.36. The most important factor affecting joint performance in New York is installation workmanship, followed closely by inspection and weather conditions at time of installation. All ratings are presented in Figure 2.37. The most promising joint types are EM-SEAL and link-slabs.

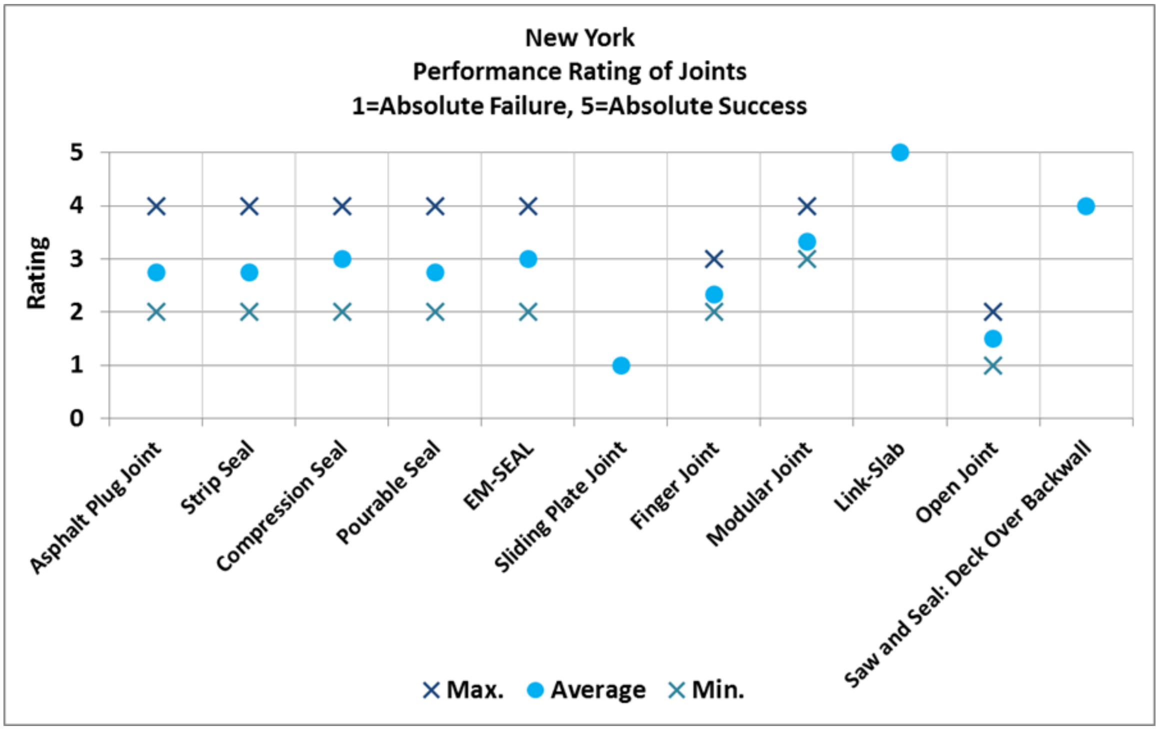


Figure 2.36: Performance Rating of Joints (New York State Survey Respondent Answers)

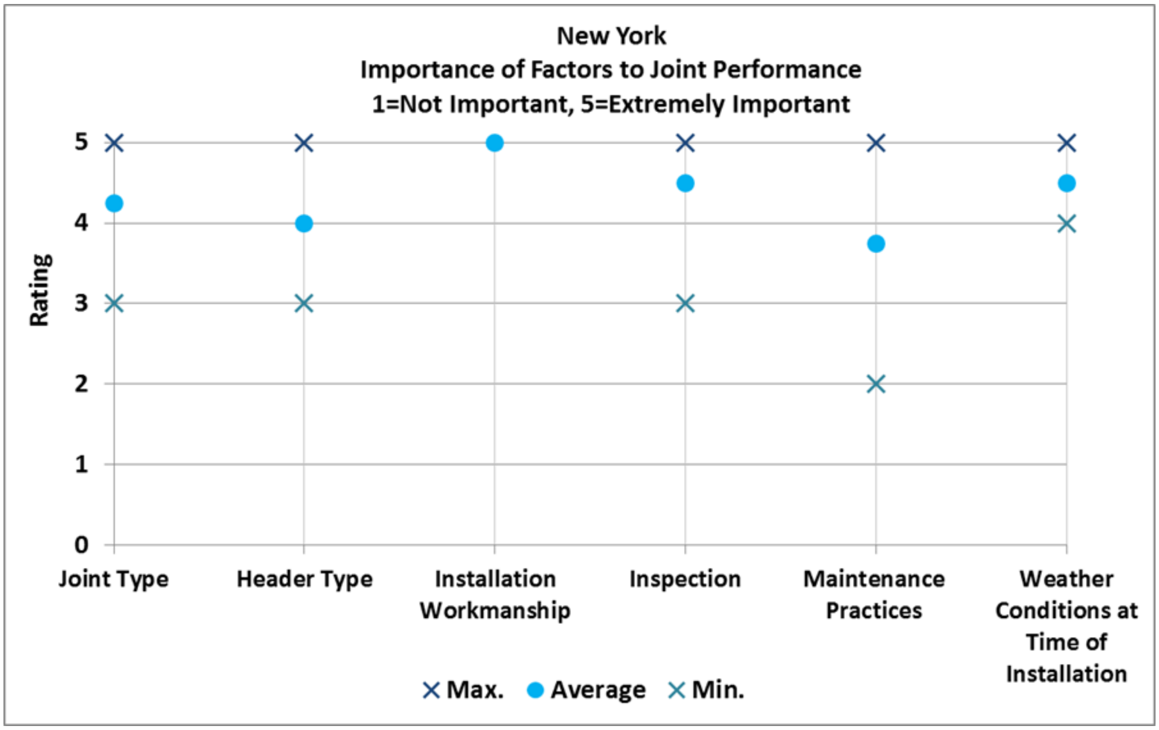


Figure 2.37: Importance of Factors to Joint Performance (New York State Survey Respondent Answers)

Current headers used in New York are normal setting concrete, elastomeric concrete, and quick setting concrete. For overnight construction, only quick setting concrete and elastomeric concrete are used. For new headers, NYSDOT standard details only specify elastomeric concrete. In the past, armored headers were used with joints such as compression seals for decades, but they were very difficult to repair and continuously subject to plow damage. Elastomeric concrete is used for compression seals and pourable seals, while quick setting and normal setting concrete are used for modular joints.

New elastomeric headers placed on old concrete decks can fail prematurely due to deteriorated deck concrete. Elastomeric concrete can also have tire friction issues, issues with rutting, and in some cases sections of the header of broken out. Some elastomeric concretes are low strength or exhibit creep so they cannot overhang the end of a deck. They are also sensitive to damp concrete installations.

Temperature is considered during joint installation. There is a table of joint opening adjustments due to temperature difference from the standard 68°F. Fabricator's charts are used to properly size the seals. Watertight integrity tests are performed after new installation and witnessed by the engineer in charge; no testing is done on repairs. Concrete deck must be dry before elastomeric concrete is placed. There is also a specified temperature range given in the NYSDOT specifications. Field splicing is generally allowed for closed cell foam seals which can be field welded to splice or extend the seal. One respondent noted that most other joint types are not allowed to be spliced.

There are installation practices that were described to positively influence joint performance: removal of all unsound concrete requiring removal of at least 2 ft. of deck on each side of joint centerline, proper cleaning of surfaces, ensuring dry surface prior to placing elastomeric concrete, waiting after the header is placed until it is completely dry to the touch before installing seal, and casting fine aggregate to surface of elastomeric concrete to provide some initial tire friction.

Installation practices that negatively impact joint performance include: installing headers and seals in short windows of time, improperly specifying joint seals (sometimes seal is not properly sized and is placed in tension when the temperature drops, resulting in bond failure between seal and header), replacing armored headers with elastomeric headers resulting in an increase in the seal width which can lead to seal failures as a result of debris build up, and placing elastomeric concrete on concrete that has not cured for at least 10 days.

The majority of respondents stated that there are standard joint and header replacement details utilized in the state, and all respondents noted that there are no standard repair details for joints and headers. The majority of respondents also noted that there is a bridge maintenance manual for the state that is separate from the design manual; this document is referred to as “Fundamentals”. Routine deck washing is performed. They would like to perform more preventive maintenance, but the maintenance group is understaffed. For anti-icing, New York predominantly uses just salt, but sometimes it is mixed with calcium chloride or magnesium chloride.

2.16.8 Pennsylvania

In Pennsylvania, current joints in service are asphalt plug joints, strip seals, compression seals, pourable seals, sliding plate joints, finger joints, modular joints, open joints, saw and seal: deck over backwall, saw and seal: over existing joints, inverted V-joints, and a combination of strip seal with an asphalt plug joint over the top of the strip seal. In new construction, the joints used are: strip seals, compression seals, pourable seals, finger joints, modular joints, saw and seal: deck over backwall and inverted V-joints. There are also finger joints being constructed off the bridge with a concrete trough detail behind the backwall.

In joint replacement projects, asphalt plug joints, strip seals, compression seals, pourable seals, inverted V-joint, and saw and seal: deck over backwall are used for both overnight replacements as well as replacements with no time constraints. Replacement joints used strictly when there are no time constraints are modular joints and saw and seal: over existing joint, while finger joints and open joints are also choices for overnight construction. Inverted-V joints were added in the “other” category of joints. This is a newer joint type that was not included in the survey. An inverted-V joint is a rubber strip seal joint with an upside-down “V” shape. The benefits of this seal shape, according to manufacturer D.S. Brown’s website, is that seal is weather, UV, ozone and tear resistant, is quickly installed, and can be used for easy rehabilitation of existing expansion joints (D.S. Brown 2015). An example of the V-Seal Expansion System is shown in Figure 2.38.

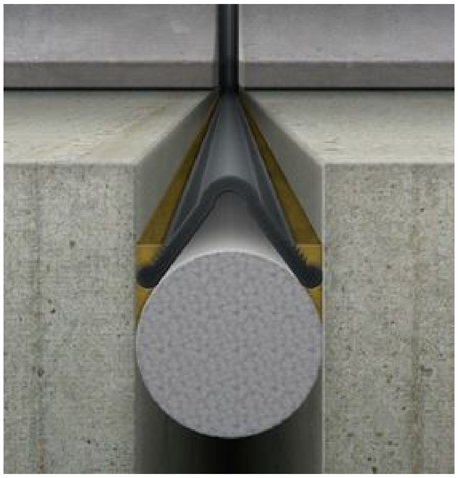


Figure 2.38: D.S. Brown V-Seal Expansion Joint System (D.S. Brown)

Joints that typically were reported to perform well with routine repair and maintenance varied throughout Pennsylvania. All respondents agree that strip seals are on this list, while the majority also chose asphalt plug joints, finger joints, and modular joints. Other responses included pourable seals, compression seals, and saw and seal: deck over backwall. Typical service life of joints is presented in Table 2.29. The shortest service life was assigned to strip seals, compression seals, pourable seals, open joints and saw and seal: over existing joint. At least one respondent selected a typical service life of zero to four years for these joints. For the strip seal, many other districts selected this joint as having one of the longest service lives, showing the variability in performance throughout the state. The longest service life was assigned to finger joints, sliding plate joints and modular joints. As previously stated, some respondents also selected strip seals. These joints were all said to have a service life over sixteen years by at least some respondents. Pennsylvania has just started using EM-SEAL for seal replacements, so its service life is not yet known.

Table 2.29: Typical Service Life of Joints (Pennsylvania Survey Respondent Answers)

Pennsylvania: Typical Service Life of Joints											
Years	**Asphalt Plug Joint**	**Strip Seal**	**Compression Seal**	**Pourable Seal**	**Sliding Plate Joint**	**Finger Joint**	**Modular Joint**	**Link-Slab**	**Open Joint**	**Seal: Deck Over**	**Seal: Over Existing**
0-4		1	1	2					1		1
5-8	5										
9-12			1	1				1			
13-16		2					1			1	
>16		2			1	4	2				

Joints that have been, or are currently being, phased out include: asphalt plug joints, compression seals, pourable seals, sliding plate joints, finger joints, and open joints. Joints that Pennsylvania would like to phase out include: modular joints and saw and seal: deck over backwall. Some respondents chose that they would like to phase out compression seals, sliding plate joints, and finger joints but have not yet started to. Compression joints weaken over time and fall through open joints. Finger joints are very difficult to maintain and replace drainage troughs. The circumstances (if any) preventing the phasing out of joints include project development and funding as well as the expense of removing the entire joint system. In general, Pennsylvania is eliminating joints when possible and/or designing semi-integral approaches.

In Pennsylvania, the definition of a successful joint varies slightly throughout the state: service life of a successful joint would range from 5 to over 15 years, and watertightness should be maintained over this service life. The joint should be maintenance free or be easy to maintain, be durable, allow for easy movement, be cost-effective, and be able to be replaced in a short time. One respondent stated, with regards to finger joints, the joint should last as long as the deck and the troughs should last at least 20 years without leaking.

Failure of a joint includes joints that allow water to leak through and damage the substructure, short life span, joint leakage in between maintenance cycles for the wearable components (neoprene seal, trough), and the steel extrusion requiring repair before the life expectancy of the seal.

In rating the performance of joints, the highest rated joints were strip seals and saw and seal: deck over backwall, while the lowest rated joints were open joints and sliding plate joints. All ratings are presented in Figure 2.39. The importance of factors on joint performance is presented in Figure 2.40. The most important factor is installation workmanship, while the least important factor is weather conditions at time of construction. The most promising new products in Pennsylvania are EM-SEAL and using elastomeric concrete for header repairs. Another respondent noted that although they are not using new products, they believe using a specialized tool for neoprene seal installation in strip seals may reduce damage to the seal during installation. This has not been implemented as of yet, but will be specifying the use of this tool for future installations. Information on this specialized tool is available from D.S. Brown.

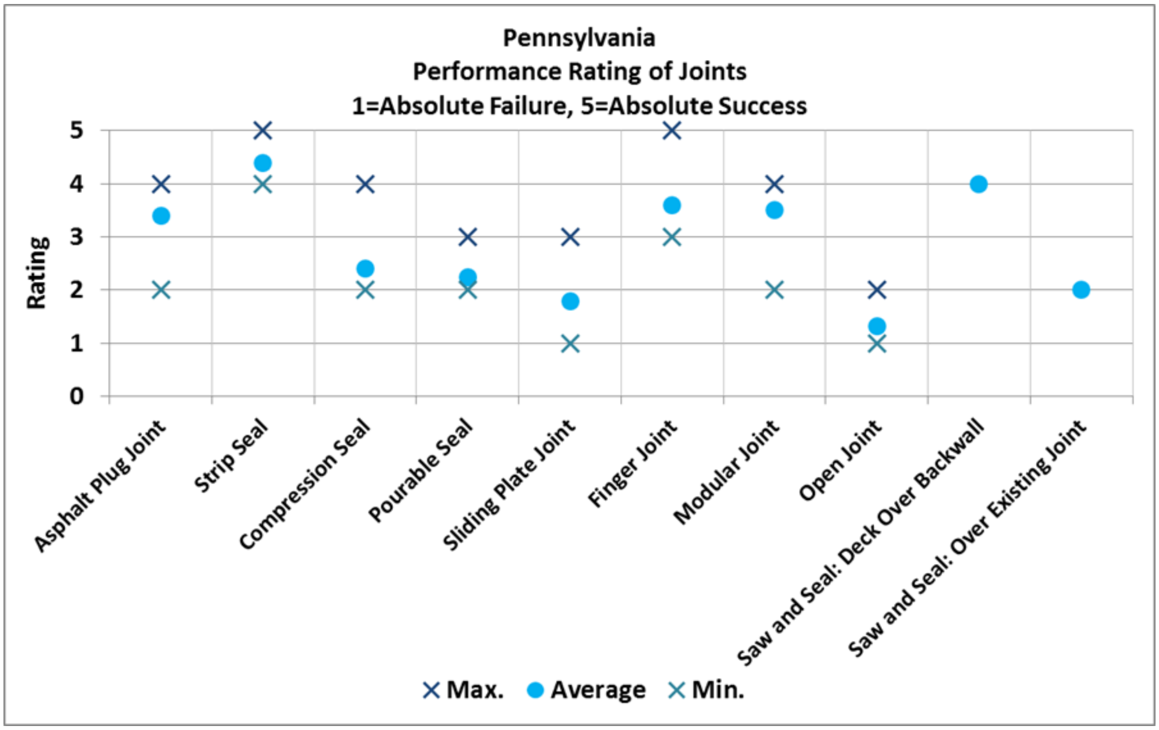


Figure 2.39: Performance Rating of Joints (Pennsylvania Survey Respondent Answers)

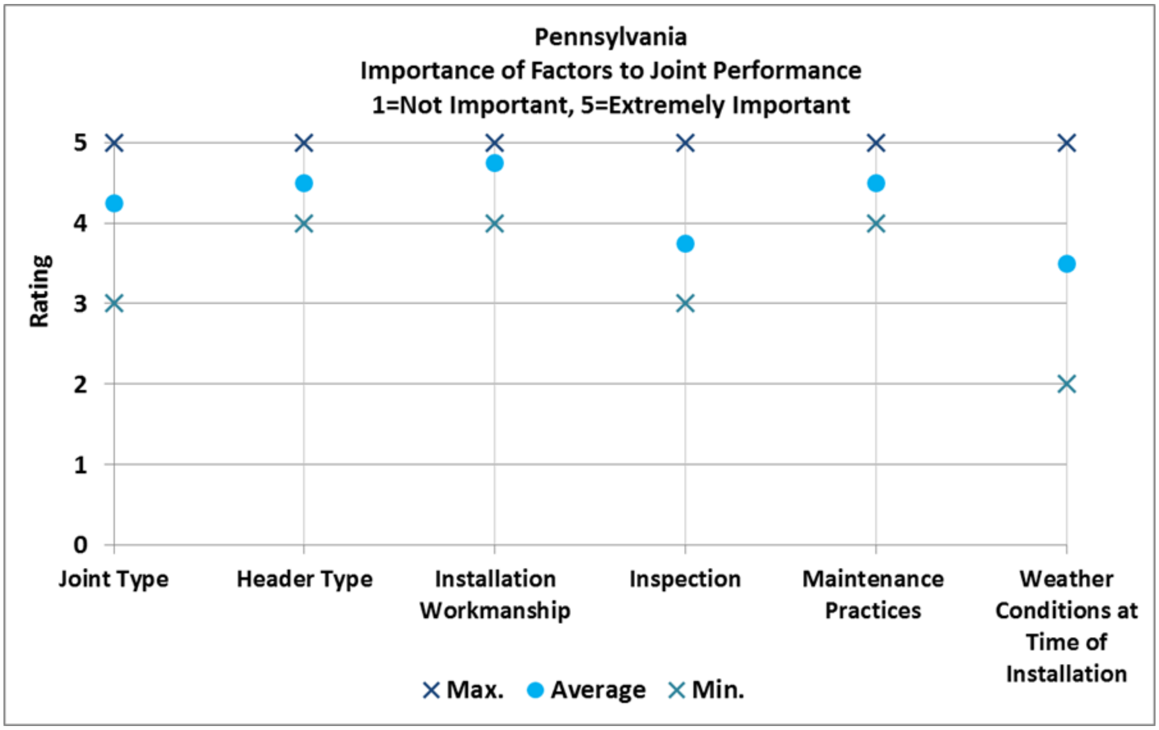


Figure 2.40: Importance of Factors to Joint Performance (Pennsylvania Survey Respondent Answers)

Headers currently in use when there are no time constraints include: armored headers, normal setting concrete, and elastomeric concrete. Headers used when overnight construction is required include quick setting concrete and elastomeric concrete, while one respondent also reported using armored header. Steel armor is used for strip seals, modular joints and finger joints. Armored headers can rust and deteriorate, get damaged by snow plows, and can experience some spalling of concrete around steel headers. In other cases armored headers have performed well. Elastomeric concrete headers are specified for repairs of deteriorated headers according to one respondent.

There are standard joint and header replacement details in Pennsylvania, but there are not standard repair details. Temperature is accounted for during joint installation. If there is a total depth joint replacement, then the distance between the joint will be adjusted to temperature prevailing at time of installation prior to pouring concrete. A temperature table is placed on bridge plans for setting the joint opening based on the installation temperature. Standard watertight testing is done after new joint installation, but is not done on repairs. Repair jobs are generally done during the summer. Any concrete work (for joint installation) would require cold weather curing measures to be used during cold weather. Field splicing of joints is allowed and is typically done for strip seals or inverted V-joints.

Having a field representative on site to provide technical assistance and following manufacturer's specifications are both felt to have positively influenced joint performance. However, it was noted that the representative is only sometimes/rarely on site for joint installation. Inverted-V (strip seal) joints are preferred joints. Many of the issues that are reported to negatively influence joint performance relate to strip seals. Steel extrusions have to be clean before joint installation. Installation of bonding compound cannot be done too far in advance of setting the seal. Pennsylvania may start requiring a specialized tool to install the neoprene gland since there have been issues in the past where use of normal hand tools ended up damaging the seal. They also recommend constructing semi-integral approach with joint located off the bridge when possible. Finger joints with concrete troughs behind the abutment have performed very well in Pennsylvania.

There is a bridge maintenance manual in Pennsylvania. Preventive maintenance is performed with annual pressure washing and cleaning of debris, deck and joints. Strip seal neoprene glands are replaced on a 10 to 15 year cycle on interstates, sometimes longer on other roadway classifications due to funding restraints. For anti-icing, salt and salt brine pretreatment are used.

2.16.9 Rhode Island

There are currently a wide range of joint types in Rhode Island, including: asphalt plug joints, strip seals, compression seals, pourable seals, sliding plate joints, finger joints, modular joints, link-slabs, saw and seal: deck over backwall, and saw and seal: over existing joint. Of those joint types, all are being used in new construction except for sliding plate joints and saw and seal: over existing joints. For replacement joints, asphalt plug joints, compression seals, link-slabs and saw and seal: deck over backwall are joint choices when there are no time constraints, as well as when overnight replacement is required. Strip seals, finger joints and modular joints are also used for replacement joints, but only when there are no time constraints. Sliding plate joints and open joints have been (or are currently being) phased out. The reasons for discontinuing them include their susceptibility to plow damage.

Of the many joint types in use in Rhode Island, the joints that typically perform adequately if routine repair and maintenance are performed are asphalt plug joints, compression seals, pourable seals, and saw and seal: deck over backwall. The typical service life of the joint types are presented in Table 2.30. The joints with the longest service life are strip seals, compression seals, and finger joints, while the shortest service lives are for the asphalt plug joint and saw and seal: over existing joint.

Table 2.30: Typical Service Life of Joints (Rhode Island Survey Respondent Answers)

Rhode Island: Typical Service Life of Joints								
Years	**Asphalt Plug Joint**	**Strip Seal**	**Compression Seal**	**Pourable Seal**	**Finger Joint**	**Modular Joint**	**Saw and Seal: Deck Over Backwall**	**Saw and Seal: Over Existing Joint**
0-4								
5-8	1							1
9-12				1		1	1	
13-16			1					
>16		1			1			

In Rhode Island, a successful joint is defined as one that is water tight and provides a smooth riding surface, with the opposite being defined as failure. Rhode Island rated the performance of joint types with results shown in Figure 2.41. None of the joints were rated an absolute success, however asphalt plug joints, pourable seal, link-slab, and saw and seal: deck over backwall were all rated as successful with a rating of 4. The joints with poor performance ratings were strip seals and modular joints (with a rating of 2) and open joints (absolute failure). Multiple factors were rated for their importance to joint performance. The single most important factor affecting joint performance, according to Rhode Island, is installation workmanship. The factor with the least importance is header type. These results are presented in Figure 2.42.

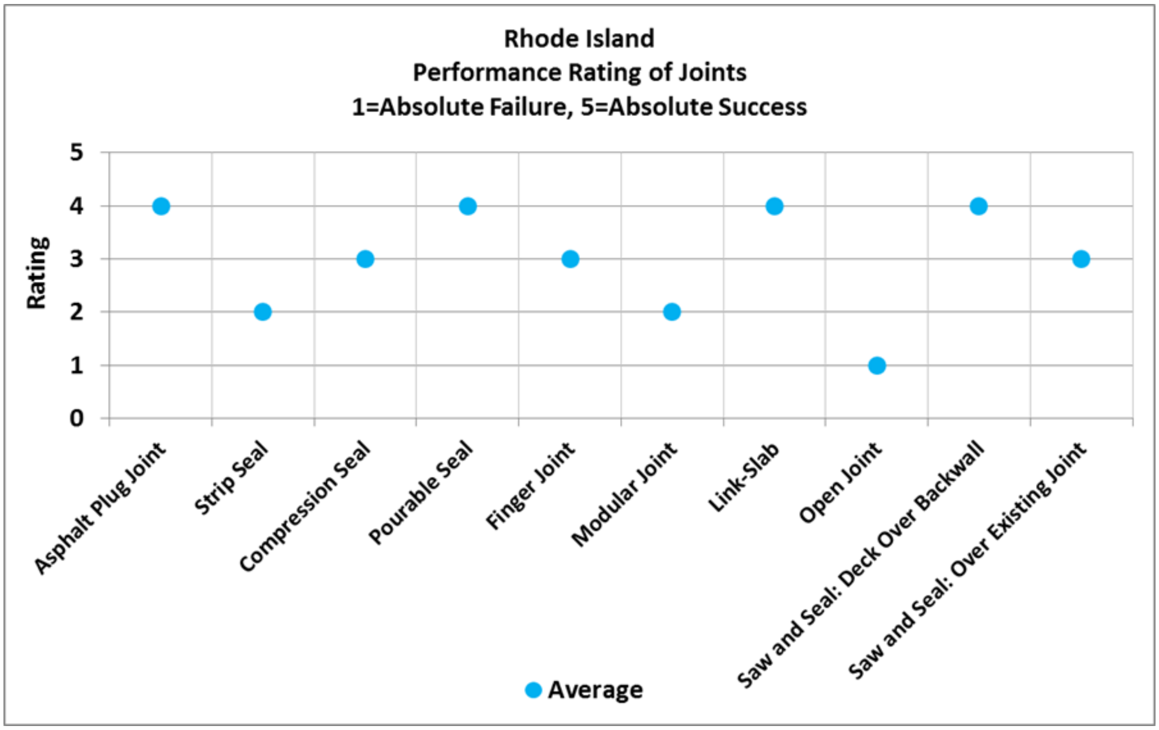


Figure 2.41: Performance Rating of Joints (Rhode Island Survey Respondent Answers)

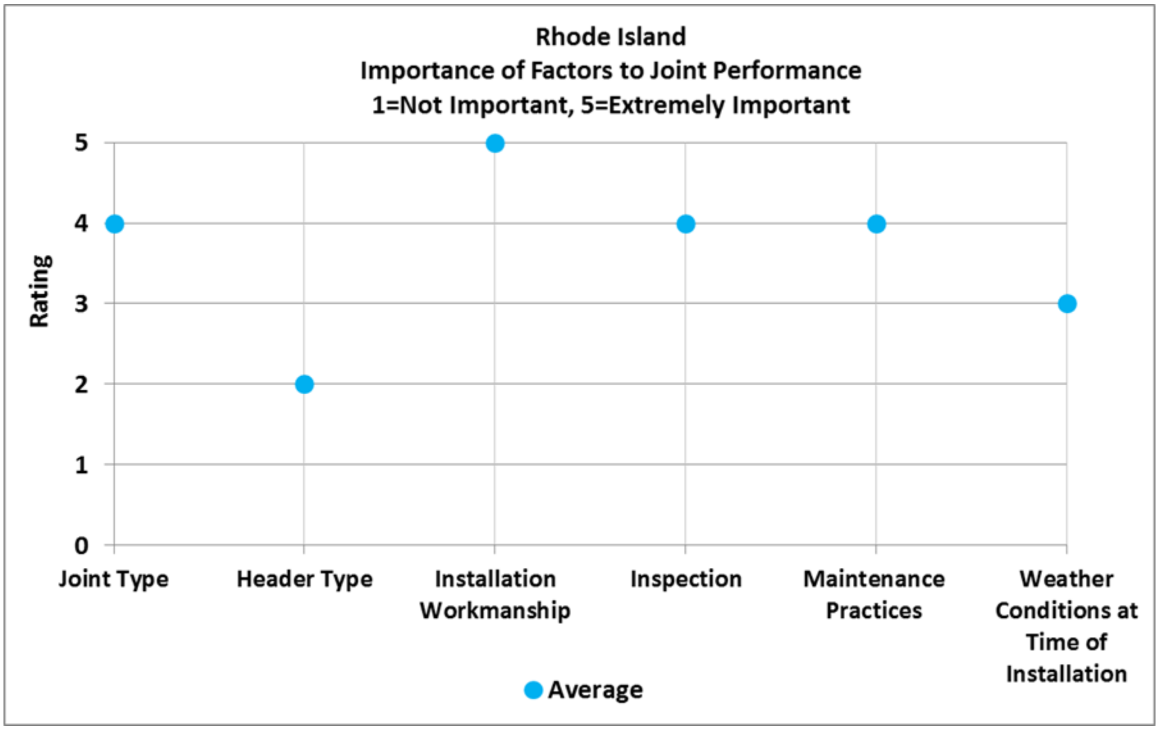


Figure 2.42: Importance of Factors to Joint Performance (Rhode Island Survey Respondent Answers)

Armored headers, normal setting concrete, and elastomeric concrete are all used in Rhode Island when there are no time constraints, and quick setting concrete is used when overnight construction is needed. Elastomeric concrete is used with strip seals, while quick setting concrete is used with poured sealant and deck over backwall (saw and seal). In high traffic volume bridges there have been issues with anchorage pulling out of elastomeric headers.

There are no standard joint or header replacement or repair details in Rhode Island. The temperature is considered when joints are installed by adjusting the opening for temperature increase or decrease from 60°F. The only weather requirement for joint installation is that the temperature must be 45°F or higher. There is no testing (such as watertight test) done to verify proper installation or repair. Field splicing is allowed on all repairs unless restricted per manufacturer recommendations. Although there is no solid evidence, it is believed that construction phasing has a negative influence on joint performance, and complete installation without phasing improves joint performance. The manufacturer's representatives are sometimes present to oversee joint work at time of construction. Rhode Island uses salt as their de-icing treatment.

2.16.10 Vermont

Vermont has a range of joint types currently in use which include: asphalt plug joints, strip seals, compression seals, sliding plate joints, finger joints, modular joints, link-slabs, open joints, and saw and seal joints. The two joint types not currently in service are pourable seal and EM-SEAL. For new construction, the joint types being used are asphalt plug joints and compression seals for small movement, and finger joints and modular joints for larger movement. For replacement projects, the only joint used for overnight construction would be the asphalt plug joint. When there are no time constraints, Vermont uses asphalt plug joints, compression seals, sliding plate joints, finger joints, link-slabs, or saw and seal: deck over backwall.

Of the joints used in Vermont, the ones that typically perform adequately if routine repair and maintenance are performed are asphalt plug joints and saw and seal: deck over backwall, while half of the respondents also added compression seals, finger joints, and link-slabs. Typical service lives of joints in Vermont are presented in Table 2.31. The joints that were unanimously assigned the longest service lives are finger joints and modular joints. The joint with the shortest service life is the asphalt plug joint.

Vermont is currently phasing out, or would like to phase out, strip seals, sliding plate joints, modular joints, open joints, and saw and seal: over existing joints.

Table 2.31: Typical Service Life of Joints (Vermont Survey Respondent Answers)

Vermont: Typical Service Life of Joints									
Years	**Asphalt Plug Joint**	**Strip Seal**	**Compression Seal**	**Finger Joint**	**Modular Joint**	**Link- Slab**	**Open Joint**	**Saw and Seal: Deck Over Backwall**	**Saw and Seal: Over Existing Joint**
0-4									
5-8	2								
9-12		1	2				1	1	
13-16									
>16				2	2	1		1	

In Vermont, success of a joint is defined as one that meets or exceeds the predicted service life without failing, and one that allows movement while also being easily maintained. Failure of a joint occurs when it allows water to reach the bearings, bridge seats or ends of the beam. It was noted that any type of mechanical joints are harder to maintain and typically much more costly.

When rating the success of joints, the only joint type rated an absolute success was the link-slab, with asphalt plug joint and finger joint highly rated as well. All ratings are presented in Figure 2.43. The importance of various factors to joint performance was rated. The most important factors were joint type, header type, installation workmanship, and maintenance practices. The ratings are shown in Figure 2.44. Manufacturer's representatives are sometimes on site for joint installation. Vermont stated that a promising new product they are using is 501 Matrix (asphalt plug joints). This system is a pre-measured, pre-packaged joint system composed of uniquely formulated polymer modified asphalt binder combined in one box with the exact ratio of select aggregate (Crafco 2015). The product eliminates field measuring, proportioning and mixing typically required with asphalt plug joints.

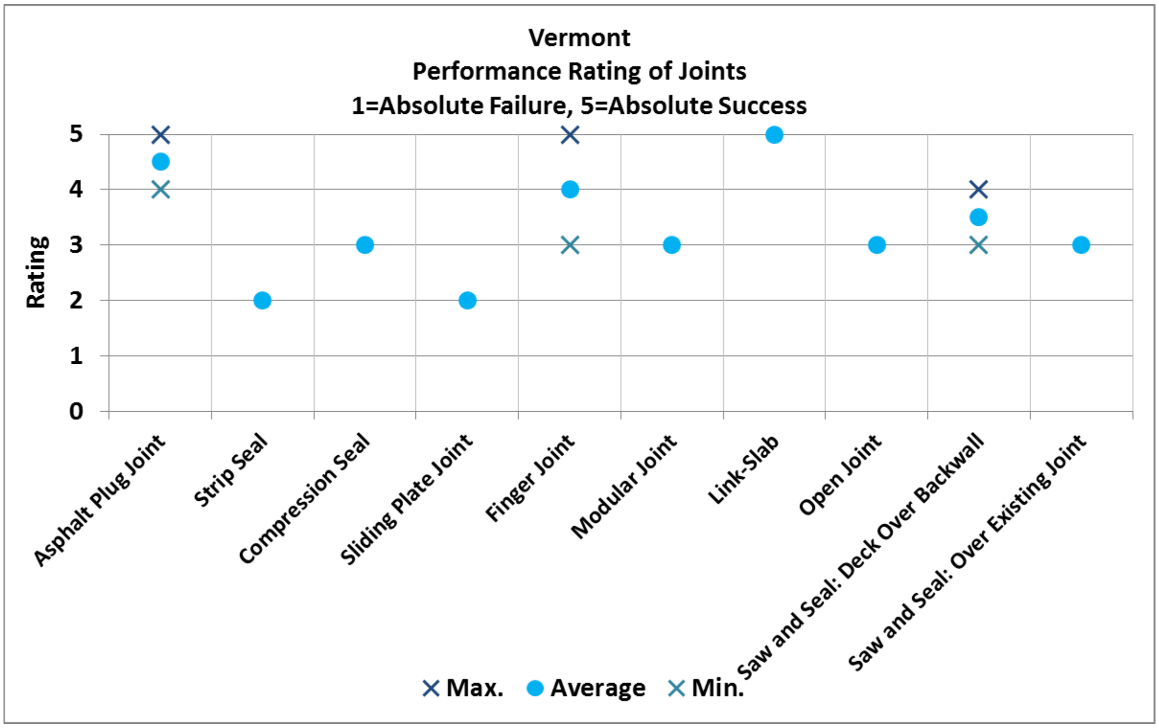


Figure 2.43: Performance Rating of Joints (Vermont Survey Respondent Answers)

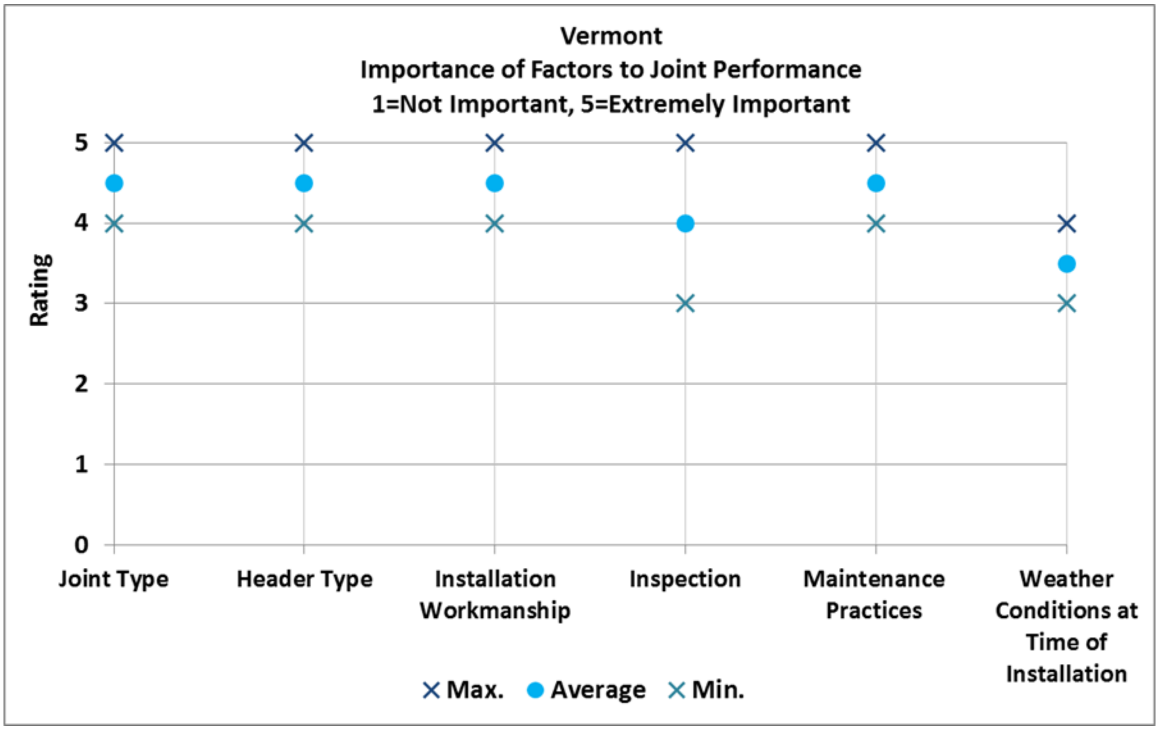


Figure 2.44: Importance of Factors to Joint Performance (Vermont Survey Respondent Answers)

Only normal setting concrete and quick setting concrete are used for headers in Vermont; quick setting being used only when overnight construction is required. For extreme cold temperatures, Vermont has approved Tech Crete as a header material. Quick setting concrete headers do not seem to last as long as normal setting concrete headers. It was also noted that most concrete headers react differently than the bituminous material surrounding them, which makes them more likely to be damaged by heavy truck traffic. For anti-icing treatments, Vermont uses salt, salt brine (includes calcium chloride and magnesium chloride) and Ice-B-Gone.

Temperature is considered for joint installation. For longer bridges, joints with troughs are adjusted to neutral temperature condition. This applies to finger joints and some modular joints. Other weather specifications include asphalt plug joints being repaired, replaced, or installed during spring, summer or fall construction. There is no testing done to verify proper installation. Field splicing is done on some repairs. Vermont performs preventive maintenance. They have a sweeping/washing program where 100% of bridges are swept each year and 50% of washable bridges are washed, including deck, joints, troughs, drains, and superstructure components. Asphalt plug joints are on a 5 to 6 year repair or replacement cycle. Joint headers are repaired as necessary. Bridge joint troughs are washed when bridges are washed.

2.17 Summary of All State Responses

The average rating of joint performance from all states is presented in Figure 2.45. These ratings present the average of each state's average ratings. The figure shows the maximum and minimum rating assigned to each joint type in the survey (considering all individual responses). According to the survey results, the joints with the best performance are link-slabs, EM-SEAL, compression seal, and saw and seal: deck over backwall. The joints with the worst performance are open joints, sliding plate joints, and pourable seals. However, these results show that the majority of joint types have a large range of performance ratings. Link-slabs have the overall best performance rating with the highest average rating as well as the least variability in performance ratings.

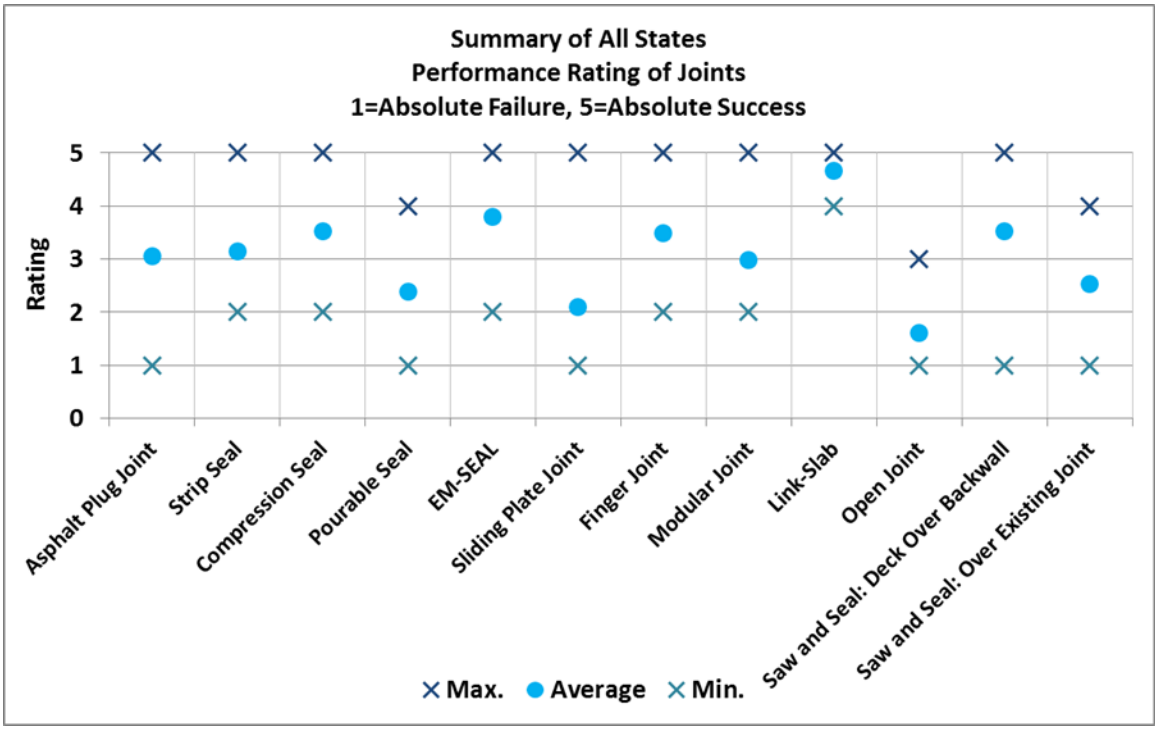


Figure 2.45: Summary of States Performance Rating of Joints

As a result of differing expectations on joint performance, success of a joint is not a direct correlation with its typical service life, and failure of a joint does not necessarily mean it has a short service life. For example, sliding plate joints and open joints have typical service lives greater than nine years, and in many cases greater than sixteen years, despite having “poor” performance compared to other joints. Those pleased with asphalt plug joint performance have an expectation of a short service life for these joints. For some of the more successful joints, such as EM-SEAL and compression seals, many states rate their service life below nine years. These examples are shown in Figure 2.46 and Figure 2.47.

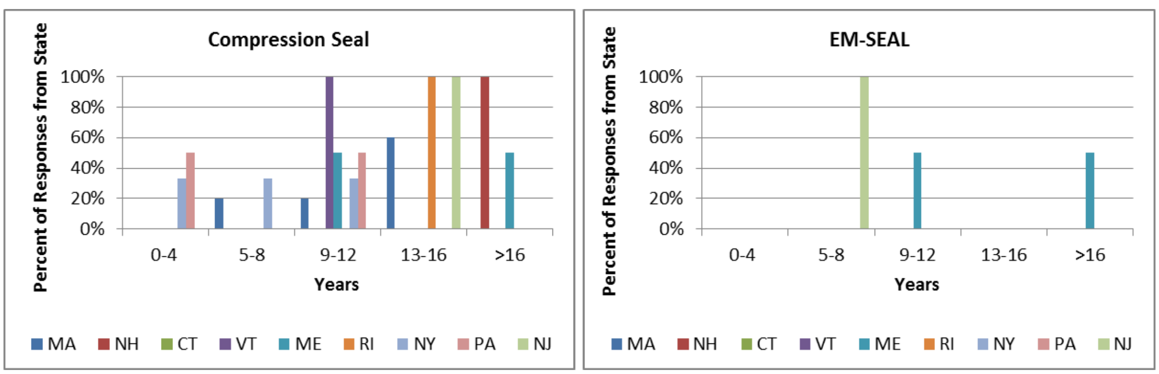
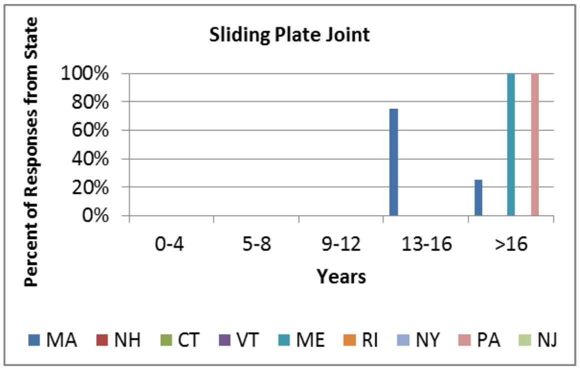


Figure 2.46: Variation in Typical Service Life of Two of the Highest Rated Joints



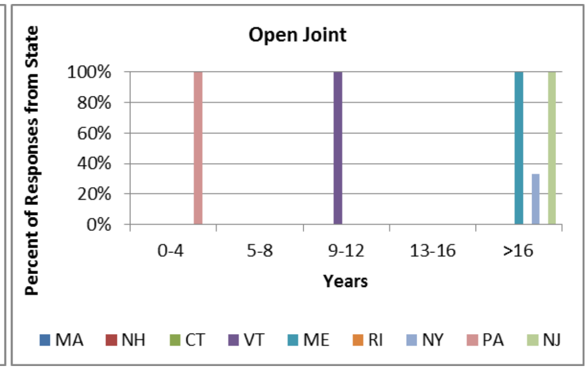


Figure 2.47: Variation in Typical Service Life of Two of the Lowest Rated Joints

Table 2.32 presents a complete list of typical service life ratings from all states. In order to fit all data in the table, the following acronyms were used to denote joint types: saw and seal: deck over backwall (SS:D), saw and seal: over existing joint (SS:O), asphalt plug joint (APJ), compression seal (CS), strip seal (SS), EM-SEAL (EM), pourable seal (PS), modular joint (MJ), sliding plate joint (SPJ), finger joint (FJ), open joint (OJ), and link-slab (LS).

Table 2.32: Typical Service Life of Joints Assigned by All Respondents

All States: Typical Service Life of Joints												
Years	**SS:D**	**SS:O**	**APJ**	**CS**	**SS**	**EM**	**PS**	**MJ**	**SPJ**	**FJ**	**OJ**	**LS**
0-4	0	1	6	2	2	1	11	0	0	0	1	0
5-8	1	1	13	2	1	1	0	1	0	0	0	0
9-12	5	1	1	6	5	1	2	3	0	1	1	1
13-16	1	0	1	5	7	0	0	4	3	2	0	2
>16	4	0	0	2	4	1	0	8	3	16	3	4

Finger joints have the longest typical service life, and also the most consistent, with 16 of the 19 respondents reporting a service life of greater than sixteen years. The common consensus on finger joints is that they could perform well, but plow damage and drainage troughs lead to many issues. All states reporting problems with finger joints noted that they are nearly impossible to maintain and often build up with debris, fail, leak, and experience other similar issues.

Definitions of success and failure were categorized and presented in Figure 2.48. Of the 26 survey respondents, 24 provided definitions for success and failure. These were compiled and quantified based on each factor noted by respondents. Therefore, the total number of factors noted by respondents is referenced rather than the total number of respondents (18 of the 26 respondents noted that joints that are watertight are critical to success, but this was 34% of the 53 total factors mentioned by the respondents).

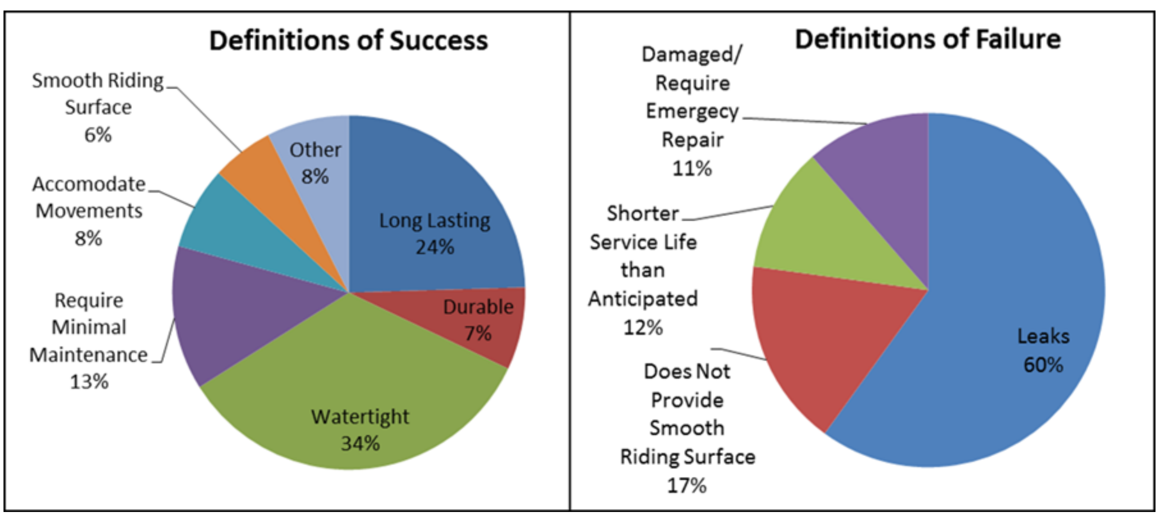


Figure 2.48: All States Definitions of Success and Failure of a Joint

The definition of a successful joint varies slightly for each state, and can also vary from respondents within a state. However, there are many similarities in what states would use to define a successful joint: joints that do not leak, that provide a smooth riding surface, require minimal to no maintenance, and do not get damaged by snow plows. In many cases, the large variation in a joint's performance rating came from the individual respondent's definition of a success and failure. For example, within Massachusetts the sliding plate joint was rated an absolute success by two respondents and a failure by another two. The difference in their definitions of success were that the two respondents rating the joint a success put the most value in a long service life, while the two rating it a failure put value in the joint being watertight.

The definitions of failure of joints included joints that leak, have seals that fall out, do not provide a smooth riding surface, and cause other issues when not properly maintained (including damage to beam ends and bearings). The category of "damaged/requires emergency repair" includes joints damaged from plows and joints that are difficult to maintain and result in costly damage when maintenance is not performed.

The joints selected by the most respondents as ones that perform adequately with routine repair and maintenance are asphalt plug joints, strip seals and compression seals. All results are shown in Figure 2.49. Open joints, saw and seal: over existing joints, and sliding plate joints received the lowest rating. Open joints and sliding plate joints also received the lowest success rating, being rated close to an absolute failure. While some joints may perform poorly without routine repair and maintenance, the respondents believe they have the ability to be a successful joint. Open joints and sliding plate joints are not successful and are not believed to have the potential to be successful by most respondents.

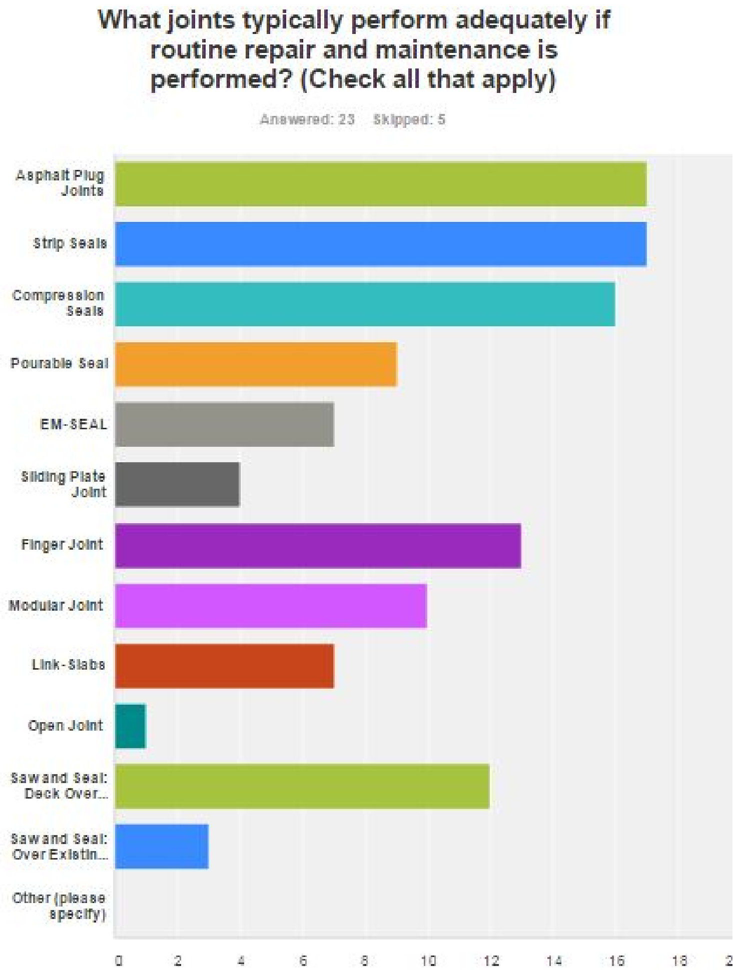


Figure 2.49: Joints that Perform Adequately with Routine Repair and Maintenance

The most important factor in influencing joint performance is installation workmanship; this factor was the highest rated when all states' ratings were averaged as shown in Figure 2.50. Furthermore, installation workmanship has the least variation in state responses, with all respondents rating the importance highly. The second most important factor is equally assigned to inspection and maintenance practices. This shows that the joint type itself is not as important as proper installation, ensuring proper installation (inspection), and maintaining the joints. However, only three states report doing watertight testing upon new installation and watertight testing is almost never done after repairs.

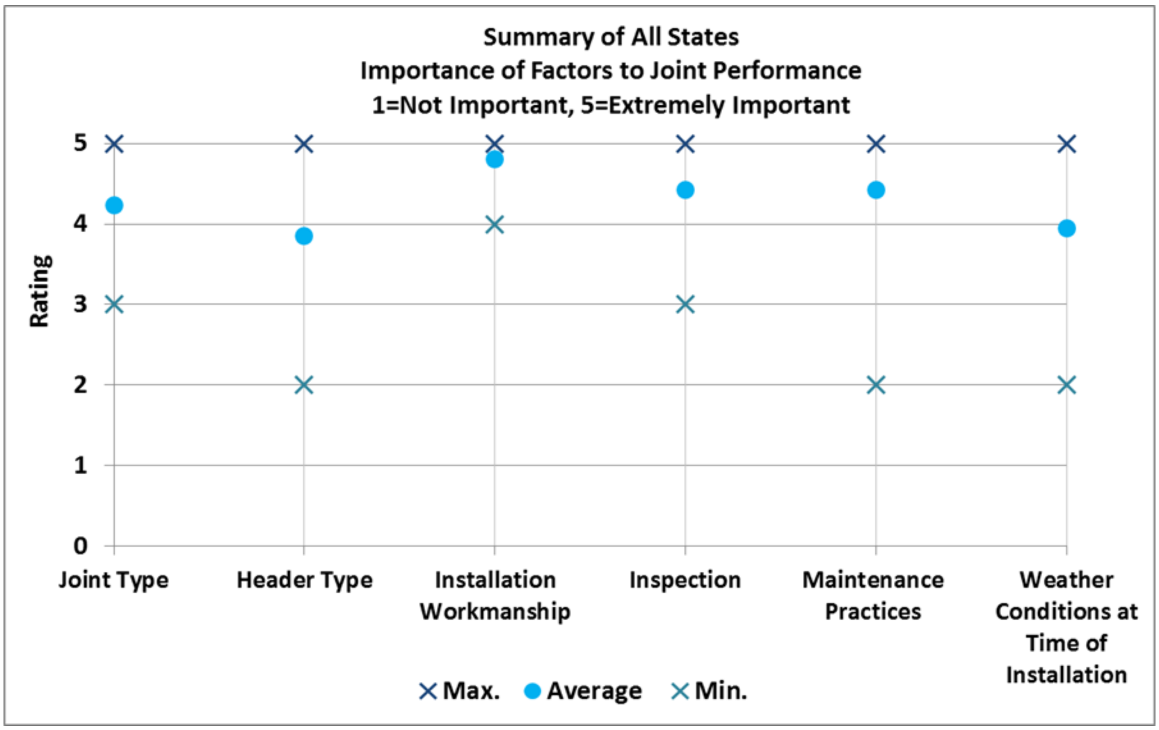


Figure 2.50: Summary of States Rating of Factors Affecting Joint Performance

Installation practices have a significant impact on joint performance, and many states gave suggestions of practices that positively and negatively impacted performance, as well as experiences where certain joints or headers perform poorly. Multiple states noted that installation of joints or elastomeric headers should not be done when the deck is damp, as this leads to early failure and adhesion issues. One of the most consistent installation practices that lead to failure is improper cleaning of surface prior to installation. Many states noted that cleaning after initial cut is made, including sandblasting, is generally included in the specifications but is often skipped due to time constraints or other issues.

Joint seals are sometimes improperly sized, according to one state, and the seal ends up being placed in tension when the temperature drops, which results in bond failure between the seal and header. Bond failure is something many states have experienced and noted as a problem. Installation of bond cannot be done too far in advance of placing the seal or this will likely result in inadequate bond.

State DOTs selected joints that perform adequately if routine repair and maintenance were performed. These answers differed from the joints rated an absolute success or an absolute failure. The joints selected for this question are ones that may have one or more issues with them if they are not maintained or repaired, but would be adequate joint choices with routine repair and maintenance.

The most popular choices were asphalt plug joints and strip seals, with 17 of the 28 respondents (61%). Compression seals were the next joint choice with 16 of 28 respondents (57%). Most states thought that these joint types were easier to replace when there were issues and less expensive than some other options (including less time consuming for installation and therefore less costly). The joint that does not perform adequately, even with routine repair and maintenance, is an open joint. Only 1 of 28 respondents chose an open joint for this question, with the other low scoring joint being a sliding plate joint (4 of 28 respondents). These two joints are also the ones rated closest to an absolute failure.

While maintenance is an important component of joint performance, many respondents stated that their state or district did not have the funds to perform as much maintenance as is needed or as they would like. Similarly, many respondents noted that maintenance groups are understaffed. 57% of respondents reported doing some type of preventive maintenance including bridge sweeping/washing (where 50% of washable bridges are washed to include deck joints, troughs, drains, and superstructure components), and annual cleaning of debris from joints. Among these respondents, it was also noted that the cleaning is not always done well, and that maintenance programs are inconsistent. While there are not enough funds available to do the level of preventive maintenance most states would like, the lack of incorporating a program for this results in significant costs to the state as time progresses. If joints are not maintained and troughs/drainage systems are not cleaned out, leaking often occurs.

There are two broad categories of maintenance: preventive maintenance and reactive maintenance. While reactive maintenance is more common among the states (repairing a failed joint when it is reported from inspection or citizens calling to report damaged joints), this approach to maintaining joints is not cost-effective. AASHTO Bridge Maintenance suggests that for maximum effectiveness of joint performance as well as the highest return on resources expended, preventive maintenance of joints should begin when the bridge is new and continue throughout its life (AASHTO 2007). Documenting joint performance starting when they are new would bring attention to any early failure issues; these could stem from improper design or construction, improper forming of joint opening (or wrong size of opening), improper seal size or placement, inadequate bonding of seal to adjacent concrete, or failure to install bond breaker (AASHTO 2007). These issues were noted by multiple states as causes of early failure of joints. If they are realized early, then the problem can be addressed and potentially prevented from happening again.

If these issues go undocumented, over time they will more than likely lead to many other problems; not only does the severity of damage from a leaking joint increase over time, but the rate of damage also increases. By the time the joint is considered completely failed (which varies depending on the state's definition of failure) and reactive maintenance is implemented, the damage from the failed joint will more than likely be significantly more costly than if the problem was realized and repaired quickly. Many joint failures result from debris build up and failure of drainage troughs which could be avoided (or at the least, the trough could perform successfully for a longer period of time), with a preventive maintenance program. Investing money into these types of programs would likely extend the service lives and performance of joints as well as minimize repairs, replacements, and costly repairs to elements damaged by failing joints.

Having a field representative on site to provide technical assistance and ensure that the contractor closely follows the manufacturer's specifications, is believed to have a positive impact on joint performance. The presence of a representative or inspector, regardless of specific interactions, gives an indication that workmanship is important to the success of the project and can result in an improved joint. There should be quality control and materials should be exactly those specified by the manufacturer. The majority of states also have problems when concrete is not removed all the way to sound concrete. It is suggested that concrete should be removed over at least 2 ft. of deck on each side of the joint centerline ensuring all unsound concrete is removed. When the substrate is in bad condition prior to installation of the header, it can result in many problems; substrate can be saturated in chloride, corroding bars and popping them up which pushes out the joint. There have been some reported anchorage issues in elastomeric headers when they are under high traffic volume, but this is not a prevalent problem.

There were many similarities in header behavior, problems, and possible solutions throughout the responses. One state noted that for reconstructed joints on existing bridges, the header durability appears to be the limiting factor in joint life; they continue stating that installing new headers over old concrete decks is not a good idea. When armored headers are replaced with elastomeric headers, the width of the seal is increased which can result in debris build up and lead to seal failure. Elastomeric concrete should not be placed on concrete that has not cured for at least 10 days. Where possible, it is suggested to completely remove and reconstruct adjacent concrete then replace joints to provide a new, clean, watertight seal. Furthermore, a successful practice has been to, after cleaning surfaces and ensuring a dry surface, wait until the header is completely dry to the touch before installing seals. Another suggestion was to cast fine aggregate to the surface of elastomeric concrete to provide some initial tire friction.

Partial replacement of joints does not provide as tight of a seal as complete replacement of joints, and replacements are generally not checked for watertightness after installation. When joints are being installed, it was noted that construction phasing is believed to have a negative impact on performance and complete installation should be performed without phasing.

Traffic volume should be a consideration in joint types. Asphalt plug joints have not performed well under high traffic volume and are not recommended for this use by manufacturers. Strip seals have performed better in high traffic volume, and EM-SEAL has proven durable in these conditions so far. These conditions also limit options for header materials since the amount of time for repairs is very limited. Quick setting concrete generally needs to be used, which does not perform as well as other header materials but meets the short time constraints. Some respondents noted that Thorac1060 BASF and CTS Cement (low permeability) perform decently as quick setting concrete options.

Cost data was collected for any joints on which the states had information. The approximate costs are given in price per linear foot, including installation and materials, but not including cost of traffic control/police. This data is presented in **Table 2.33**. The most expensive joint types are modular joints and finger joints, which also tend to last longer than other joint types. Saw and seal joints are the least expensive, followed by EM-SEAL and asphalt plug joints. Strip seals are slightly more expensive when installed, but to just replace the seal they are among the least expensive. However, the cost of these joints is not the only consideration. Traffic impacts are often a top priority for state

DOTs. There was no information available for the cost of link-slabs, however many respondents noted that the reduced maintenance demands in these joint types, and lack of associated issues, makes them a desirable option. Even if the upfront cost is greater than other joints, they should still be considered for the many benefits they offer over other joint types.

Table 2.33: Approximate Cost Data for Joint Types Provided by Survey Respondents

Joint Type	**Cost per Linear ft.**
Finger Joints	$1375-$1750
Pourable Seal	$300 (including header)
Compression Seal	$450
Strip Seal	$300-$800, $75 to replace seal
Saw and Seal Deck over Backwall	$15-$25
Saw and Seal over EM-SEAL	$60
Asphalt Plug Joint	$120-$200
Modular Joint	$1750-$4600
EM-SEAL	$90

As far as successful new products, EM-SEAL was at the top of many states' list. This seal has not been in use for very long but has already shown promise in its performance. One of the benefits of EM-SEAL is that it comes with vertical pieces making it easy to maneuver up and over parapets and curbs, a detail that is difficult and generally leaks when done with other joint types. EM-SEAL has also demonstrated success when incorporated with other joint types, such as the modified asphalt plug joint which uses EM-SEAL underneath the asphalt plug. While time will tell how well these joints hold up, they have initially been successful.

Vermont commented that they use 501 Matrix asphalt plug joints that have been very promising. The Crafco 501 Matrix Asphaltic Plug Joint System comes pre-measured and pre-packaged and is hot-applied. It is composed of a unique polymer modified asphalt binder combined in a box with the exact ratio of select aggregate (*7*). Using a product like this eliminates contractor interaction with material and ensures high quality control. One Massachusetts District (District 3) has switched to using pre-mixed asphalt plug joints as well and has reported that the success of these joints is higher than previous asphalt plug joints that were mixed on site, noting an increase in overall quality of the joints.

Other states have had success with Inverted-V strip seals and Silicoflex joints. Many states noted that link-slabs have been highly successful; generally the only issue is finding the time and money to install these types of joints. These joint types have performed very well without needing much maintenance and do not have the same leakage problems as many other joint types. Other states have tried “modified asphalt plug joints” by combining the plug joint with strip seals, EM-SEAL, or other seal types. Pennsylvania noted that they have been using finger joints with concrete troughs behind the abutments and that these have performed very well. Another two states have started using heavy steel angles for joint armor and steel plates/rebar hoops for anchorages; they note that they are heavy duty joints but use readily available materials and welding details which are relatively simple. Finally, another state suggests using a specialized tool for neoprene seal installation in strip seals, stating that normal hand tools tend to lead to damage/tears.

When it comes to choosing the best joint type, there is no right answer. Many factors need to be considered and while no option may be perfect, the information collected from these states should give insight into which joints perform well in various conditions and meet various needs. Some joints, such as asphalt plug joints and strip seals, do not have a long service life if they are not maintained, but they provide relatively quick installation at comparatively low costs and can be repaired more quickly than other joint types. If time is not an issue, considering an option like link-slabs or saw and seal: deck over backwall would be worthwhile to take the time installing since they require minimal maintenance and can remain in service for longer periods of time without issues. For joint types accommodating large movement, most states agree that finger joints are the better option if they can figure out a solution to the trough problems. Pennsylvania suggested that putting the finger joints with concrete troughs behind the abutments would be a viable option. Overall, the joint type should be chosen based on a number of factors it can accommodate.

Attention also needs to be placed first and foremost on the installation practices, as was pointed out by all states. Without proper installation, the service life will not be as long as expected and other issues could present themselves before failure, such as leaking of joints which affects many other potentially costly areas of the bridge. All states reported that installation practice is the single most important factor affecting joint performance. Other factors rated highly important as well, including inspection, maintenance and weather at time of installation.

Some of the most important factors that need to be addressed during installation include proper cleaning of joint opening prior to installation, proper sizing of joint opening and proper timing of placing bond to ensure adequate adhesion. To ensure proper installation, inspectors should be aware of all specifications the manufacturer provides and ensure that they are completed. Most states report that the manufacturer’s representative is rarely on site during joint installation, so it is critical that whoever is overseeing the project understands the proper steps and ensures their completion.

When it comes to choosing the best joint type, many factors should be considered. Beyond expansion needs, states need to consider factors such as traffic demands, cost, and time. With most joints, the ability to perform adequately stems from their installation, inspection, quality control of the product, and maintenance practices. No joint is perfect, but could have improved performance throughout their service life if these measures were taken.

2.18 Summary and Conclusions

This chapter presented a literature review of previous joint research, organized information on the existing bridge joint inventory in Massachusetts, compiled joint information and practices from meetings with the six MassDOT district offices, compiled information from survey responses by Connecticut, Maine, Massachusetts, New Hampshire, New Jersey, New York, Pennsylvania, Rhode Island and Vermont, and summarized all survey responses.

While IABs are the highway bridge of choice across the country, it is not always possible to construct them due to design limitations and changing from traditional jointed bridges takes time. Therefore, it is important to understand what causes problems with joint performance and what bed practices are to avoid these problems.

The purpose of this research was to determine best practices with bridge expansion joints and headers from Massachusetts and states in and around New England. Joints that are damaged and not functioning properly lead to costly issues that extend far beyond the joint itself; superstructure and substructure elements can be damaged by corrosive materials carried by water leaking through joints. While preventive maintenance would be an ideal way to prolong joint life, as well as enhance joint performance over its service life, none of the states are able to incorporate the level of maintenance they would like to due to a lack of funding. Lack of maintenance is a main cause of joint failure which leads to much greater repair and maintenance costs for the superstructure and substructure elements. An evaluation of overall life-cycle bridge costs would be worthwhile and

DOTs that are practicing regular maintenance attribute this to better joint life and performance.

An ideal joint would be one that remained in service and performed without issue for a long period of time without needing maintenance. However, there is no perfect joint. Individual joint types have advantages and disadvantages associated with them that should be considered when selecting a joint type for a project. The definition of a successful joint varies between states and between individuals within states. Some respondents define a successful joint as one that could remain in service for a long period of time, while others define that a successful joint should also remain watertight through its service life. Others noted that each joint type has its own expected durability and this should be considered in defining success. Others indicated that some leakage is acceptable as the joint ages.

The highest rated bridge joints from all states were link-slabs, EM-SEAL, compression seals, saw and seal: deck over backwall and finger joints. However, only link-slabs had consistent high ratings, while all other joints had a wide range of success ratings based on individual respondents' definitions. Open joints and sliding plate joints received the lowest performance ratings, with respondents noting problems including leaking (as expected) and difficulty to repair. The majority of respondents would like to phase these out, or have phased them out already. This was also mentioned by individuals of some joints which were rated highly by others.

Typical service lives of joints varied between respondents as well. Pourable seals have the shortest reported service life of zero to four years, but some states still use these joints because they can be quickly installed, are less expensive than many other joints, and can be repaired quickly. Similarly, asphalt plug joints have a short reported service life, with the majority of respondents stating they are in service five to eight years. However, these joints are also quick to install and relatively inexpensive, resulting in their continued use in the majority of states.

With asphalt plug joints, some states reported that quality control of the product is a big contributor to the joint performance. Vermont and one district in Massachusetts have started using pre-mixed, pre-bagged asphalt plug joint products that eliminate contractor interaction with the material and have reported that these have been successful so far and perform better than asphalt plug joints mixed on site. Another new technique with this joint type reported by some respondents is using a “modified asphalt plug joint” where EM-SEAL is used beneath the plug joint in an effort to increase watertightness of the joint. So far, these have performed well but are still a new practice.

All states reported that installation practice is the single most important factor affecting joint performance. Other factors rated highly important as well, including joint and header types, inspection, maintenance practices, and weather at time of installation. The majority of practices reported that negatively impact joint performance were issues during the installation of the joint.

Some of the most important factors that need to be addressed during installation include proper cleaning of joint opening prior to installation, proper sizing of joint opening and proper timing of placing bond to ensure adequate adhesion. To ensure proper installation, inspectors should be aware of all specifications the manufacturer provides and ensure that they are followed. Most states report that the manufacturer's representative is rarely on site during joint installation, so it is critical that whoever is overseeing the project understands the proper steps and ensures their completion. Training of contractors, installers, and site engineers would be beneficial to ensuring proper installation and maintenance. Currently, the on-site engineers have different levels of experience and knowledge; therefore, a statewide training could ensure uniform standards are being upheld during installation.

Overall, in order for there to be consistent practices throughout a state, decision making would have to be heavily centralized. This research has shown that within a state each district has various constraints, inventories, traffic demands, and local contractor and inspector experience/training. Therefore, there is benefit to allowing localized decision making, but this is not consistent between new installation and repair or replacement of joints. While variability in practice throughout a state is not an issue in itself, it does make it difficult to determine any statewide conclusions.

When it comes to choosing the best joint type, many factors should be considered. Beyond expansion needs, states need to consider other factors such as traffic demands, cost, and time. With most joints, the ability to perform adequately stems from their installation, inspection, quality control of the product, and maintenance practices. No joint is perfect, but all could have improved performance throughout their service life if these measures were taken.

2.19 Recommendations for Implementation

Based on the research conducted through district meetings and survey responses, the following recommendations are proposed to be considered for future bridge joint practice in Massachusetts to improve joint and header performance, save cost on frequent repairs and replacements, minimize costly damage to other structural components, and extend the service life of joints:

- Address joint installation and expected performance during pre-construction meetings
- Initiate statewide joint/header installation training of contractors, installers, inspectors and on-site engineers
- Develop a consistent program for preventive maintenance, including routine cleaning of joints and drainage troughs along with designated funding for these activities
- Require watertight testing on all closed joint types for both replacement and repair of joints
- Have manufacturer representative on-site for installations when possible
- Determine a way to warranty joint performance for a period of time post-construction
- Streamline process for adding new products to approved product list for new construction

- Include information specific to joint performance in a searchable database, such as PONTIS, and on inspection reports

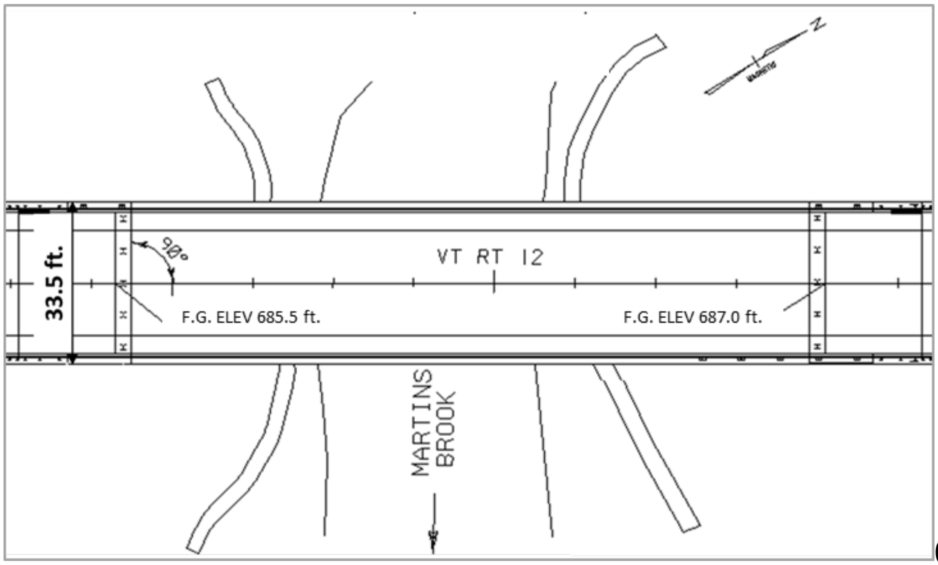
CHAPTER 3

DESCRIPTION OF BRIDGES

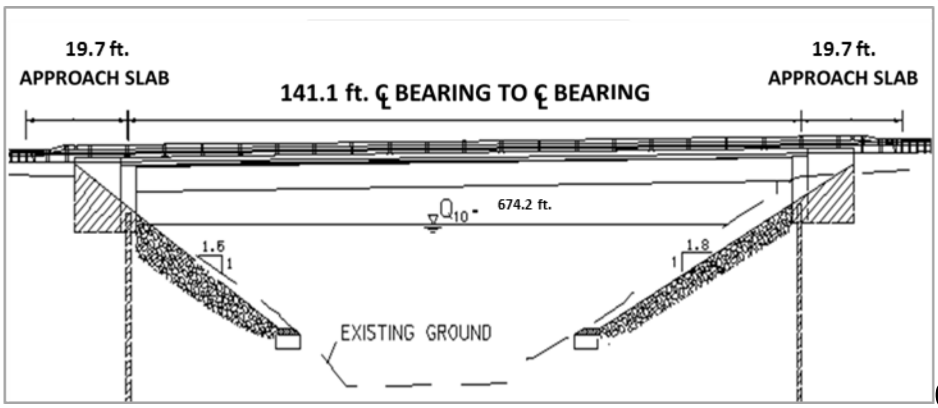
This chapter provides details on the three IABs instrumented from construction for research conducted at the request of the Vermont Agency of Transportation (VTrans). Details of the finite element models of the bridges are provided in Chapter CHAPTER 4, and the long term response of the bridges is presented in Chapter CHAPTER 5.

3.1 Details of Middlesex Bridge

The Middlesex Bridge is located over Martin's Brook on VT12 in Middlesex, Vermont. The bridge is a straight girder non-skewed single span IAB with a bearing to bearing length of 141.0 ft (43.0 m) and a width of 33.5 ft (10.2 m) to outside of fascia. Five plate girders support an 8.7 in (220 mm) concrete deck. Each girder has a 46.1x0.6 in (1170x14 mm) web, with top and bottom flange plates of 20.1x1.0 in (510x25 mm) and 20.1x2.1 in (510x54 mm), respectively. Girders start at 3.28 ft (1.00 m) from fascia and are evenly spaced at 6.70 ft (2.05 m).The abutment is 3.28 ft (1.00 m) thick with a height of 13.12 ft (4.00 m) and 13.78 ft (4.20 m) from bottom of abutment to top of concrete at the fascia and center of roadway, respectively. Wingwalls are integral with the abutment with a thickness of 1.48 ft (0.45 m) and extend 9.84 ft (3.00 m) orthogonal to the abutment. Five HP12x84 (HP310x125) steel piles support each abutment wall; the piles are embedded 3.28 ft (1.00 m) into the bottom of the abutment and extend approximately 29.53 ft (9.00 m) below the abutment. Figure 0.1, Figure 0.2 and Figure 0.3 show a plan and elevation view of the bridge, plan and elevation view of abutment and a deck section view, respectively.



(a)



(b)

Figure 0.1 Middlesex Bridge (a) plan view (b) elevation view

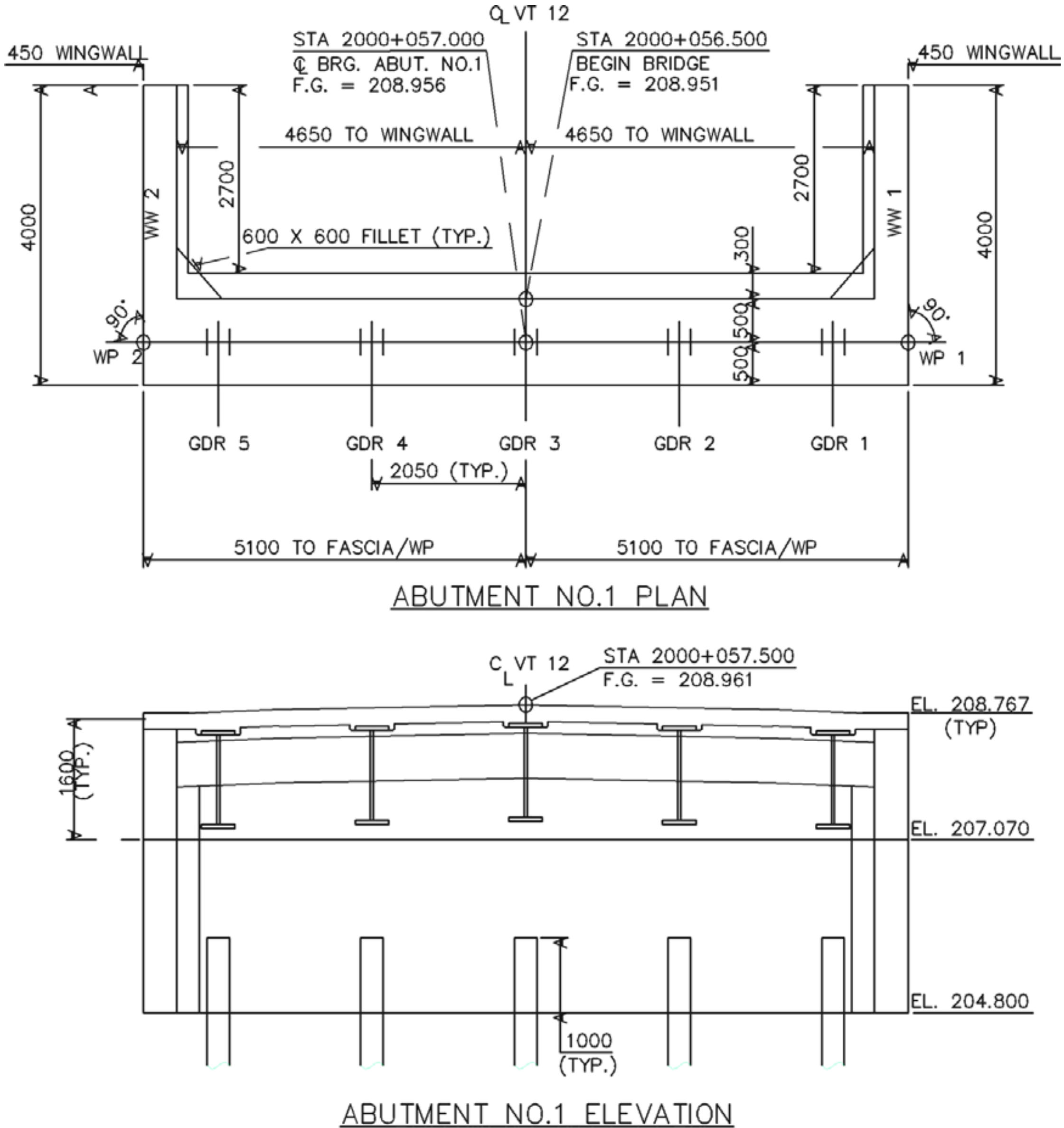


Figure 0.2: Plan and Elevation View of Abutment 1at Middlesex Bridge

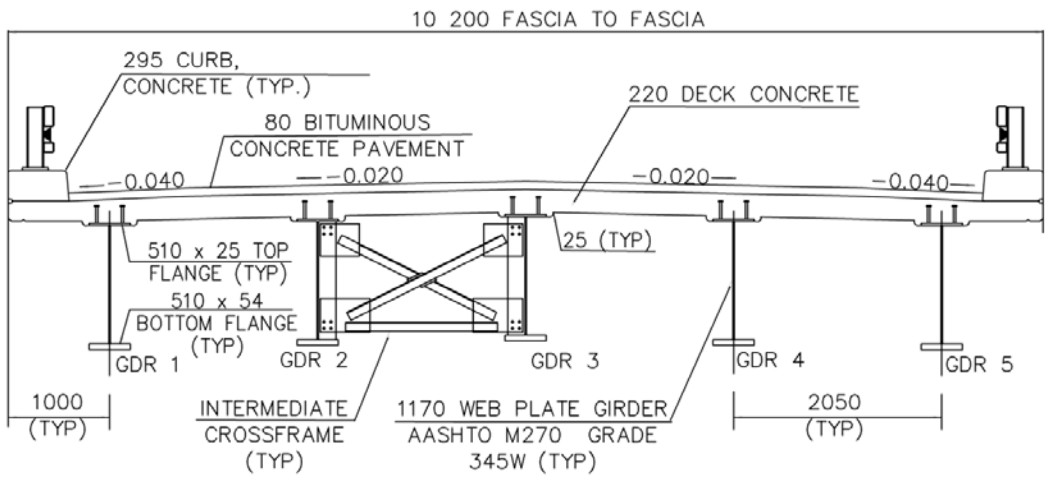
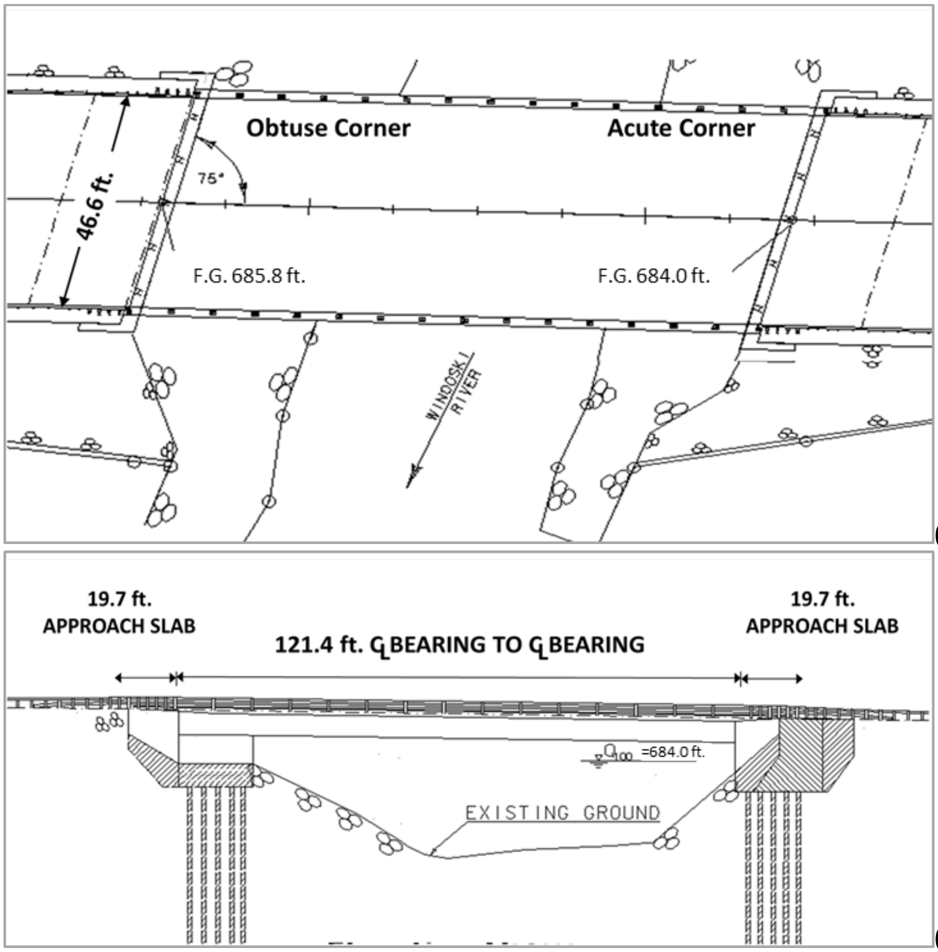


Figure 0.3: Middlesex Bridge Deck Section

3.2 Details of East Montpelier Bridge

The East Montpelier Bridge is located over the Winooski River on US2 in East Montpelier, Vermont. The bridge is a straight girder IAB with a 15 degree bridge skew. The bearing to bearing length is 121.4 ft (37.0 m) and the width is 46.6 ft (14.2 m) to outside of fascia. Five plate girders support an 8.7 in (220 m) concrete deck. Each girder web is 53x0.6 in (1346x16 mm) with top and bottom flange plates of 18x0.87 in (457x22 mm) and 18x1.6 in (457x41 mm), respectively. Starting 3.61 ft (1.10 m) from each fascia, the girders are evenly spaced at 8.84 ft (3.00 m) across the bridge. The abutment is 2.95 ft (0.90 m) thick with a height of 12.80 ft (3.90 m) and 13.3 ft (4.10 m) from the bottom of abutment to top of concrete at the fascia and center of roadway, respectively. Wing walls are integral with the abutment, with a thickness of 1.48 ft (0.45 m) and extend 9.19 ft (2.80 m) from the centerline of the abutment at a 15 degree skew. Five HP12x84 (HP310x125) steel piles support each abutment; the piles are embedded 1.97 ft (0.60 m) into the bottom of the abutment and extend approximately 125 ft (38 m) below the abutment. Figure 0.4, Figure 0.5 and Figure 0.6 show a plan and elevation view of the bridge, plan and elevation view of abutment and a deck section view, respectively.



(a)
(b)

Figure 0.4 East Montpelier Bridge (a) plan view (b) elevation view

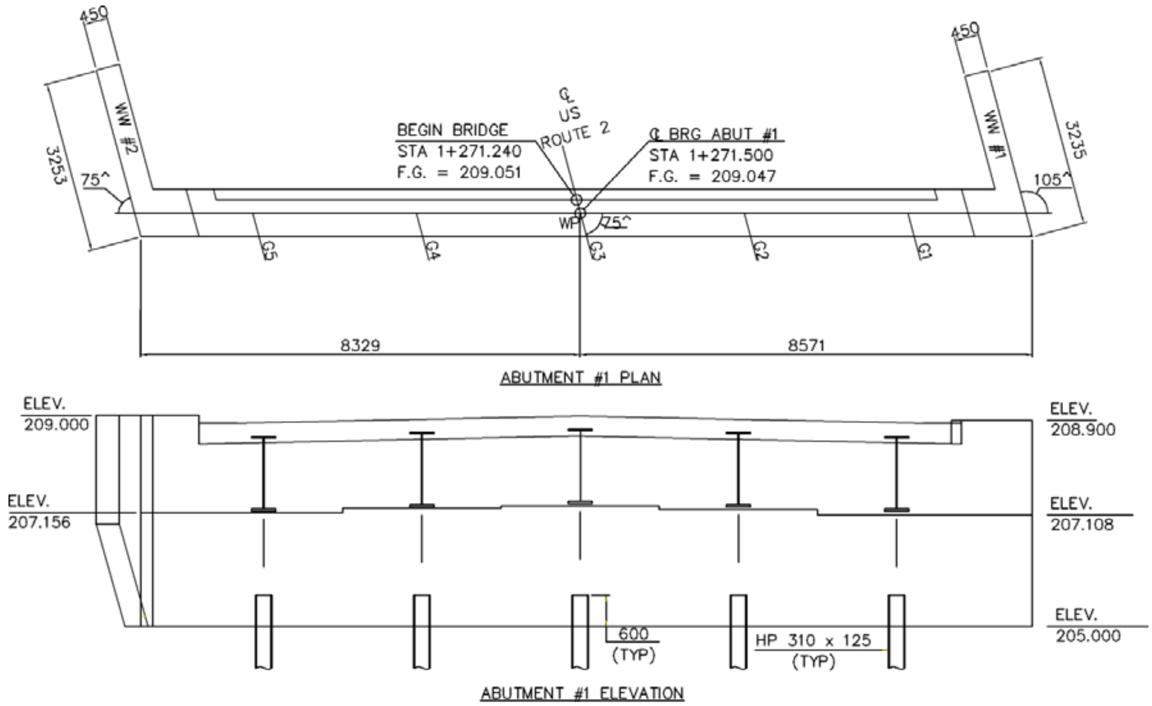


Figure 0.5: Plan and Elevation View of Abutment 1 at East Montpelier Bridge

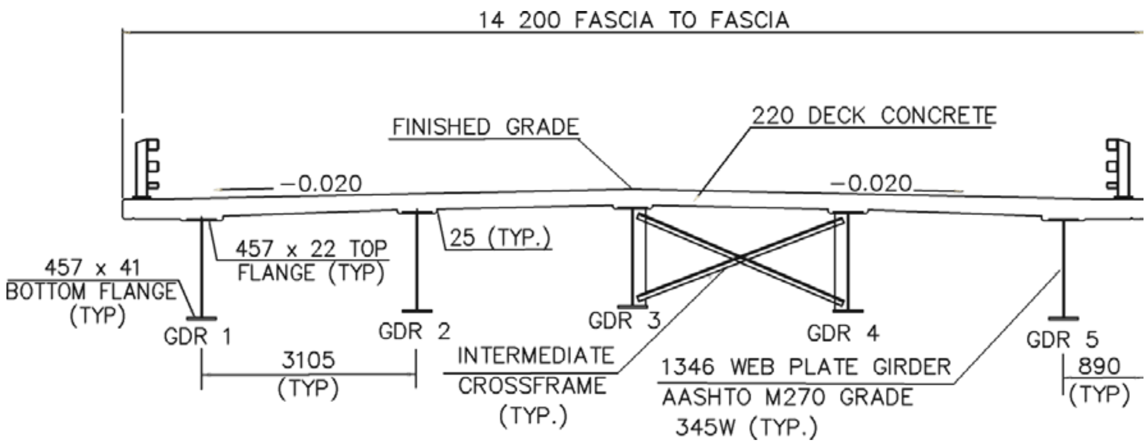
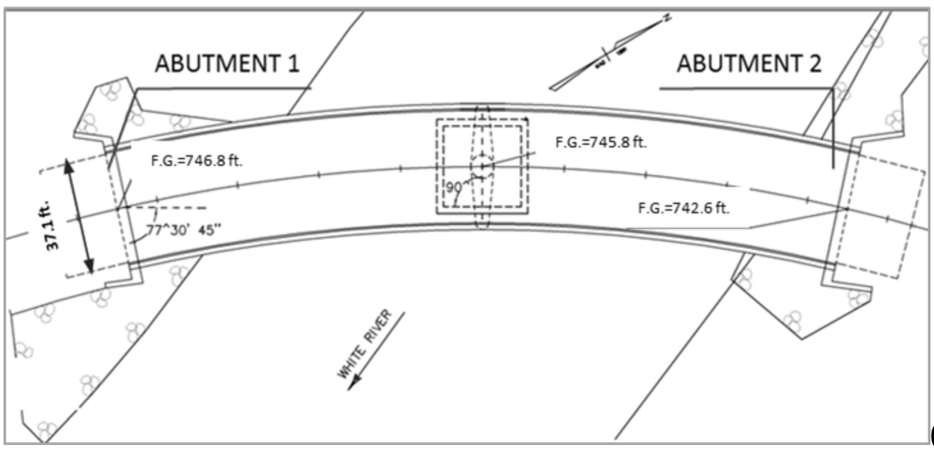


Figure 0.6: East Montpelier Bridge Deck Section

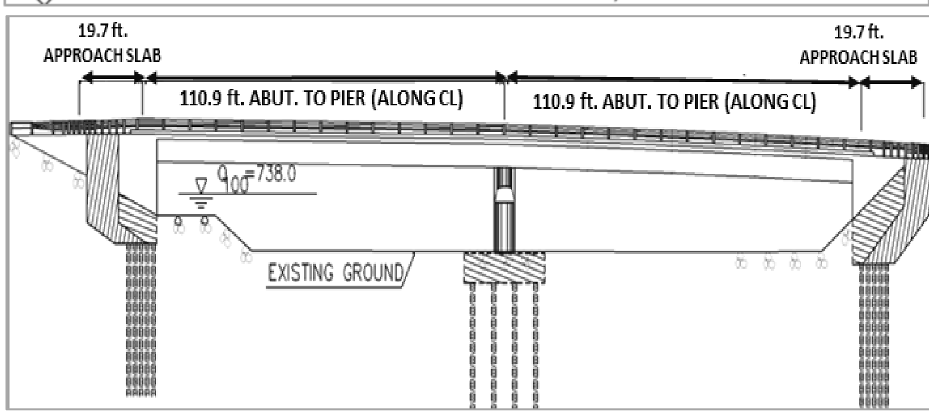
3.3 Details of Stockbridge Bridge

The Stockbridge Bridge is a curved girder IAB located on VT Route 100, crossing the White River in Stockbridge, Vermont. The bridge length is 221.0 ft (67.6 m) along its curved centerline, with an 11.25 degree of curvature along the bridge alignment, and a width of 37.1 ft (11.3 m) to the fascia. This two-span bridge includes a center pier with guided bearings on top of the pier cap positioned to support each steel girder. There is a superelevation of 6 percent at road level and a vertical elevation difference of 4.3 ft (1.3 m) between start and end of the bridge. The bridge is composite with an 8 in (203 mm) thick reinforced concrete and five built-up steel plate girders. The girders are spaced at 6.70 ft (2.36 m) supporting the deck. The girder web dimensions are constant at 46x0.6 in (1170x16mm). Flange dimensions differ among girders and vary along the span (cross sections are presented in Part I). Abutments are 3.00 ft (0.90 m) thick with an average depth of 20.70 ft (6.30 m) Wing walls have a thickness of 1.50 ft (0.45 m) and extend 10.00 ft (3.00 m) and 14.00 ft (4.30 m) from the centerline of Abutment 1 and Abutment

2, respectively. The wing walls are oriented at 85 degrees and 110 degrees from the abutment and have a tapered bottom face. The abutments are supported on five HP 14x117 (HP 360x174) steel piles that are embedded 2.00 ft (0.60 m) into the bottom of the abutment and extend approximately 75.5 ft (23.0 m) below the abutment. Geofoam material was applied at the abutment backwalls prior to backfilling to reduce earth pressures on abutments. Figure 0.7, Figure 0.8, Figure 0.9 and Figure 0.10 show a plan and elevation view of the bridge, plan and elevation view of abutment and two deck section views, respectively.



(a)



(b)

Figure 0.7 Stockbridge Bridge (a) plan view (b) elevation view

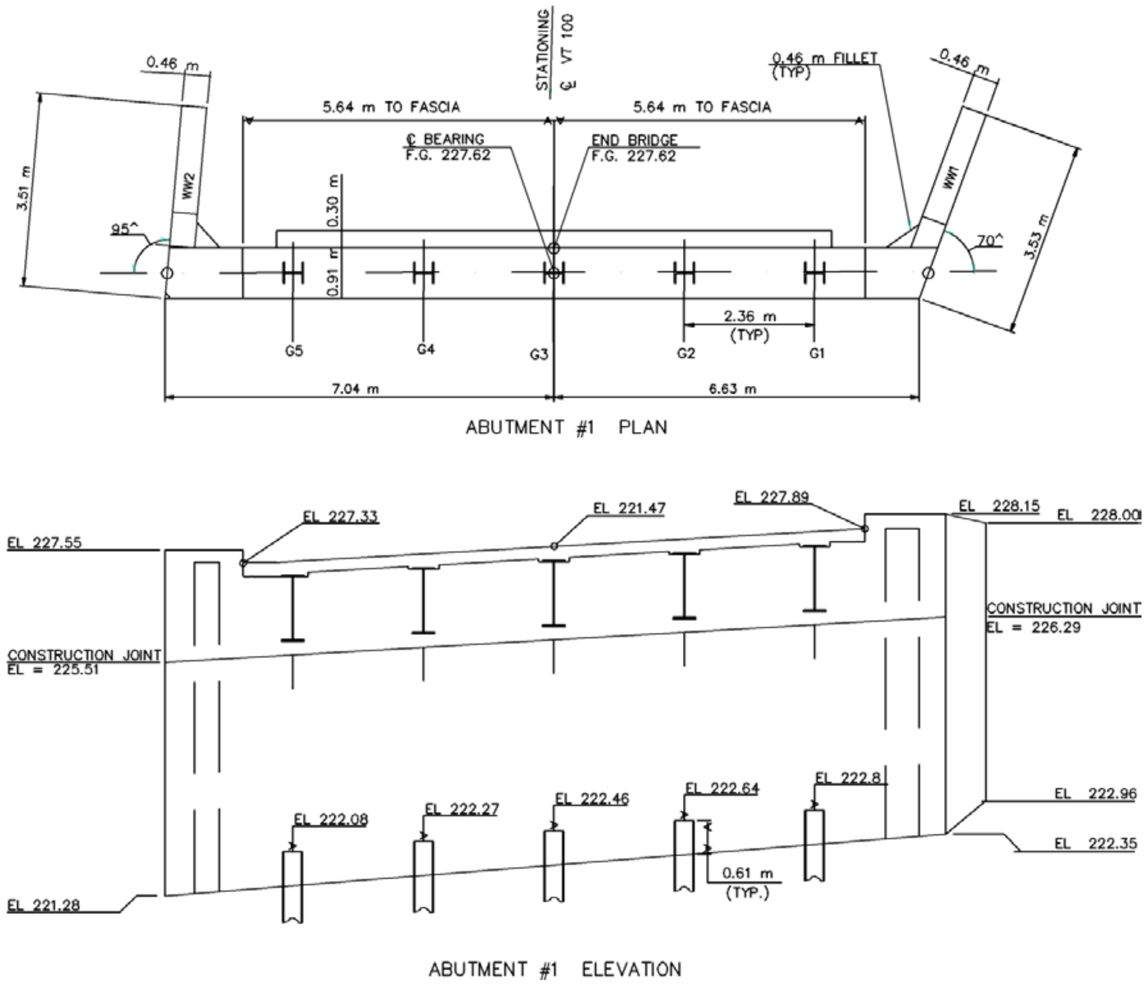


Figure 0.8: Plan and Elevation View of Abutment 1 at Stockbridge Bridge

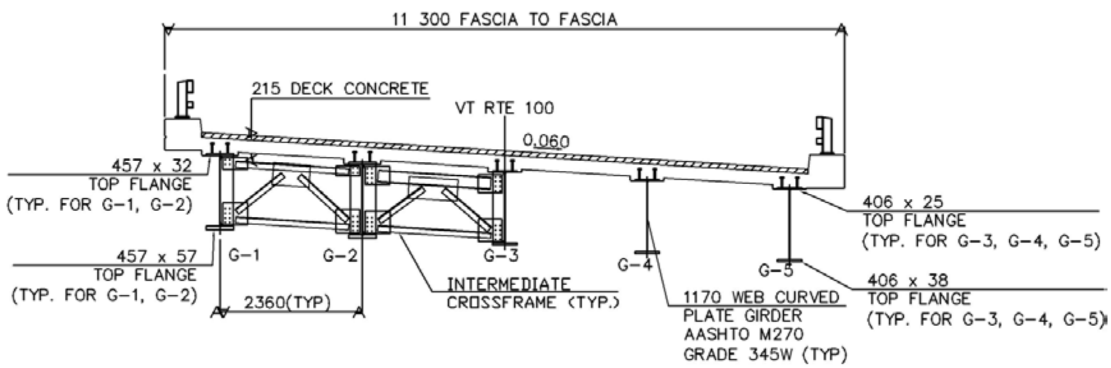


Figure 0.9: Stockbridge Deck Section 1

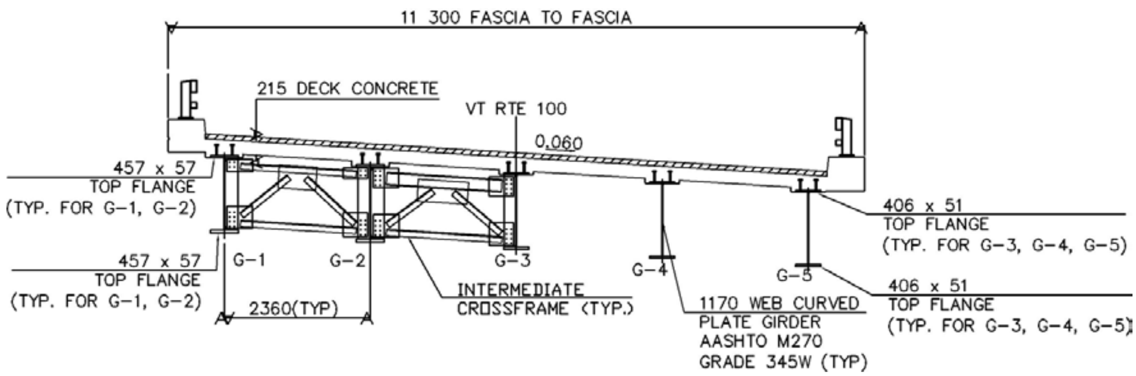


Figure 0.10: Stockbridge Deck Section 2

3.4 Instrumentation of Bridges

All three IABs were constructed with an array of gages and data logging equipment. The instrumentation consists of Geokon gages and multiplexers and Campbell Scientific CR1000 and CR10X data loggers. The data loggers allowed for automated data collection and remote access to data. For the long-term monitoring of the bridges, readings were taken every 6 hours for over five years. All gages include internal thermistors which capture the temperature at each reading.

Locations of instrumentation and call-outs for gages specific to each bridge can be found in Appendix A and Appendix B. A summary of gage types and total number of gages at each bridge are presented in Table 0.1 and Table 0.2. Overview of instrumentation in plan view and abutment instrumentation are shown in Figure 0.11 and Figure 0.12 for Middlesex, Figure 0.13 and Figure 0.14 for East Montpelier, and Figure 0.15 and Figure 0.16 for Stockbridge.

Table 0.1: Summary of Gage Types

GAGE TYPE	GAGE TYPE	GAGE MODEL	MONITORED RESPONSE
Pier Strain Gage	Vibrating Wire	4200	Strains in pier column (only at Stockbridge Bridge)
Girder Strain Gage	Vibrating Wire	4050	Strains on girders
Pile Strain Gage	Vibrating Wire	4000	Strains on piles
Earth Pressure Cells	Vibrating Wire	4810 & 4815	Abutment Backfill Pressures
Displacement transducers	Vibrating Wire	4420	Abutment displacements
Inclinometers (Uniaxial)	Vibrating Wire	6350	Pile deformations
Tiltmeters (Uniaxial)	Vibrating Wire	6350	Abutment Rotations
Inclinometers (Biaxial)	MEMS	6150	Pile deflections
Tiltmeters (Biaxial)	MEMS	6160	Abutment Rotations (only at Stockbridge Bridge)

Table 0.2: Total Number of Gages in Each Bridge

	Strain Gage			Pressure Cell			Displacement Transducer	Tiltmeter	Inclinometer	Total Number of Gages
	Pile	Girder	Pier	Abutment	Wingwall	Reference				
Middlesex	37	18	NA	11	1	1	4	2	9	83
East Montpelier	32	20	NA	12	1	1	5	2	16	89
Stockbridge	60	24	8	16	4	1	6	2	10	131

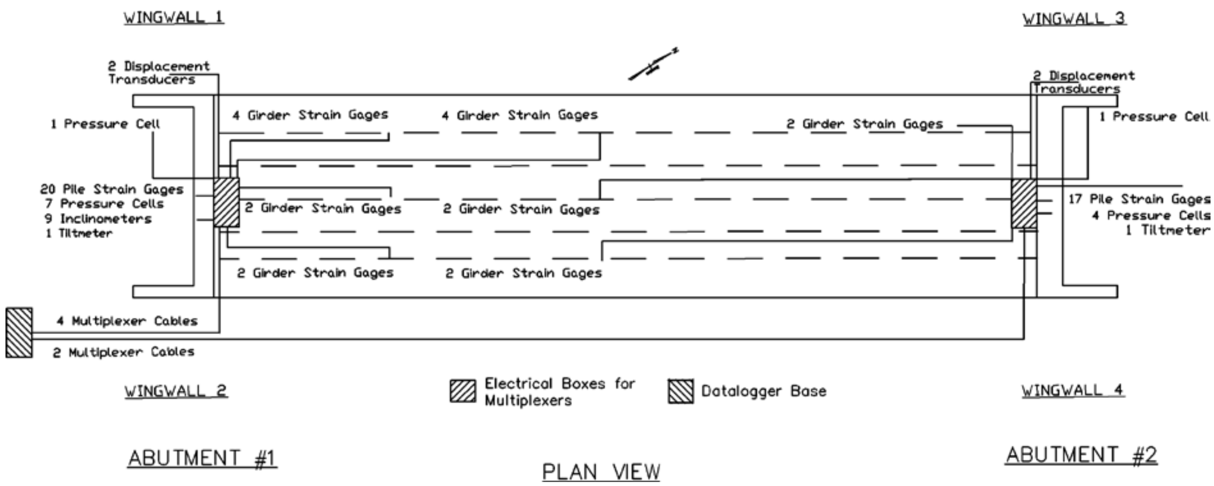


Figure 0.11: Overview of Instrumentation in Middlesex Bridge

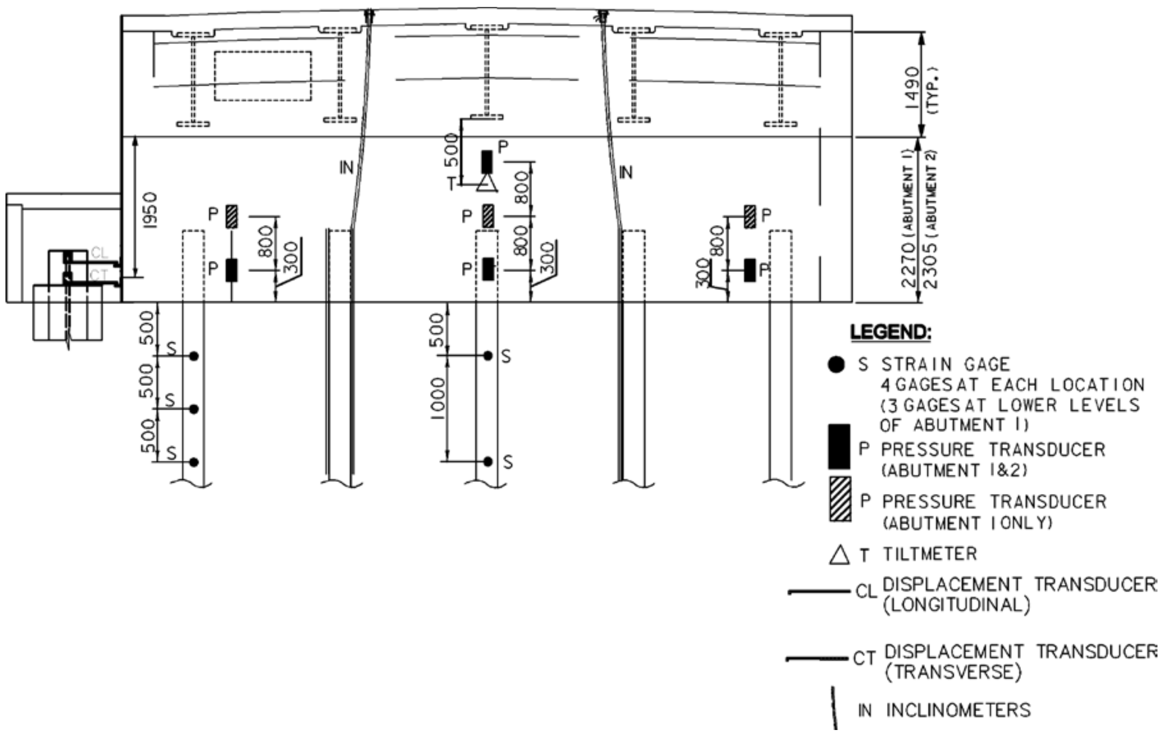


Figure 0.12: Abutment Instrumentation at Middlesex Bridge

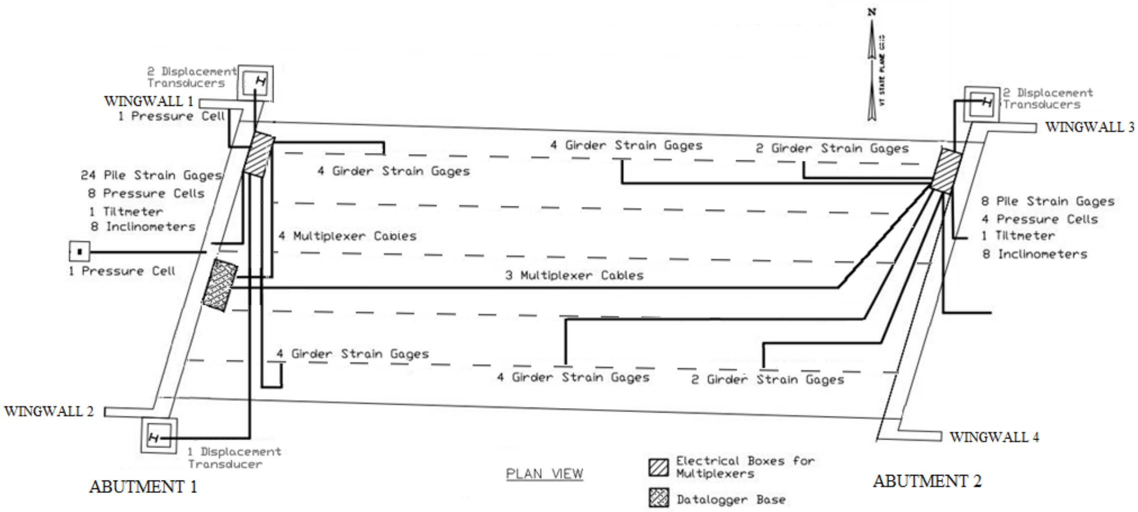


Figure 0.13: Overview of Instrumentation in East Montpelier Bridge

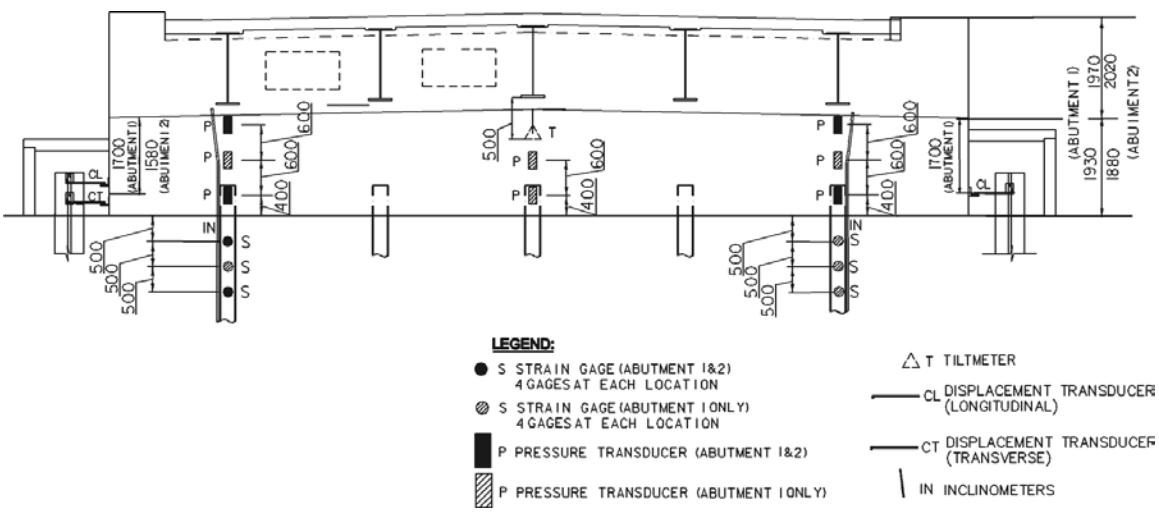


Figure 0.14: Abutment Instrumentation at East Montpelier Bridge

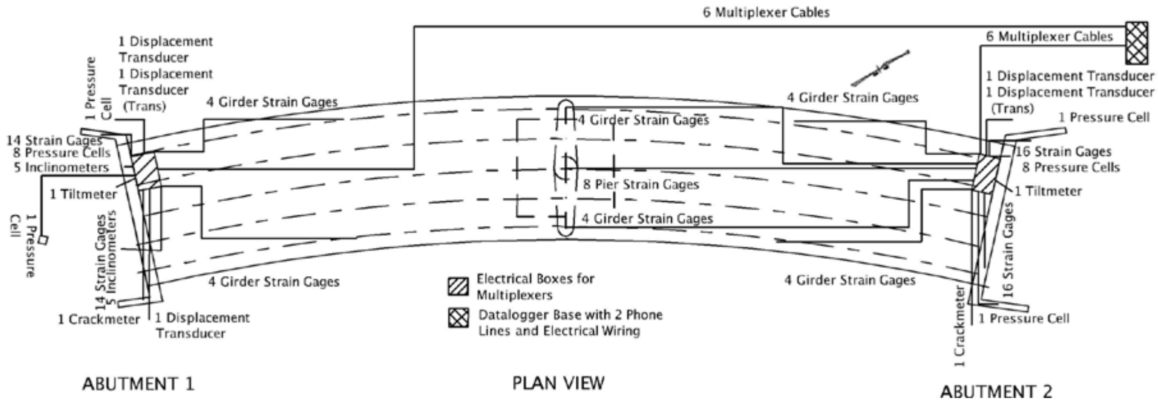


Figure 0.15: Overview of Instrumentation at Stockbridge Bridge

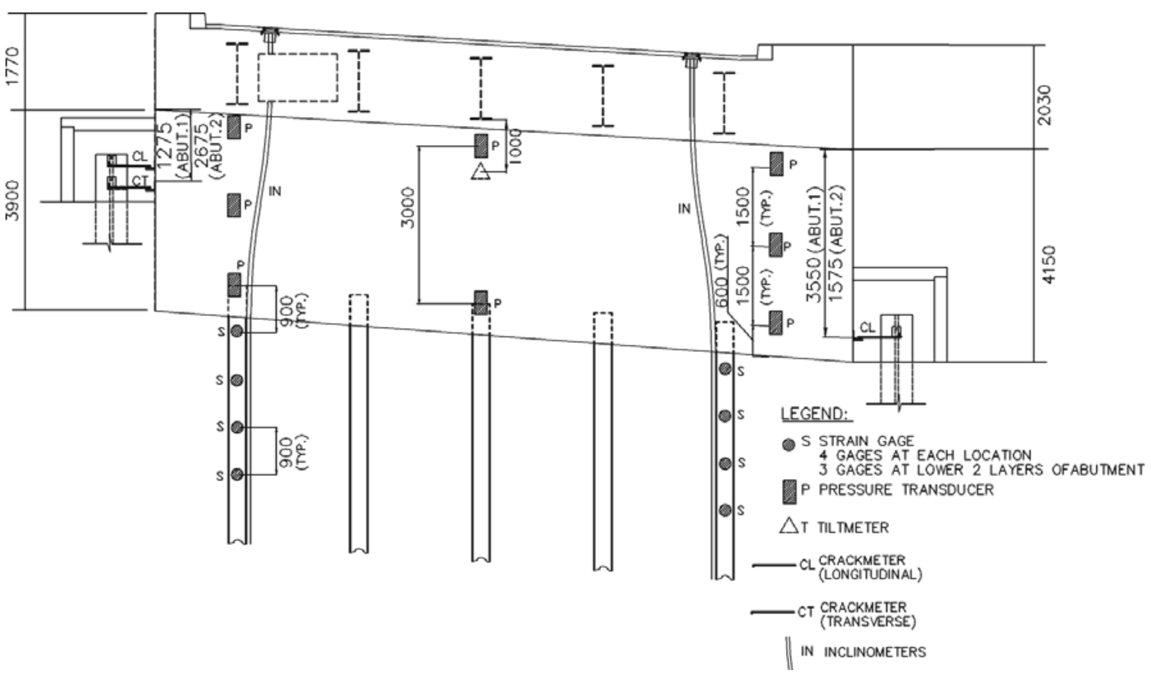


Figure 0.16: Abutment Instrumentation at Stockbridge Bridge

CHAPTER 4

FINITE ELEMENT MODELS

Three dimensional finite element models (FEMs) were created for each bridge (described in Chapter CHAPTER 3) using SAP2000. Original FEMs including nominal properties were created and outlined in previous research (Kalayci 2012) and modified as needed for this project. The steel elements (girders, piles, cross-bracing) were modeled using two-node frame elements and section properties corresponding to the actual sections used at the individual bridges. Steel girder properties were calculated at the center of the top flange. Concrete elements (deck, abutments, and wingwalls) were modeled using four-node thin shell elements. Cracked section properties were assumed for all concrete elements (0.35 I_g for deck and abutments and 0.70 I_g for the Stockbridge bridge pier column). Frame and shell elements have six degrees of freedom at each node.

Material properties are presented in Table 0.1.Class A concrete was used for the deck and for the part of the abutment above the construction joint (located near bottom of steel girders). Class B concrete was used for all other concrete elements. Composite action between the girders and the deck was simulated by use of rigid links defined between each deck node and coincident girder nodes. Transfer of moments at each girder-abutment connection was enabled by constraining all degrees of freedom of these links. A schematic of the superstructure showing the cross section modeling used for all three bridges is shown in Figure 0.1. The full bridges modeled in FEMs are shown in Figure 0.2.

Soil was modeled using nonlinear Winkler springs. The pile springs were modeled in two perpendicular directions at 1 ft. increments along the pile depth to simulate soil around the pile. The force-deformation (p-y) curves for these pile soil springs were defined using the hyperbolic tangent method described in American Petroleum Institute (API) standards (API 1993) (Equation 0.1).

$$F = A * p_u * \tanh\left(\frac{k_1 * z * y}{A * p_u}\right) * L \qquad \textbf{Equation 0.1}$$

where A = empirical correction factor (Equation 0.2), D= pile diameter, p_u = estimated ultimate lateral soil resistance, k_1 = soil strength modulus, z = soil depth from top of soil later to the specified node, y = horizontal deflection, L = length of pile section.

$$A = 3 - 0.8\left(\frac{z}{D}\right) \geq 0.9 \qquad \textbf{Equation 0.2}$$

Behind the abutments, backfill was modeled with springs oriented orthogonal to abutment and wingwalls. The spring resistance was defined by p-y curves which were calculated for various soil depths. Each p-y curve had a corresponding tributary area where the effective horizontal earth pressure was estimated using Equation 0.3.

$$\boldsymbol{F = K * \sigma'_v * w * h} \qquad \textbf{Equation 0.3}$$

where F = effective lateral soil resistance on the defined object with corresponding dimensions of w (width) by h (height) of tributary area, K = lateral earth pressure coefficient (calculated following the report by Barker et al. (1991)), and s'_v= effective vertical earth pressure.

Soil was assumed as dense for the compacted backfill (internal friction angle of 40° and density of 140 pcf (2243 kg/m^3)). The p-y curves were offset by an initial active pressure that was recorded by field data at the end of construction/beginning of long term monitoring. It should be noted that as a result of the Geofoam installed behind the abutments at the Stockbridge Bridge, the FEM best modeled field behavior when abutment soil springs were neglected (due to the effectiveness of Geofoam at minimizing earth pressures), therefore all subsequent FEMs of the Stockbridge Bridge presented in this research do not include backfill.

For the soil around the piles, original properties were based on soil boring logs from the bridge construction sites; this soil was modeled as medium dense (internal friction angle of 35° and density of 130 pcf (2082 kg/m^3)). An example of the p-y soil curves and where the springs are applied in the models is presented in Figure 0.3.

The FEMs described in this chapter, using the original soil properties assumed at the bridges, will be referred to as "Nominal FEMs". Over time, changing responses at the bridges were not captured by the Nominal FEMs and FEMs were calibrated to field data to create "Matched FEMs". The Matched FEMs required changing soil spring properties in the models; this was done by adjusting the p-y curves systematically using different internal friction angles and soil density values to define various soil types. In some cases, soil needed to be removed over a specific depth of the pile in order to match field data. In these cases, soil-springs were assigned zero stiffness at corresponding depths. Specific details of “Matched FEMs” and soil conditions used in the models will be discussed in Chapter CHAPTER 5.

When models were created for parametric studies, the nominal models of the straight and skewed bridges served as the basis for the parametric FEMs with similar material and soil properties and varying geometry.

Table 0.1: Material Properties used in FEMs[1]

	Strength [MPa (ksi)]	Elastic Modulus [MPa (ksi)]	Shear Modulus [MPa (ksi)]	Coeff. of Thermal Expansion [m/m/^{o}C (in./in./^{o}F)]	Poisson’s Ratio	Unit Weight [kg/m^3 (lb/ft^3)]
Concrete (Class A)	28 (4.0)	25 (3605)	10 (1502)	9.9E-6 (5.5E-6)	0.2	2400 (150)
Concrete (Class B)	24 (3.5)	235 (3372)	10 (1405)	9.9E-6 (5.5E-6)	0.2	2400 (150)
Steel	345 (50.0)	200 (29000)	77 (11153)	11.7E-6 (6.5E-6)	0.3	7850 (490)

[1]Courtesy of Kalayci (2012)

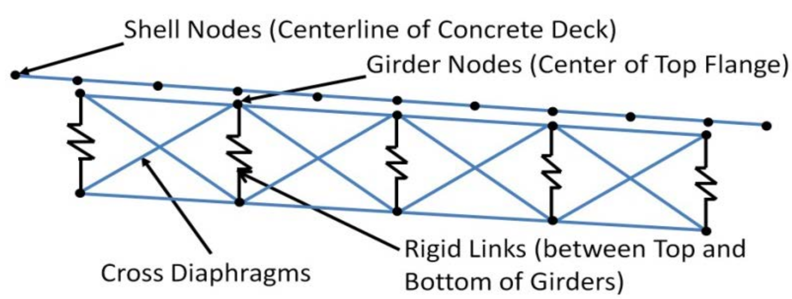
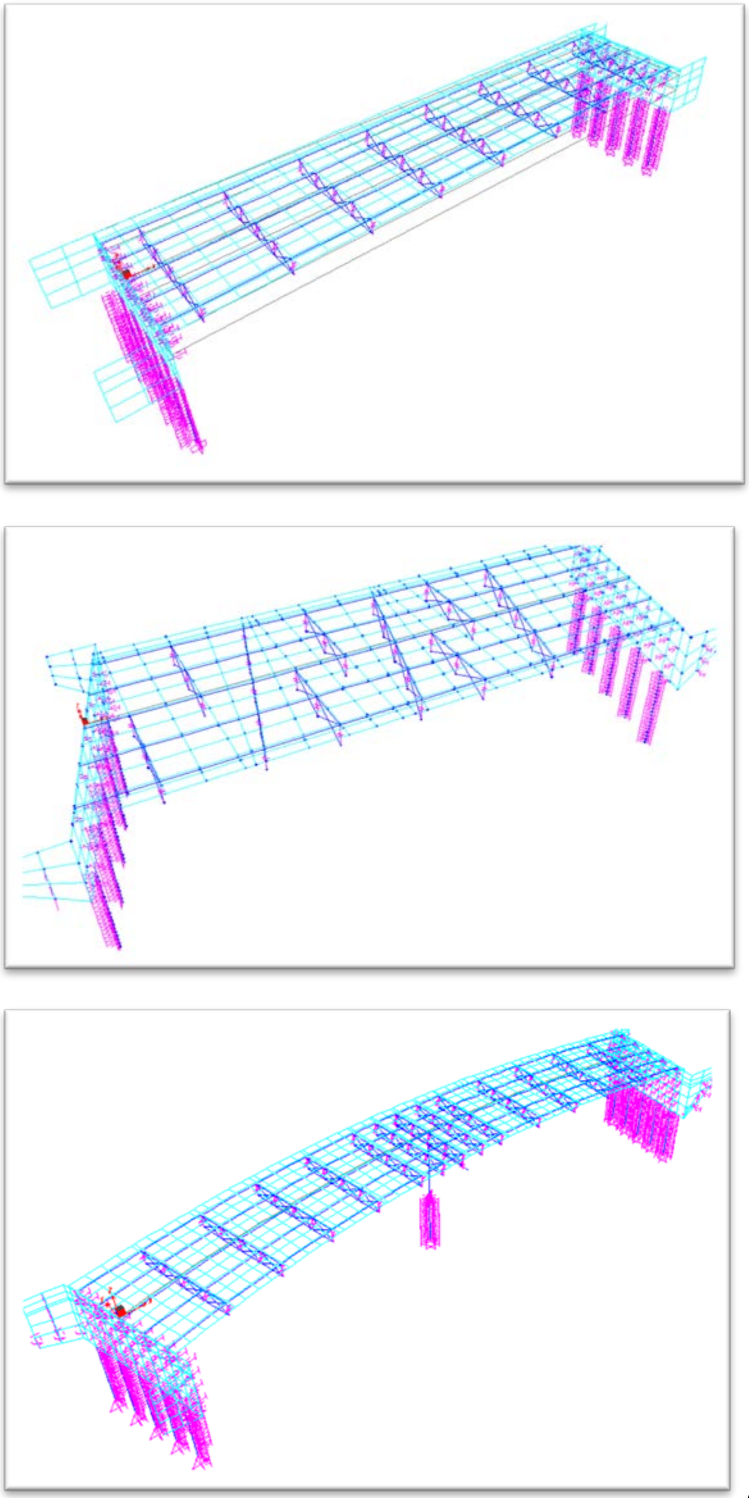


Figure 0.1: Node and Elements for Bridge Superstructure FEMs (Cross Section)[1]

[1]Courtesy of Kalayci (2012)



(a)

(b)

(c)

Figure 0.2: FEMs of the Three Monitored Vermont IABs (a) Middlesex (b) East Montpelier (c) Stockbridge

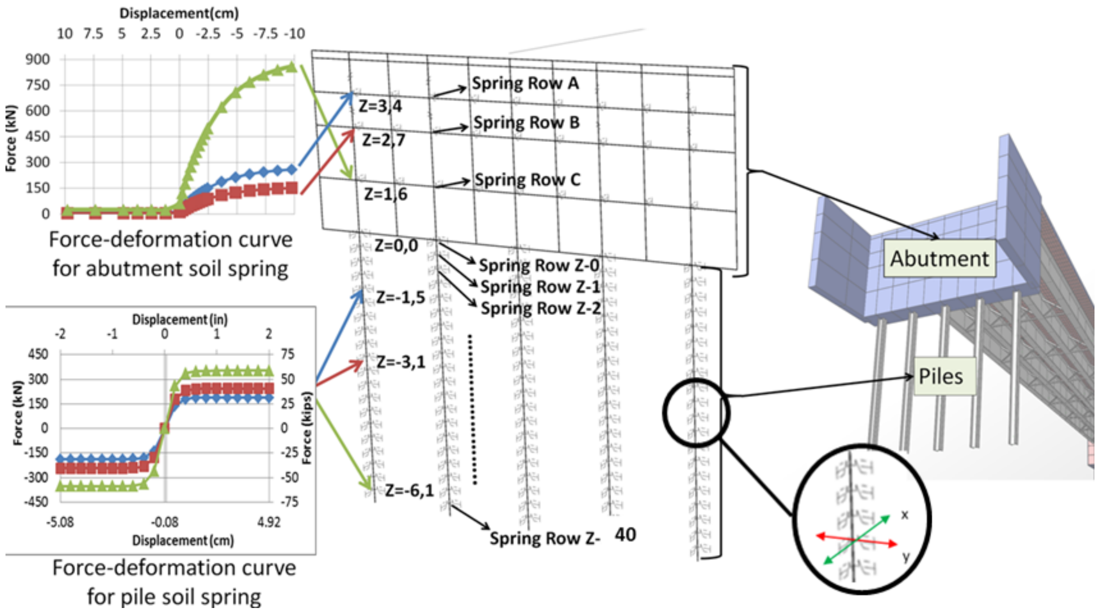


Figure 0.3: Example of Soil-Spring p-y Curves and Location in FEMs

CHAPTER 5

LONG TERM RESPONSE OF BRIDGES

This chapter presents long term field data collected from the Middlesex Bridge, East Montpelier Bridge, and Stockbridge Bridge. Unless otherwise noted, the data presented are due to thermal effects only. A description of gage labeling and corresponding gage locations is provided in the appendix. The long term monitoring of the three bridges began at the completion of construction: December 04, 2009 for the Middlesex Bridge, November 24, 2009 for the East Montpelier Bridge, and November 02, 2009 for the Stockbridge Bridge. Data is presented from the completion of construction in 2009 through the end of May, 2015 for all three bridges.

5.1 Temperature

Bridge ambient temperature is calculated by averaging the temperature recorded by the interior bottom flange gages of the girders such that they are not subject to direct sunlight. The ambient temperature over the monitoring period is shown in Figure 5.1, Figure 5.2 and Figure 5.3 for Middlesex, East Montpelier, and Stockbridge, respectively. Electrical storms caused extended periods of data acquisition interruptions in all three bridges during 2011 due to equipment damage. The temperature recorded at the point when long term monitoring initiated is referred to as the “reference temperature”. Theoretically, if the bridge response is purely elastic, all gage readings would be zero for subsequent points in time when the reference temperature occurs. The bridge reference temperature is 45.3°F (7.4°C), 46.0°F (7.8°C), and 45.4F (7.4°C) for Middlesex, East Montpelier and Stockbridge, respectfully.

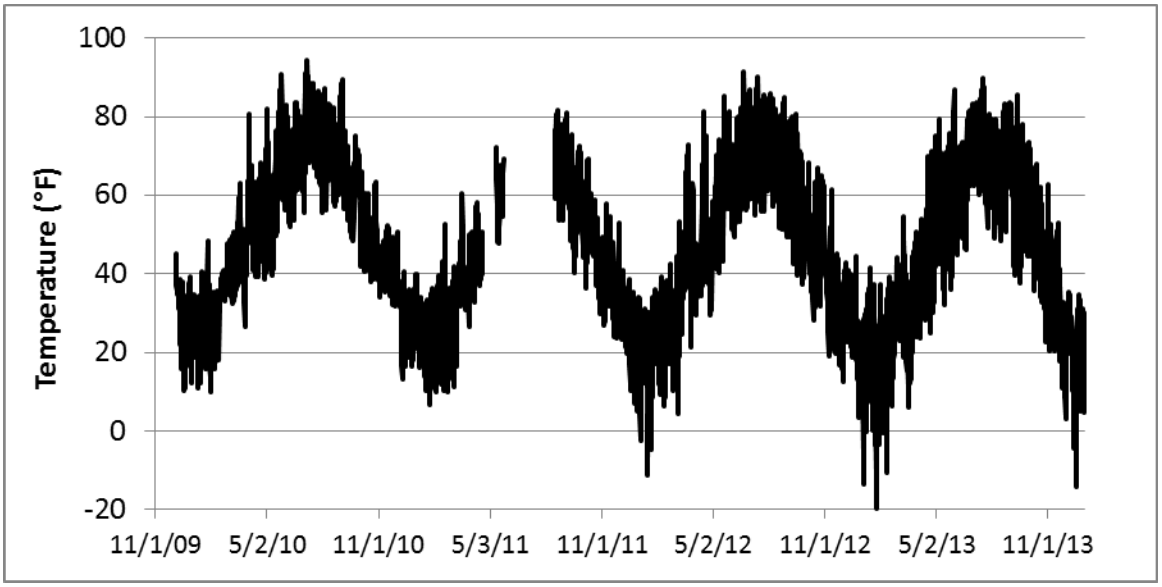


Figure 5.1: Middlesex Ambient Bridge Temperature

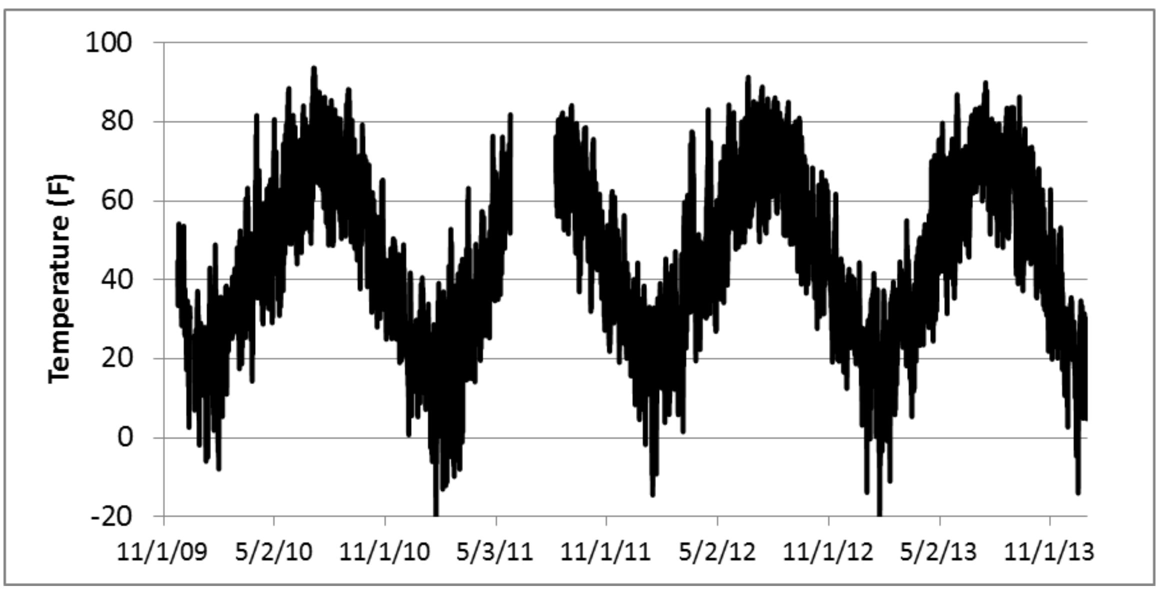


Figure 5.2: East Montpelier Ambient Bridge Temperature

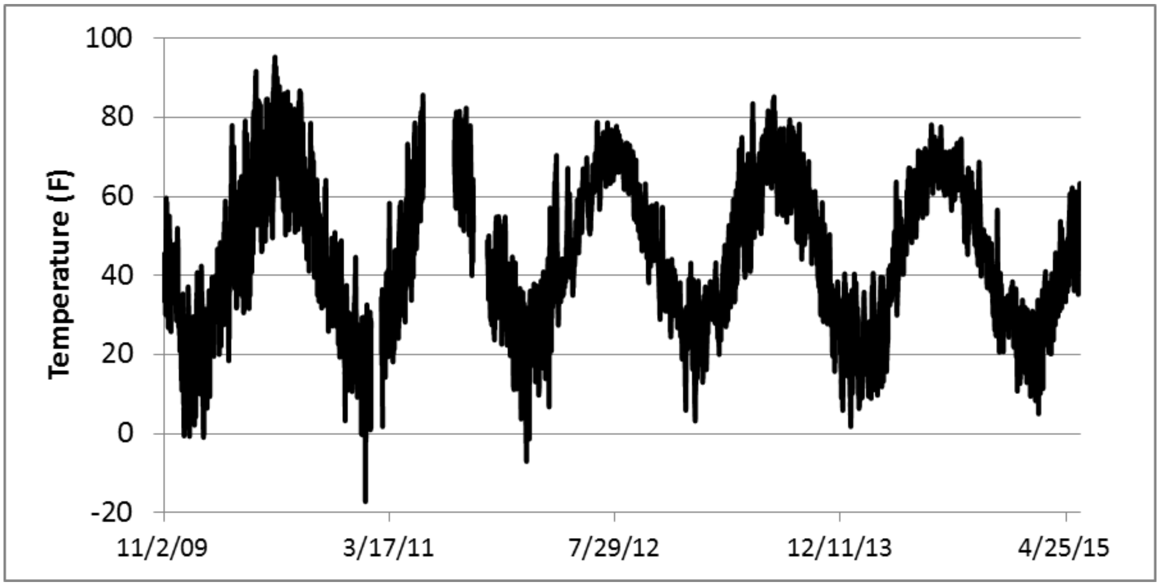


Figure 5.3: Stockbridge Ambient Bridge Temperature

5.2 Girder Stresses

All stress values presented in this section correspond to those induced by thermal effects only. Girder stresses for the Middlesex Bridge are presented in Figure 5.4, Figure 5.5 and Figure 5.6. Girder stresses for the East Montpelier Bridge are presented in Figure 5.7, Figure 5.8 and Figure 5.9. Girder stresses for the Stockbridge Bridge are presented in Figure 5.10, Figure 5.11 and Figure 5.12. All girder stresses shown are those induced by thermal effects only.

First, the flange stresses for the corresponding bridge are presented with temperature, and then shown over time. A negative value indicates compressive stress and a positive value indicates tensile stress. Bottom flange stresses are consistently greater than top flange stresses at all three bridges due to the elastic neutral axis located near the slab/girder interface. Temperature has little effect on top flange stresses which indicates that the girders act compositely for both positive and negative bending.

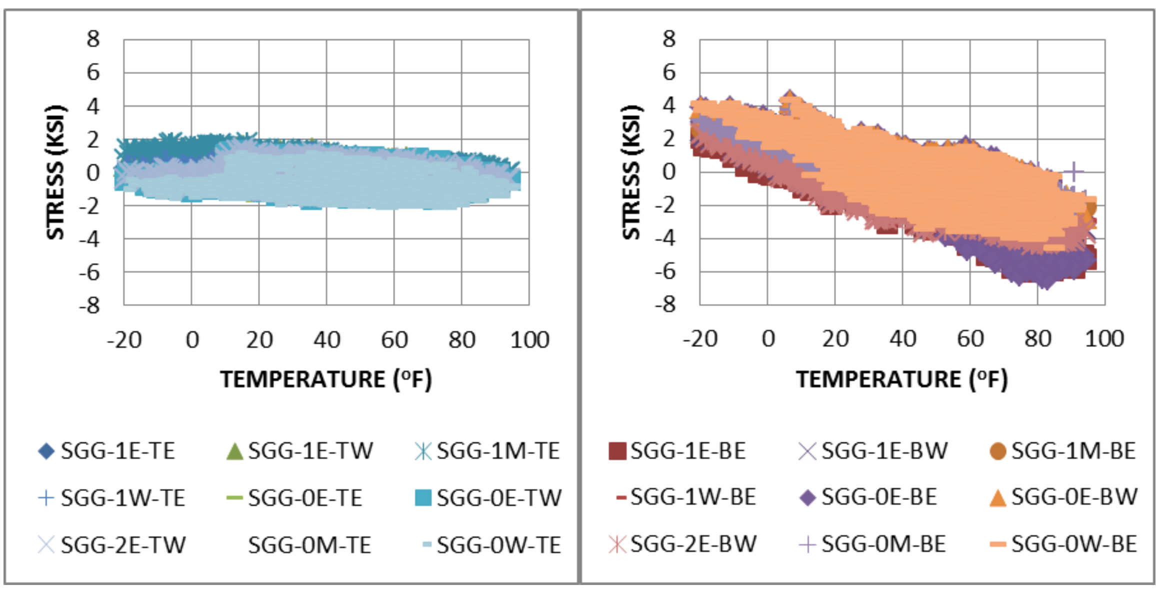
For the Middlesex Bridge, maximum compressive stress on the top flange was -2.1 ksi (-14.5 MPa) and maximum tensile stress was 1.7 ksi (11.7 MPa). The bottom flange maximum compressive stress was -6.4 ksi (-44.1 MPa) and maximum tensile stress was 4.3 ksi (29.6 MPa).

For the East Montpelier Bridge, the maximum top flange compressive stress was -5.2 ksi (-35.9 MPa) and maximum tensile stress was 2.8 ksi (19.3 MPa). For the bottom flanges, the maximum compressive stress was -6.7 ksi (-46.2 MPa) and the maximum tensile stress was 5.7 ksi (39.3 MPa).

For the Middlesex and East Montpelier Bridges, an increase in temperature caused an increase in compressive stresses on the bottom flanges. Girder stresses across the abutment are consistent; the upstream, middle, and downstream girder have comparable stress values at any given point in time as seen for the Abutment 1 end. These stresses are also comparable at midspan of the bridge and at the Abutment 2 end, demonstrating the constant induced girder moment along the bridge for a given thermal change, as expected from finite element analysis.

The two-span Stockbridge Bridge had similar girder stresses to the other two bridges. The top flange stresses range from -2.9 ksi (-20.0 MPa) to 2.7 ksi (18.6 MPa), while the bottom flange stresses range from -6.5 ksi (44.8 MPa) to 4.9 ksi (33.8 MPa). Girder stresses with corresponding temperature are shown in the three instrumented locations for this two span bridge: Abutment 1 girder ends, midspan, and Abutment 2 girder ends. Midspan gages are located at 113.2 ft (34.5 m) and 105.0 ft (32.0 m) from Abutment 1 for upstream and downstream girder, respectively. Girder stresses have been consistent year to year.

Overall, thermally induced girder stresses were minimal. For all bridges, the bottom flange stresses were greater than top flange stresses due to the elastic neutral axis being located near the girder/slab interface. Temperature had minimal influence on top flange stresses indicating that girders act compositely for both positive and negative bending. For the single span bridges, bottom flange stresses consistently decreased with increasing temperature. Stresses were comparable between the three bridges.



(a) (b)

Figure 5.4: Middlesex Girder Stresses with Temperature (a) top flange gages (b) bottom flange gages

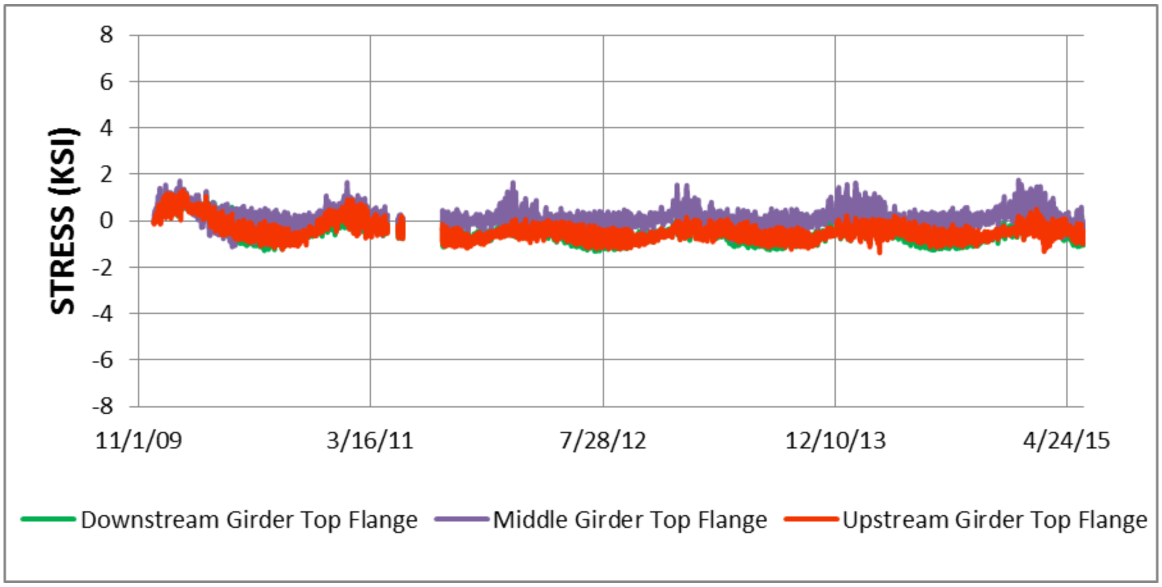


Figure 5.5: Middlesex Top Flange Girder Stresses at Abutment 1

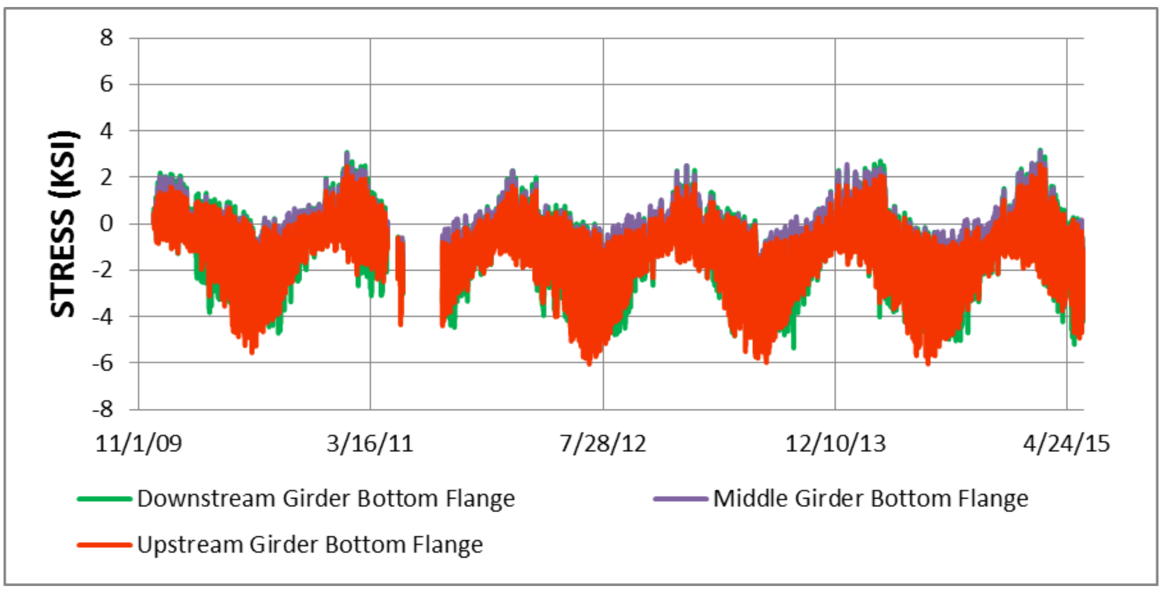
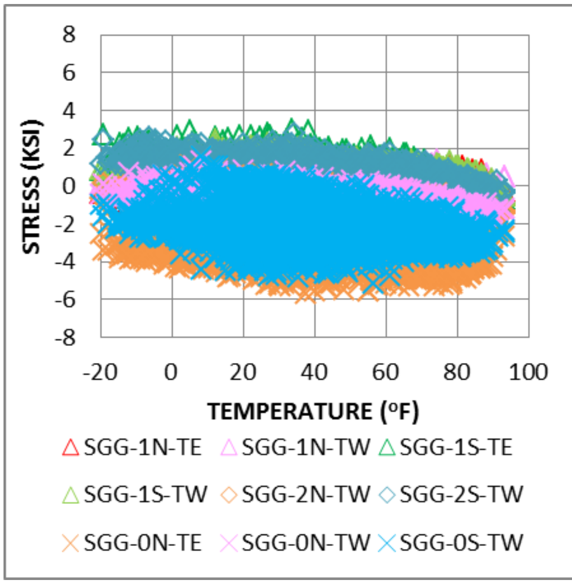
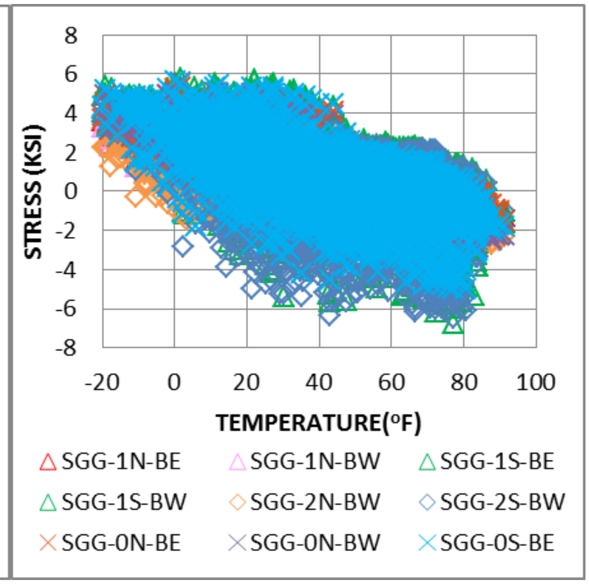


Figure 5.6: Middlesex Bottom Flange Girder Stresses at Abutment 1



(a)



(b)

Figure 5.7: East Montpelier Girder Stresses (a) top flange (b) bottom flange

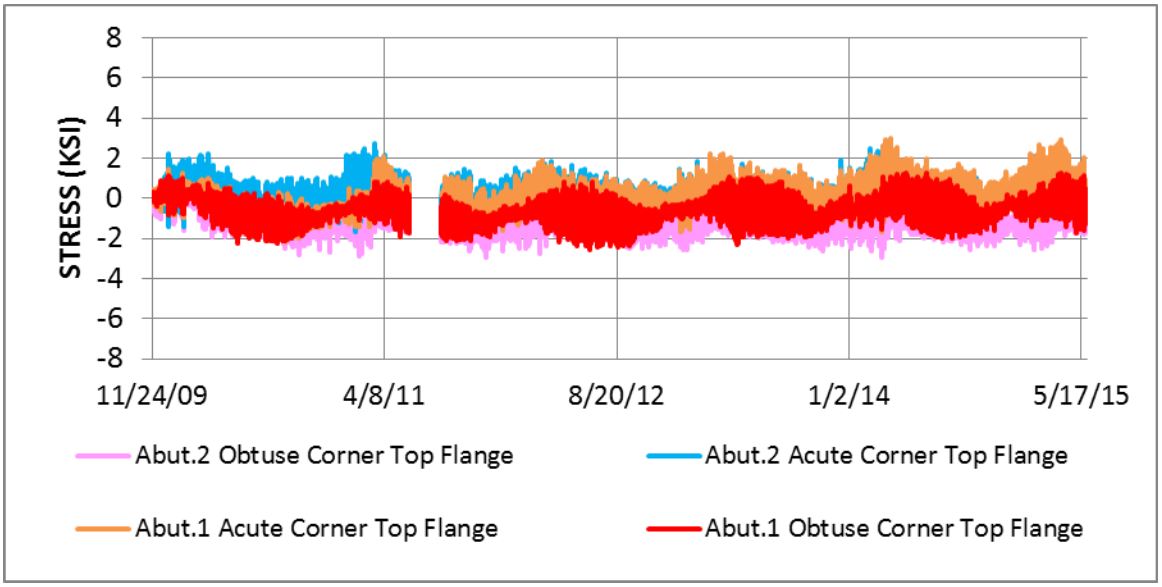


Figure 5.8: East Montpelier Bridge Girder Stresses at Obtuse and Acute Corner Girder Ends Top Flange Gages

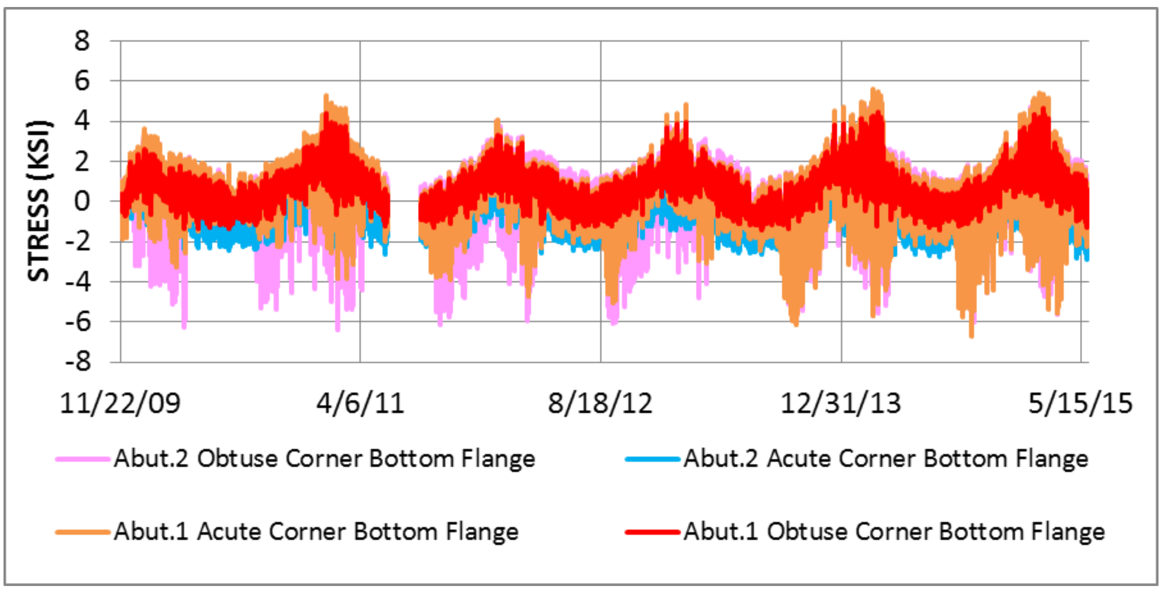


Figure 5.9: East Montpelier Bridge Girder Stresses at Obtuse and Acute Corner Girder Ends Bottom Flange Gages

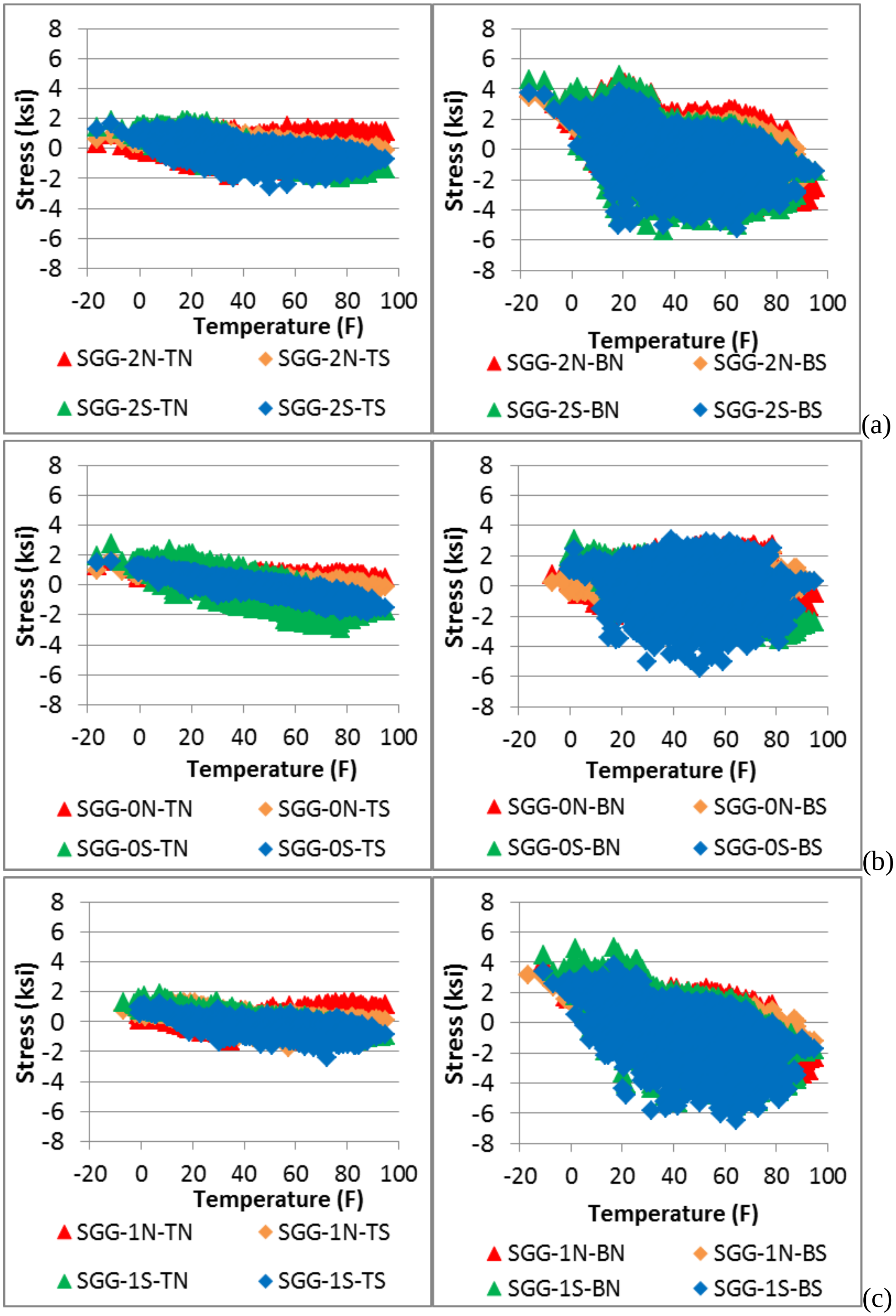


Figure 5.10: Stockbridge Girder Stresses vs. Temperature – (left) Top Flange Gages (right) Bottom Flange Gages -- (a) Abutment 1 (b) midspan and (c) Abutment 2

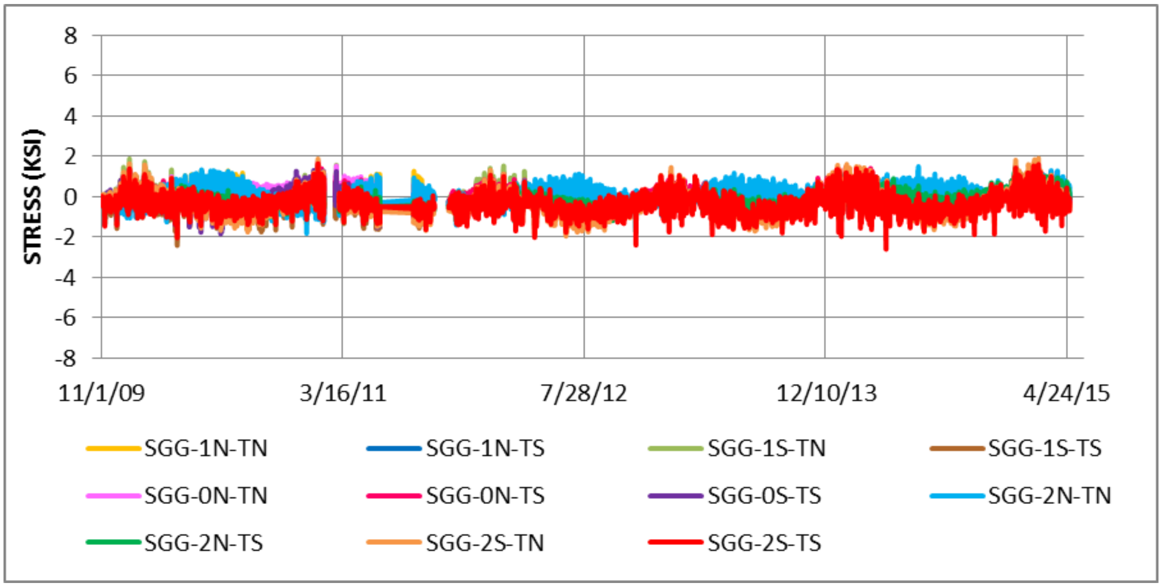


Figure 5.11 Stockbridge Girder Stresses Top Flange Gages

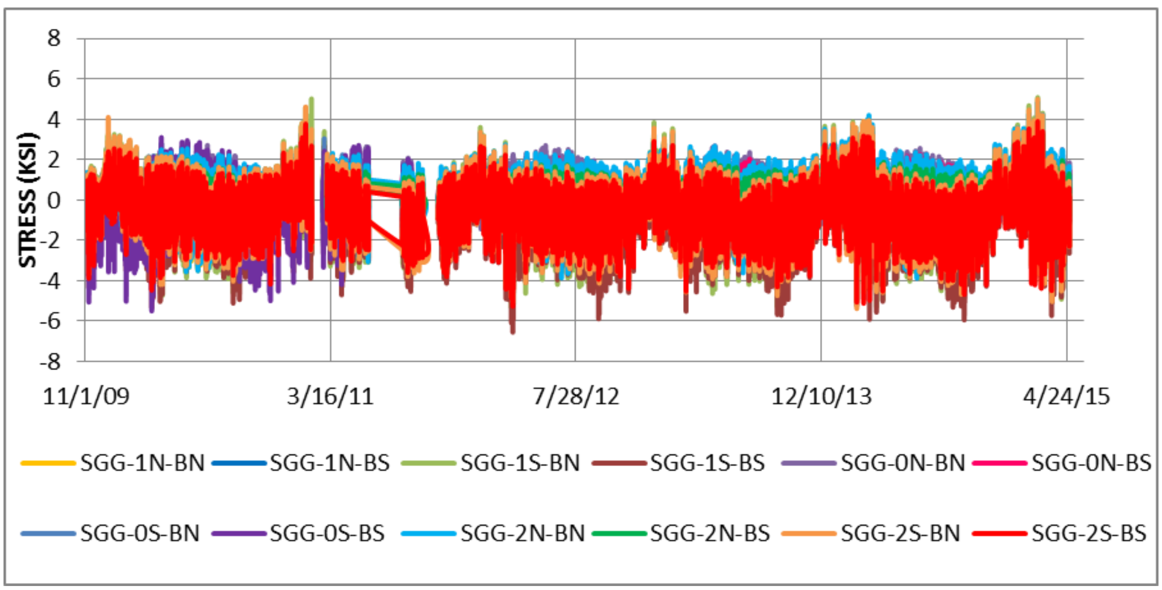


Figure 5.12 Stockbridge Girder Stresses Bottom Flange Gages

5.3 Abutment Top Longitudinal Displacement

Top of abutment displacements are those induced by thermal effects only, and refers to displacement at the top of the girder. Top of abutment displacement is shown in

Figure 5.13, Figure 5.14 and Figure 5.15 for the Middlesex, East Montpelier and Stockbridge Bridge, respectively. For longitudinal displacements, sign convention is consistent for all three bridges; a negative displacement at Abutment 1 indicates movement towards the backfill while a positive value indicates movement towards the river, while for Abutment 2 the opposite sign convention is used in order to visualize the abutments expanding away from each other and contracting towards each other. There was a shift in displacement towards the backfill recorded over time for all three bridges for at least one location.

At the straight Middlesex Bridge, Abutment 1 experienced a permanent shift towards the backfill following the flooding that occurred in 2011. This unsymmetrical response is not something that would be predicted in FEM or in design. After the initial shift towards the backfill, the peak to peak displacement remained similar year to year, with the top of the abutment essentially oscillating around a new zero position. Abutment 2 also experienced a gradual, and slight, increase in displacement towards the backfill overtime. The maximum displacement recorded at the Middlesex Bridge was 0.70 in (17.8 mm).

The East Montpelier Bridge showed a more uniform shift towards the backfill where both abutments shifted equally. This shift increased over time, while the top of the abutment still contracted past zero during the winter months. The response of this bridge was more symmetric than the straight bridge, which is not something that would be predicted. The obtuse and acute corner displacements are nearly identical showing that for longitudinal direction, the top of abutment displacement of the 15 degree skewed bridge could be approximated as a straight bridge. The maximum displacement recorded at the East Montpelier Bridge was 0.55 in (14.0 mm).

The curved Stockbridge Bridge only showed an increase in displacement at the top of the upstream location at Abutment 2. All other locations experienced very consistent year to year readings. The average of the downstream and upstream corner displacements of Abutment 2 is similar to the readings at Abutment 1. At Abutment 1, the displacement at the upstream and downstream locations is nearly identical. The maximum displacement recorded at the Stockbridge Bridge is 0.83 in (21.1 mm), while Abutment 1 had a maximum of 0.66 in (16.8 mm). The Stockbridge Bridge had the most consistently elastic behavior year to year compared with the straight Middlesex Bridge and skewed East Montpelier Bridge. The top of abutment displacement of this curved bridge could be accurately estimated assuming a straight bridge response.

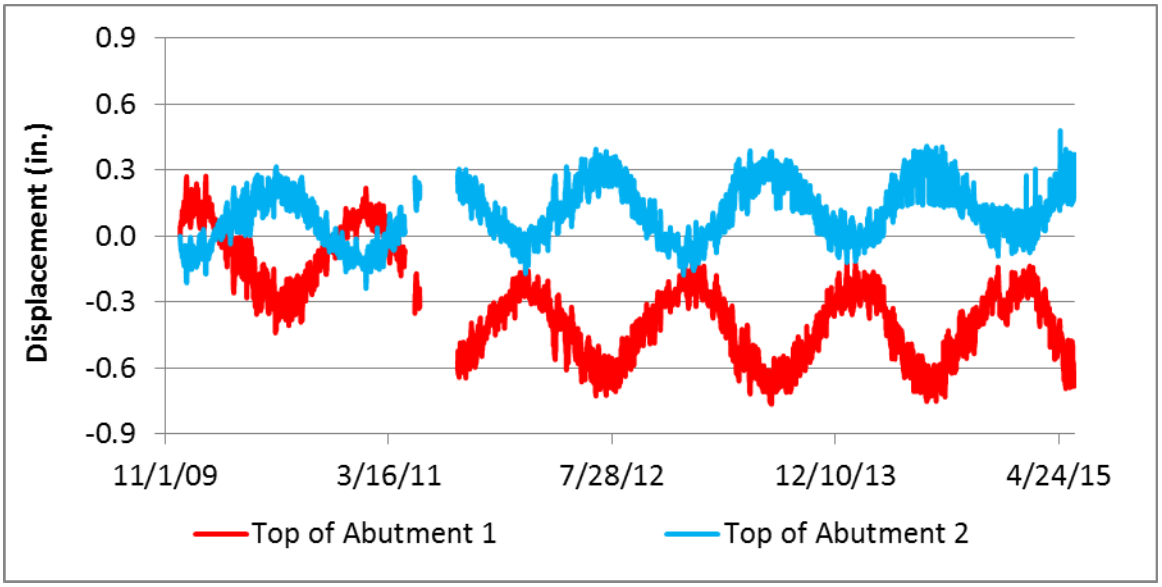


Figure 5.13: Middlesex Longitudinal Top of Abutment Displacements

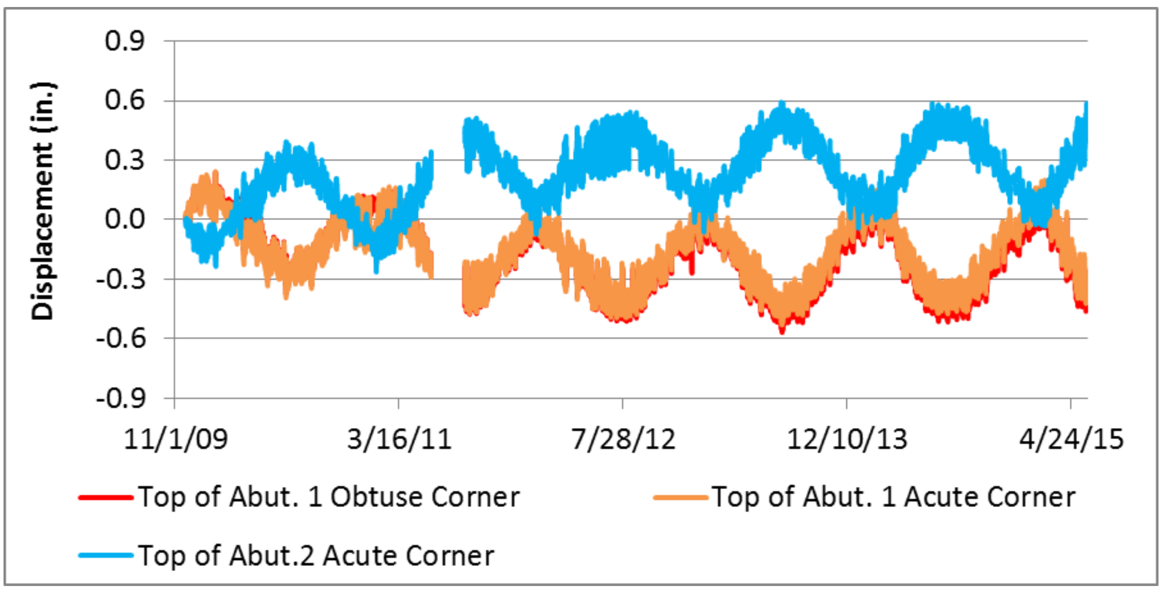


Figure 5.14: East Montpelier Longitudinal Top of Abutment Displacements

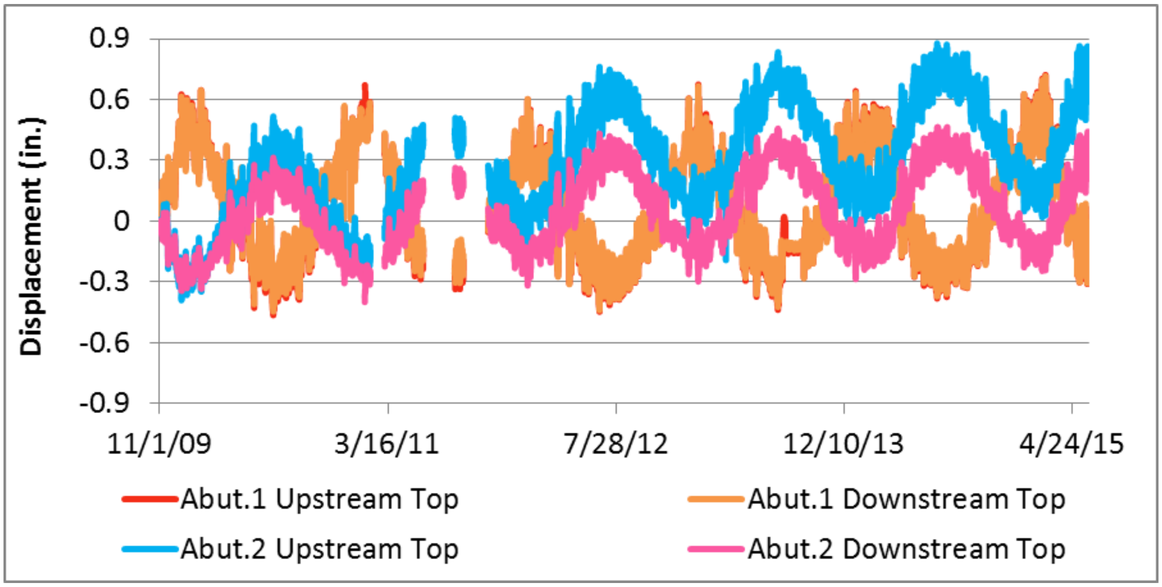


Figure 5.15: Stockbridge Longitudinal Top of Abutment Displacements

5.4 Abutment Bottom Longitudinal Displacement

Bottom of abutment displacements are those due to thermal effects only. Bottom of abutment displacement was lower than top of abutment displacement at all three bridges due to abutment movement being a combination of translation and rotation. Bottom of abutment displacement is shown in Figure 5.16, Figure 5.17, and Figure 5.18 for the Middlesex, East Montpelier, and Stockbridge Bridge, respectively.

At the Middlesex Bridge, the shift in displacement at the bottom of Abutment 1 is similar to that at the top of the abutment with less magnitude. Again, the peak to peak (displacement from minimum to maximum temperature in a given year) response following this shift was fairly consistent. The maximum displacement recorded was 0.41 in (10.4 mm) at Abutment 1, and 0.18 in (4.6 mm) at Abutment 2.

The East Montpelier Bridge has symmetric bottom of abutment displacements between the two abutments. Over time, the obtuse corner has slightly greater expansion values at both abutments. Peak to peak response is similar over time despite the increase in displacement towards the backfill. The maximum displacement recorded was 0.34 in (8.6 mm).

At the Stockbridge Bridge, the bottom of abutment displacement looks very similar to that seen at the Middlesex Bridge, despite the Middlesex Bridge being a straight bridge and shorter than the two-span curved Stockbridge Bridge. The maximum displacement at the abutment that shifted towards the backfill at Stockbridge (Abutment 2) is 0.42 in (10.7 mm), while the maximum displacement at Abutment 1 was 0.18 in (4.6 mm).

The variation in response from introducing 11.25 degrees of curvature or 15 degrees of skew to an IAB is no greater than the variability that can occur from unpredictable field conditions as seen at the straight Middlesex Bridge. Displacement at the bottom of the abutment was less than the top of abutment at all three bridges as a result of abutment rotation.

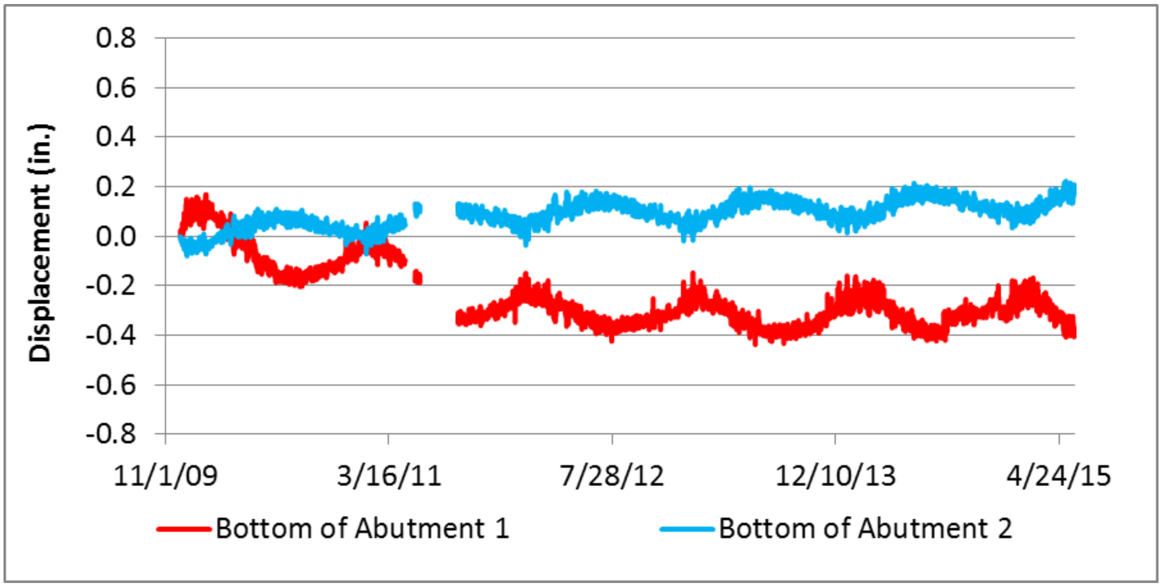


Figure 5.16: Middlesex Longitudinal Bottom of Abutment Displacements

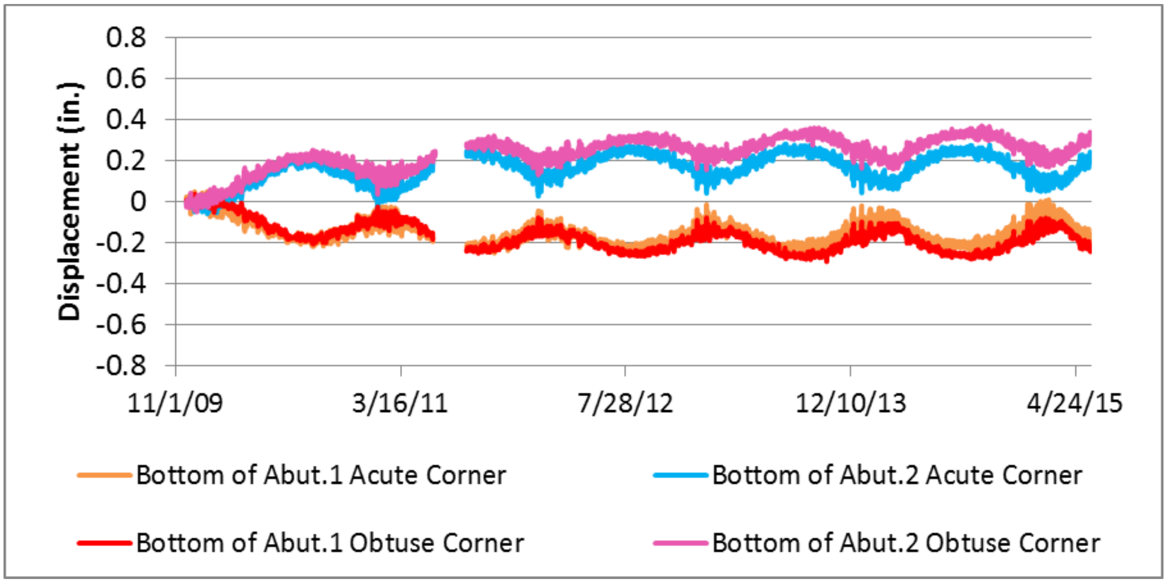


Figure 5.17: East Montpelier Longitudinal Bottom of Abutment Displacements

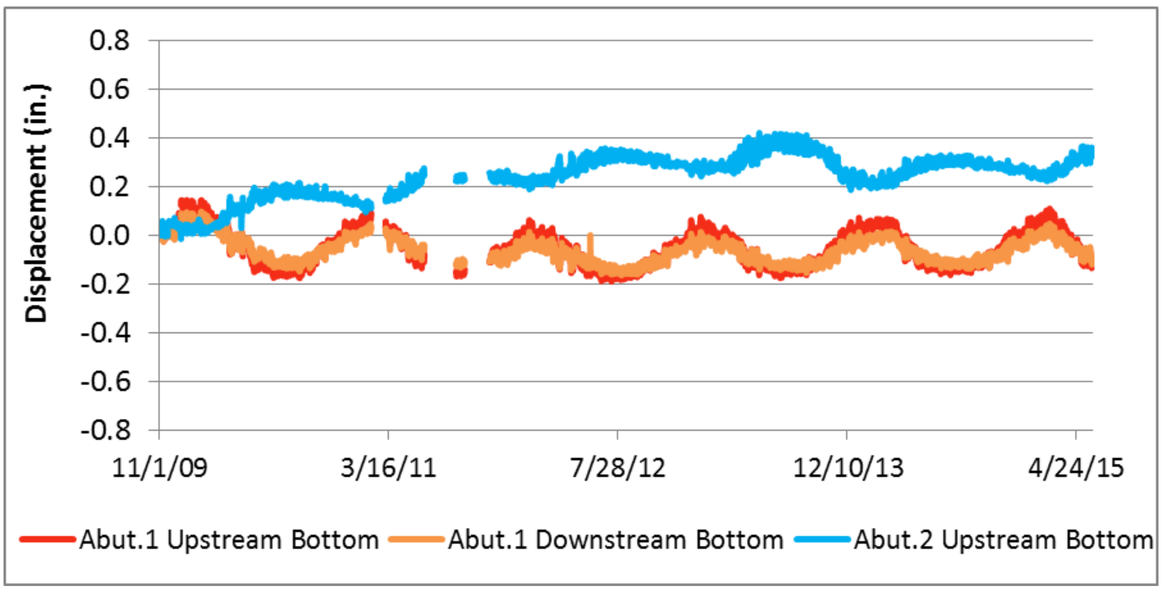


Figure 5.18: Stockbridge Longitudinal Bottom of Abutment Displacements

5.5 Abutment Bottom Transverse Displacement

Transverse displacements presented are those induced by thermal loads only.

Transverse displacements at the bottom of abutment (top of the pile) are presented in

Figure 5.19, Figure 5.20 and Figure 5.21 for the Middlesex, East Montpelier and Stockbridge Bridge, respectively.

Transverse displacements were small at the Middlesex Bridge and the Stockbridge Bridge though they did both display a slight transverse shift over time. At the East Montpelier Bridge, the bridge exhibited seasonal rotation in plan view with the majority of displacement occurring at the acute corner. This will be addressed in more detail in later sections.

The transverse displacement was only monitored at one side of the Middlesex and Stockbridge Bridges (therefore the net transverse displacement of the bridge cannot be determined). The maximum peak to peak transverse displacement at the top of the pile (occurring over the change from minimum to maximum temperature in a given year) was 0.10 in (2.5mm) and 0.16 in (4.1mm) at the Middlesex and Stockbridge Bridge, respectively. At the East Montpelier Bridge, the maximum peak to peak transverse displacement (sum of obtuse and acute corner) was 0.25 in (6.4 mm) and 0.18 in (4.6 mm) at Abutment 1 and Abutment 2, respectively.

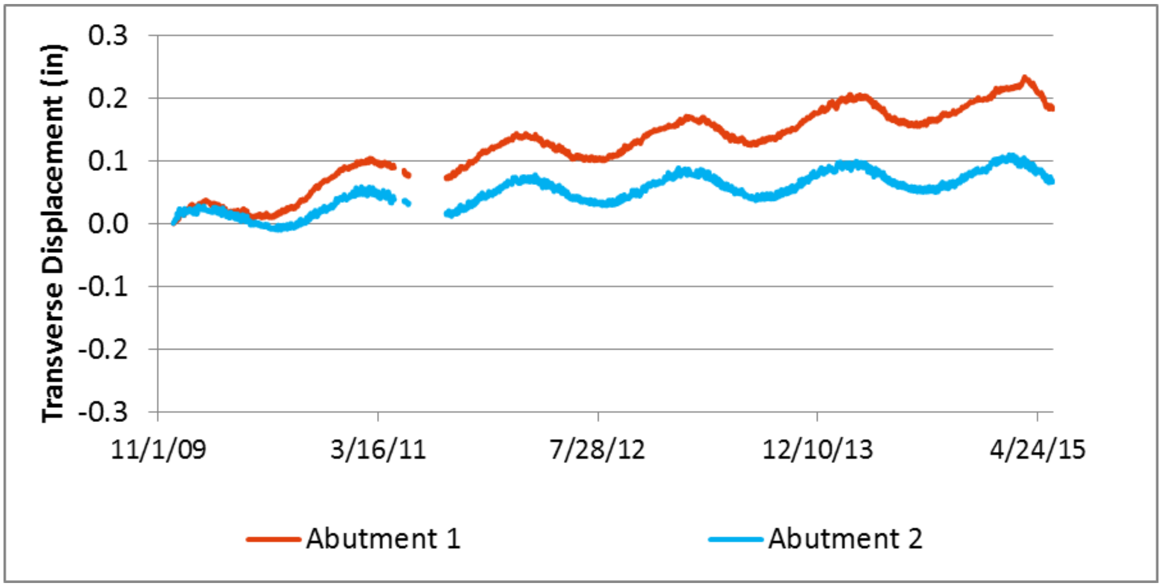


Figure 5.19: Middlesex Transverse Displacement

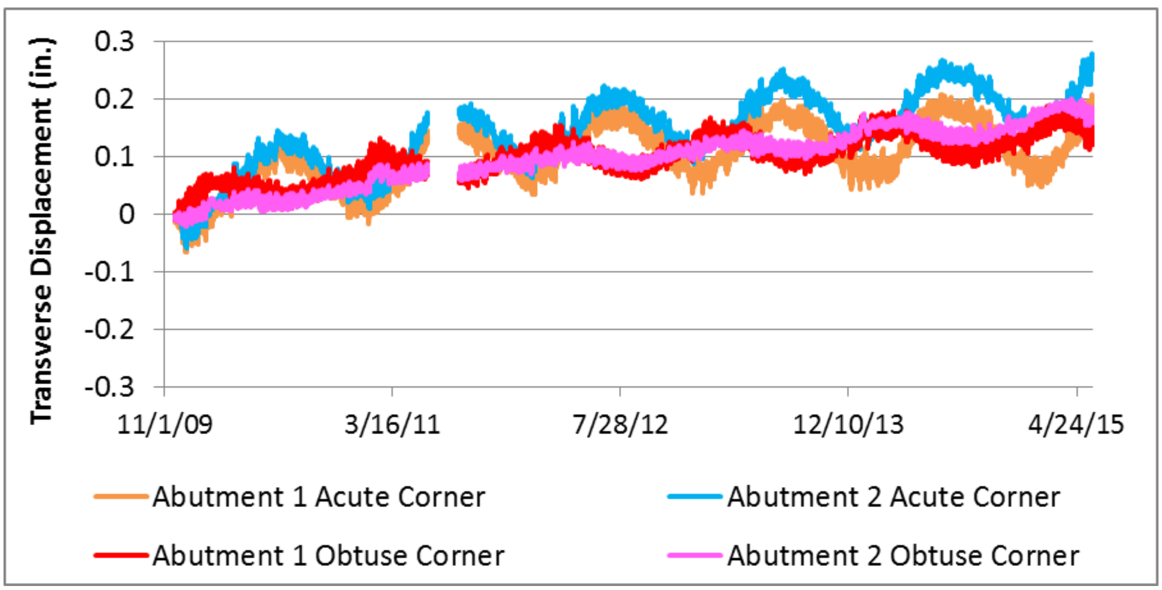


Figure 5.20: East Montpelier Transverse Displacement

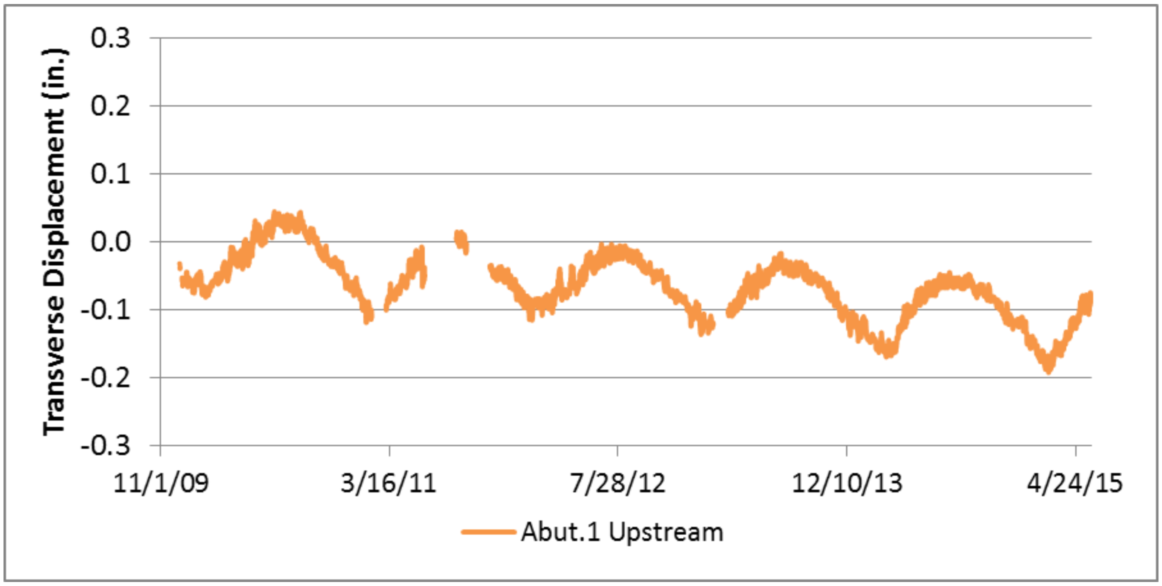


Figure 5.21: Stockbridge Transverse Displacement

5.6 Abutment Rotation

Abutment rotations presented in this section are the rotations resulting from thermal effects. A positive rotation value indicated rotation towards the river (bridge contraction) while a negative rotation value indicates rotation towards the backfill (bridge expansion) for both abutments. Abutment rotations are presented in Figure 5.22, Figure 5.23, and Figure 5.24 for the Middlesex, East Montpelier, and Stockbridge Bridge, respectively.

All three bridges had a difference in abutment rotations where one abutment had greater rotation under expansion while the other had greater rotation under contraction. Maximum rotation under expansion was similar between all three bridges (within 0.01 degrees of each other). The curved two-span Stockbridge Bridge had the greatest fluctuation in rotations (going between expansion and contraction at either abutment), which is consistent with the highly elastic response in top of abutment displacement. This bridge had the greatest rotation under contraction, which was about twice the contraction rotation recorded at the other two bridges.

At Middlesex, the maximum rotation under bridge contraction was 0.08 degrees at Abutment 2, maximum rotation under expansion was -0.12 degrees at Abutment 1. At the East Montpelier Bridge the maximum abutment rotation under contraction was 0.07 degrees at Abutment 1, while maximum rotation under expansion was -0.13 degrees at Abutment 2. At the Stockbridge Bridge maximum rotation under expansion was 0.15 degrees, while under contraction it was -0.11 degrees.

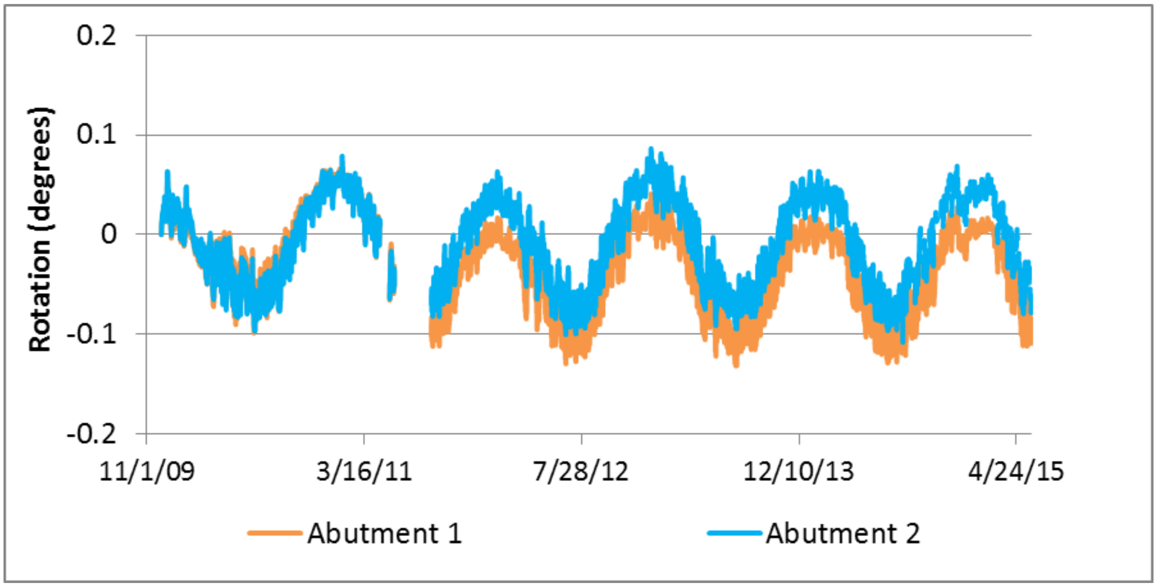


Figure 5.22: Middlesex Bridge Abutment Rotation

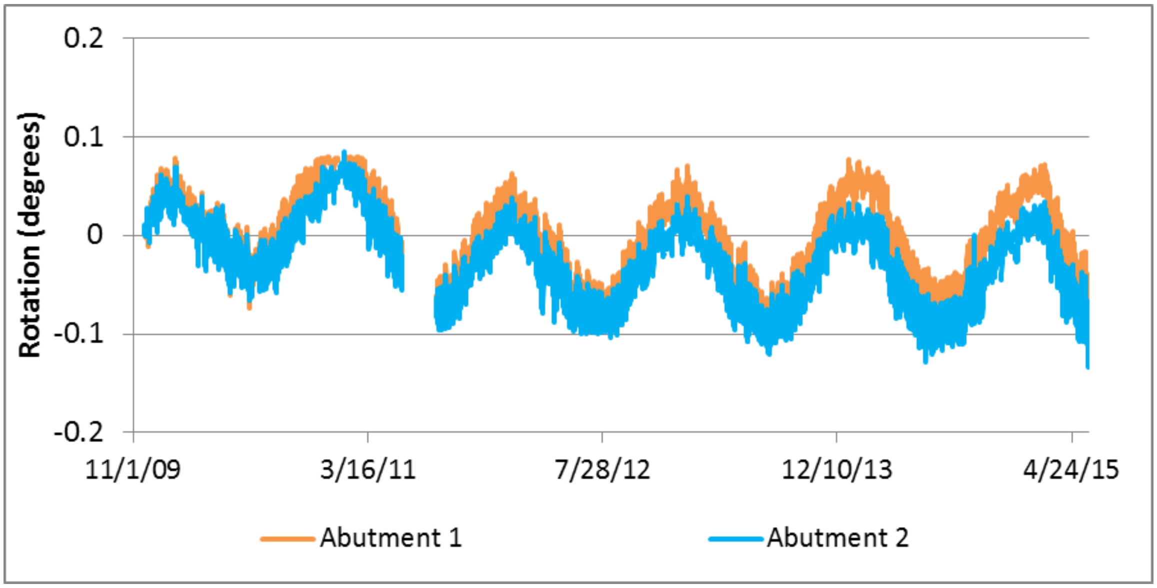


Figure 5.23: East Montpelier Bridge Abutment Rotation

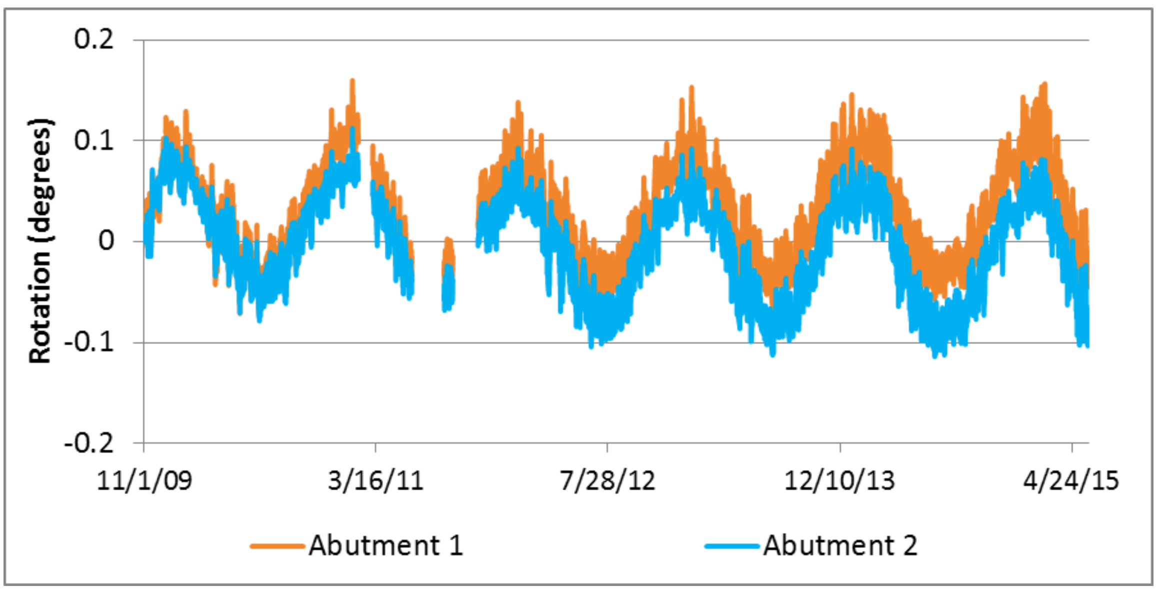


Figure 5.24: Stockbridge Bridge Abutment Rotation

5.7 Earth Pressure

Pressures presented in this section are the changes in pressure since construction due to thermal loading. Earth pressures behind the abutments are presented in Figure 5.25, Figure 5.26 and Figure 5.27 for the Middlesex, East Montpelier, and Stockbridge Bridge, respectively. Additionally, Figure 5.28 shows the wing wall pressures recorded at all three bridges for comparison.

Earth pressures behind the abutments at the Middlesex Bridge are consistent over time. The maximum thermally induced passive earth pressure is minimal with a maximum value of 5.2 psi (35.9 kPa). The earth pressures are comparable across the abutments.

Earth pressures at the skewed East Montpelier Bridge show variation across the abutment and between the two abutments. Passive pressure was greatest behind the obtuse corner at Abutment 1. The maximum thermally induced passive pressure is 13.9 psi (95.8 kPa) at the top pressure cell in the obtuse corner of Abutment 1. The earth pressure distribution of the skewed bridge is addressed in more detail in a later section.

The curved Stockbridge Bridge, with 11.25 degrees of curvature and two spans totaling 222.1 ft (67.7), had extremely minimal earth pressures behind the abutments with a maximum thermally induced passive pressure value of only 2.8 psi (19.3 kPa). Additionally, the pressures were very consistent year to year. The curved bridge did not experience any notable difference in pressure distribution across the abutments. The consistently minimal pressures are likely a result of the Geofoam installed behind the abutment walls. It appears that this material is extremely effective in minimizing earth pressures and is also likely the reason for the consistent response observed under cyclic loading.

The wing wall pressures were the greatest at the Stockbridge Bridge with a maximum thermally induced value of 13.2 psi (91.0 kPa) behind the upstream corner of Abutment 2, while the downstream corner of Abutment 1 had similar maximum values. At the East Montpelier and Middlesex Bridge, maximum wing wall pressure was about 5.0 psi (34.5 kPa).

Positive pressure values should be considered additive to construction pressures, while negative pressures indicate lower total pressure values than those recorded at the completion of construction. While pressures tend to initially increase with an increased displacement, the pressure is less in subsequent seasons when comparable displacements occur. This response shows that soil ratcheting does not occur at any of the three bridges.

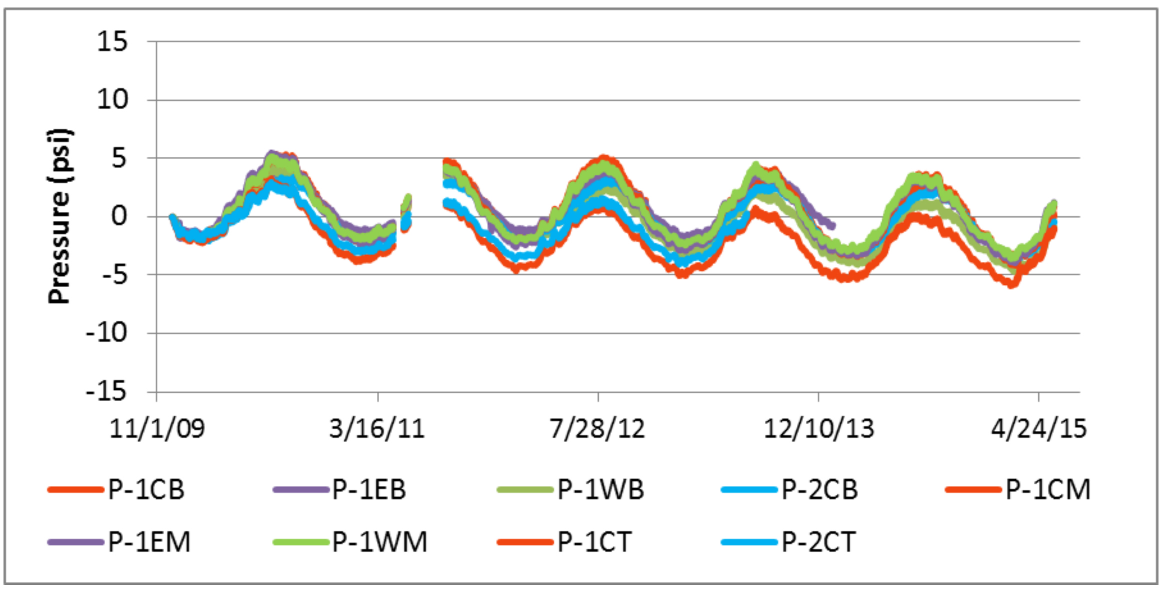


Figure 5.25: Middlesex Bridge Earth Pressures behind Abutments

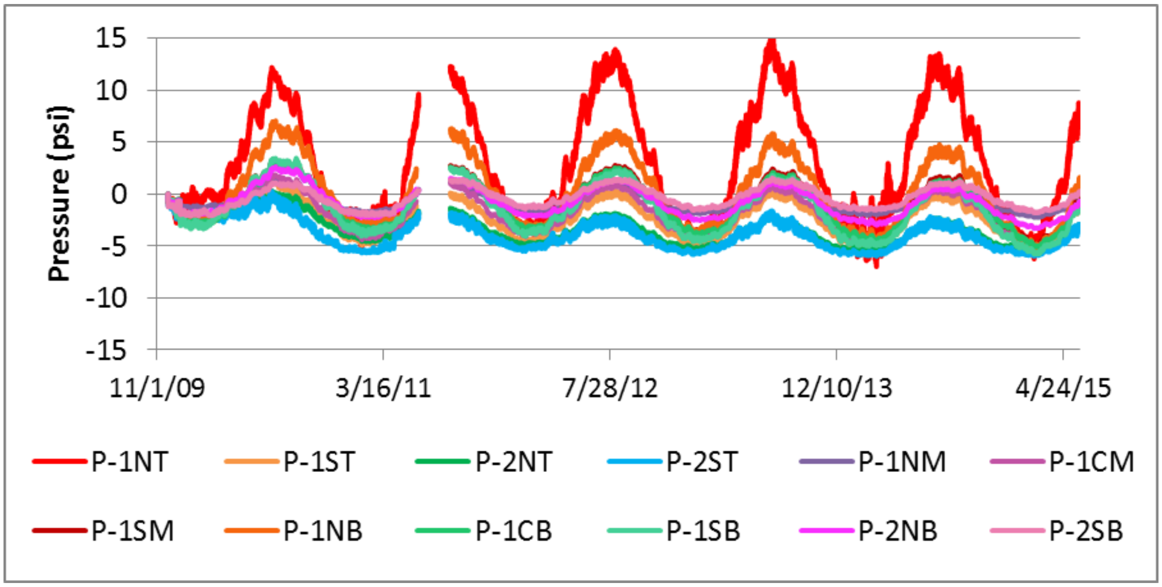


Figure 5.26: East Montpelier Bridge Earth Pressures behind Abutments

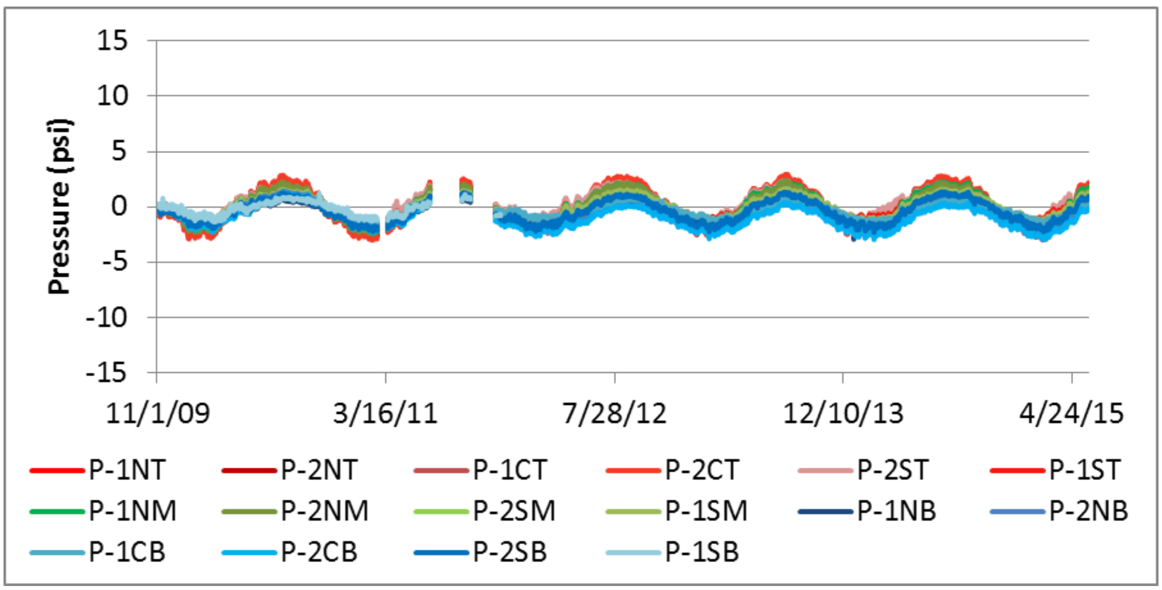


Figure 5.27: Stockbridge Bridge Earth Pressures behind Abutments

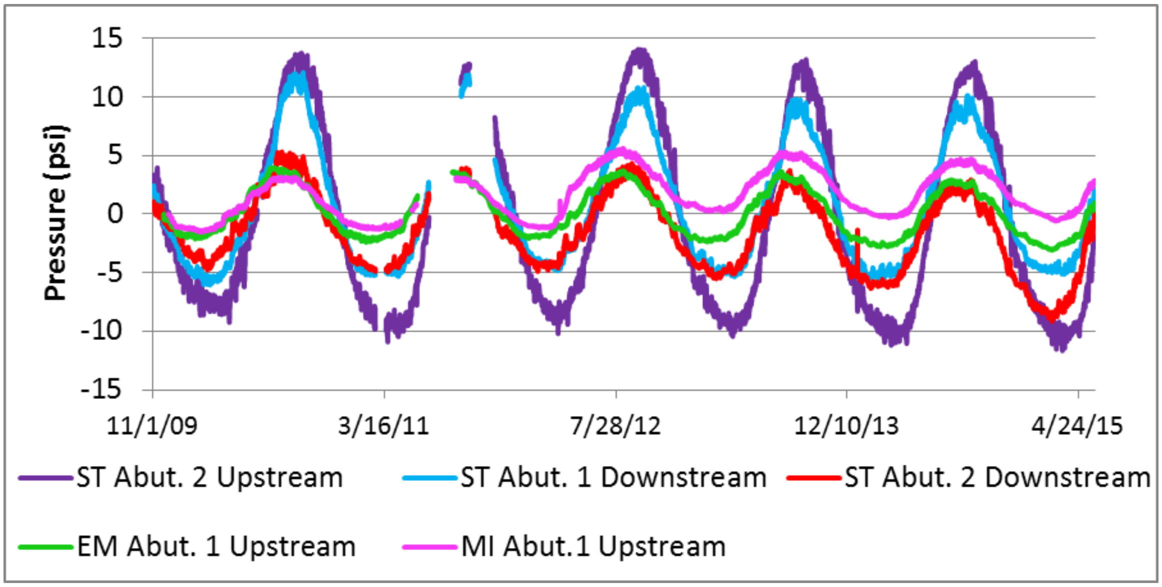


Figure 5.28: Wing Wall Pressures at all three Bridges

5.8 Substructure Displacement at Maximum and Minimum Temperatures

The substructure displacements presented in this section are those resulting from thermal effects only. This section presents the complete substructure displacement on yearly maximum and minimum temperatures recorded at the three bridges, which can be seen in Figure 5.29. Data for all three bridges is shown for Abutment 1 upstream.

At the Middlesex Bridge, the shift in displacement appears to cause a greater change in response to bridge contraction where in later years, once the bridge has stabilized; neither the bottom nor top of abutment contracts to the original zero position. The East Montpelier Bridge shows a lag in response of the pile which does not fully contract in later years, while the top of abutment still contracts past its zero position.

The most consistent response was observed at the Stockbridge Bridge. The substructure displacement on the warmest day in 2011 is the only day in the Stockbridge data that appears different; however this is likely due to the fact that the warmest day of 2011 was not captured by gages since the electrical storm caused power outages. The Stockbridge Bridge also has notably greater displacements under bridge contraction than are seen at the other two bridges. Pile response at this bridge appears to behave more elastically, with no notable lag in pile recovery from its expanded deflected shape and no shift in displacement towards the backfill over time. The consistency in response at the Stockbridge Bridge could be a result of using Geofoam behind the abutments, and could also be a result of the Stockbridge Bridge being a two-span IAB compared to the other two single span IABs.

The substructure displacement does not only vary at a similar temperature year to year, but it also varies depending on when that temperature occurs in a given season. Figure 5.30 shows substructure displacements at cold and warm temperatures from 2010 through 2015; displacements are shown for maximum and minimum temperatures, as well as displacement at a comparable warm and cold temperature at the beginning and end of the season. Once again, the most variable response was observed at the straight Middlesex Bridge which showed significant variation in both abutment and pile displacements depending on when a temperature occurred. The skewed East Montpelier Bridge had less variability than Middlesex, which could be a result of the more gradual shift towards the backfill at this bridge, but the pile response still showed a lag in recovery from its expanded deflected shape resulting in both seasonally and yearly differences in displacement at similar temperatures. At the Stockbridge Bridge, the variation in response was primarily observed at the top of the abutment, while the pile response was far more consistent at comparable temperatures within a season and year to year. The seasonal differences in displacement were more apparent in the first two years after construction, and appear to have stabilized over time.

Substructure displacement does not only depend on temperature but also depends on when the temperature occurs in a season, this would not be predicted in a static thermal analysis.

The single span straight bridge showed the greatest inconsistencies in field response under comparable temperatures both seasonally and annually. The skewed East Montpelier Bridge similarly showed variation in response depending on when a given temperature occurred, and a lag in pile recovery. The most complex of the three bridges, the two-span 222.1 ft (67.6 m) curved Stockbridge Bridge, showed far more consistency at similar warm and cold temperatures both annually and seasonally.

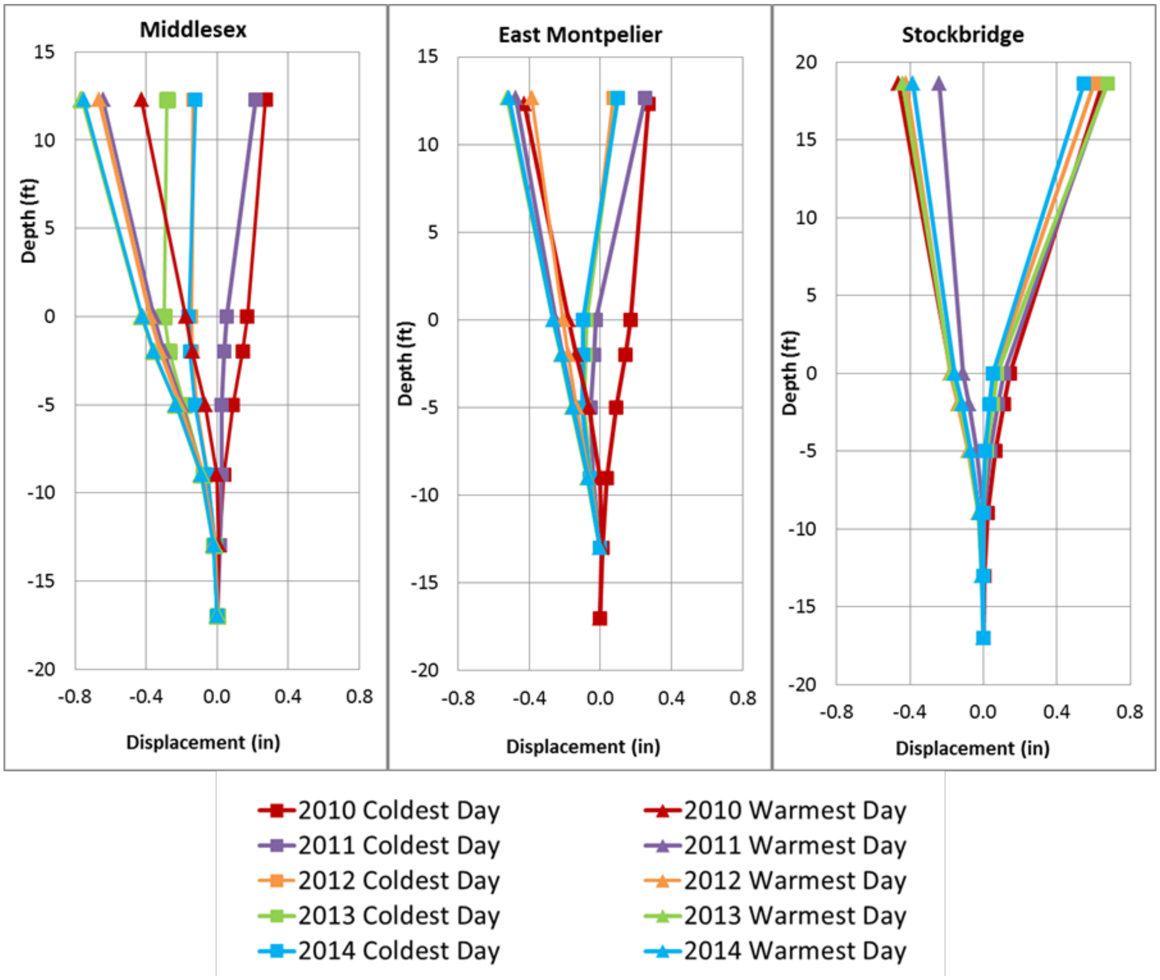


Figure 5.29 Substructure Displacement for Yearly Maximum and Minimum Temperatures (all three bridges)

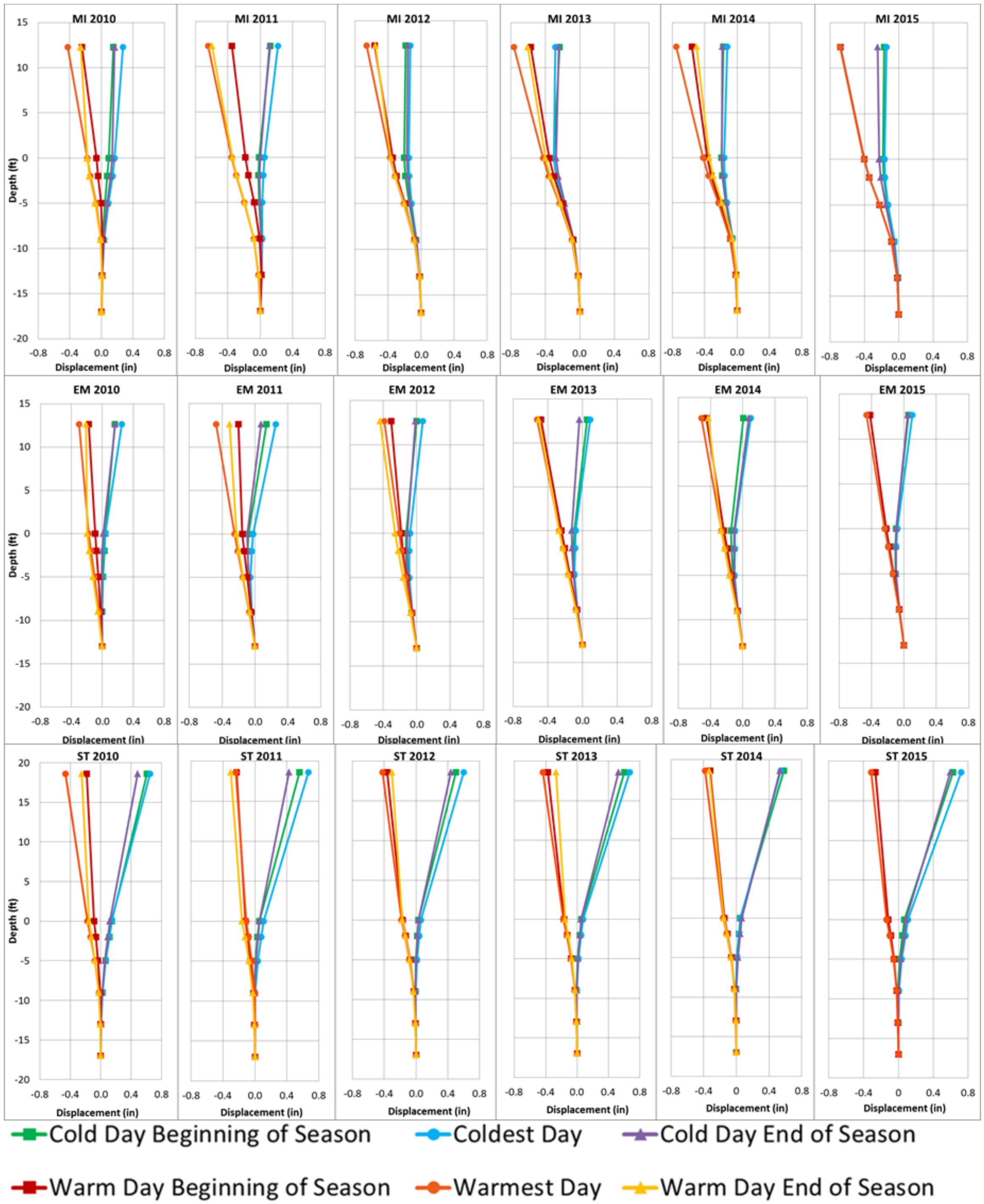


Figure 5.30: Seasonal Substructure Displacement

5.9 Nominal Finite Element Model Results

The long term results presented throughout this chapter have highlighted the complex response of IABs under cyclic thermal loading. The response of these bridges is complicated to predict due to the large number of variables present in the field. The non-linear, time-dependent response of soil-structure interaction, extreme weather conditions, lag in response of the pile, and shifts in displacement over time are all factors that are extremely difficult to predict. The nominal FEMs created in SAP2000 for these three bridges used the initial field conditions (based on soil boring logs) in the calculations for pile soil springs, and assumed dense backfill properties for compacted backfill (except for the nominal FEM of Stockbridge Bridge which does not include backfill soil springs as a result of Geofoam minimizing the effects of backfill). A detailed description of the FEMs and nominal soil modeling was presented in Chapter CHAPTER 4.

While FEMs could be useful in obtaining results that were not captured through instrumentation, it is first necessary to ensure that the FEM results matched the field response that was recorded through instrumentation. This section presents a comparison of displacements predicted via nominal FEMs to displacements recorded in field data to determine the accuracy of the models. Thermal analyses were performed by applying a thermal load to the deck and girders; since field data was set to zero at the start of long term monitoring, the temperature applied in the FEMs was the difference between the reference temperature of the bridge and the corresponding maximum (or minimum) temperature recorded.

Yearly maximum and minimum temperatures were analyzed in the nominal FEMs and the top and bottom of abutment displacements were compared to field data as seen in Figure 5.31 and Figure 5.32 for Middlesex, Figure 5.33 and Figure 5.34 for East Montpelier, and Figure 5.35 and Figure 5.36 for Stockbridge.

For all three bridges, the first year maximum and minimum temperature displacements at the top of the abutment were a reasonable comparison to the values predicted in the FEMs although the FEM did underestimate the top of abutment displacement under bridge contraction at the Stockbridge Bridge. After the initial year, FEM results were not a good match since a static FEM could not predict the shift towards the backfill. At the bottom of the abutment, the FEM results were not a good match for any of the bridges due to the lag in pile response where it does not fully recover from its expanded deflected shape.

To further investigate the accuracy of the nominal FEM, the complete deflected shape under maximum and minimum temperatures for the first two years of monitoring are shown with the field data deflected shapes for 2010 through 2013. These plots can be seen in Figure 5.37 and Figure 5.38 for the three bridges for warm and cold temperatures, respectively. These results are significant as they show that the FEM nominal conditions resulted in much more restraint to expansion than was observed in the field. Under bridge contraction, the FEM overestimates values at the top of pile and abutment for Middlesex and East Montpelier and underestimates contraction for Stockbridge. Since the maximum and minimum temperatures were similar year to year, these plots clearly show that with time the nominal FEM results are an increasingly less accurate representation of field response with the exception of the Stockbridge Bridge which demonstrated more consistent yearly response than the other two bridges.

Overall, these results demonstrated that an FEM based on original “nominal” site conditions, when analyzed at temperatures corresponding to maximum and minimum temperatures in the field, did not accurately predict displacements. Therefore, extracting any results from the nominal FEMs would be an inaccurate representation of forces being realized in the field. The FEM runs each thermal analysis from an existing un-deformed shape and is unable to account for a lag in response from expansion to contraction. Furthermore, the severe weather and flooding in 2011 resulted in shifts in displacement that would not be accounted for. It also appeared that the nominal soil conditions that were assumed in the FEM were not accurate and the soil in the field was providing significantly less restraint to expansion.

In light of these results, steps were taken to calibrate the FEMs to match observed field response. While calibrating an FEM to field data is not a practical approach that could be used in design (since having existing field data is necessary to compare with model results), this process can still be used to provide valuable insight into variations that are not captured in a nominal FEM. Furthermore, by accurately calibrating the model to match the measured field response, other values can be determined through the model for locations that instrumentation does not capture.

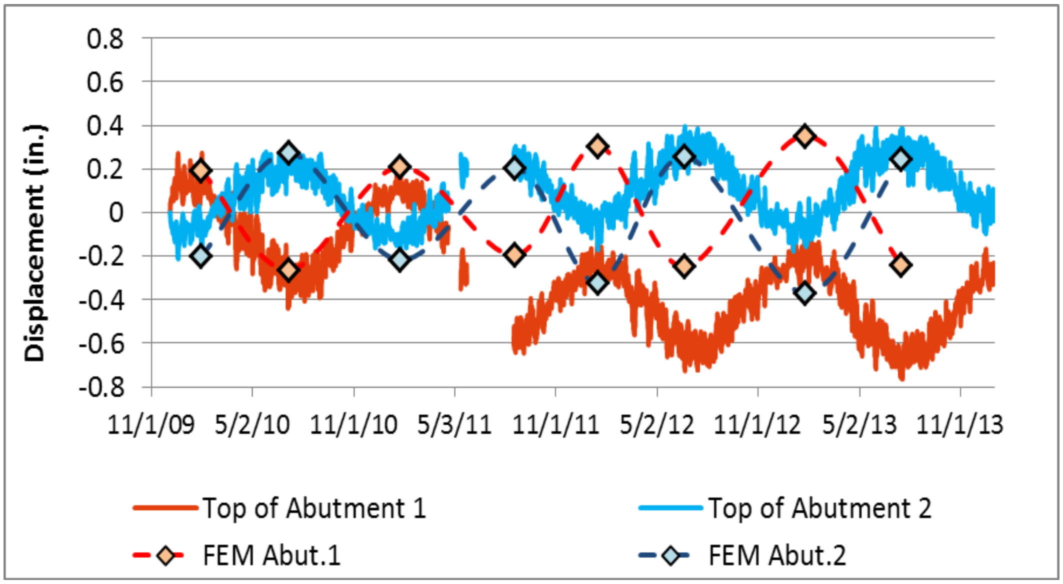


Figure 5.31: Middlesex Top of Abutment Displacement with FEM

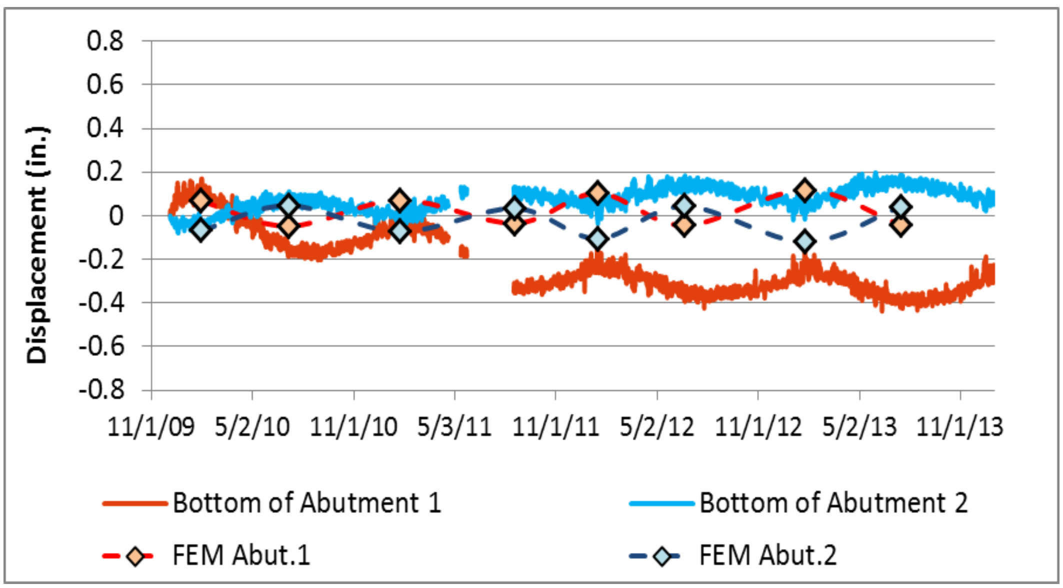


Figure 5.32: Middlesex Bottom of Abutment Displacement with FEM

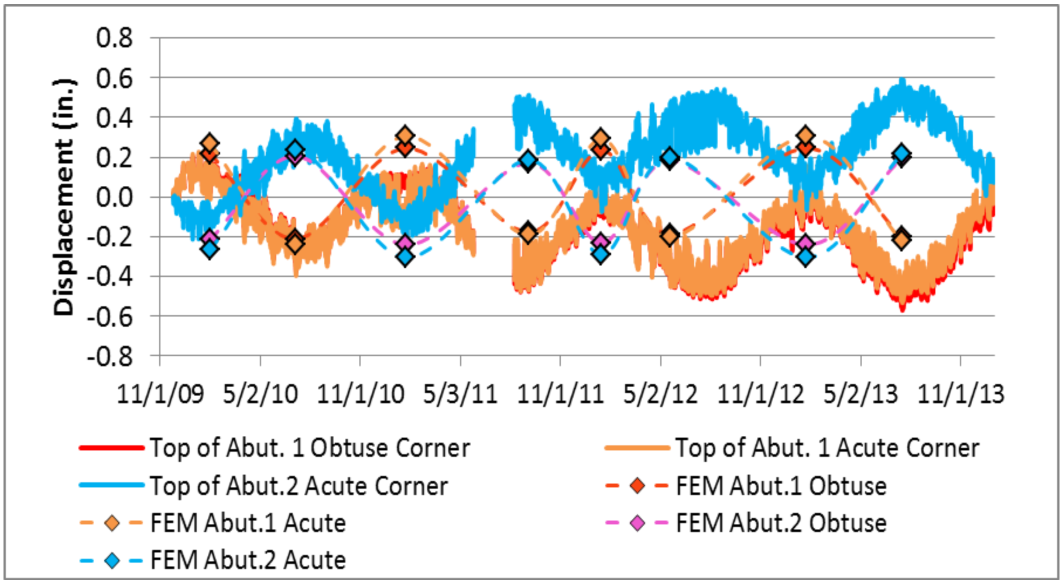


Figure 5.33: East Montpelier Top of Abutment Displacement with FEM

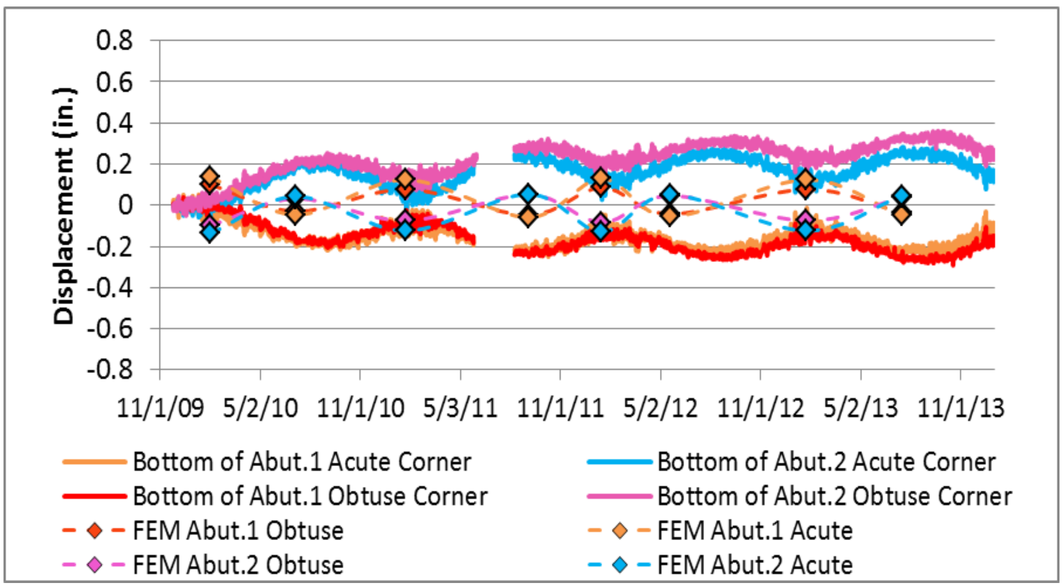


Figure 5.34: East Montpelier Bottom of Abutment Displacement with FEM

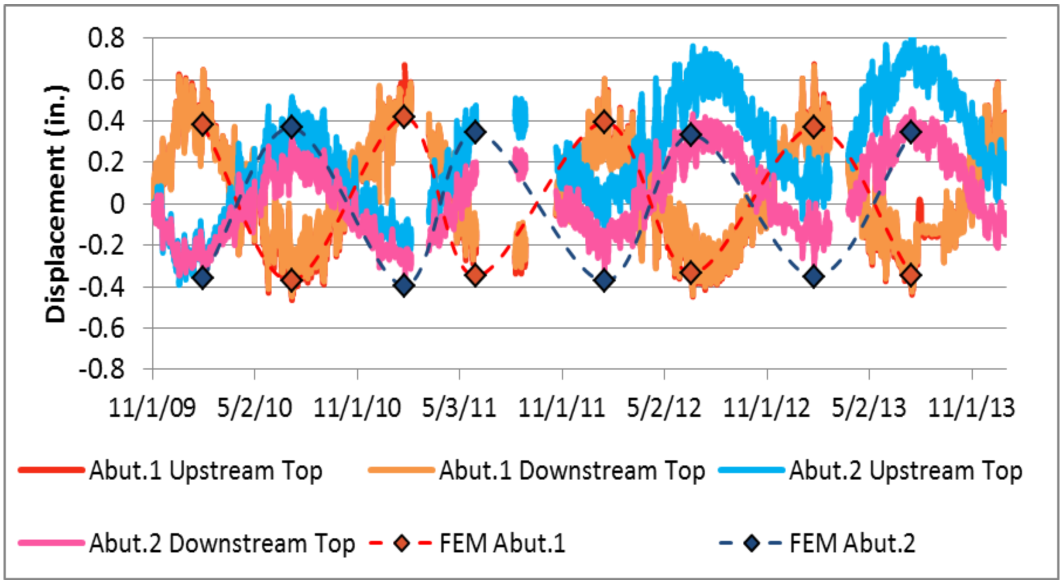


Figure 5.35: Stockbridge Top of Abutment Displacement with FEM

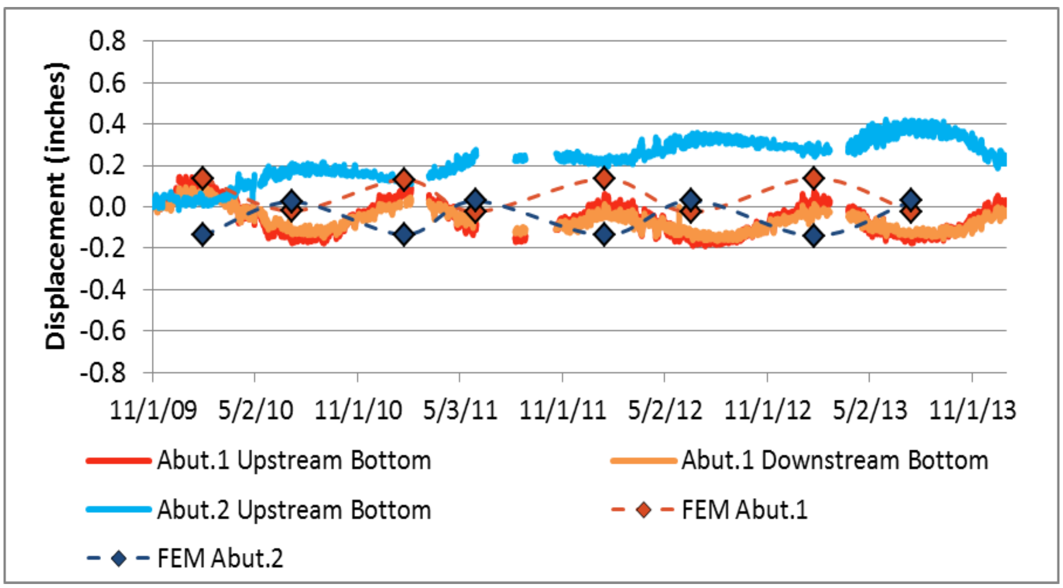


Figure 5.36: Stockbridge Bottom of Abutment Displacement with FEM

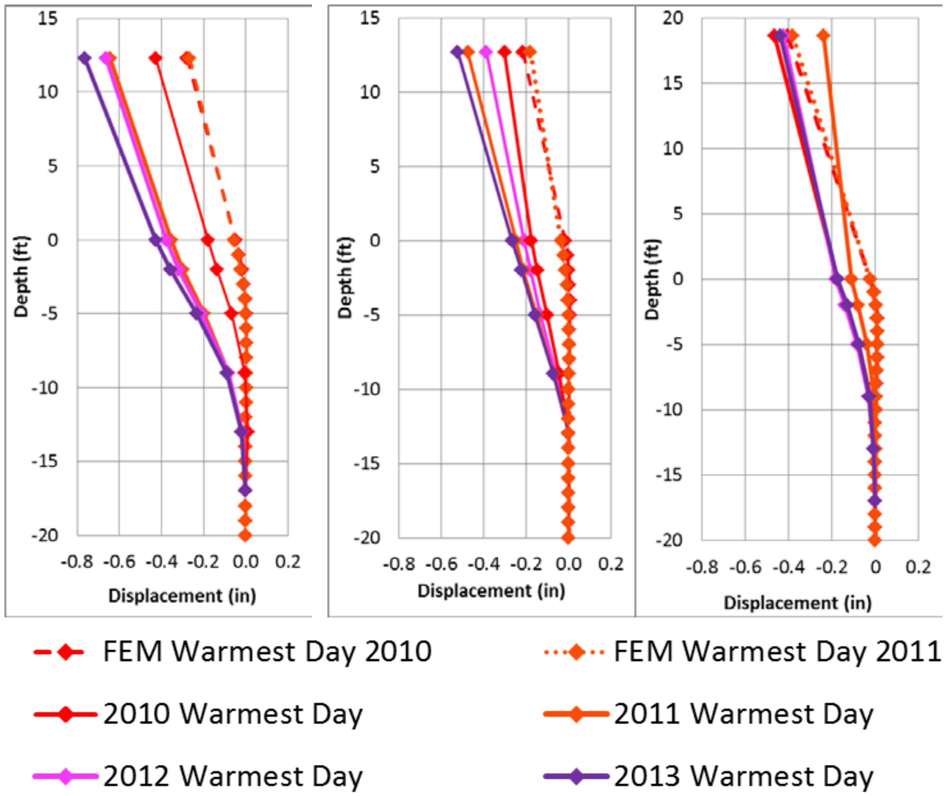


Figure 5.37: Complete Deflected Shape Warmest Day with FEM using Original Soil Conditions Left to Right: Middlesex, East Montpelier, Stockbridge

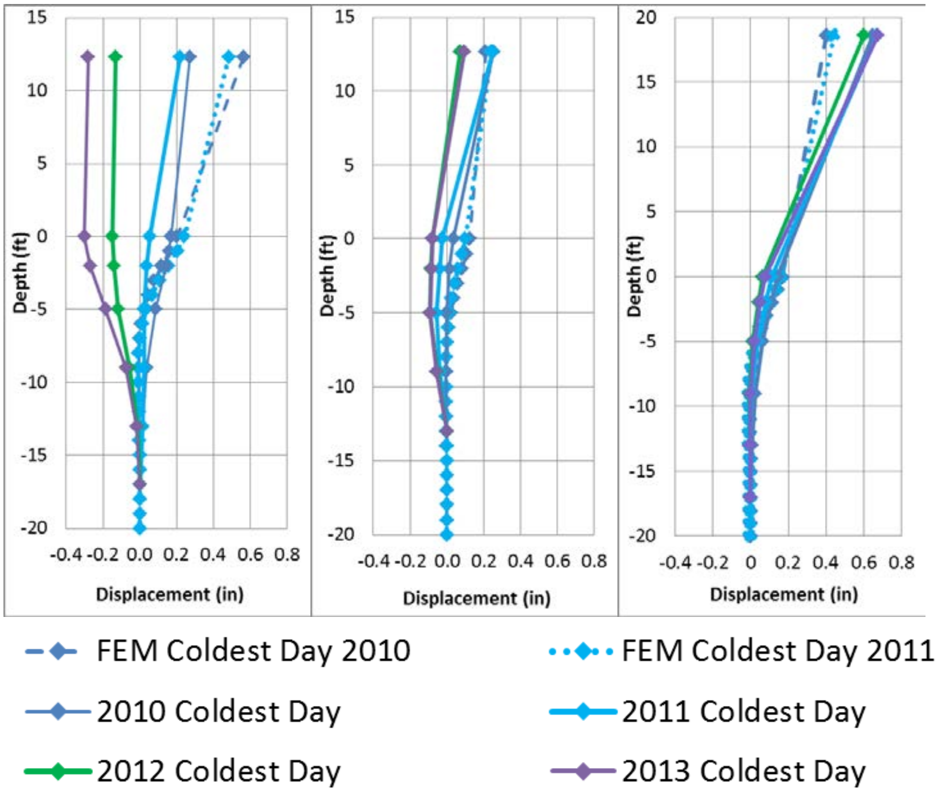


Figure 5.38 Complete Deflected Shape Warmest Day with FEM using Original Soil Conditions Left to Right: Middlesex, East Montpelier, Stockbridge

5.10 Matched Finite Element Models

The first step to matching FEM results to field displacements was to determine how the complete deflected shape would compare to field data if the FEM results matched the top of abutment displacement. As the nominal FEM results showed, when the maximum temperature from the field was applied as a thermal load in the FEM, the displacement was underestimated. Therefore, the thermal load applied to the FEMs was adjusted to match the displacement at the top of the abutment. Essentially the ratio of the displacement to thermal load in the nominal FEM was used to determine the thermal load necessary to match the field displacement. This process was done for the warmest day of the first year of monitoring and compared to the corresponding complete deflected shape from field data to determine how well the substructure displacements matched once the top of abutment displacement matched. The results of matching top of abutment displacement for all three bridges are shown in Figure 5.39 and Figure 5.40 for the warmest and coldest day of the year in 2010, respectively. As previous sections have shown, the Middlesex and East Montpelier Bridge displacements change significantly in later years. The results of matching displacement at the top of the abutment worked the best for the Stockbridge Bridge; however the results at the Middlesex and East Montpelier Bridge still show significant differences in pile deflected shape when the top of abutment displacement is matched. Therefore, further steps had to be taken to calibrate the FEM results to match field data.

The overall trend in Middlesex and East Montpelier plots shows that the nominal FEM was providing far more soil restraint than what was actually being mobilized in the field. Recall that for the Stockbridge Bridge, Geofoam was installed behind the abutments to minimize earth pressures and as a result the nominal FEM for this bridge neglected backfill soil. The Middlesex and East Montpelier Bridge both had dense backfill soil modeled, as well as medium dense soil around the piles. Therefore, in order to match the displacement observed at these bridges the soil spring calculations were revised by using different friction angles and density values. This process was done iteratively. By refining the soil spring properties and adjusting thermal loads to match displacement at the top of the abutment “Matched” FEMs were created for the three bridges.

The “Matched” FEM results are shown with corresponding field data for the warmest and coldest day of the year for 2010 through 2015; Figure 5.41 and Figure 5.42 for the Middlesex Bridge, Figure 5.43 and Figure 5.44 for the East Montpelier Bridge, and Figure 5.45 and Figure 5.46 for the Stockbridge Bridge. A table of the FEM Matched soil conditions, and corresponding soil friction angle and density used in modeling the soil springs, are shown in Table 5.1 and Table 5.2.

For the Middlesex and East Montpelier Bridge, the backfill soil that matched field response under warm temperatures was a loose backfill, and the soil around the pile was significantly looser as well. At the Middlesex Bridge, the soil around the top 13.0 ft (4.0 m) of the pile ended up being very loose, with loose soil the remainder of the depth. At East Montpelier, in order to match the field response, there were no soil springs modeled in the first 7.0 ft (2.1 m) of the pile, with loose soil below. At the Stockbridge Bridge, the nominal model did not include backfill since the Geofoam was assumed to minimize the backfill pressures.

The Stockbridge Bridge had “matched” soil conditions that differed from the Middlesex and East Montpelier Bridges; where the Stockbridge Bridge had very loose soil around the top 10 ft (3.0 m) of the pile in the first year of matching, and loose soil for the next four years which shows the soil provided some more resistance to expansion in later years unlike the other two bridges.

The FEM Matched conditions for cold temperatures had some key differences and had more differences between the bridges. The main difference in modeling Matched soil conditions was seen in the Middlesex Bridge. This bridge had unique results because of the highly concentrated offset in displacement towards the backfill. In later years, the bridge never fully contracted; therefore in order to “match” the deflected shape in the field, a temperature increase had to be applied to the bridge with no backfill soil and no soil in the top 8.0 ft (2.4 m) of the pile. This result was only seen at the Middlesex Bridge. For the East Montpelier Bridge, pile soil was dense in the first year, and in the next four year the soil was very dense but had to be modeled nearly rigid at the top of the pile in order to simulate the restraint that was observed in the field. The soil properties for the nearly rigid condition were also used at the Stockbridge Bridge, where similarly the top 1.0 ft (0.30 m) of the pile had a very significant amount of restraint to contraction. The soil friction angle and density for the rigid condition are not presented because they are not realistic of an actual type of soil. The fact that “matching” at the two bridges required this rigid modeling condition indicates potentially freezing soil conditions at the top of the pile.

Considering Matched FEM conditions for expansion, very loose backfill and essentially no soil restraint around the top 10 ft (3.0 m) of the piles was the trend of the matched soil conditions at the three bridges. While each of the bridges required slightly different changes in modeling the soil to match field response “perfectly”, the overall trend is that the soil around the top portion of the pile provides minimal resistance upon cyclic thermal loading. This could be due to a gap forming around the pile where the soil does not fill back in, or due to other time-dependent factors of soil response under cyclic thermal loading. The behavior of the soil merits further investigations to determine what causes the change in response compared to what would be expected based on soil boring logs.

For bridge contraction, the case of the Middlesex Bridge is unique since there was a large offset in displacement therefore modeling bridge contraction at this abutment actually required applying a temperature increase since bridge remained in an expanded condition (see plots for clarification). For the East Montpelier Bridge and Stockbridge Bridge, results for matched soil conditions were both required the soil around the top of the pile to be modeled as nearly rigid to match the contracted shape. This is not an actual soil "type" but rather could be an indication that soil is freezing and restricting contraction at the top of the pile.

This part of the research project substantiates the importance of field data as the field response of the three bridges differed from the response that was predicted in the nominal FEM. These results highlighted how soil response changes under cyclic thermal loading, and that thermal analyses with corresponding temperatures to field data did not match displacements due to the shift in displacement towards the backfill that occurred in the field. The field data provided the information needed to calibrate FEMs and gain a better understanding of how soil response changed over time. The use of field data and FEMs complement each other since the field data is needed to validate the FEM results. By analyzing how changing the soil properties in the FEM matches field response, the FEM gives insight into the behavior of the soil. Once the soil was matched, by the second year the Matched soil conditions remained constant and matched field response for all subsequent years. This provides validation of the accuracy of the matched soil condition, and also shows that bridge response has stabilized by the end of the second year. The matched FEMs can be used to extrapolate bridge response not captured by instrumentation in the field.

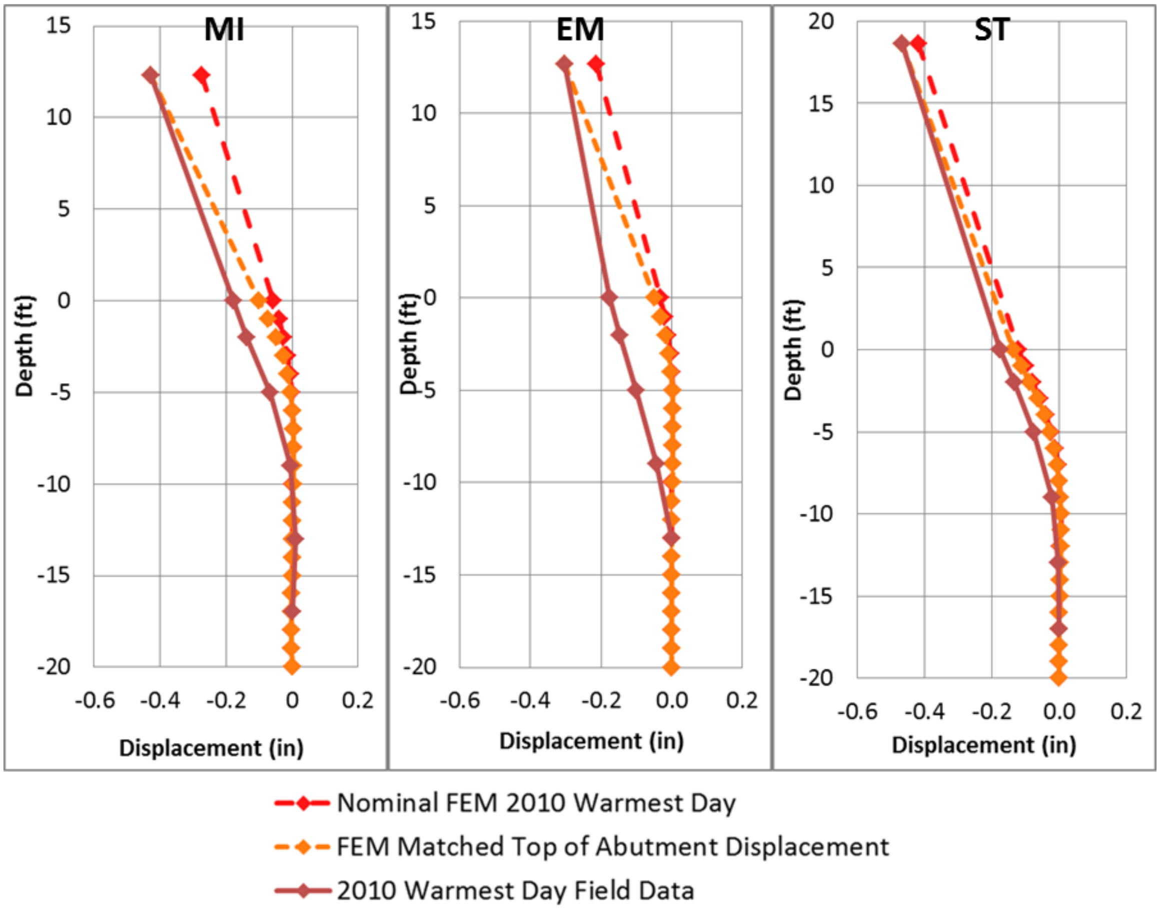


Figure 5.39: FEM Warmest Day 2010- Matched Top of Abutment Displacement in FEMs

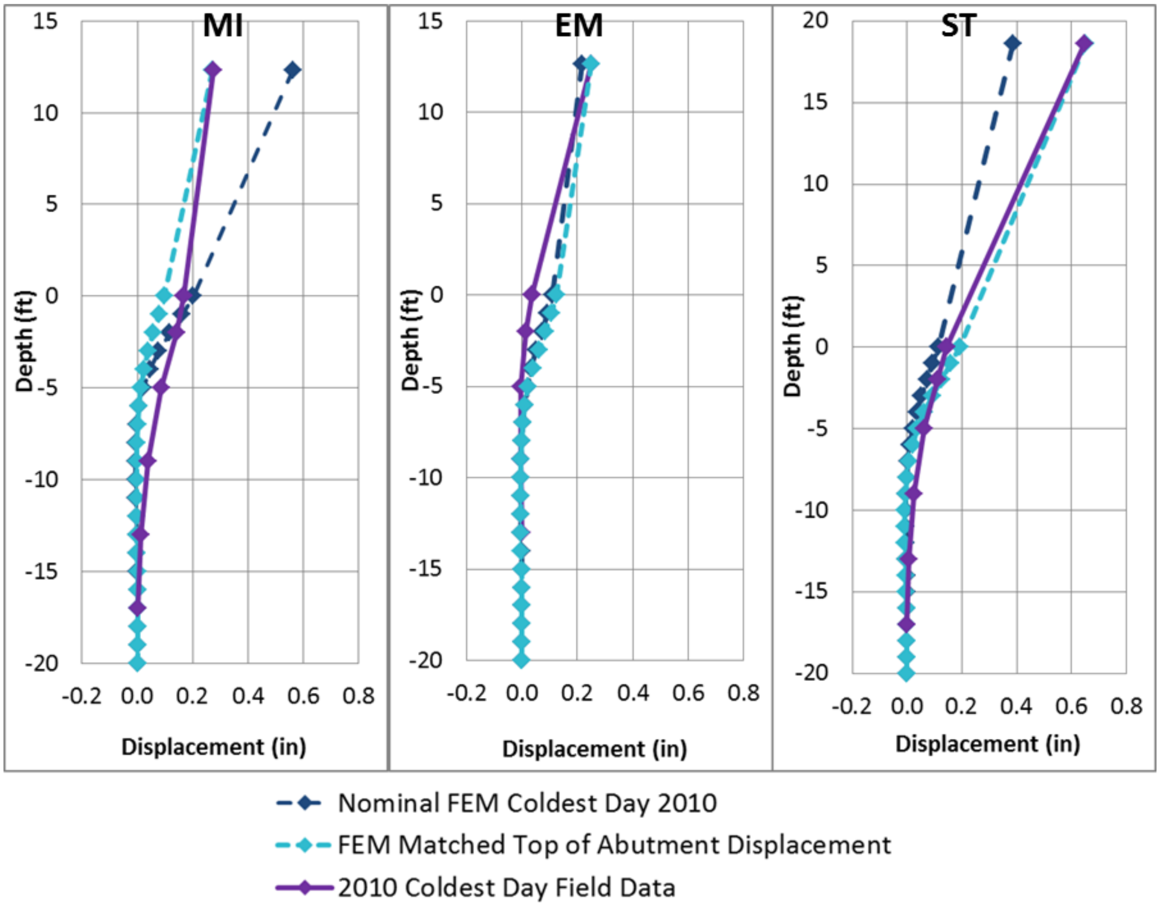


Figure 5.40: FEM Coldest Day 2010- Matched Top of Abutment Displacement in FEMs

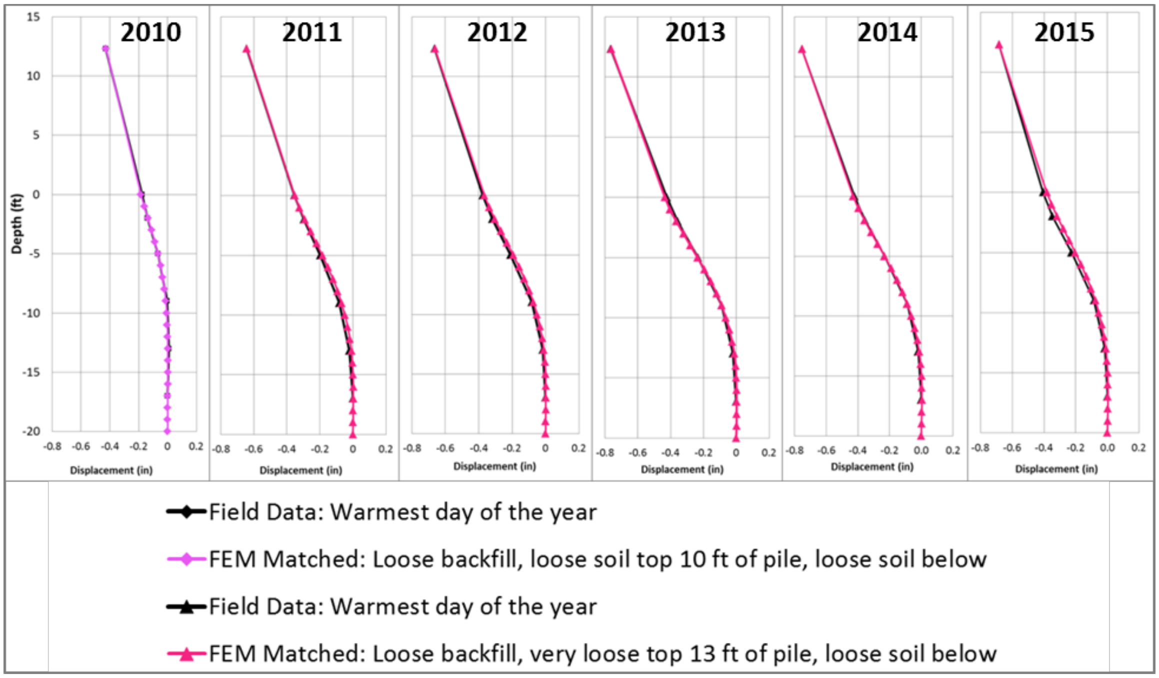


Figure 5.41: Middlesex Bridge FEM Matched Warmest Day of the Year 2010-2015

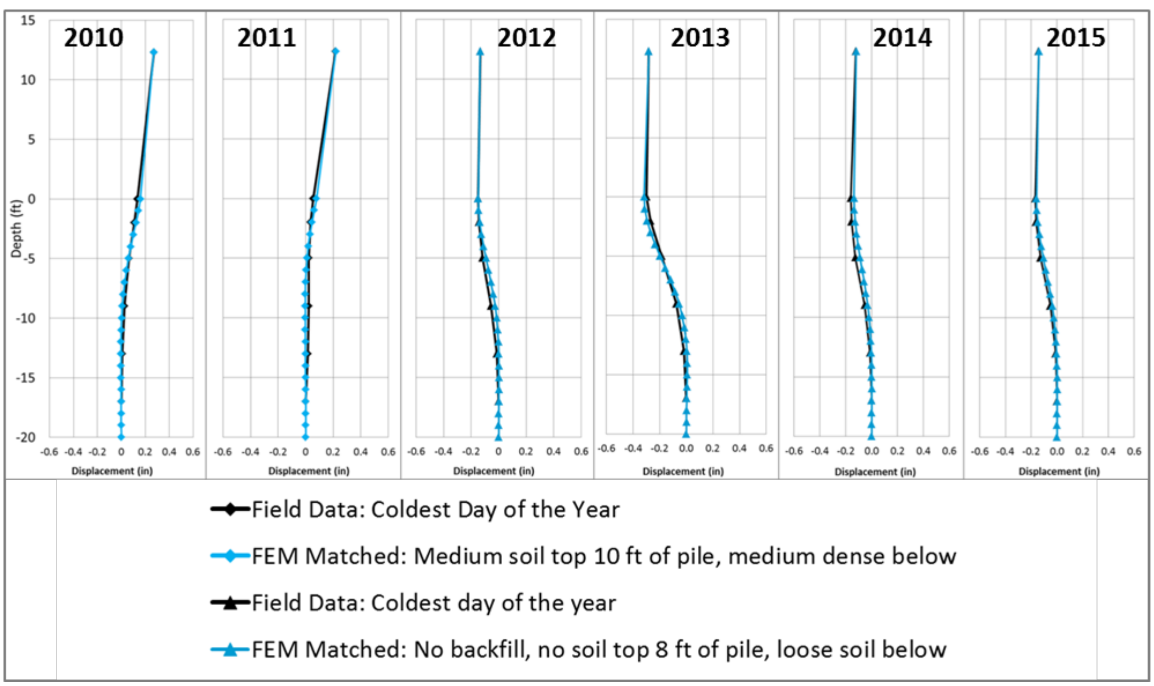


Figure 5.42: Middlesex Bridge FEM Matched Coldest Day of the Year 2010-2015

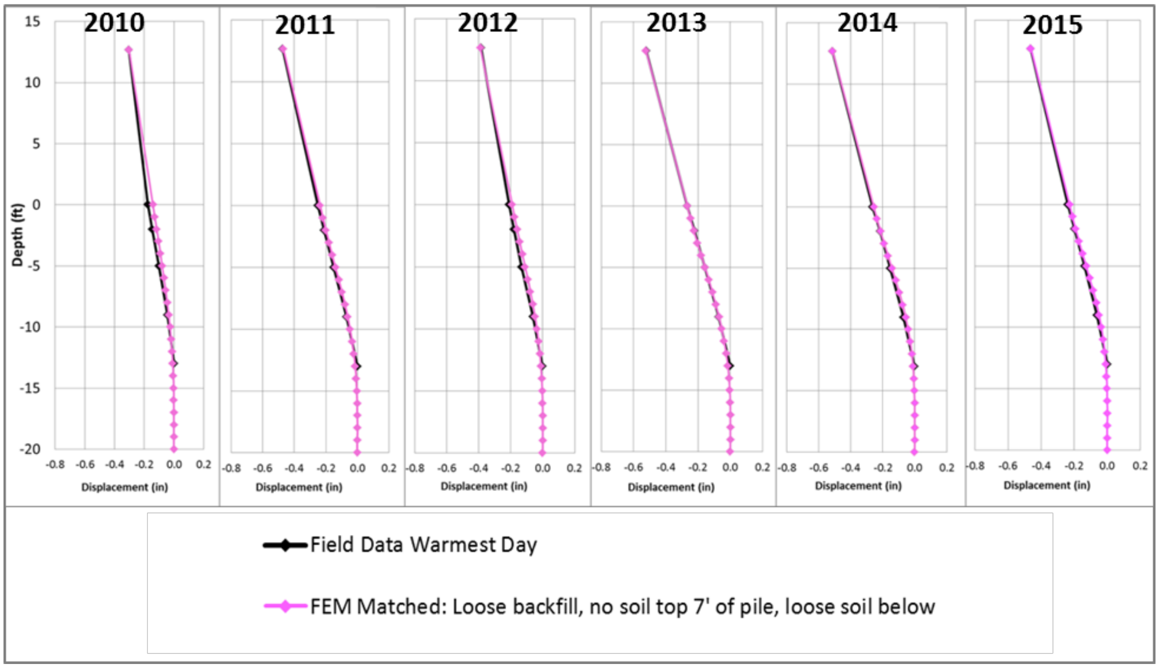


Figure 5.43: East Montpelier Bridge FEM Matched Warmest Day of the Year 2010-2015

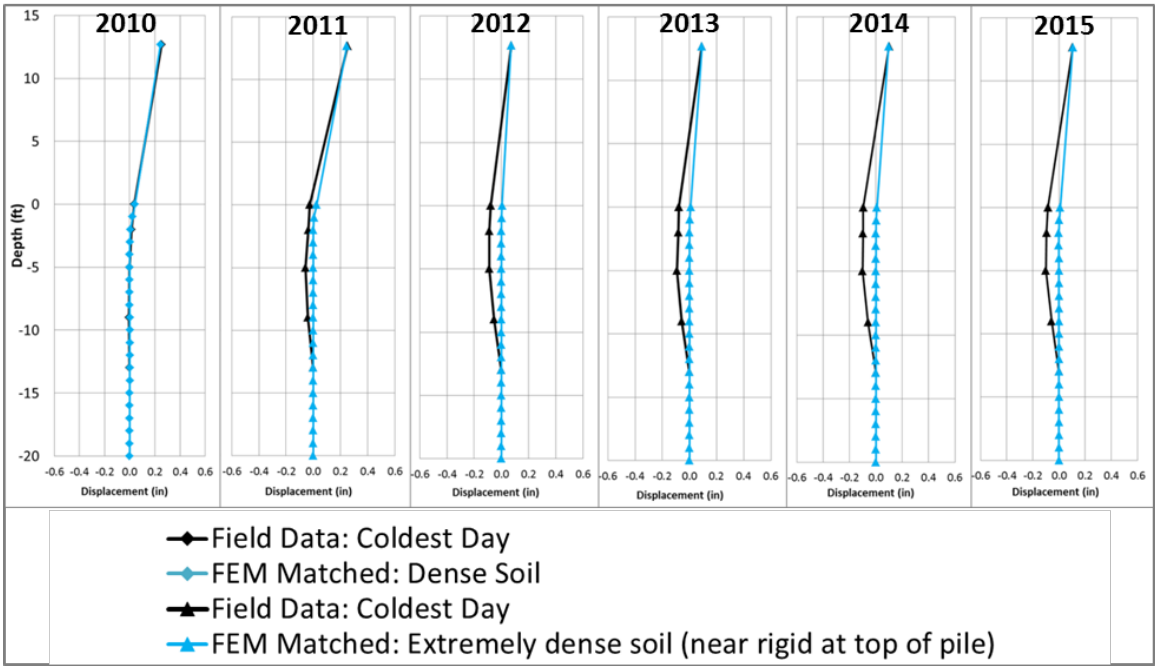


Figure 5.44: East Montpelier Bridge FEM Matched Warmest Day of the Year 2010-2015

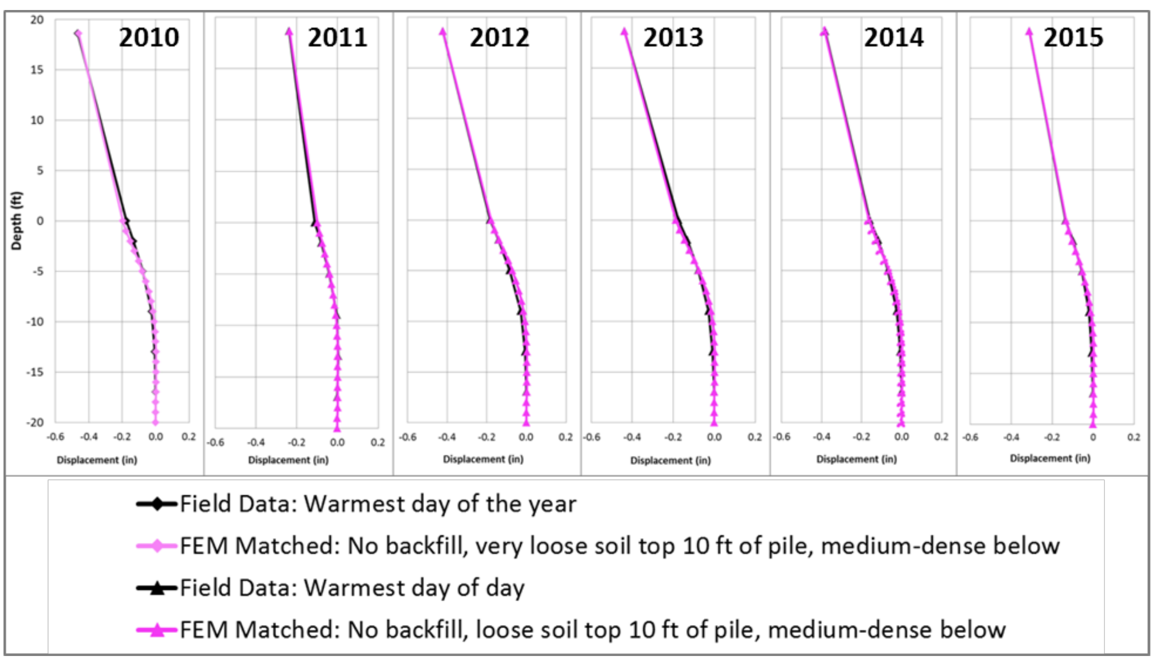


Figure 5.45: Stockbridge Bridge FEM Matched Warmest Day of the Year 2010-2015

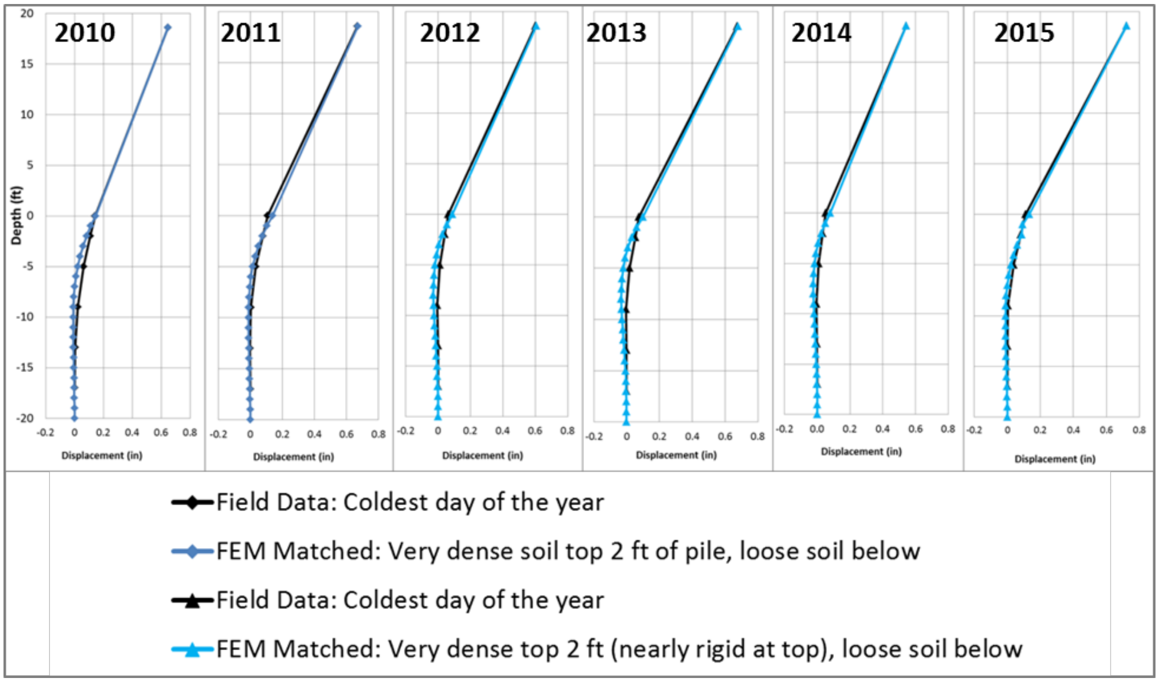


Figure 5.46: Stockbridge Bridge FEM Matched Coldest Day of the Year 2010-2015

Table 5.1: Soil Properties in FEM Matched Models Warm Temperatures

Bridge	Year	Warmest Day of the Year Matched FEM Soil Conditions	Soil Friction Angle, φ and Density, g lb/ft³ (kg/m³)
Middlesex	2010	Loose Backfill, very loose soil top 10 ft (3.1m) of pile, loose soil below	Loose Backfill: φ=20°, g=85 (1361) Very Loose Soil: φ=18°, g=95 (1522) Loose Soil: φ=25°, g=110 (1762)
	2011-2015	Loose backfill, very loose soil top 13 ft (4m) of pile, loose below	Loose Backfill: φ=20°, g=85 (1361) Very Loose Soil: φ=18°, g=95 (1522) Loose Soil: φ=25°, g=110 (1762)
East Montpelier	2010-2015	Loose backfill, no soil top 7 ft (2.1m) of pile, loose soil below	Loose Backfill: φ=20°, g=85 (1361) Loose Soil: φ=18°, g=95 (1522)
Stockbridge	2010	No backfill, very loose soil top 10 ft (3.1m) of pile, medium dense soil below	Very Loose Soil: φ=18°, g=95 (1522) Med.-Dense Soil: φ=35°, g=135 (2162)
	2011-2015	No backfill, loose soil top 10 ft (3.1m) of pile, medium dense below	Loose Soil: φ=25°, g=110 (1762) Med.-Dense Soil: φ=35°, g=135 (2162)

Table 5.2: Soil Properties for FEM Matched Cold Temperatures

Bridge	Year	Coldest Day of the Year Matched FEM Soil Conditions	Soil Friction Angle, φ and Density, g lb/ft³ (kg/m³)
Middlesex	2010-2011	Medium soil top 10 ft (3.1m) of pile, medium-dense soil below	Medium Soil: φ=30°, g=130 (2082) Med.-Dense Soil: φ=35°, g=135 (2162)
	2012-2015	No backfill, no soil top 8 ft (2.4m) of pile, loose soil below	Loose Soil: φ=25°, g=110 (1762)
East Montpelier	2010	Dense soil	Dense Soil: φ=40°, g=140 (2243)
	2011-2015	Extremely dense soil (near rigid at top)	Very Dense Soil: φ=45°, g=150 (2403)

Stockbridge	2010-2011	Very dense soil top 2 ft (0.6m) of pile, loose soil below	Very Dense: φ=45°, g=150 (2403) Loose Soil: φ=25°, g=110 (1762)
	2012-2015	Very dense top 2 ft (0.6m) (with top 1 ft (0.3m) nearly rigid), loose soil below	Very Dense: φ=45°, g=150 (2403) Loose Soil: φ=25°, g=110 (1762)

5.11 Pile Bending Moments

The piles at the three bridges were instrumented with strain gages. However, upon studying the Matched FEMs it was determined that the pile bending moment calculated from field data may not be an accurate representation of the maximum pile bending moment in the field. It was selected to place gages away from the geometry change at the top of pile and instrument a location of expected linear strain distribution due to St. Venant's Principle. Therefore, the strain gages were placed about 1.6 ft (0.49 m) from the top of the pile. In examining the shape of the bending moment diagram from the FEMs, it was determined that the point of inflection of the pile could be quite close to the location of the top strain gage, therefore the moment calculated from the strain gage could potentially be much lower than the actual maximum moment in the pile. Figure 5.47 shows an example of the location of the point of inflection of the pile compared to the gage location for both Nominal FEM and Matched FEM results from 2013. The results clearly show that the gage location is not at the point of maximum moment, and furthermore demonstrates how the bending moment diagram changes based on the soil conditions. The Matched FEM results in this figure show that with minimal backfill restraint and very loose soil conditions, the pile undergoes double curvature under expansion whereas with dense backfill and medium-dense soil the pile undergoes single curvature. Since the field data results are likely not an accurate representation of maximum pile moments, the Matched FEMs were used to determine maximum pile bending moments at each of the three bridges.

The results of the maximum pile bending moments from Matched FEMs show that, considering moments about the weak axis, the maximum bending moment is similar between the straight Middlesex Bridge, skewed East Montpelier Bridge and curved two-span Stockbridge Bridge with absolute maximum values of46.6 kip-ft (63.1 kN-m), 68.0 kip-ft (92.1 kN-m) and 67.8 kip-ft (92.0 kN-m) at the three bridges, respectively.

Matched FEM results for strong axis bending moments showed that bending about the transverse axis resulted in minimal strong axis bending moments at the straight Middlesex Bridge as well as the curved Stockbridge Bridge. The skewed East Montpelier Bridge had significant transverse bending moments in the piles, which were greater than or equal to the weak axis bending moments at the bridge and also greater than bending moments in the other two bridges with a maximum strong axis bending moment of 68.0 kip-ft (92.2 kN-m).

(a) (b)

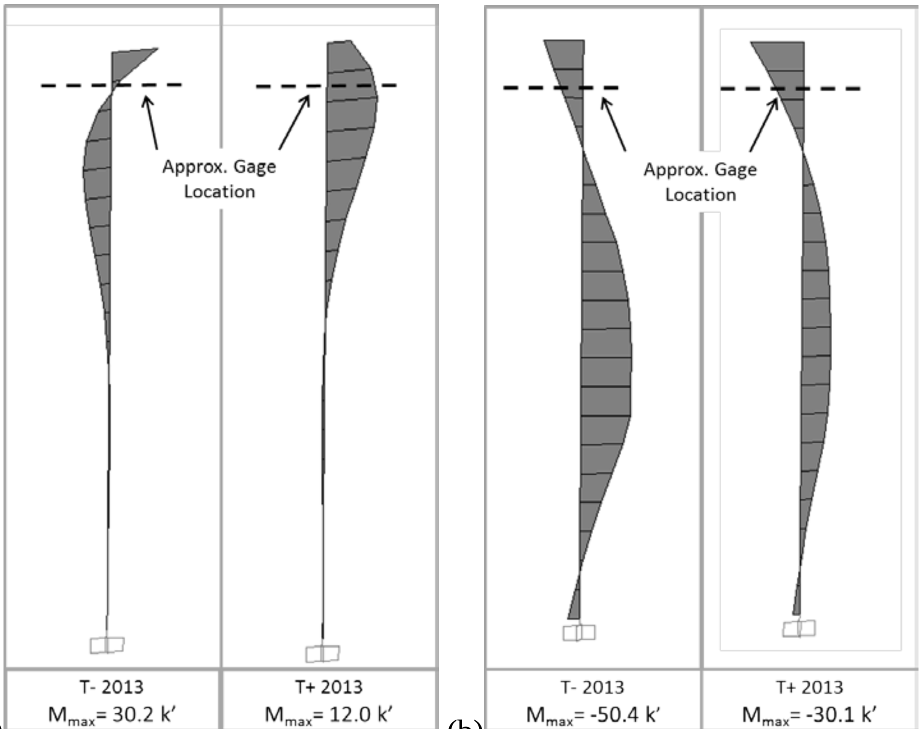


Figure 5.47: Middlesex Pile Bending Moment Diagram for 2013 Coldest and Warmest Day from Nominal FEM (a) and Matched FEM (b)

Table 5.3: Maximum Weak Axis Pile Bending Moments from Matched FEMs

Bridge	Matched FEM Results for Maximum Pile Weak Axis Bending Moment kip-ft (kN-m)	
	Bridge Expansion	Bridge Contraction
Middlesex*	30.1 (40.8)	46.6 (63.1)
East Montpelier*	35.5 (48.1)	-68.0 (-92.1)
Stockbridge**	51.8 (70.4)	-67.8 (-92.0)

*M_{yy}=144.2 kip-ft, M_{py}=221.7 kip-ft (HP12x84) weak axis yield moment and plastic moment, respectively

**M_{yy}=247.9 kip-ft, M_{py}=380.8 kip-ft (HP14x117) weak axis yield moment and plastic moment, respectively

Table 5.4: Maximum Strong Axis Pile Bending Moments from Matched FEMs

Bridge	Matched FEM Results for Maximum Pile Strong Axis Bending Moment kip-ft (kN-m)	
	Bridge Expansion	Bridge Contraction
Middlesex*	5.2 (7.0)	6.2 (8.3)
East Montpelier*	68.0 (92.2)	-43.2 (-58.6)
Stockbridge**	-14.2 (19.3)	12.5 (17.0)

*M_{yy}=441.7 kip-ft, M_{py}=500.0 kip-ft (HP12x84) strong axis yield moment and plastic moment, respectively

** M_{yy}=716.7 kip-ft, M_{py}=808.3 kip-ft (HP14x117) strong axis yield moment and plastic moment, respectively

5.12 Detailed Comparison of the Middlesex and East Montpelier Bridges

This following sections focus on the response of the Middlesex and East Montpelier Bridge to highlight similarities and differences between the straight and skewed bridges, give greater insight into their long term response, and determine any significant differences in the skewed bridge response that would not be accounted for if assuming a straight bridge design.

5.12.1 Abutment Top Longitudinal Displacement

Top of abutment displacement refers to the displacement calculated at the top of the girder. Data is presented for both bridges in Figure 5.48. Abutment displacement is often estimated using the equation of thermal expansion shown in Equation 5.1.

$$\delta = \alpha \times \Delta T \times L \qquad \textbf{Equation 5.1}$$

Where δ=total displacement (mm) (in), a= coefficient of thermal expansion (6.5*10-6/°F) (11.7*10-6/°C), DT=change in temperature from maximum to minimum yearly value (°F) (°C), and L= bridge length (in) (mm). This equation was used to calculate expected displacement values at the two bridges using the difference from the peak cold to peak warm temperature (each year) as the change in temperature in Equation 4.1. For field data, the difference in displacement from the peak cold to peak warm temperature was calculated for each abutment, and the sum of these displacements (net displacement of the bridge) is compared to the values predicted using Equation 4.1; this information can be seen in Table 5.5.

For both bridges, a negative displacement at Abutment 1 indicates movement towards backfill (expansion) and a positive value indicates movement towards river (contraction) while Abutment 2 sign convention is opposite. The data presented for displacements are only due to thermal fluctuations. For Middlesex, data is shown for the upstream side of both abutments. East Montpelier only has data for the acute corner at Abutment 2, while both obtuse and acute corner data are presented for Abutment 1. On each of the plots, markers are used to highlight dates seasonal peak warm and cold temperatures.

The Middlesex Bridge experienced a shift in displacement towards the backfill that was concentrated at Abutment 1. To verify this data, the displacement values were compared from crackmeters, tiltmeters and inclinometers. Tiltmeters and displacement transducers attached to the abutment wall recorded the abutment displacement and rotation whereas inclinometers installed on the piles registered the pile incremental rotations. The displacements along the substructure were calculated by combining the two sets of gage readings. The displacements presented in this section are based on the pile inclinometers and the tiltmeters. To confirm the shift in displacement, calculations from the crackmeter and tiltmeter were used and verified that the shift was recorded by all gages.

Similar to the Middlesex Bridge, the East Montpelier Bridge also displayed an increase in displacement towards the backfill; however this shift was more symmetrical between the two abutments than the straight bridge. The skew of the East Montpelier

Bridge did not affect longitudinal displacements; the displacement at the obtuse and acute corner was almost identical.

The maximum net displacement (sum of both abutments) at the top of the abutments was 1.32 in (33.5 mm) and 1.28 in (32.5 mm) at the Middlesex and East Montpelier Bridge, respectively. The maximum difference between the thermal expansion equation and the net displacement from field data was 0.21 in (5.3 mm) and 0.25 in (6.35 mm) at the Middlesex and East Montpelier Bridge, respectively. The equation of thermal expansion was not always a good prediction of the net abutment displacement, which is likely a result of the increasing displacement towards the backfill. In typical designs, abutment rotation is neglected and the top of abutment displacement is calculated using Equation 4.1, with the displacement equally split between the two abutments and assigned to the top of the pile for pile design.

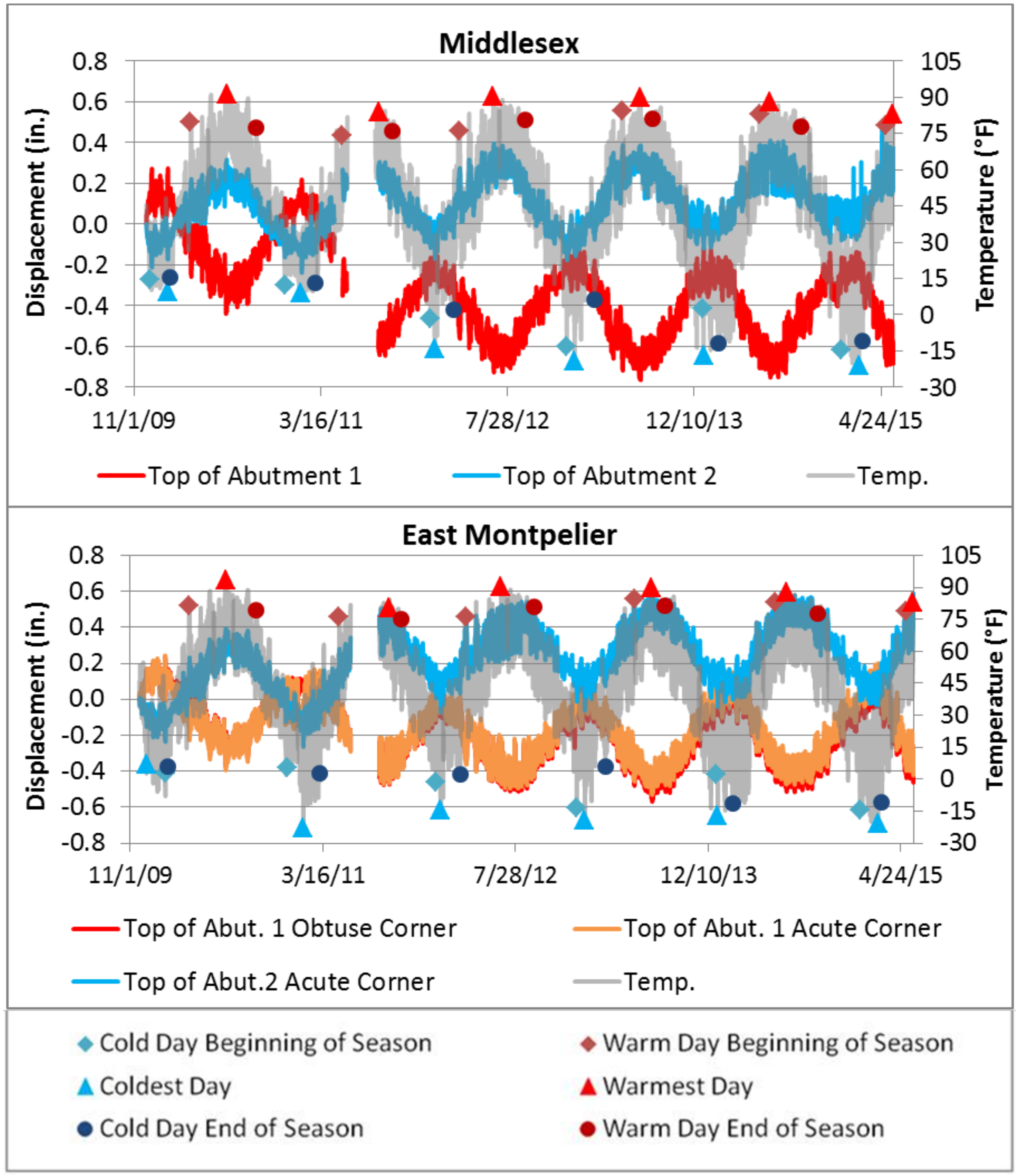


Figure 5.48: Longitudinal Top of Abutment Displacement at the Middlesex (top) and East Montpelier Bridge (bottom)

Table 5.5: Comparison of Abutment Displacement to Thermal Expansion Equation Prediction

Year	Peak to Peak Temperature °F (°C)		Peak to Peak (from Min. to Max. Temp.) Displacement in. (mm)			
			Middlesex		East Montpelier	
	MI	EM	Thermal Expansion Equation	Field Data: Sum of Top of Abut. Displacement	Thermal Expansion Equation	Field Data: Sum of Top of Abut. Displacement
2010	84.2 (46.8)	99.0 (55.0)	0.93 (23.6)	1.14 (29.0)	0.94 (23.9)	1.09 (27.7)
2012	105.1 (58.4)	105.1 (58.4)	1.16 (29.5)	1.15 (29.2)	1.00 (25.4)	1.10 (27.9)
2013	109.0 (60.6)	109.0 (60.6)	1.20 (30.5)	1.17 (29.7)	1.03 (26.2)	1.28 (32.5)
2014	105.0 (58.3)	105.0 (58.3)	1.16 (29.5)	1.16 (29.5)	0.99 (25.1)	1.20 (30.5)
2015	104.0 (57.8)	104.0 (57.8)	1.14 (29.0)	0.97 (24.6)	0.98 (24.9)	1.12 (28.4)

5.12.2 Abutment Bottom Transverse Displacement

The transverse displacement for the straight Middlesex Bridge was minimal, as would be expected in design. Transverse displacement in this section is focused on the skewed East Montpelier Bridge. Displacement data is presented for the bottom of the abutment (top of pile) for all four corners of the bridge in Figure 5.49. For Abutment 1, positive displacement indicates movement towards downstream, while positive values for Abutment 2 indicate movement towards upstream. The transverse displacement over time highlights a significant difference in behavior of a skewed bridge compared to a straight bridge. There is a net displacement towards downstream at Abutment 1 and towards upstream at Abutment 2 resulting in a net rotation of the bridge in plan view. The movement is primarily concentrated at the acute corners, which increase in displacement while the bridge expands, while the displacement at the obtuse corners decreases (with a smaller magnitude) showing that the obtuse corners tend to resist the rotation of the bridge. Transverse displacements do not recover completely from season to season which has resulted in a net rotation of the bridge.

A schematic drawing demonstrating how the skewed abutments introduce a rotation in plan view of the bridge is shown in Figure 5.50 where the light shade of blue shows the deformed shape under bridge expansion. The resultant soil pressure on skewed abutments has a normal force component as well as a component parallel to the abutment since the abutment is not perpendicular to the centerline of the bridge, and the normal force couple causes rotation of the bridge (England et al. 1995). The rotation of skewed bridges has been noted in other studies, which have also found the tendency of rotation towards the acute corners (Hoppe and Gomez 1996) (Frosch and Lovell 2011). The effect of transverse displacements on pile bending moments is discussed in the next section.

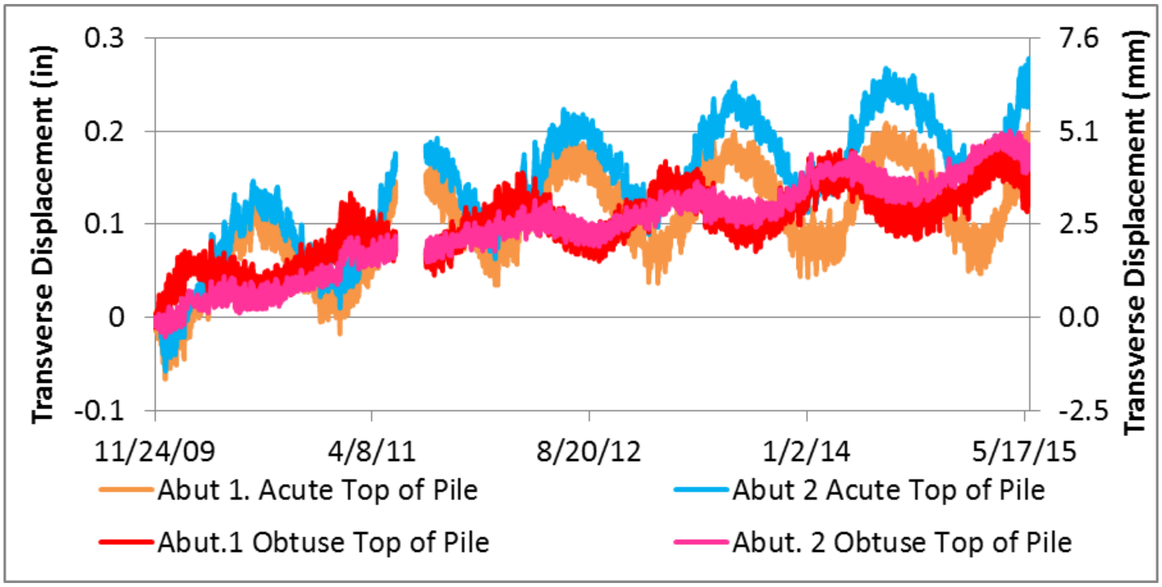


Figure 5.49: Transverse Displacement at East Montpelier Bridge

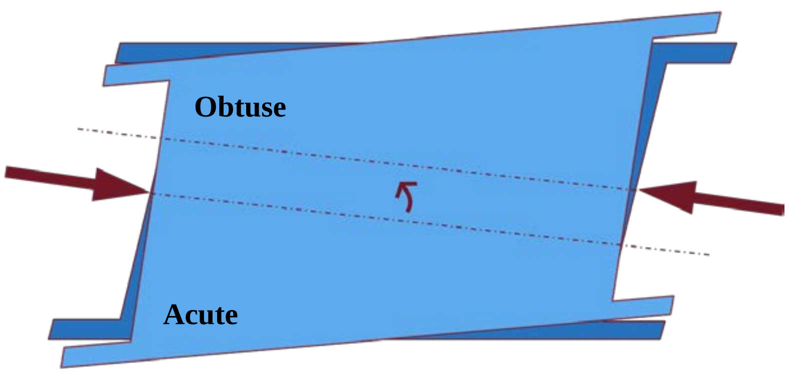


Figure 5.50: Schematic of Seasonal Rotation in Plan View of East Montpelier Bridge

5.12.3 Pile Bending Moments

Pile bending moments from field data were calculated using values from four strain gages on the pile flanges at three depths along the pile: 1.6 ft (0.49 m), 3.3 ft (1.0 m), and 4.9 ft (1.5 m). All pile bending moments are due to thermally induced loads and do not include bending moments from construction. Piles are HP12x84 (HP310x125) and are oriented about weak axis (weak axis resisting longitudinal bending). Bending moments from field data were plotted with pile bending moments obtained from the Nominal and Matched FEMs and are presented for the warmest and coldest day of the year in 2010 and 2014.

For the Middlesex Bridge, the bending moment plots are shown in Figure 5.51. The values from the FEMs are shown for the upstream pile; only weak axis bending moments are shown since strong axis bending moments were minimal. For the East Montpelier Bridge, both weak and strong axis bending moment plots are shown in Figure 5.52 and Figure 5.53, respectively. Strain gages on the obtuse corner pile were damaged over time; therefore these figures show data for the acute corner pile.

The field data validates the accuracy of the Matched FEMs compared to the Nominal FEMs. For East Montpelier, the cold weather field data shows that the response was somewhere between the Nominal and Matched FEM for 2010 weak axis bending, however the maximum bending moment is about the same for this case. Since the field data bending moments could potentially be near the point of inflection, these values may not always be representative of the maximum bending moment. Therefore, maximum pile bending moments were obtained from the Matched FEMs for both bridges. Maximum weak axis and strong axis bending moments from the Matched FEM are presented for the yearly maximum and minimum temperature in Table 5.6 and Table 5.7, respectively. Since field data is missing for periods of 2011 when power was lost, values are not presented for Matched FEMs for 2011 because peak values may not have been captured.

In the tables, the Middlesex Bridge reports data for the upstream pile while data for East Montpelier is presented for both the acute and obtuse corner piles.

The maximum pile bending moments at the Middlesex Bridge occur about the weak axis, which is expected for a straight bridge. The East Montpelier Bridge, with a skew angle of only 15 degrees, had transverse bending moments that were greater than, or equal to, longitudinal bending moments.

The maximum thermally induced weak axis bending moment obtained from the Matched FEM was 46.6 k-ft (63.1 kN-m) and -68.0 k-ft (-92.1 kN-m) at the Middlesex and East Montpelier Bridge, respectively. The maximum strong axis bending moment was 6.2 k-ft (8.3 kN-m) and 68.0 k-ft (92.2 kN-m) at the East Montpelier Bridge. The transverse bending moments in the skewed bridge were greatest under bridge expansion, with values consistently greater at the obtuse pile.

These results show that transverse pile bending can result in moments comparable to, or exceeding, longitudinal pile bending moments. The plan rotation of the skewed bridge (and corresponding transverse pile bending) is the main difference in response from a straight bridge. While this did not lead to issues in the East Montpelier Bridge, it is something that should be considered in greater skew angles or longer bridge lengths.

A simple comparison of FEMs was done for the skewed East Montpelier Bridge by analyzing a thermal analysis with the integral wingwalls, and then analyzing the same thermal load without the presence of wingwalls. The results showed that wingwalls provide some resistance to the bridge rotation in plan view, resulting in less transverse displacement and lower transverse bending moments.

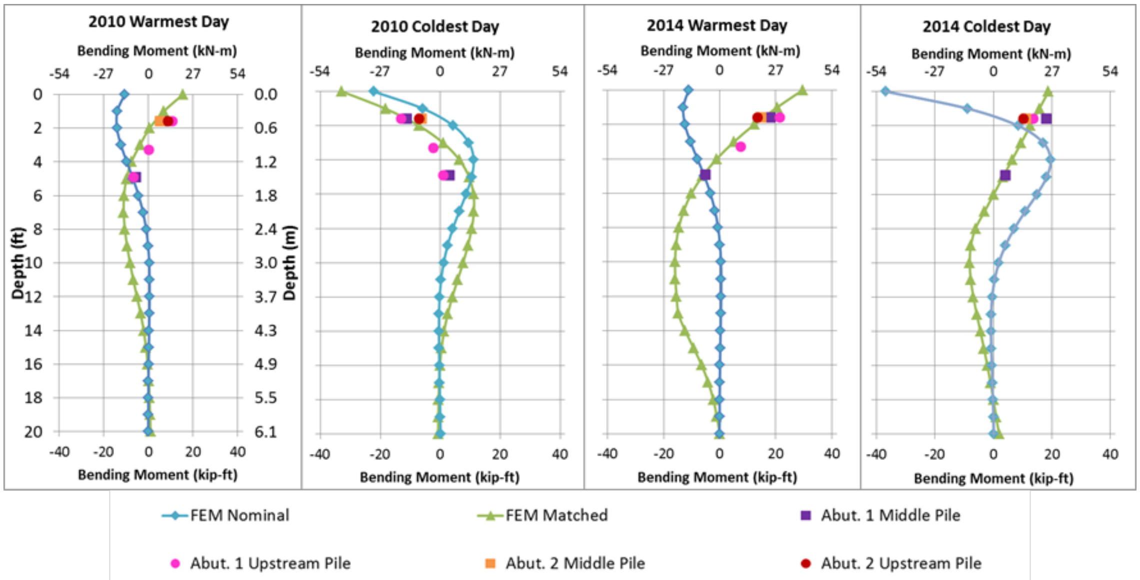


Figure 5.51: Middlesex Weak Axis Pile Bending Moment Comparison of Field Data, Nominal and Matched FEMs

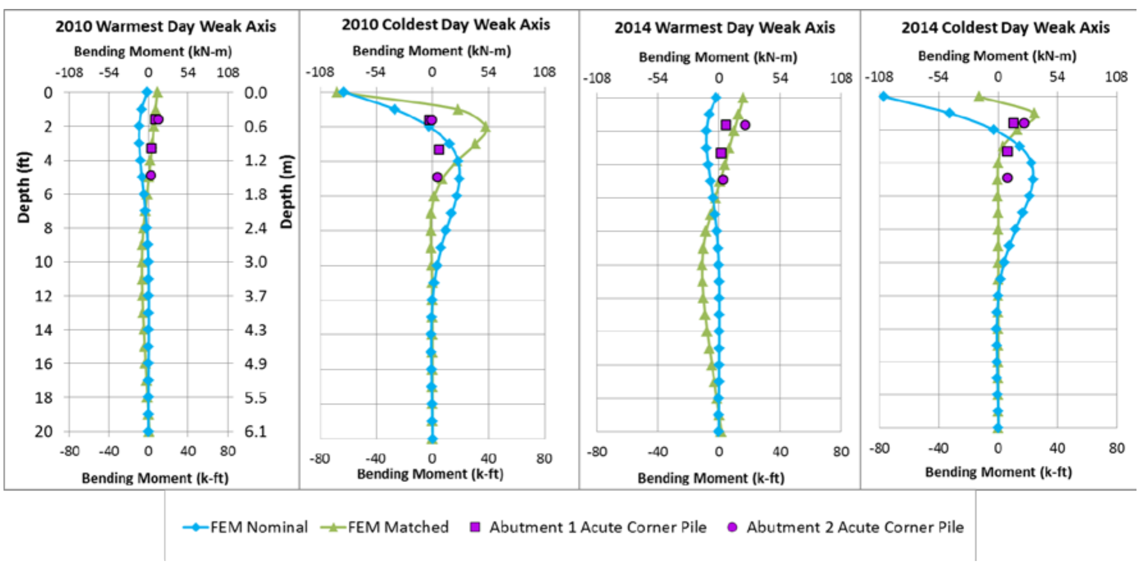


Figure 5.52: East Montpelier Acute Corner Weak Axis Pile Bending Moment Comparison with Field Data: Nominal and Matched FEMs

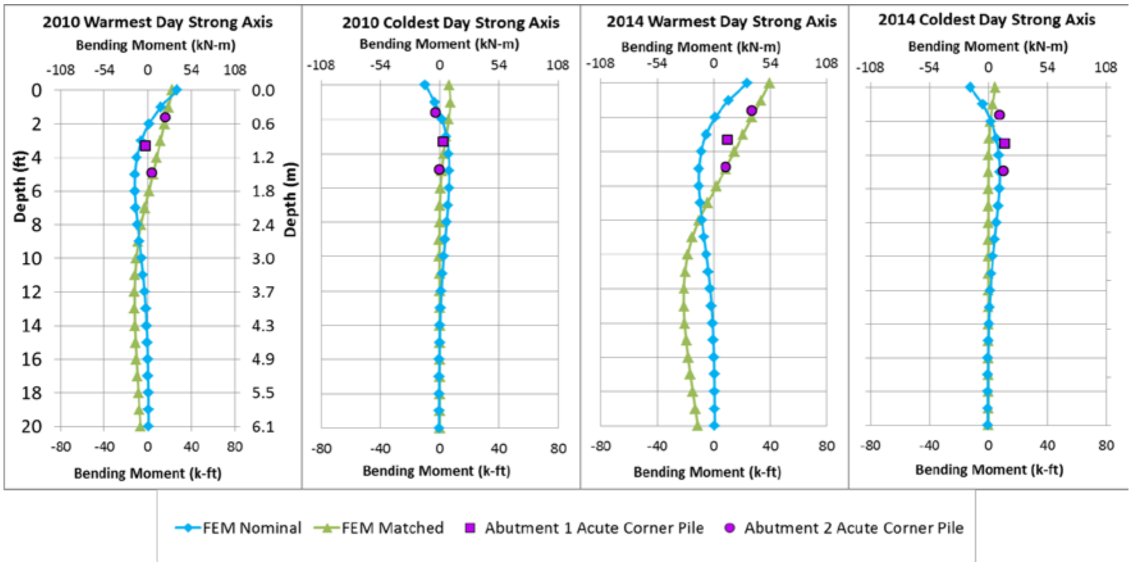


Figure 5.53: East Montpelier Acute Corner Strong Axis Pile Bending Moment Comparison with Field Data: Nominal and Matched FEMs

Table 5.6: Maximum Weak Axis Pile Bending Moment

	Matched FEM Maximum Weak Axis BM kip-ft (kN-m)		
	Middlesex	East Montpelier	
		Acute	Obtuse
2010 Coldest	-32.88 (-44.57)	-67.95 (-92.13)	24.73 (33.53)
2010 Warmest Day	15.51 (21.02)	9.17 (12.43)	-5.91 (-8.02)
2012 Coldest	21.65 (29.36)	17.43 (23.64)	11.52 (15.62)
2012 Warmest Day	24.92 (33.79)	11.83 (16.04)	-7.73 (-10.48)
2013 Coldest	46.57 (63.14)	22.30 (30.23)	14.87 (20.16)
2013 Warmest Day	30.08 (40.78)	16.18 (21.94)	11.04 (14.97)
2014 Coldest	18.84 (25.54)	24.36 (33.03)	16.29 (22.09)
2014 Warmest Day	29.52 (40.03)	15.87 (21.52)	10.79 (14.63)
2015 Coldest	22.65 (30.71)	26.06 (35.34)	17.46 (23.68)
2015 Warmest Day	26.10 (35.39)	14.17 (19.21)	35.46 (48.08)

Table 5.7: Maximum Strong Axis Pile Bending Moment

	Maximum Strong Axis BM kip-ft (kN-m)		
	Middlesex	East Montpelier	
		Acute	Obtuse
2010 Coldest Day	-2.1 (-2.90)	7.29 (9.88)	-43.19 (-58.6)
2010 Warmest Day	3.29 (4.47)	22.70 (30.8)	38.47 (52.2)
2012 Coldest Day	2.74 (3.73)	3.29 (4.46)	-12.49 (-16.9)
2012 Warmest Day	4.42 (6.01)	29.54 (40.04)	50.03 (67.8)
2013 Coldest Day	6.13 (8.34)	4.41 (5.98)	-16.06 (-21.8)
2013 Warmest Day	5.14 (6.99)	40.36 (54.7)	67.99 (92.2)
2014 Coldest Day	2.36 (3.21)	4.88 (6.62)	-17.58 (-23.8)
2014 Warmest Day	5.07 (6.89)	39.62 (53.7)	66.79 (90.6)
2015 Coldest Day	2.88 (3.91)	5.27 (7.15)	-18.83 (-25.5)
2015 Warmest Day	4.58 (6.23)	35.46 (48.1)	59.89 (81.2)

5.12.4 Earth Pressure

Earth pressure values were recorded behind the abutment walls at various depths. The assumed fully passive and active pressure values for the two bridges are shown in Table 5.8, for the three depths of pressure cells. The earth pressures presented in this section include the pressure recorded at the end of construction in order to observe how the total pressure values compare to assumed fully passive pressure.

For the Middlesex Bridge, Figure 5.54 shows the earth pressures behind Abutment 1 at locations behind the upstream, downstream and center of abutment. The displacement at the top and bottom of abutment is also shown, which is recorded at the upstream corner. The greatest passive pressure occurs in the middle row of pressure cells with a maximum value of 11.8 psi (81.2 kPa), while pressure values were slightly less for top and bottom pressure cells. Pressures were consistent across the abutments. During bridge contraction, values were close to assumed active pressure.

For the East Montpelier Bridge, Figure 5.55 and Figure 5.56 show similar plots of pressure and corresponding top and bottom of abutment displacement for the obtuse and acute corner, respectively. The longitudinal displacement values at the obtuse and acute corners are comparable over time. However, the earth pressure distribution is different. The rotation of the bridge under expansion builds up pressure behind the obtuse corner where the maximum passive pressure value is 22.6 psi (155.9 kPa). At the acute corner, the maximum passive pressure is only 8.2 psi (56.8 kPa). The horizontal variation in earth pressures in skewed bridges has also been noted in previous research. A field study on a Maine IAB with a 20 degree skew angle reported that the backfill pressure behind the obtuse corner of the bridge was significantly greater than the acute corner, having almost 3 times greater pressures. They also noted that the variation across the abutment (horizontal) was greater than the vertical variation (Sandford and Elgaaly 1993). Vertical variation at the East Montpelier Bridge was observed at the obtuse corner; however this variation was less than the horizontal variation. Active pressure distribution is more consistent in both horizontal and vertical distributions with active pressures decreasing to about zero for both obtuse and acute corners over time.

The greatest pressure at the obtuse corner occurs in the top row of pressure cells (which are 7.2 ft (2.2m) below the top of the abutment). While the maximum passive pressure of 22.6 psi (155.8 kPa) is within 2.5 psi (17.3 kPa) of fully passive pressure at the top depth of pressure cells, the abutments are designed for the maximum fully passive pressure of 38.3 psi (264.3 kPa) which occurs at the bottom of the abutment therefore the design is still conservative. Passive pressure at the acute corner is well below design fully passive pressure.

Previous research has noted a potentially serious concern of soil ratcheting in IABs (England et al. 1995), however the results from this study show that for these two bridges soil ratcheting did not occur and there were, in fact, lower passive pressure values over time with corresponding displacements.

While the 15 degree skew angle did lead to variation in pressure distribution behind the abutment, this is not something that necessarily needs to be considered separately from the design of a straight bridge. Maximum passive pressure in the skewed bridge occurs in the top row of pressure cells while the abutments are designed for fully passive pressure at the bottom of the abutment. Designing the abutments for the maximum fully passive pressure value is a conservative design approach and neither the straight nor skewed bridge had passive pressure values exceeding design values. Designing without integral wing walls would likely reduce the pressure concentration observed in skewed bridges, however this is not recommended as the abutment pressures did not exceed design values and the wingwalls provide restraint from plan rotation of the bridge and reduce transverse pile bending.

Table 5.8: Assumed active and fully passive pressure at pressure cell locations

	Assumed Pressure Values, psi (kPa)					
	Middlesex			**East Montpelier**		
	Depth ft (m)	Active	Fully Passive	Depth ft (m)	Active	Fully Passive
Top	7.2 (2.2)	1.6 (11.0)	21.9 (151.1)	8.1 (2.5)	1.8 (12.4)	25.1 (173.2)
Middle	10.2 (3.1)	2.2 (15.2)	30.6 (211.1)	10.0 (3.1)	2.3 (15.9)	31.7 (218.7)
Bottom	12.8 (3.9)	2.9 (20.0)	39.3 (271.2)	12.0 (3.7)	2.8 (19.3)	38.3 (264.3)

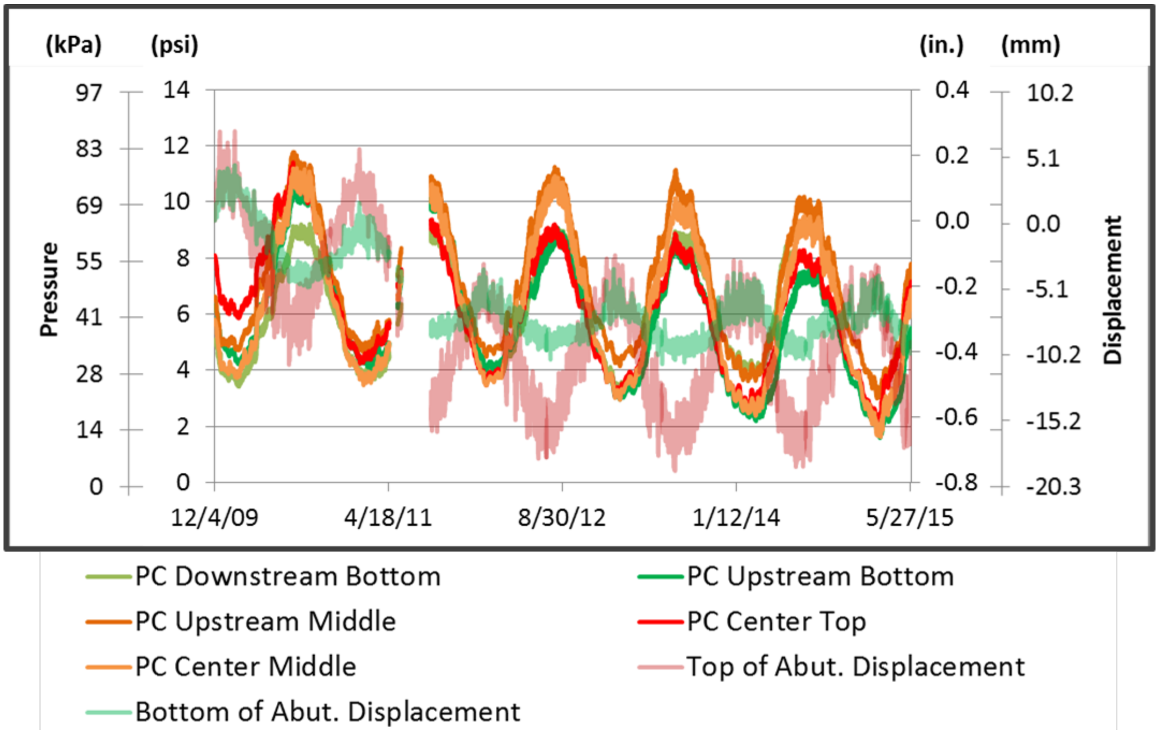


Figure 5.54: Earth Pressures Behind Abutment 1 at Middlesex Bridge with Displacement

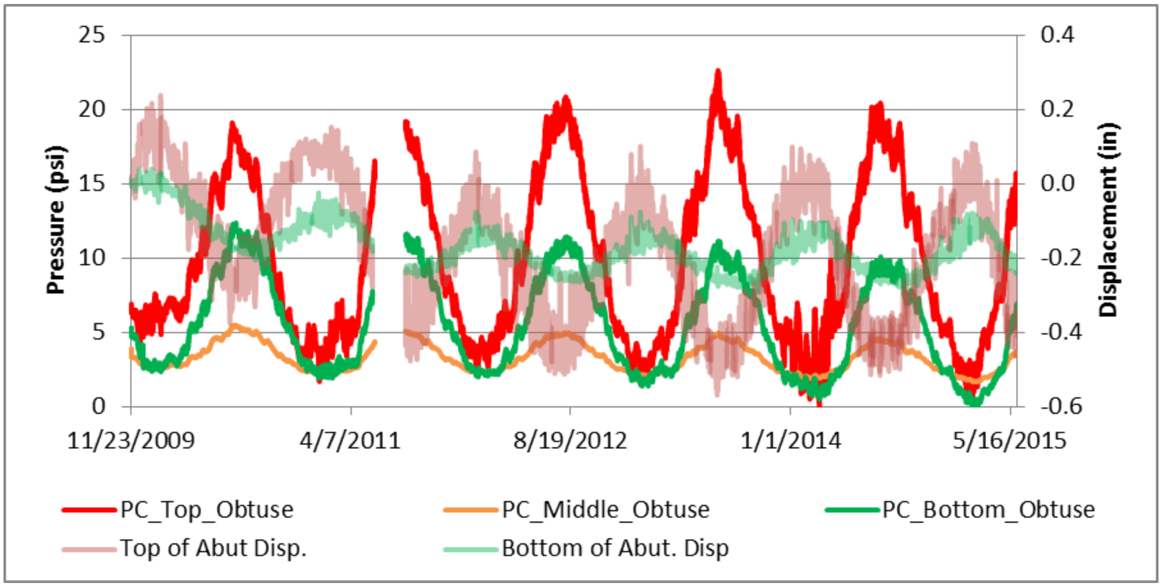


Figure 5.55: Earth Pressures Behind Abutment 1 Obtuse Corner at East Montpelier Bridge with Displacement

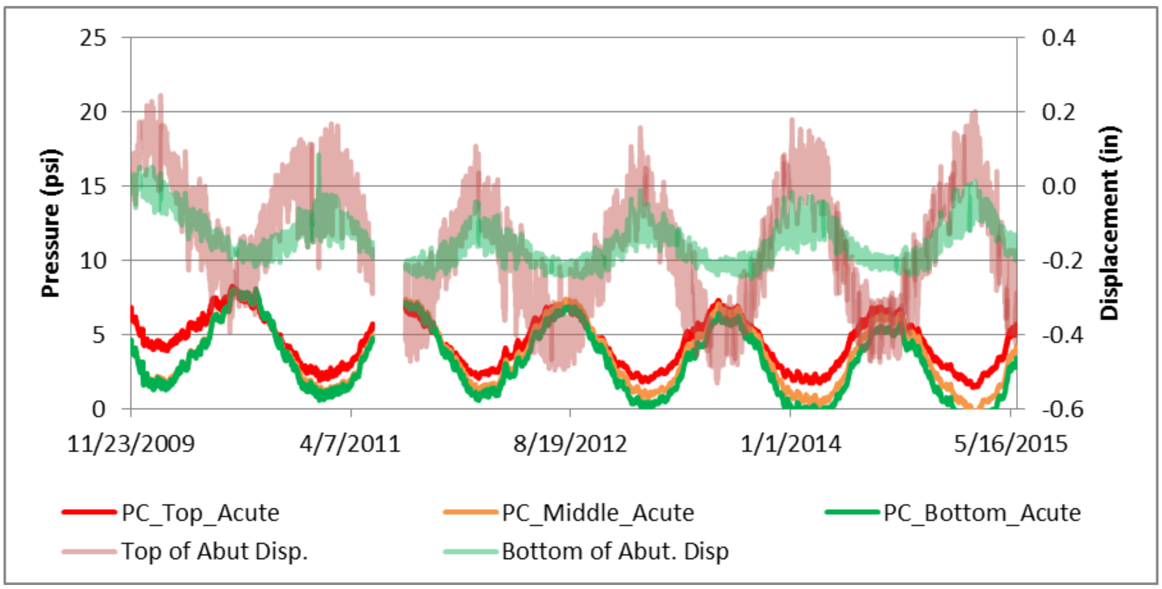


Figure 5.56: Earth Pressures Behind Abutment 1 Acute Corner at East Montpelier Bridge with Displacement

5.13 Summary and Conclusions

The Vermont Agency of Transportation (VTrans) initiated a program of field instrumentation and analysis to evaluate the performance of three IABs beginning at bridge construction, and monitored over four years. The bridges have an increasingly complex geometry, a straight girder non-skew bridge geometry, a straight girder 15 degree skew bridge geometry, and a curved girder two-span continuous structure with 11.25 degrees of curvature. A comparison of the thermally induced girder stresses, top and bottom of abutment longitudinal and transverse displacement, abutment rotation, earth pressures, seasonal substructure displacement, and pile bending moments was given for the three IABs. FEM results were compared to measured longitudinal displacements using nominal and matched FEMs that incorporate the changes in soil properties that arose during the 5 years of monitoring.

Top of abutment longitudinal displacement could be reasonably predicted using a simplified two-dimensional analysis. The 15 degree skew angle and 11.25 degrees of curvature did not affect longitudinal displacements. There was negligible difference in longitudinal displacement of the acute and obtuse corner of the skewed bridge. The greatest variability in longitudinal response occurred at the straight single-span Middlesex Bridge, with an unsymmetrical response between the two abutments that would not be expected in design. A shift toward the backfill was noted over time at all three bridges, although the shift was most prominent at the straight bridge and least prominent at the two-span curved bridge.

Bottom of abutment displacement was greater than top of abutment displacement at all three bridges due to abutment rotation. Piles are designed for the maximum displacement from thermal expansion (which would occur at the top of the abutments) and neglect abutment rotation. By neglecting abutment rotation, the design accounts for increased displacement which was observed due to the shift towards the backfill, therefore this is not an overly conservative design method.

Seasonal substructure displacement data showed that displacements are not only depended on temperature, but also depend on when the temperature occurs in a given season. This behavior is not something that is predicted in a static thermal analysis. The seasonal variation in response stabilized over time. The straight and skewed bridges showed a permanent offset of the pile towards the backfill demonstrating that IABs response to cyclic thermal loading is not a straightforward as typical design programs such as L-pile would imply.

Transverse displacements were minimal at the straight and curved bridges. The skewed East Montpelier Bridge experienced transverse displacements concentrated at the acute corners that resulted in plan rotation of the bridge and a net rotation over time.

Thermally induced earth pressures were minimal at the straight and curved bridges. The Geofoam appeared to be highly effective at minimizing earth pressures. These two bridges showed consistent pressure distribution across the abutments. The skewed East Montpelier Bridge showed variation in pressure distribution horizontally and vertically across the abutment. The obtuse corner had passive pressure values about three times greater than those at the acute corner. Maximum passive pressure occurred in the top row of pressure cells at the obtuse corner. Cumulative passive pressure at this location was close to assumed fully passive pressure at the corresponding depth. However, this value is below the maximum fully passive pressure used in design (which occurs at the bottom of the abutment). Pressure values decreased over time in both bridges while displacement towards the backfill increased over time which shows soil ratcheting did not occur at these bridges

Designing the abutments for fully passive pressure at the straight Middlesex Bridge and the two-span curved Stockbridge Bridge with Geofoam behind the abutments appears to be an overly conservative design based on the low passive pressure values observed at these bridges.

The Nominal FEM (which had soil springs based on boring log information and assumption of compacted backfill response), did not accurately predict field displacements. A static thermal analysis does not account for variations observed in field response such as the shift toward the backfill and complex soil response to cyclic thermal loading. Matching the FEMs to field response required changing the soil spring properties; in general, the soil provided far less restraint than what was originally assumed.

Pile bending moments in the two-span curved bridge were fairly minimal in the transverse direction, while longitudinal bending moments were similar to those at the straight, single-span bridge. Transverse displacements in the15 degree skewed bridge generally resulted in strong axis pile bending moments that were equal to, or greater than, the bending moments about the weak axis.

Overall, for the case of the Stockbridge Bridge, the 11.25 degrees of curvature did not result in any variations in response from what would be expected in a straight bridge. The Geofoam effectively minimized earth pressures, and may also be a reason for the extremely elastic and consistent response year to year. The response of this bridge was more consistent than the straight or skewed single span bridges. These results suggest that designing IABs with curvature of 11.25 degrees can safely be done assuming a straight bridge response; however, it should be noted that the presence of Geofoam and the fact that this is a two-span bridge introduce additional factors that could have played a role in response.

The 15 degree skew angle of the East Montpelier Bridge did result in transverse response, and earth pressure distribution, that differed from a straight bridge. Transverse bending of the piles can be significant, even with a relatively low skew angle of 15 degrees, therefore this should be considered in design. The variation in earth pressure, and significant passive pressure behind the obtuse corner, are accounted for if designing the abutments for fully passive pressure and no additional considerations due to skew need to be addressed. Designing skewed abutments with integral wing walls helps to resist the seasonal rotation of the bridge and reduce transverse bending moments; therefore the use of integral wing walls is recommended.

The greatest variation in response occurred at the straight single-span Middlesex Bridge which highlights the fact that field variability can result in unpredictable response even in a seemingly straightforward design. While this did not result in any structural issues, it demonstrates the complex response of IABs and soil-structure interaction and the difficulty in predicting response of these bridges.

CHAPTER 6

PARAMETRIC STUDY ON EFFECTS OF PILE ORIENTATION

This chapter presents a parametric study on effects of pile orientation pile bending moments in single span IABs of varying length and skew angles. The study of the skewed East Montpelier Bridge highlighted the significance of transverse bending of piles in skewed bridges. Even with a relatively small skew angle of 15 degrees, the East Montpelier Bridge had transverse pile bending moments that were greater than or equal to the bending moments about the weak axis. Since there are no uniform recommendations on the orientation of piles in IABs, this parametric study analyzed FEMs of various lengths and skew angles under multiple thermal loads to determine what factors could influence the choice of pile orientation, and to determine what orientation, if any, is the best choice for avoiding pile yielding at the pile-abutment interface.

6.1 Introduction

Despite the popularity of IABs, uniform design recommendations are lacking. One area that has no uniform standard in the design of IABs is the optimal orientation of H-piles. Pile orientation is significant in that the choice relates to differences in design philosophy where the design intent can be to provide minimal restraint to thermal movement, to minimize forces in the pile, to provide resistance to out of plane movements, or a combination of these. Preferences on pile orientation, however, are typically based on general design intent and past performance of bridges rather than a direct comparison of resulting performance. This paper presents a parametric study using three-dimensional finite element models (FEMs), focusing on the effects of thermal loads with two pile orientations on steel girder IABs with varying lengths and skew angles. The results in this chapter apply to thermal response of the models, reporting displacements of abutment and piles, pile bending moments about the weak and strong axes, and the ratio of maximum moment to the yield moment of the piles.

6.2 Background

In a 2004 survey, out of 39 responding states there was no consensus on the preferred orientation of the pile (Maruri and Petro 2004). The two main pile orientations used across the country are weak axis (web parallel to the abutment centerline) and strong axis (web perpendicular to the abutment centerline) (Figure 6.1). The preference of these two orientations varies greatly throughout the country: 33 percent of states oriented the piles with the strong axis parallel to the abutment centerline, 46 percent oriented the piles with the weak axis parallel to the abutment centerline, 8 percent left it to the discretion of the Engineer, and the remaining 13 percent did not comment or noted use of symmetric piles (Maruri and Petro 2005). This is an aspect of IAB design that still, eleven years after the survey, has no uniform design standard. Sherafati (2011) reported the strong axis orientation should be used with straight bridges, but curved bridges should consider multiple factors in determining orientation. Olson et al. (2013) reported that orientation of pile web perpendicular to the abutment centerline results in more displacement capacity but reduced lateral stiffness and substantially reduced weak-axis bending of the pile regardless of skew therefore recommending strong axis always be used. On the contrary, Mistry (2005) reported a single row of flexible piles with weak axis orientation should always be used. The effect of pile orientation has been gaining attention across the country, especially when considering application to skewed IABs where transverse movements can become significant.

6.3 Finite Element Models

Three -dimensional FEMs were created using SAP2000. Three bridge lengths were chosen to cover a typical range of IAB designs: 15.2 m (50 ft), 30.5 m (100 ft), and 45.7 m (150 ft). Each model of the three bridge lengths were modeled at four skew angles: 0° (straight bridge), 15°, 30°, and 45°. Each model was analyzed for piles with web parallel to abutment centerline (weak axis pile orientation), and piles with web perpendicular to abutment centerline (strong axis pile orientation) as shown in Figure 6.1. The 45.7 m (150 ft) bridge with 45° skew is presented in Figure 6.2. In total, 24 independent geometric models were analyzed. All FEMs were based on models calibrated to field data from IAB bridges used in previous research (Civjan et al. 2014).

The frame elements in the models were constructed with Grade 345 (50 ksi) steel using 2-node elements with six degrees of freedom per node and with W-shapes for girders and HP sections for piles. The sections used for the three bridge lengths are provided in Table 6.1. The pile choices for this study are consistent with typical piles used in IABs in many states; Massachusetts' Bridge Manual states these as acceptable pile sections meeting the criteria in LRFD Bridge Design Specifications 6.9.4.2 and 6.12.2.2.1; flange local buckling will not precede the sections reaching the plastic range (MassDOT 2013). These pile sizes were also similar to piles used in IABs in Vermont where the bridges ranged from 36.9 m (121 ft) to 43 m (141 ft) (Civjan et al. 2014). Piles were modeled to be 12.2 m (40 ft) long which was shown to be of sufficient length to not influence the results. Five piles support each abutment, distributed evenly at 2.3 m (7.7 ft) spacing.

The bridge abutments and decks were modeled of 4-node thin shell elements with six degrees of freedom at each node. All FEMs used 27.6 MPa (4000 psi) concrete for the shell elements. Abutment thickness for the IABs in this parametric study was based on the aforementioned bridges in Vermont with a thickness of 0.91 m (3.0 ft) for all three bridge lengths. Abutment height ranged from 2.74 (9.0 ft) (for the 15.2 m (50 ft) and 30.5 m (100 ft) bridge) and 3.58 m (11.75 ft) (for the 45.7 m (150 ft) bridge) which were also similar to the Vermont bridges. The bridge width for all three bridges in this study was 36 ft which was not only similar to the Vermont bridges but also the width used in another parametric study on IABs from Olson et al. (2013). The size of structural components remained constant with all skew angles in order to limit influence of factors other than those under consideration for this study.

Composite action between the deck and girders was simulated by making use of rigid links between girder nodes and coincident deck nodes (slip at the interface through shear studs was neglected which was assumed reasonable for the service load conditions studied). Rigid links were also used to simulate the depth of the girder embedded into the abutment, restricting all degrees of freedom in order to transfer all forces from the girders to the abutments. The deck sections were modeled as thin shell elements, 152.4 mm (6 in.) thick. Abutment wingwalls were not included in the models.

Soil structure interaction was modeled using non-linear discrete Winkler springs. Equations for pile soil springs were determined using API standards (API 1993). For the abutment springs, coefficients of lateral earth pressure were calculated following the report by Barker et al. (1991). All equations were presented in Chapter CHAPTER 4. Backfill soil springs were oriented perpendicular to the abutment, ignoring transverse friction components at the backfill. The purpose of this study is to highlight pile response; therefore variables such as soil friction and wingwalls were not included as they are highly variable between structures and would reduce the reported forces resisted by the piles. It was also assumed that pile resistance would be significantly higher than soil friction effects. Pile soil springs were oriented in both orthogonal directions to the pile to simulate soil surrounding the piles and assigned at 0.30 m (1 ft) intervals. Soil conditions can be highly variable. The properties used in this study are based on nominal conditions at several IABs studied in Vermont and were held constant in the analysis to highlight bridge length and skew effects on pile response. Behind the abutments, dense soil was assumed (friction angle=45°, density=22 kN/m^3 (140 lb/ft^3)) while around the piles medium-dense soil was assumed (friction angle=30°, density=20.4 kN/m^3 (130 lb/ft^3)). An example of the soil spring curves and where they are applied is shown in Figure 6.3.

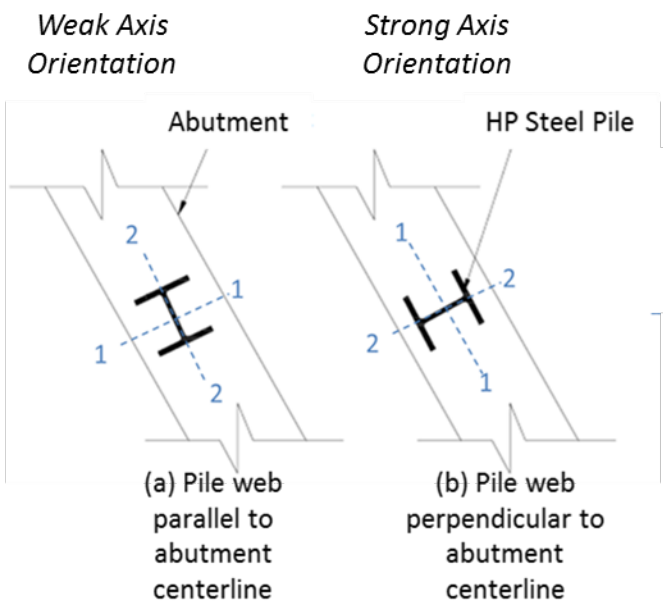


Figure 6.1: Pile orientations investigated in study

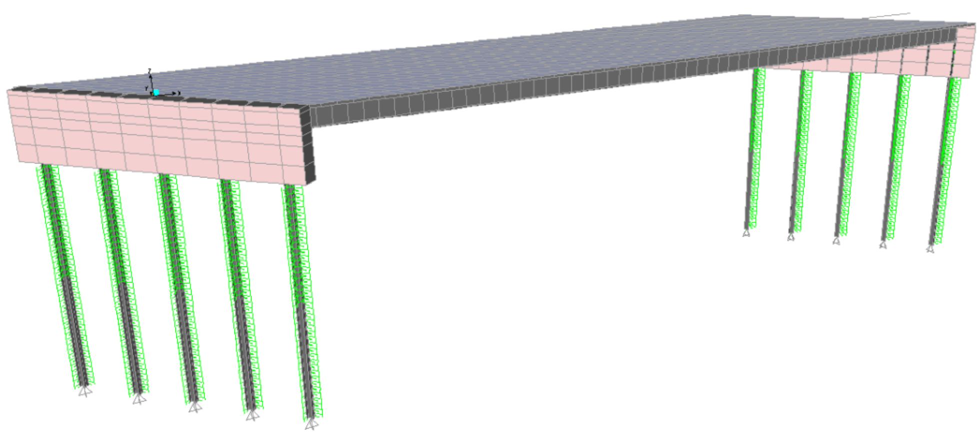


Figure 6.2: Finite Element Model example (L=45.7 m (150 ft), Skew=45°)

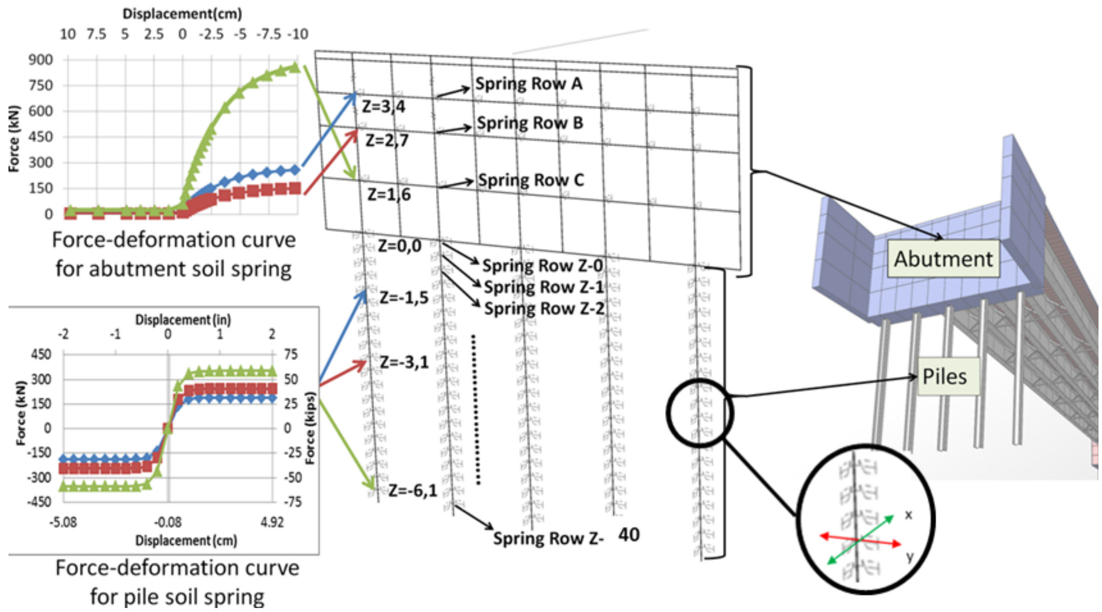


Figure 6.3: Soil spring curves

Table 6.1. Member sizes

	Bridge Length		
	15.2 m (50 ft)	**30.5 m (100 ft)**	**45.7 m (150 ft)**
Girders	W30x124	W30x124	W40x593
Piles	HP10x57	HP10x57	HP12x84

When skew is introduced, the corners of the abutment are referred to as the "acute" and "obtuse" corner, in reference to the angle they form with the bridge. Results presented in the proceeding sections will be the results for "Abutment 1", while the results are similarly mirrored at "Abutment 2". The call-out for these corners and abutments, as well as the sign convention that is used for displacements, is presented in

Figure 6.4. Displacements presented for the top and bottom of the abutment are in terms of the global axis (see longitudinal and transverse orientation in Figure 6.4). However, bending moments presented in later sections are about the weak and strong axes of the pile, which is oriented with abutment skew angle.

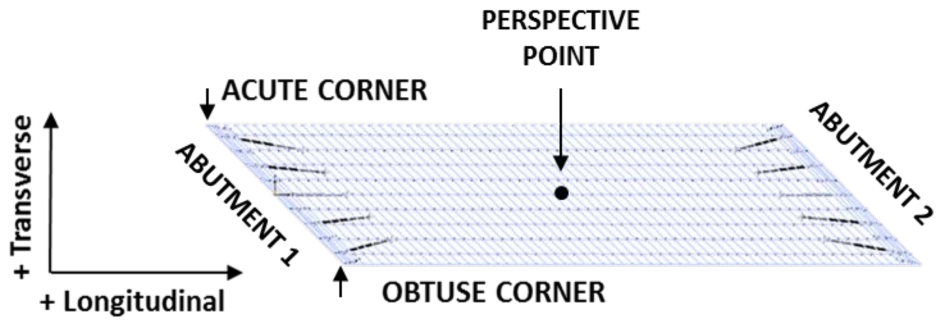


Figure 6.4: Call-out orientation and sign convention for displacements

6.4 Thermal Loads

Thermal loads were applied to both the deck and girders in the FEMs. This was done by assigning an area load of the thermal value to all of the deck shell elements, and a frame load of the thermal value to all girder frame elements. Four thermal loads were considered: T+41.7°C (T+75°F), T-41.7°C (T-75°F), T+50°C (T+90°F), and T-66.7°C (T-120°F) where +/- indicate an increase/decrease from the assumed reference temperature. These temperatures were chosen to be applicable to design conditions for many of the states using IAB's in the 2004 survey (Maruri and Petro 2005). Many states in the northern region of the United States have minimum and maximum design temperatures of -34.4°C (-30°F) and 48.9°C (120°F), for a thermal range of 83.3°C (150°F) per AASHTO design temperatures (AASHTO 2010). Thermal load of ±41.7°C

(±75°F) assumes a construction temperature at the mean of the minimum and maximum design temperatures and can be used to directly compare effects of bridge expansion and contraction at a similar load. Construction temperature in the context of this chapter is the temperature recorded when the abutments and deck attain strength to provide girder to abutment continuity. The assumed construction temperature of 7.2°C (45°F) would be typical of construction being completed near the end of a construction season. However, it was also acknowledged that construction temperature could vary. Realistic minimum and maximum bridge construction temperatures were assumed to range from -1.1°C (30°F) to 32.2°C (90°F). These assumptions were the basis for temperature loads referenced to design temperatures for thermal changes of T+50°C (T+90°F) and T-66.7°C (T-120°F).

Throughout this chapter, the thermal analyses of T±41.7°C (T±75°F) will be presented to directly compare bridge behavior under bridge expansion and contraction considering the same temperature increase and decrease. Next the thermal analyses T+50°C (T+90°F) and T-66.7°C (T-120°F) will be presented to understand how these non-symmetric maximum temperatures ranges affect results.

6.5 Displacements

Displacements of the bridges are first discussed as these define the overall response to thermal load. These displacements and related rotations of the abutments define the end conditions at the top of the piles. Similarly, the restraint provided by the pile orientation affects these movements. Expected longitudinal and transverse displacements for the temperature ranges evaluated were calculated using the equation of unrestrained thermal expansion:

$$\delta = \alpha L \Delta T$$

Equation 6.1

Where,

a=coefficient of thermal expansion ($11.7*10^{-6}/°C$) ($6.5*10^{-6}/°F$)

DT= Change in bridge temperature (°C) (°F)

L= Bridge span length (mm) (in) (longitudinal displacement) or superstructure span width (mm) (in.) (transverse displacement)

Expected displacements calculated for the two sets of thermal analyses are provided in Table 6.2 and are used as comparison to observed results from the FEMs.

6.5.1 Results: T+41.7°C (T+75°F) and T-41.7°C (T-75°F)

Longitudinal and transverse displacements from FEM results are presented in Figure 6.5 for the specified temperature increase and decrease. Results are shown at the top and bottom of the abutment, which also defines the abutment rotation. Each plot includes weak (square markers) and strong (diamond markers) axis pile orientation and results at the acute (blue shades) or obtuse (red shades) corner of the deck for each bridge length considered. Abutment and deck movements are nearly rigid body movements, so longitudinal, transverse and rotational movements are dependent on each other.

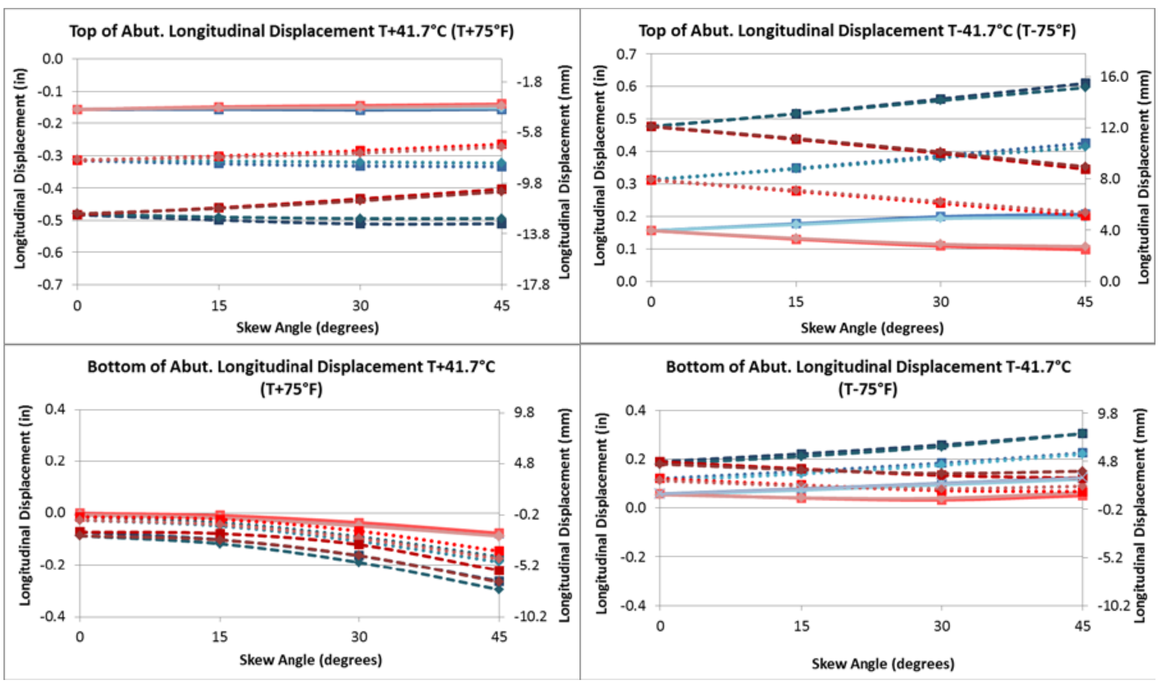
For top of abutment longitudinal displacement, the difference between strong and weak axis pile orientation is negligible for both expansion (temperature increase) and contraction (temperature decrease). Equation 6.1 provides a reasonable estimate of straight bridge longitudinal displacement. As the bridge length increases skew has a greater effect on longitudinal displacement for both contraction and expansion. Under bridge contraction, the difference in displacement at the acute and obtuse corner as skew increases is symmetric; the acute corner contracts with increasing skew while the obtuse corner expands at a similar rate indicating plan rotation about the perspective point (seen in Figure 6.4). Under bridge expansion non-symmetry of displacements occurs at the top of the abutment; the acute corner displacement essentially remains constant with increasing skew while the obtuse corner displacements are reduced to accommodate bridge rotation. The non-symmetric response under bridge expansion indicates backfill restraint across the abutment, becoming concentrated at the acute corners as the span length increases and resulting in a lower average longitudinal displacement.

Longitudinal displacement at the bottom of the abutment is significantly less than at the top of abutment as a result of abutment rotation and cannot be predicted by Equation 6.1. General results are similar to those at the top of the abutment though displacements are more significantly reduced under bridge expansion as skew decreases. Therefore, while top of abutment displacement at the acute corner essentially stays the same under bridge expansion with increasing skew, the bottom of abutment displacement at the both corners increases with increasing skew. Although pile orientation does not affect top of abutment longitudinal displacement, the orientation does affect longitudinal displacement at the bottom of the abutment; weak axis orientation results in greater displacements under bridge contraction (less abutment rotation) while strong axis orientation results in slightly greater displacements under bridge expansion.

For transverse displacements, the piles and rigidity of the abutment provide restraint against transverse thermal expansion at the ends of the bridge, while at other locations along the edge of the bridge displacement at 0° skew is similar to that calculated by Equation 6.1. Top of abutment transverse displacement is greater under bridge contraction due to the greater plan rotation of the abutment as was noted in the longitudinal displacements. At 45° skew the transverse displacements under contraction are over twice the amount they are under expansion. Bridge length and skew angle impacts transverse displacement behavior. Transverse displacements increase with bridge length and indicate plan rotation of the bridge more than thermal effects in the transverse direction. Pile orientation does affect transverse displacement; strong axis orientation results in greater displacement under bridge expansion and weak axis orientation results in greater displacements under bridge contraction (similar to effect of pile of orientation on bottom of abutment longitudinal displacement). For top of abutment transverse displacement, displacements are consistently in the positive direction (moving toward the acute corner) regardless of expansion or contraction. For bottom of abutment transverse displacement, the displacement behavior differs depending on whether the bridge is expanding or contracting. Under bridge expansion, transverse displacement at both corners is larger than at the top of the abutment, while under contraction the two corners have opposite signs with a negligible average displacement.JMN

Table 6.2. Expected displacement T+41.7°C (T+75°F), T-41.7°C (T-75°F), T+50°C (T+90°F) and T-66.7°C (T-120°F)

Bridge Span Length m (ft)	Expected Total Displacement mm (in)							
	Longitudinal (per abutment)				Transverse (total)			
	T+41.7°C (T+75°F)	T-41.7°C (T-75°F)	T+50°C (T+90°F)	T-66.7°C (T-120°F)	T+41.7°C (T+75°F)	T-41.7°C (T-75°F)	T+50°C (T+90°F)	T-66.7°C (T-120°F)
15.2 (50)	3.8 (0.15)	-3.8 (-0.15)	4.6 (0.18)	-5.8 (-0.23)	5.3 (0.21)	-5.3 (-0.21)	6.4 (0.25)	-8.6 (-0.34)
30.5 (100)	7.4 (0.29)	-7.4 (-0.29)	8.9 (0.35)	-11.9 (-0.47)	5.3 (0.21)	-5.3 (-0.21)	6.4 (0.25)	-8.6 (-0.34)
45.7 (150)	11.2 (0.44)	-11.2 (-0.44)	13.5 (0.53)	-17.8 (-0.70)	5.3 (0.21)	-5.3 (-0.21)	6.4 (0.25)	-8.6 (-0.34)



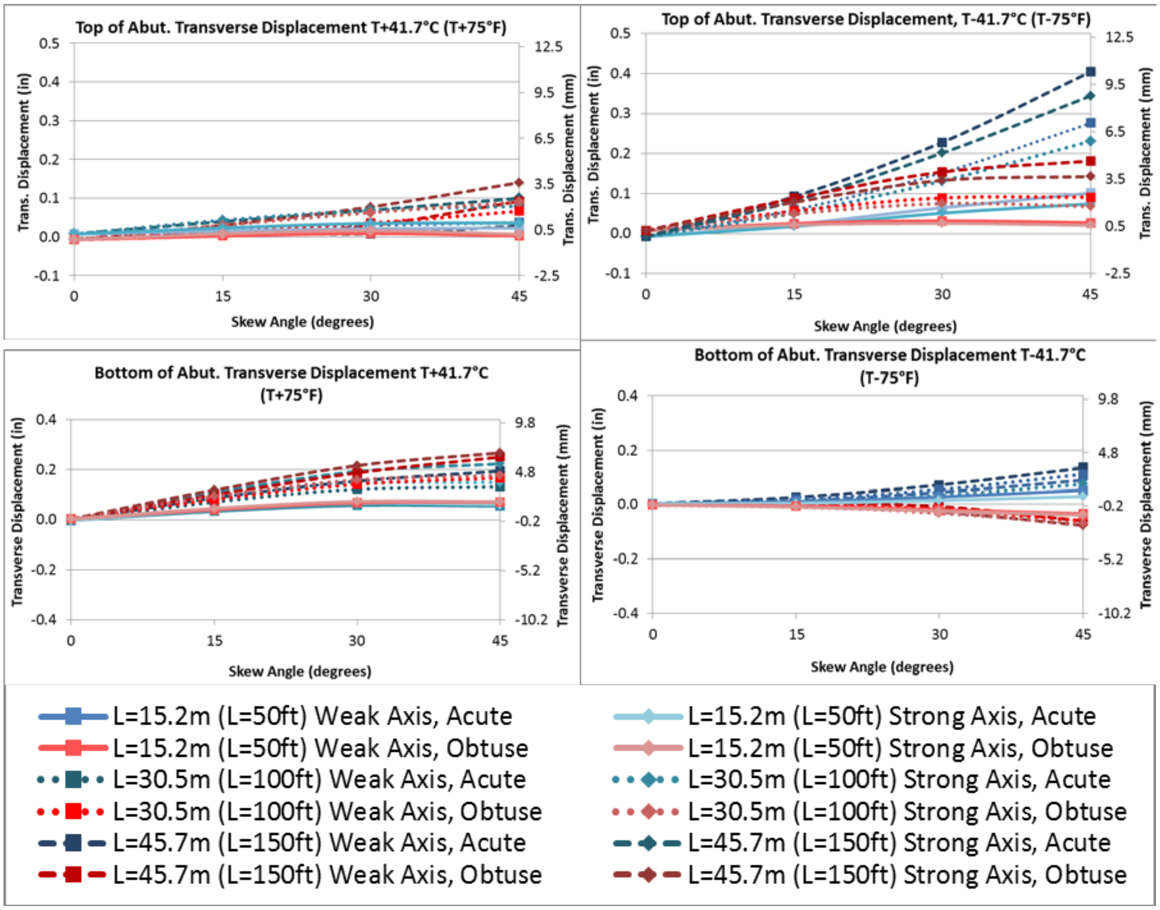


Figure 6.5: Longitudinal and transverse displacements for T+41.7°C (T+75°F) (left) and T-41.7°C (T-75°F) (right)

6.5.2 Results: T+50°C (T+90°F) and T-66.7°C (T-120°F)

The following temperature changes consider two separate possible construction temperatures, therefore an IAB would have the potential to experience only one of these thermal cases depending on the temperature at completion of construction, and maximum and minimum design thermal ranges. Longitudinal and transverse displacements for top and bottom of the abutment are presented for bridge expansion of T+50°C (T+90°F) and bridge contraction of T-66.7°C (T-120°F) in Figure 6.6 .The overall displacement behavior is similar to those presented for the temperatures presented in Figure 6.5, with magnitudes of displacement values increasing proportionally to the increase in temperature applied, indicating that all soil response is in the elastic range of the non-linear behavior and that elastic springs would be appropriate for many designs. Under these two temperature changes, the bottom of abutment transverse displacement is greater (once skew is introduced) under expansion of T+50°C (T+90°F) than under the contraction of T-66.7°C (T-120°F), even though the contraction temperature is 16.7°C (30°F) greater than expansion. This is a result of the twisting of the bridge that occurs at higher skews when the backfill is restricting longitudinal displacement.

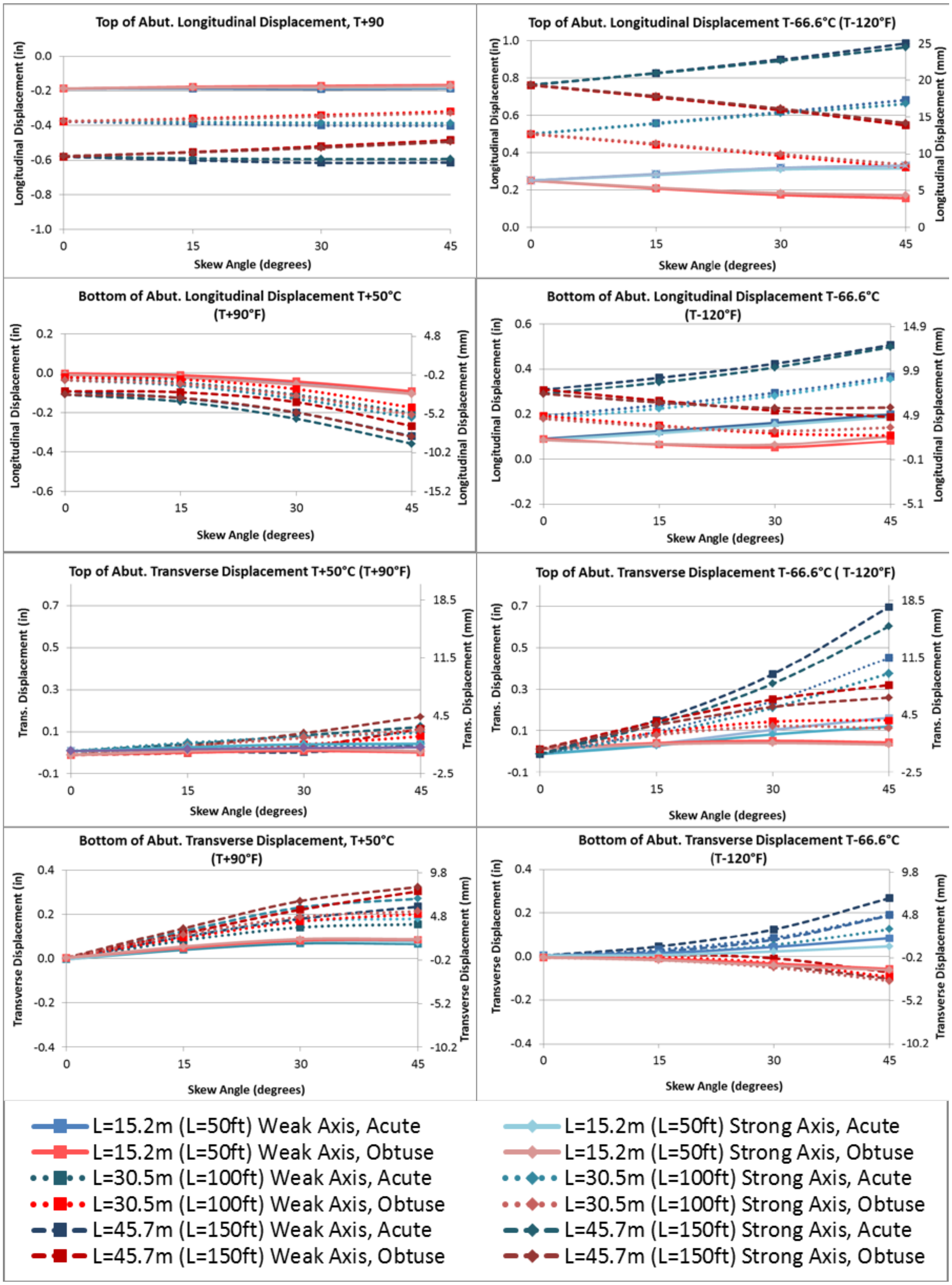


Figure 6.6: Longitudinal and transverse displacements for T+50°C (T+90°F) (Left) and T-66.7°C (T-120°F) (Right)

6.6 Pile Bending Moments

Pile moments are a critical part in the design of IABs. Depending on the design philosophy the designer may decide that pile yielding should be avoided, while others may introduce a plastic hinge at the yielded location and design accordingly. The focus of this chapter is to address the influence of factors such as pile orientation and skew on pile moments assuming minimal yielding of the piles and results are reported in terms of yield moment.

Post yield behavior is specifically not included in the analysis to keep the focus on the influence of geometry on the pile moments. Since results only include thermal load these would need to be superimposed on results from construction, dead and live load. At the completion of construction, similar monitored IABs had pile moments due to dead load and construction effects that were approximately 10 percent of the yield moment (Civjan et al. 2014). For this study it was assumed that stresses from construction, dead load and live load would be around 15 percent of the yield moment. Therefore, for the purpose of identifying cases that may result in pile yielding, the authors propose that the maximum pile moment to yield moment ratio due to thermal effects should remain below 85%.

Since these analyses do not model post yield behavior, no load redistribution with yielding occurs. All results exceeding this percentage of pile yielding are therefore not realistic, but could be realized in piles of higher yield strength or geometry. The results are meant to show trends in moments with increasing relative stiffness and orientation which is not as clear once non-linear material properties are introduced. The linear-elastic analyses are considered suitable for the purpose of examining criteria to avoid pile yielding.

Finally, a simplified interaction equation was used to compare effects of biaxial bending of the piles. The equation adds the ratio from each bending axis of the non-factored maximum pile moment from thermal analysis to the yield moment since yielding is the limiting criteria of this study. The same 85% benchmark is used to account for moments from other load effects.

6.6.1 Results: T+41.7°C (T+75°F) and T-41.7°C (T-75°F)

Bending moment results from thermal analyses of temperature increase and decrease of 41.7°C (75°F) are presented in Figure 6.7 through Figure 6.12. Figure 6.7 and Figure 6.8 present maximum positive and negative pile bending moment as a function of bridge length. Markers are also shown on the figures to signify 85% of the pile yield capacity for the three bridge lengths about the axis of bending reported.

Considering the weak axis bending moments (Figure 6.7), the weak axis orientation of piles results in minimal weak axis moment for bridge expansion and larger moments for bridge contraction that are still less than 85% of the yield moment. Piles with strong axis orientation result in decreased maximum weak axis moment for bridge contraction. From a design perspective the longitudinal movement is assumed to dominate, but this shows that transverse displacements induce significant weak axis moments. When the bridge expands, backfill restricts expansion longitudinally which results in significantly higher transverse pile moments. Therefore, strong axis pile orientation results in greater transverse displacements at the top of pile (Figure 6.5) and results in weak axis pile moments exceeding 85% of the yield moment in longer spans with 45 degree skew. Moments also approach this criterion for 30 degrees of skew.

For strong axis bending moments (Figure 6.8: the results follow a similar pattern to weak axis results. The major difference is that the pile capacity is much greater about the strong axis and resulting moments end up being a lower percentage of the yield moment. Therefore, for all conditions studied, resulting strong axis pile moments were below 85% yield capacity regardless of pile orientation. This initial data would indicate that weak axis pile orientation would reduce pile moments as a percentage of yield moment and be preferred for design, but that conclusion is specific to the case of similar increase and decrease in temperature.

Further pile bending moment comparison plots are presented in Figure 6.9: and Figure 6.10. These plots present the ratio of the maximum bending moment to the yield moment about the corresponding axis for the skews investigated. These results are shown for obtuse and acute corners. To compare the benchmark of 85% yield, a red indicator line is shown on all plots to mark this boundary. Note that the longest span has a larger pile section and corresponding yield moment capacities about each axis.

These plots more clearly show the importance of transverse moments in the bridge response. For straight structures, the pile moments are dominated by longitudinal response as expected. As the skew increases the transverse moments (strong axis moment for weak axis orientation and vice versa) increase significantly. These maximum transverse pile moments exceed moments that result from bending in the direction of the longitudinal axis for bridge skews as low as 15 degrees. The piles therefore play a significant role in restraining rotation of the bridge in plan view.

For bridge contraction, higher skew results in an increase in the longitudinal pile moments at the acute corner and a decrease at the obtuse corner (with an exception at 45 degrees for the latter). Concurrent out of plane moments are somewhat higher at the obtuse corner. For bridge expansion, the out of plane moments are nearly twice the values for similar conditions under contraction. In plane moment trends are very different from those noted for bridge contraction (as were the longitudinal displacements) with less disparity between corners and higher values at the obtuse corner. Olsen et al. (2013) reports that the acute corner is typically the critical corner to consider for design. However, the results of this study show that the pile moment in the primary direction at the acute corner is critical under bridge contraction while bridge expansion when backfill is present results in critical moment at the obtuse corner. Moments in the transverse direction are consistently slightly higher at the obtuse corner with the exception of strong axis orientation under bridge contraction.

The difference in the behavior of piles under temperature increase and decrease of the same value ±41.7°C (±75°F) is due to the effects of backfill. To verify this, a thermal analysis was run without backfill for the 45.7 m (150 ft) bridge with 30° skew as an example case to compare the bridge expansion and contraction of the same temperature with and without backfill. The resulting weak axis pile bending moment diagrams along the depth of the pile is presented in Figure 6.11. These results show that without the presence of backfill, the behavior under bridge expansion is a mirror of the behavior under bridge contraction. Therefore, for less compacted backfill conditions than assumed in this study results would fall between those reported for expansion and contraction. Olsen et al. (2013) also reported that backfill plays a substantial role in the response of IAB piles to thermal expansion. Previous research has shown that over time, backfill behavior in IABs may significantly change (Civjan et al. 2013) (Civjan et al. 2014). With cyclic seasonal thermal loading data from monitored bridges indicated soil properties similar to loose backfill or no backfill at all, while initial field conditions had dense backfill. Therefore, rather than assuming that the initial backfill condition will remain constant , considering situations with and without backfill and using these results as boundary values may be appropriate in determining the optimal pile design.

When biaxial bending was evaluated using the interaction equation previously described, it was determined that the sections only exceeded 85 percent of yield capacity under expansion of 41.7°C (75°F). The critical cases under temperature increase occurred when strong axis orientation was used resulting in excessive moments for both corners in the 30.5 m (100 ft) and 45.7 m (150 ft) bridges with 45 degree skew , as well as at the obtuse corner for the 115.2 m (50 ft) bridge at 30 degrees.

A two-span analysis of the 30.5 m (100 ft) bridge was conducted by putting roller supports at each girder node at mid-span of the bridge. The results of this study showed that multiple span IAB behavior differs significantly from that of single span behavior. The difference in behavior occurs along the longitudinal axis. Transverse bending of the piles in the two-span bridge was similar to the single span bridge but slightly greater in magnitude.

Overall, for the bridges considered under an equal temperature increase and decrease of ±41.7°C (±75°F), weak axis orientation of the piles is less likely to result in pile yielding. Under temperature increase piles at both acute and obtuse corners have lower bending moment stresses with weak axis orientation. Under temperature decrease, with increasing skew angles the preferable pile orientation differs between the acute and obtuse corner as a result of the twisting of the bridge in plan view (about the perspective point); this difference is greater with increasing bridge lengths. Regardless of pile orientation the critical moments were found to be about the weak axis of the pile. When backfill is neglected, results show that the performance under expansion is a mirror of the performance under contraction. The results of comparing an equal increase and decrease in temperature highlight the impact of backfill conditions on pile response and the importance of considering transverse moments for bridges with a skew.

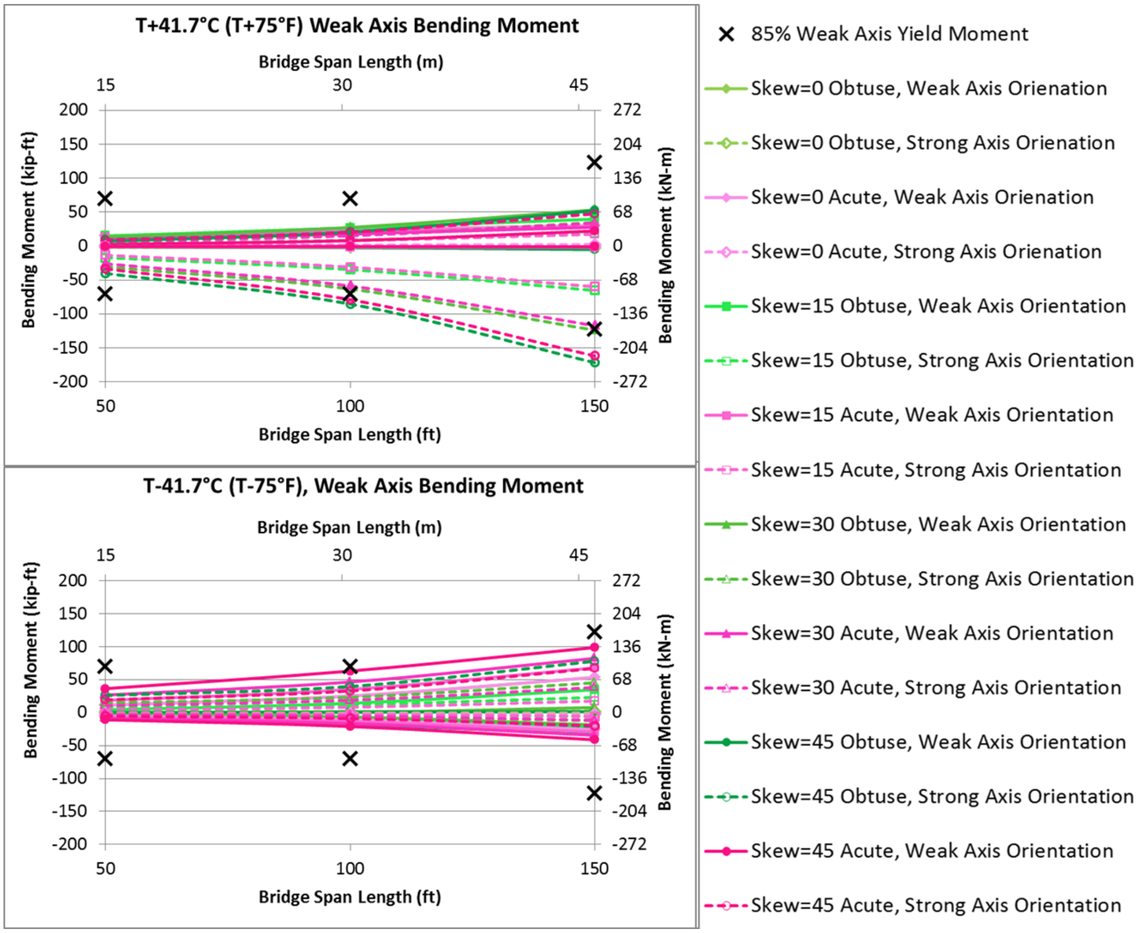


Figure 6.7: Weak axis bending moments for T+41.7°C (T+75°F) (Top) and T-41.7°C (T-75°F) (Bottom)

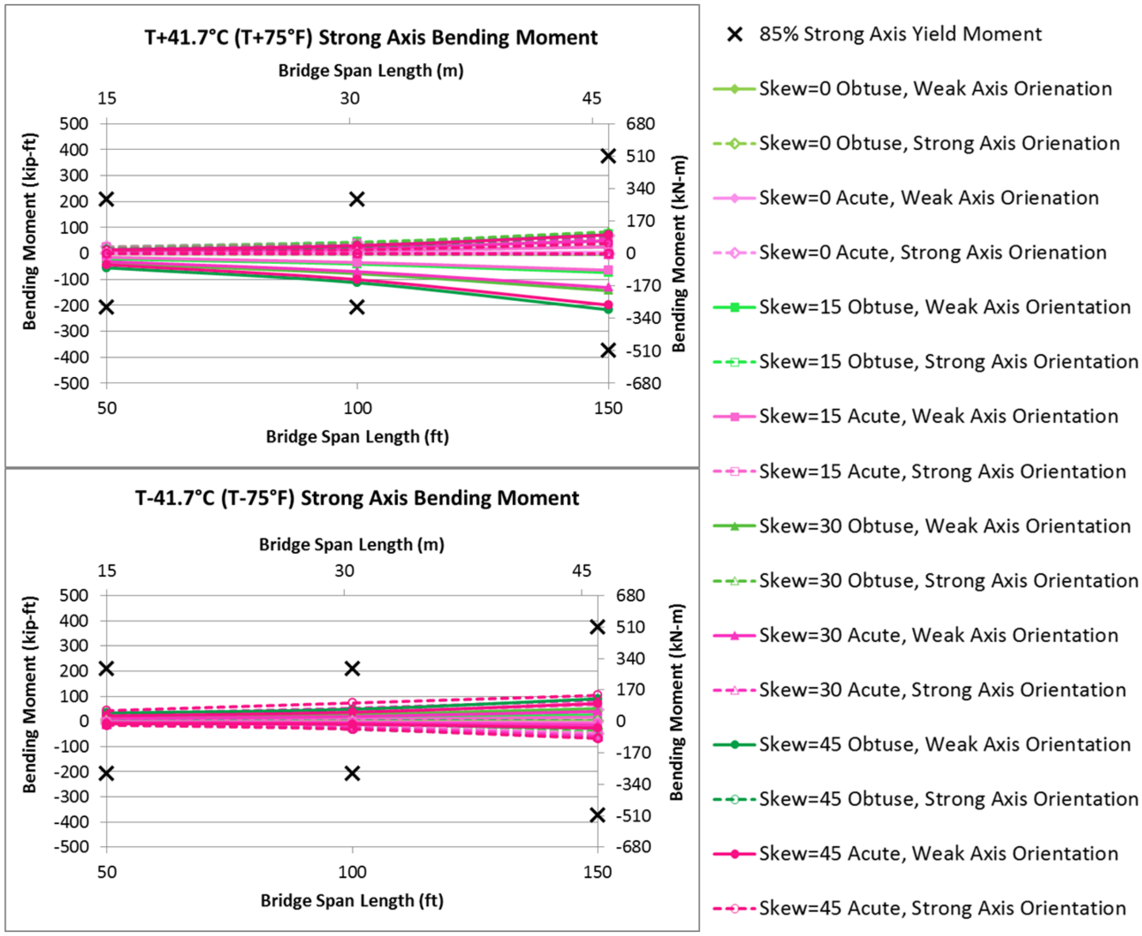


Figure 6.8: Strong axis bending moments for T+41.7C (T+75°F) (Top) and T-41.7C (T-75°F) (Bottom)

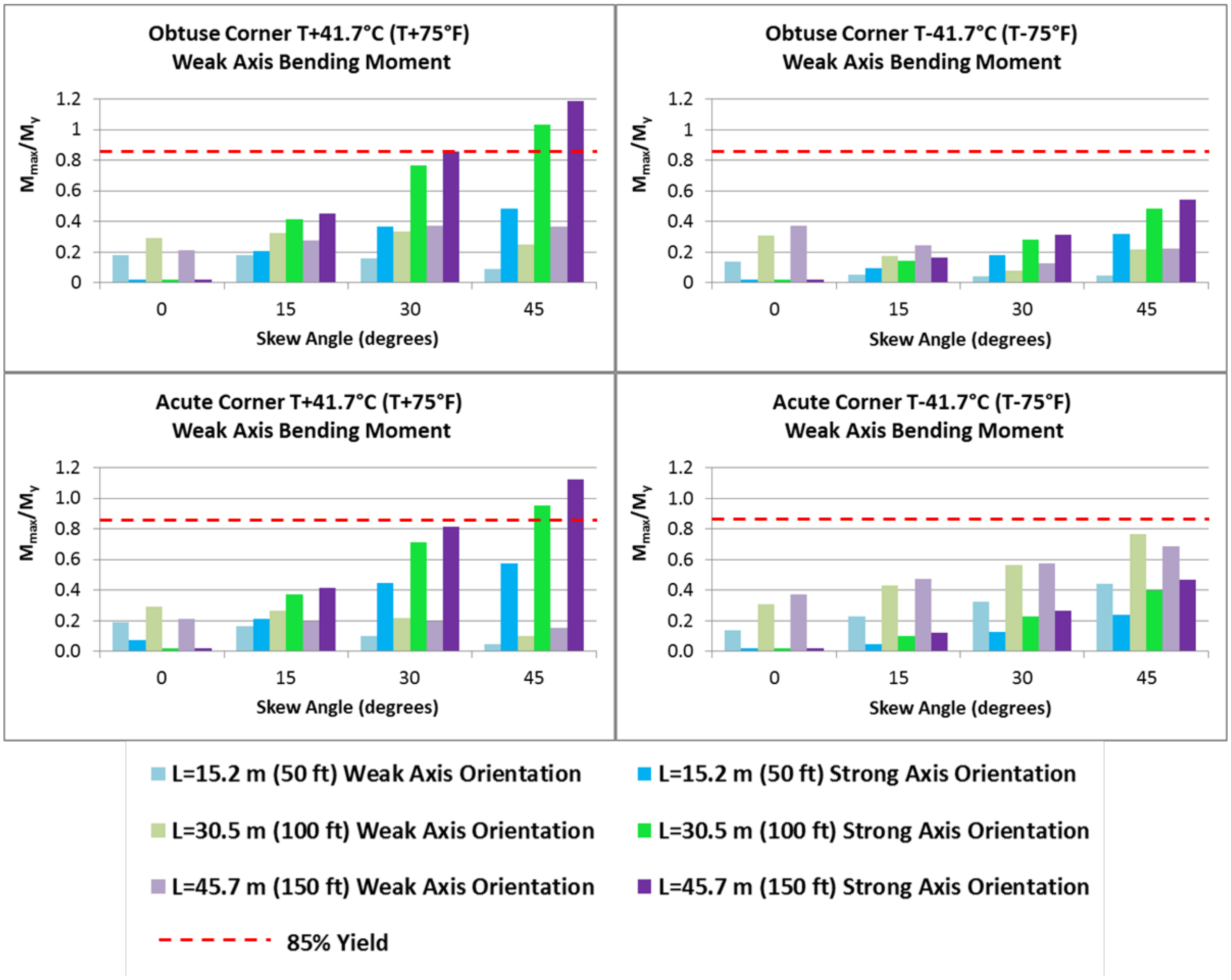


Figure 6.9: Ratio of maximum weak axis bending moment to yield moment for T+41.7°C (T+75°F) (Left) and T-41.7°C (T-75°F) (Right)

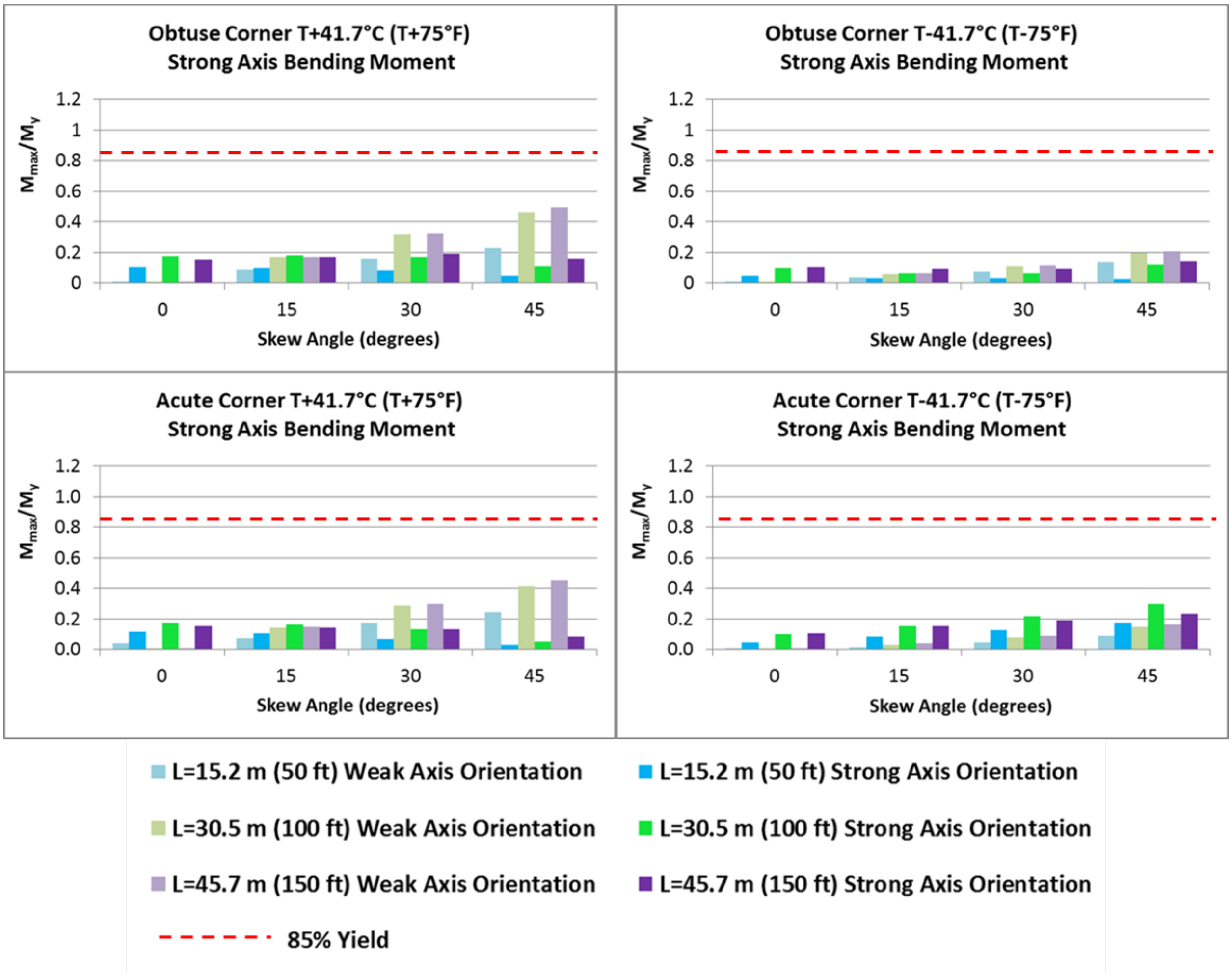


Figure 6.10: Ratio of maximum strong axis bending moment to yield moment for T+41.7°C (T+75°F) (Left) and T-41.7°C (T-75°F) (Right)

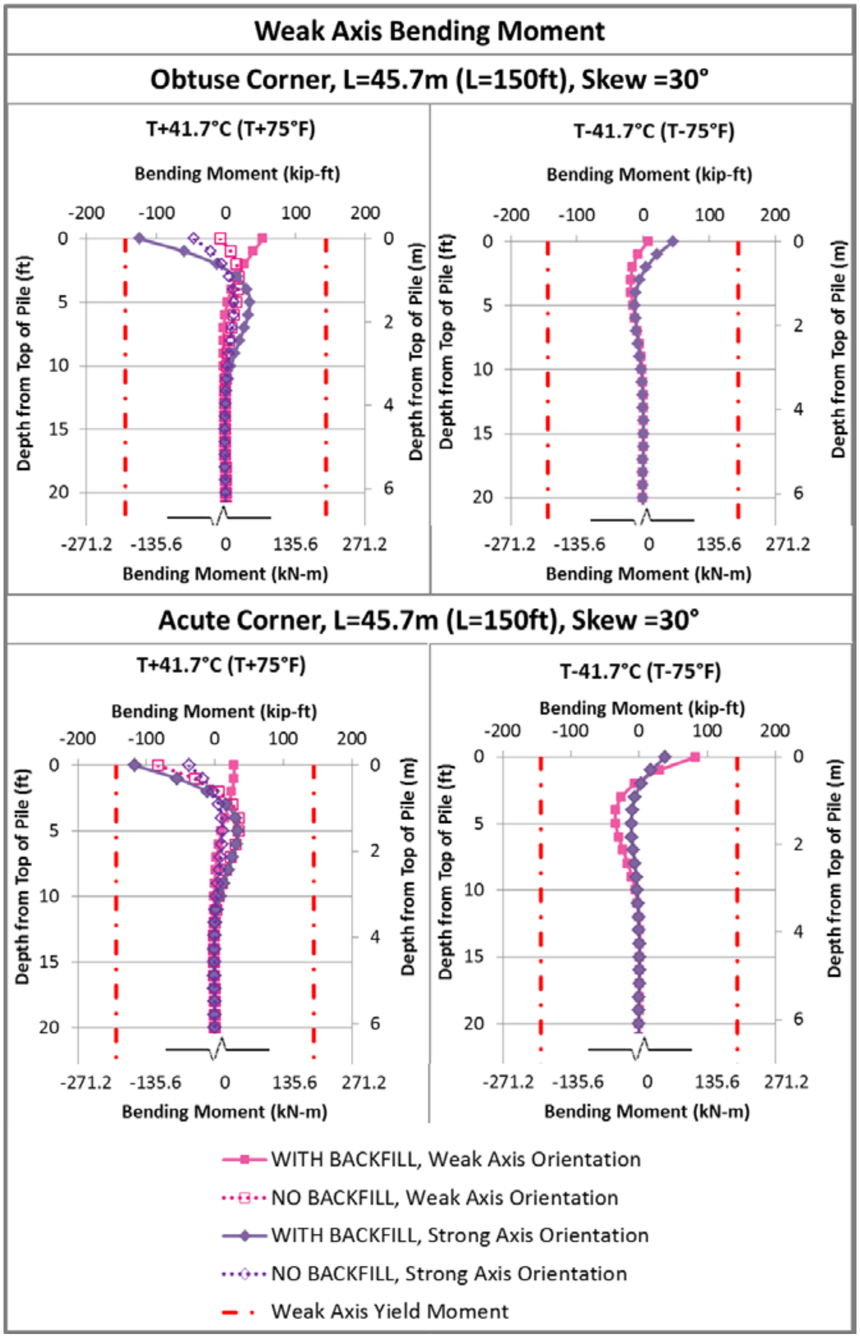


Figure 6.11: Weak axis bending moment diagram along depth of pile comparing behavior under temperature increase with backfill and without backfill present

6.6.2 Results: T+50°C (T+90°F) and T-66.7°C (T-120°F)

Data with the same format as the previous section were created for the thermal results of T+50°C (T+90°F) and T-66.7°C (T-120°F) and shown in Figure 6.12 and Figure 6.13. The relationship between maximum bending moment and temperature change is essentially linear which shows the soil is responding linearly; this was determined by taking the ratio of maximum moment to temperature change for each corner, bridge length, and skew angle. For a given bridge length at a specified corner, the difference in this ratio between the skew angles is within 10 percent; while directly comparing the ratio at a given skew angle the difference is less than 5 percent. Therefore, if another temperature change is of interest, the maximum moment can be approximated.

The results show that optimal pile orientation is largely dependent on expected thermal range and construction temperature. For bridges that will experience a significant increase in temperature (either constructed at a lower temperature or in a region with mild winters), weak axis pile orientation is preferable. The IABs analyzed for this study showed that lengths up to 45.7 m (150 ft), with skews up to 45° can be accommodated for significant temperature increases when weak axis orientation is used. For bridges that will also experience a significant decrease in temperature, similar weak axis pile moments can occur for either orientation of the piles. Neither pile orientation can be said to consistently reduce pile moments for the bridges and temperature ranges considered. Strong axis bending moments presented in Figure 6.13 show that no pile yielding is expected about the strong axis for any bridge lengths, skew angles, or pile orientation analyzed.

When the interaction of biaxial bending was considered, the results show combined moments exceeding 85 percent yield for both temperature increase and temperature decrease. For temperature increase, strong axis orientation results in yielding in both corner piles of the 30.5 m (100 ft) and 15.2 m (50 ft) bridges for both 30° and 45° skew; weak axis orientation only results in yield at the obtuse corner of the 15.2 m (50 ft) bridge with 45° skew. Similar results are seen under temperature decrease of 120°F where strong axis orientation results in yielding at both corners of the longer bridges with 45° skew. With weak axis orientation, yielding occurs only at the acute corner in the longer bridges but in this case occurs for the 30° skew as well. Therefore, while yielding can be observed with both orientations, the strong axis orientation is more critical since both corners are exceeding yield at the same time. On the other hand, while weak axis orientation can also result in yield, this is concentrated at one corner with the other corner well below yield. In cases where a designer accounts for pile yielding an inelastic analysis would be necessary to determine how forces are redistributed and is worth further study.

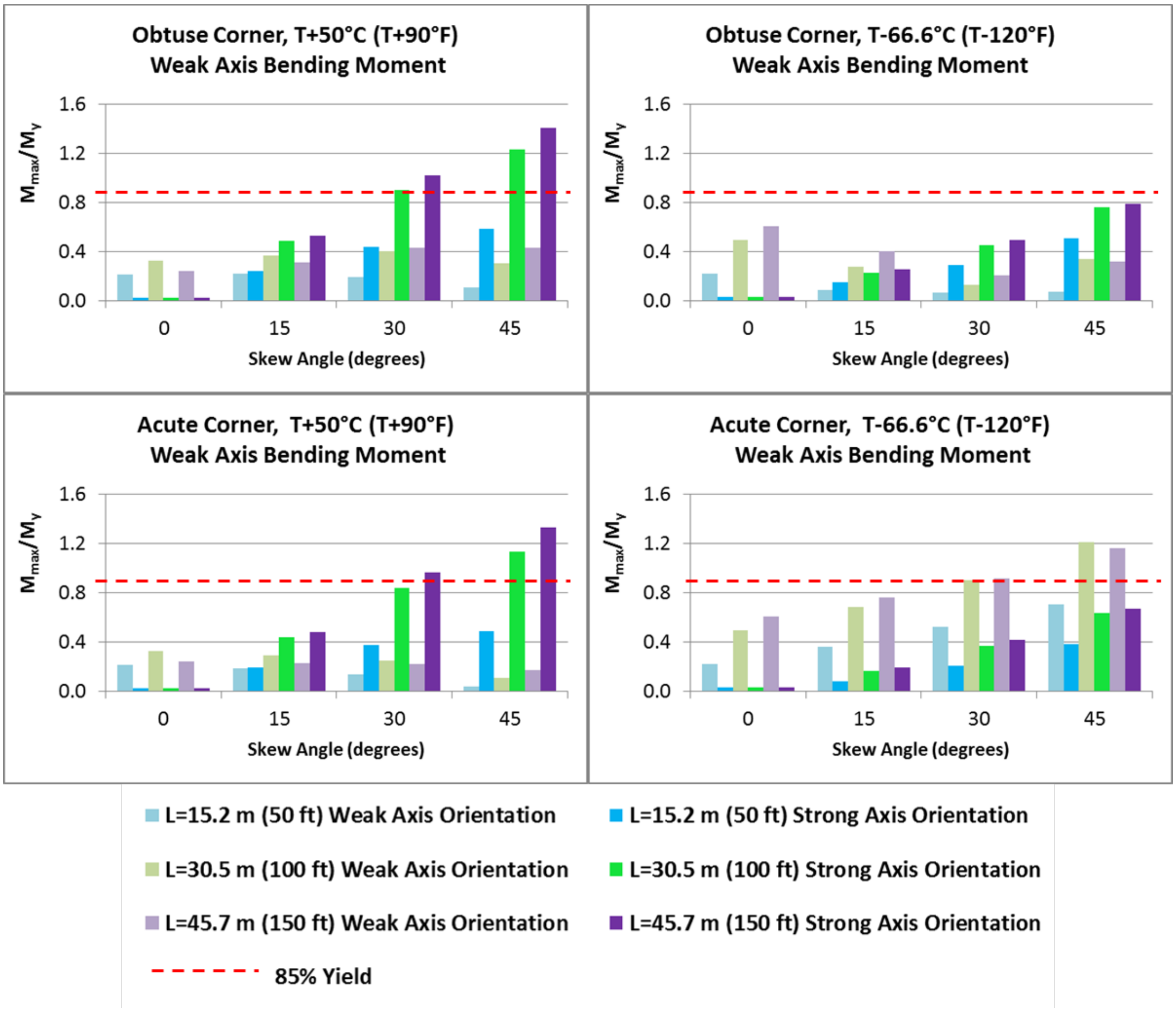


Figure 6.12: Ratio of Maximum Weak Axis Bending Moment to Yield Moment for T+50°C (T+90°F) (Left) and T-66.7°C (T-120°F) (Right)

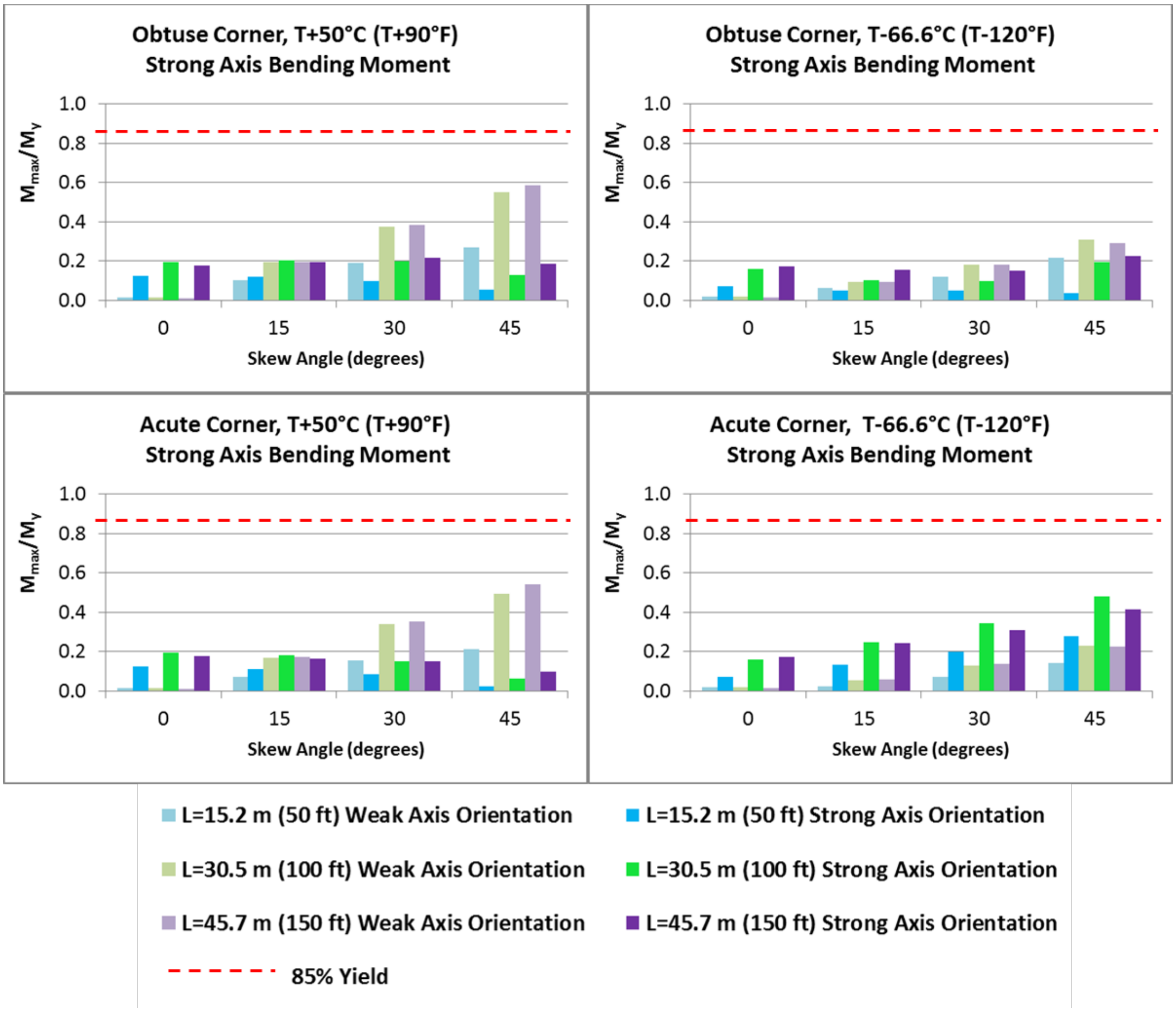


Figure 6.13: Ratio of Maximum Strong Axis Bending Moment to Yield Moment for T+50°C (T+90°F) (Left) and T-66.7°C (T-120°F) (Right)

6.7 Summary and Conclusions

This chapter reported results from a parametric study on effects of pile orientation on single span IABs. The results of this study are specific to the bridge geometry and assumptions considered. A two-span analysis showed that results differ greatly from single span IABs and results from this analysis would not be applicable to multiple spans. The factors considered in this study were bridge lengths of 15.2 m (50 ft), 30.5 m (100 ft), and 45.7 m (150 ft) with skew angles of 0°, 15°, 30°, and 45°. Two pile orientations were considered: pile web parallel with abutment centerline (weak axis orientation) and pile web perpendicular to abutment centerline (strong axis orientation). All bridge section and pile sizes were based on existing designs. Thermal analyses were first conducted doing a direct comparison of an equal temperature increase and decrease of 41.7°C (75°F), followed by thermal analyses using extreme thermal ranges with temperature increase of 50°C (90°F) and temperature decrease of 66.7°C (120°F). The analyses performed for this research only included thermal loads in order to directly compare the effects of temperature changes, and would need to be superimposed on construction, dead and live load results. The authors were specifically interested in evaluating design conditions that would lead to pile yielding under service load. Further study on post-yield redistribution of pile forces would be useful but was not included in this study.

Pile orientation was found to have little effect on longitudinal bridge displacements. Pile and backfill restraint resulted in abutment rotation and less displacement at the bottom of the abutment. Top of abutment displacement could be estimated with calculations based on the coefficient of thermal expansion. The abutment rotation was minimally affected by the pile orientation. Pile orientation did effect transverse displacements. The resulting pile moment was dependent on the pile stiffness. For straight bridges it was found that a strong axis pile orientation resulted in a lower percentage of yield moment in the pile, though the weak axis orientation resulted in a lower value of moment and piles would not be expected to yield for either orientation. The latter may be preferable to minimize force transfer into the abutment and avoid the potential for concrete cracking.

When skew was introduced the critical moments were about the weak axis of the pile, even when piles were oriented about their strong axis. Significant transverse moments in the piles were introduced due to transverse displacements resulting from plan rotation of the bridge and should be considered in design. The likelihood of pile yielding and ideal pile orientation is greatly dependent on the construction temperature which would define the expected maximum thermal increase and decrease. For symmetric temperature increase and decrease the weak axis pile orientation would be less likely to result in pile yielding while for the non-symmetric values considered there was no clear advantage to either pile orientation. For all cases the strong axis moment was not expected to cause yielding of the pile.

While previous studies states the acute corner pile is the critical pile (6), this was not the case under expansion when dense backfill was present. With the presence of backfill, the obtuse corner was the critical pile under expansion while the acute pile was the critical pile under contraction. Behavior under temperature increase was notably different than that observed under a temperature decrease due to influence of the backfill. When backfill materials were neglected the expansion and contraction response was similar. A difference noted between the identical temperature increase and decrease was that under bridge expansion the acute and obtuse corners have similar behavior, however, under bridge contraction the behavior at the corners differs. A simple interaction equation was used to determine the effects of bi-axial bending of the piles; when yielding occurred under contraction at the acute pile (weak axis orientation) the obtuse pile was well below yield but when yielding occurred under expansion (strong axis orientation) both corners either yielded or were close to yielding which would reduce the ability for forces to redistribute.

The results of this study highlight factors that should be considered in designing single span IABs and determining the optimal pile orientation to be used. The appropriate pile orientation and design depends on backfill conditions, construction temperature and maximum and minimum design temperatures. While an exact construction temperature is not necessary, having an estimate of the temperature is important in order to analyze potential maximum increase and decrease in temperature. In general it is recommended to conduct a three dimensional FEM to account for transverse pile moments from skew effects, which are especially critical for bridges with skews of greater than 15 degrees. The presence of wingwalls and soil friction will likely reduce the reported moments and should also be incorporated into an analysis when they are present in a design. Due to the influence of backfill on bridge expansion the designer should consider bounding these properties to account for potential variations in soil response during the life of the structure. Overall, choosing the optimal pile orientation should be done in consideration of these factors and understanding how these factors will affect pile response could potentially result in allowing greater IAB lengths or skew angles.

CHAPTER 7

CONCLUSION

Integral abutment bridges have become the highway bridge of choice across the country. Traditional jointed bridges have many issues associated with bridge joints that have resulted in the preference of eliminating bridge joints whenever possible. IABs offer lower construction costs and lower maintenance costs while remaining in service for longer periods of time. Despite the popularity of IABs, there are no uniform design standards and designs are often left up to individual state's experience. Design standards vary across the country for maximum span length, skew angle, curvature, pile orientation, among other design considerations.

The Vermont Agency of Transportation (VTrans) initiated a program of field instrumentation and analysis to evaluate the performance of three IABs beginning at construction, and monitored for over five years. The bridges are of increasing complexity, a straight girder non-skew bridge, a straight girder 15 degree skew bridge, and a curved girder two-span continuous structure with 11.25 degrees of curvature. Three-dimensional finite element models (FEMs) of the three bridges were created using SAP2000.

This dissertation presented a comparison of the long-term performance from over five years of data of the three instrumented IABs. The results highlighted that the straight single-span bridge exhibited the greatest variation in field response from what would be expected in design highlighting the complex nature of soil-structure interaction and the difficulties in predicting the response of IABs. The two-span Stockbridge Bridge with

11.25 degrees of curvature and 221 ft (67.6 m) total length had the most consistent year to year response, and did not exhibit any behavior that significantly differed from a straight bridge response. This bridge did include Geofoam behind the abutments which likely contributed to its favorable response. The 15 degree skew of the East Montpelier Bridge did not affect longitudinal displacements, which could be approximated assuming a straight bridge response. The responses that were affected by skew were transverse displacements which resulted in seasonal rotation of the bridge and strong axis bending moments greater than or equal to those about the weak axis. Earth pressure distribution in skewed abutments also differs from a straight bridge response where passive earth pressures are highest behind the obtuse corner of the bridge. Designing the abutments for fully passive pressure accounted for the build-up of pressure that occurred due to skew.

The comparison of nominal FEMs to field data showed that models assuming soil conditions based on soil boring logs and compacted backfill assumptions did not accurately predict field response at any of the bridges. Matched models were calibrated to field response. Soil conditions in the matched models suggest that soil does not provide significant restraint under expansion which could indicate a softening of soil response upon cyclic thermal loading. Thermal analyses corresponding to field data did not accurately match displacements observed in the field as the static thermal analysis does not account for the shift towards the backfill observed at the bridges over time.

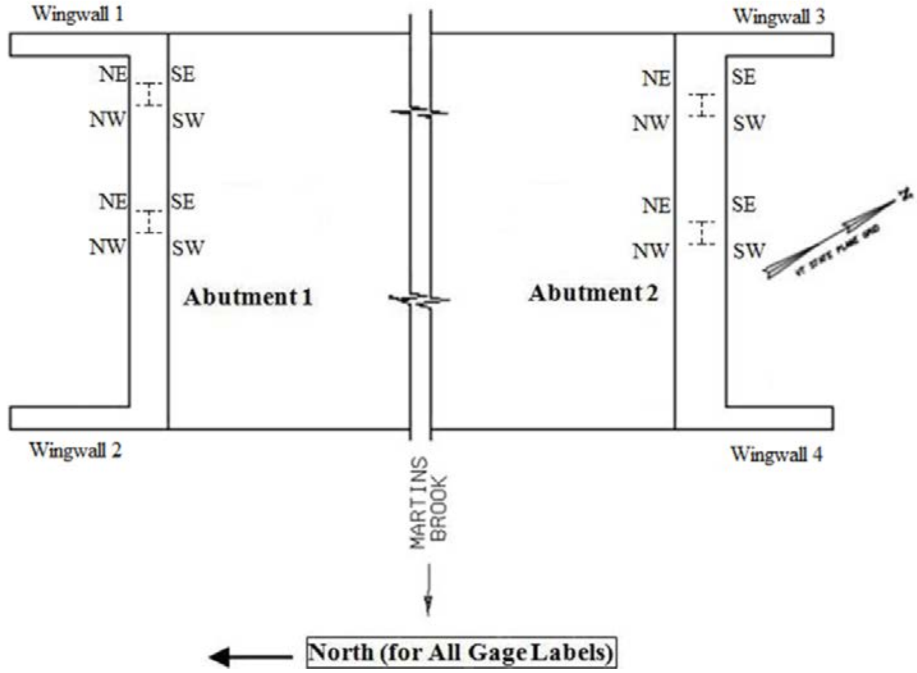
There is no consensus on preferred orientation of piles in IABs. The significant transverse bending in the 15 degree skew bridge highlighted the importance of understanding the effects of pile orientation in skewed bridges. A parametric study was presented which analyzed varying lengths and skew angles of single span IABs under multiple thermal loads with both weak axis and strong axis orientation. The purpose of the parametric study was to determine which pile orientation is optimal for avoiding pile yielding. Results of the study show there is not one optimal pile orientation but rather optimal orientation depends on other factors. Beyond the effects of length and skew angle, choice of orientation should consider expected temperature range, construction temperature, and backfill conditions. It is recommended to conduct a three dimensional FEM to account for transverse pile moments from skew effects, which are especially critical for bridges with skews of greater than 15 degrees.

While IABs are the highway bridge of choice, they cannot always be constructed due to limitations in design. Furthermore, the change from traditional jointed bridges to IABs takes time. Therefore, it is important to understand the problems associated with bridge joints and best practices in order to improve the performance of bridge expansion joints and avoid the costly damage associated with the failure of joints. Interviews with DOT personnel and survey responses from nine states in and around New England were compiled to determine what factors have the greatest impact on joint performance. While all states would prefer to have a preventative maintenance program, this is not a feasible option for most states. Definitions of a successful joint varied widely resulting in a lack of consensus of the most successful joints. However, open joints were generally rated poorly as these joint types tend to result in costly corrosion damage when not functioning properly. The most important factor effecting joint performance, according to the survey, is installation practice. Training of DOT personnel was recommended to ensure proper installation techniques.

APPENDIX A

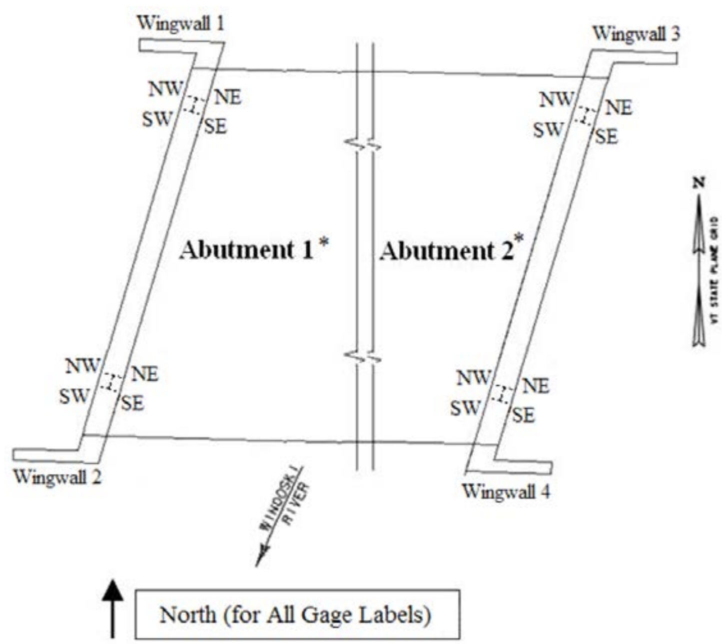
GAGE LABELING

Gage labeling is not directly related to abutment numbers for a couple of logistical reasons. First, instrumentation planning took place prior to development of structural drawings for two bridges. Therefore, final labeling of Abutment 1 and Abutment 2 was not determined, nor were compass directions (North-South-East-West) known by the research team. Gage labels starting with "1" indicated the abutment nearest the datalogger, which was typically the more heavily instrumented abutment to minimize cable lengths. Second, during construction a few gages were interchanged due to contractor preference for multiplexer locations. This avoided the need for splicing of predetermined cable lengths for these gages. For these reasons labeling of gages are not always consistent with the directions and numbering indicated in the structural drawings. This section describes the final as-built locations of gages for each bridge. Gages are labeled according to directions and abutment numbering. Differences from structural drawing callouts are noted. Gage labels are described per structural drawing position in Appendix B, along with channel locations in each multiplexer and datalogger.



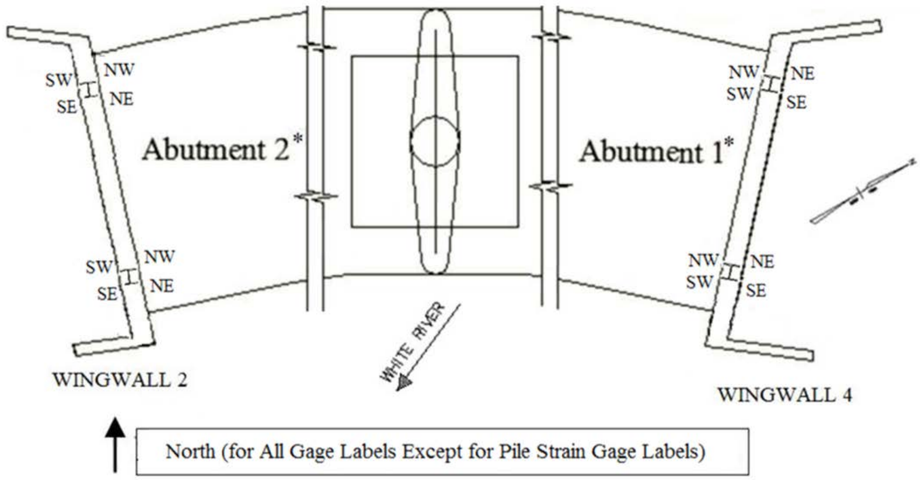
*Abutment numbering is consistent with structural drawings

Figure A-0.1: Gage Labeling for Middlesex Bridge



*Abutment numbering is consistent with structural drawings

Figure A-0.2: Gage Labeling for East Montpelier Bridge



*Abutment numbering is NOT consistent with structural drawings

Figure A-0.3: Gage Labeling for Stockbridge Bridge

APPENDIX B

INSTRUMENTATION CHANNEL LISTS & GAGE LOCATIONS

As noted in Appendix A, gage labeling is not always consistent with bridge orientation and structural drawing callouts. This section describes the final as-built locations of gages for each bridge. Gages are labeled according to directions and abutment numbering presented in Appendix A. Gage labels are fully described by data acquisition multiplexer channel number and descriptive location related to structural drawings.

Data acquisition system is composing of 16-channel multiplexers and 6-channel dataloggers. Each VW gage is connected to a single multiplexer channel while each MEMS gage (biaxial gage) is connected to two different channels.

a) Middlesex Bridge

As-built gage information for the Middlesex Bridge is given in Table B-0.1 and Table B-0.2 for Abutment 1and Abutment 2 multiplexers, respectively.

Table B-0.1: Abutment 1 Multiplexer Channel List and Gage Locations

Mux	Channel	Gage Label	Gage Type	Model	Location
1	1	CM-1ET	Disp. Transducer	4420	Abutment 1, Upstream, Transverse, 1.95 m (6.43 ft) below construction joint
1	2	CM-1E	Disp. Transducer	4420	Abutment 1, Upstream, Longitudinal, 1.95 (6.43 ft) below construction joint
1	3	P-1CT	Earth Pressure Cell	4815	Abutment 1, Center of Abutment, 1.9 m (6.2 ft) above the bottom of abutment,
1	4	P-1CM	Earth Pressure Cell	4815	Abutment 1, Center of Abutment, 1.1 m (3.6 ft) above the bottom of abutment,
1	5	P-1CB	Earth Pressure Cell	4815	Abutment 1, Center of Abutment, 0.3 m (1.0 ft) above the bottom of abutment,
1	6	P-1EM*	Earth Pressure Cell	4815	Abutment 1, Downstream, 1.1 m (3.6 ft) above the bottom of abutment, 0.8 m (2.6 ft) away from abutment-wingwall connection
1	7	P-1EB1*	Earth Pressure Cell	4815	Abutment 1, Downstream, 0.3 m (1.0 ft) above the bottom of abutment, 0.8 m (2.6 ft) away from abutment-wingwall connection
1	8	P-1WM1*	Earth Pressure Cell	4815	Abutment 1, Upstream, 1.1 m (3.6 ft) above the bottom of abutment, 0.8 m (2.6 ft) away from abutment-wingwall connection
1	9	P-1WB1*	Earth Pressure Cell	4815	Abutment 1, Upstream, 0.3 m (1.0 ft) above the bottom of abutment, 1.0 m (3.3 ft) away from abutment-wingwall connection
1	10	SGG-1E-TE	Girder Strain Gage	4050	Upstream Girder, Upstream-Top Flange, 4.5 m (14.75 ft) from the end of steel girder section at Abutment 1.
1	11	SGG-1E-BE	Girder Strain Gage	4050	Upstream Girder, Upstream-Bottom Flange, 4.5 m (14.75 ft) from the end of steel girder section at Abutment 1.
1	12	SGG-1E-TW	Girder Strain Gage	4050	Upstream Girder, Downstream-Top Flange, 4.5 m (14.75 ft) from the end of steel girder section at Abutment 1.
1	13	SGG-1E-BW	Girder Strain Gage	4050	Upstream Girder, Downstream-Bottom Flange, 4.5 m (14.75 ft) from the end of steel girder section at Abutment 1.
1	14	SGG-1M-TE	Girder Strain Gage	4050	Interior Girder, Upstream-Top Flange, 4.5 m (14.75 ft) from the end of steel girder section at Abutment 1.
1	15	SGG-1M-BE	Girder Strain Gage	4050	Interior Girder, Upstream-Bottom Flange, 4.5 m (14.75 ft) from the end of steel girder section at Abutment 1.
1	16	SGG-1W-TE	Girder Strain Gage	4050	Downstream Girder, Upstream-Top Flange,

* Gage label is not consistent with other callouts. Gage locations switched to match cable length with actual multiplexer location.

					4.5 m (14.75 ft) from the end of steel girder section at Abutment 1.

Table B-1: Abutment 1 Multiplexer Channel List and Gage Locations (cont.)

Mux	**Channel**	**Gage Label**	**Gage Type**	**Model**	**Location**
2	1	SGG-1W-BE	Girder Strain Gage	4050	Downstream Girder, Upstream-Bottom Flange, 4.5 m (1.6 ft) from the end of steel girder section at Abutment 1.
2	2	SGG-0E-TE	Girder Strain Gage	4050	Upstream Girder, Upstream-Top Flange, 22.7 m (74.5 ft) from the end of steel girder section at Abutment 1.
2	3	SGG-0E-BE	Girder Strain Gage	4050	Upstream Girder, Upstream-Bottom Flange, 22.7 m (74.5 ft) from the end of steel girder section at Abutment 1.
2	4	SGG-0E-TW	Girder Strain Gage	4050	Upstream Girder, Downstream-Top Flange, 22.7 m (74.5 ft) from the end of steel girder section at Abutment 1.
2	5	SGG-0E-BW	Girder Strain Gage	4050	Upstream Girder, Downstream-Bottom Flange, 22.7 m (74.5 ft) from the end of steel girder section at Abutment 1.
2	6	TM-1M	Tiltmeter (Uniaxial)	6300	Abutment 1, Center of Abutment, 0.5 m (1.6 ft) from the bottom of middle girder.
2	7	IN-1E-1	Inclinometer (Uniaxial)	6300	Abutment 1, Pile below Girder 2, Flange facing towards the centerline of the roadway, 0.0-0.6 m (0-2 ft) below the bottom of Abutment 1.
2	8	IN-1E-2	Inclinometer (Uniaxial)	6300	Abutment 1, Pile below Girder 2, Flange facing towards the centerline of the roadway, 0.6-1.5 m (2-5 ft) below the bottom of Abutment 1.
2	9	IN-1E-3	Inclinometer (Uniaxial)	6300	Abutment 1, Pile below Girder 2, Flange facing towards the centerline of the roadway, 1.5-2.7 m (5-9 ft) below the bottom of Abutment 1.
2	10	IN-1E-4	Inclinometer (Uniaxial)	6300	Abutment 1, Pile below Girder 2, Flange facing towards the centerline of the roadway, 2.7-4.0 m (9-13 ft) below the bottom of Abutment 1.
2	11	IN-1E-5	Inclinometer (Uniaxial)	6300	Abutment 1, Pile below Girder 2, Flange facing towards the centerline of the roadway, 4.0-5.2 m (13-17 ft) below the bottom of Abutment 1.
2	12	IN-1W-1	Inclinometer (Uniaxial)	6300	Abutment 1, Pile below Girder 4, Flange facing towards the centerline of the roadway, 0.0-0.6 m (0-2 ft) below the bottom of Abutment 1.
2	13	IN-1W-2	Inclinometer (Uniaxial)	6300	Abutment 1, Pile below Girder 4, Flange facing towards the centerline of the roadway, 0.6-1.5 m (2-5 ft) below the bottom of Abutment 1.
2	14	IN-1W-3	Inclinometer (Uniaxial)	6300	Abutment 1, Pile below Girder 4, Flange facing towards the centerline of the roadway, 1.5-2.7 m (5-9 ft) below the bottom of Abutment 1.
2	15	IN-1W-4	Inclinometer (Uniaxial)	6300	Abutment 1, Pile below Girder 4, Flange facing towards the centerline of the roadway, 2.7-4.0 m (9-13 ft) below the bottom of Abutment 1.
2	16	P-R	Reference Pressure Cell	4815	Under Approach Slab on Abutment 1 side, Facing towards the centerline of the roadway, 6 m (19.7 ft) away from the bridge end, 3.3 m (10.8 ft) away the centerline of roadway (towards upstream), 1.0 m (3.0 ft) below approach slab,

Table B-1: Abutment 1 Multiplexer Channel List and Gage Locations (cont.)

Mux	Channel	Gage Label	Gage Type	Model	Location
3	1	SG-1M-1NE	Pile Strain Gage	4000	Middle Pile (below Girder 3), 0.5 m (1.6 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.
3	2	SG-1M-1SE	Pile Strain Gage	4000	Middle Pile (below Girder 3), 0.5 m (1.6 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.
3	3	SG-1M-1SW	Pile Strain Gage	4000	Middle Pile (below Girder 3), 0.5 m (1.6 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.
3	4	SG-1M-1NW	Pile Strain Gage	4000	Middle Pile (below Girder 3), 0.5 m (1.6 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.
3	5	SG-1M-3NE	Pile Strain Gage	4000	Middle Pile (below Girder 3), 1.5 m (4.9 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.
3	6	SG-1M-3SE	Pile Strain Gage	4000	Middle Pile (below Girder 3), 1.5 m (4.9 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.
3	7	SG-1M-3SW	Pile Strain Gage	4000	Middle Pile (below Girder 3), 1.5 m (4.9 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.
3	8	SG-1M-3NW	Pile Strain Gage	4000	Middle Pile (below Girder 3), 1.5 m (4.9 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.
3	9	SG-1E-1NE	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 0.5 m (1.6 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.
3	10	SG-1E-1SE	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 0.5 m (1.6 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.
3	11	SG-1E-1SW	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 0.5 m (1.6 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.
3	12	SG-1E-1NW	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 0.5 m (1.6 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.
3	13	SG-1E-2NE	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 1.0 m (3.3 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.
3	14	SG-1E-2SE	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 1.0 m (3.3 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.
3	15	SG-1E-2SW	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 1.0 m (3.3 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.
3	16	SG-1E-2NW	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 1.0 m (3.3 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.

Table B-1: Abutment 1 Multiplexer Channel List and Gage Locations (cont.)

Mux	Channel	Gage Label	Gage Type	Model	Location
6	1	SG-1E-3NE	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 1.5 m (4.9 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.
6	2	SG-1E-3SE	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 1.5 m (4.9 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.
6	3	SG-1E-3SW	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 1.5 m (4.9 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.
6	4	SG-1E-3NW	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 1.5 m (4.9 ft) from bottom of Abutment 1 77 mm (3.0 in) from the edge of flange.

Table B-0.2: Abutment 2 Multiplexer Channel List and Gage Locations

Mux	Channel	Gage Label	Gage Type	Model	Location
4	1	CM-2ET	Displacement Transducer	4420	Abutment 2, Upstream, Transverse, 1.95 m (6.43 ft) below construction joint
4	2	CM-2E	Displacement Transducer	4420	Abutment 2, Upstream, Longitudinal, 1.95 (6.43 ft) below construction joint
4	3	P-2CT	Earth Pressure Cell	4815	Abutment 2, Center of Abutment, 1.9 m (6.2 ft) above the bottom of abutment, 0.8 m (2.6 ft) away from abutment-wingwall connection
4	4	P-2CB	Earth Pressure Cell	4815	Abutment 2, Center of Abutment, 0.3 m (1.0 ft) above the bottom of abutment, 0.8 m (2.6 ft) away from abutment-wingwall connection
4	5	P-2EB	Earth Pressure Cell	4815	Abutment 2, Upstream, 0.3 m (1.0 ft) above the bottom of abutment, 0.8 m (2.6 ft) away from abutment-wingwall connection
4	6	P-2WB	Earth Pressure Cell	4815	Abutment 2, Downstream, 0.3 m (1.0 ft) above the bottom of abutment, 0.8 m (2.6 ft) away from abutment-wingwall connection
4	7	PW-1E	Earth Pressure Cell	4815	Wingwall 3 (Abutment 2, Upstream), 1.2 m (3.9 ft) above the bottom of wingwall, 1 m (3 ft) away from abutment-wingwall connection
4	8	SGG-2E-TW	Girder Strain Gage	4050	Upstream Girder, Downstream-Top Flange, 4.5 m (14.75 ft) from the end of steel girder section at Abutment 2.
4	9	SGG-2E-BW	Girder Strain Gage	4050	Upstream Girder, Downstream-Top Flange, 4.5 m (14.75 ft) from the end of steel girder section at Abutment 2.
4	10	SGG-0M-TE	Girder Strain Gage	4050	Center Girder, Upstream-Top Flange, 22.7 m (74.5 ft) from the end of steel girder section at Abutment 1.
4	11	SGG-0M-BE	Girder Strain Gage	4050	Center Girder, Upstream-Top Flange, 22.7 m (74.5 ft) from the end of steel girder section at Abutment 1.
4	12	SGG-0W-TE	Girder Strain Gage	4050	Downstream Girder, Upstream-Top Flange, 22.7 m (74.5 ft) from the end of steel girder section at Abutment 1.
4	13	SGG-0W-BE	Girder Strain Gage	4050	Downstream Girder, Upstream-Bottom Flange, 22.7 m (74.5 ft) from the end of steel girder section at Abutment 1.
4	14	SG-2M-1NE	Pile Strain Gage	4000	Middle Pile (below Girder 3), 0.5 m (1.6 ft) from bottom of Abutment 2 77 mm (3.0 in) from the edge of flange.
4	15	SG-2M-1SE	Pile Strain Gage	4000	Middle Pile (below Girder 3), 0.5 m (1.6 ft) from bottom of Abutment 2 77 mm (3.0 in) from the edge of flange.

Table B-2: Abutment 2 Multiplexer Channel List and Gage Locations (cont.)

Mux	Channel	Gage Label	Gage Type	Model	Location
5	1	SG-2M-1SW	Pile Strain Gage	4000	Middle Pile (below Girder 3), 0.5 m (1.6 ft) from bottom of Abutment 2, 77 mm (3.0 in) from the edge of flange.
5	2	SG-2M-1NW	Pile Strain Gage	4000	Middle Pile (below Girder 3), 0.5 m (1.6 ft) from bottom of Abutment 2, 77 mm (3.0 in) from the edge of flange.
5	3	SG-2M-3NE	Pile Strain Gage	4000	Middle Pile (below Girder 3), 1.5 m (4.9 ft) from bottom of Abutment 2, 77 mm (3.0 in) from the edge of flange.
5	4	SG-2M-3SE	Pile Strain Gage	4000	Middle Pile (below Girder 3), 1.5 m (4.9 ft) from bottom of Abutment 2, 77 mm (3.0 in) from the edge of flange.
5	5	SG-2M-3SW	Pile Strain Gage	4000	Middle Pile (below Girder 3), 1.5 m (4.9 ft) from bottom of Abutment 2, 77 mm (3.0 in) from the edge of flange.
5	6	SG-2E-1NE	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 0.5 m (1.6 ft) from bottom of Abutment 2 77 mm (3.0 in) from the edge of flange.
5	7	SG-2E-1SE	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 0.5 m (1.6 ft) from bottom of Abutment 2 77 mm (3.0 in) from the edge of flange.
5	8	SG-2E-1SW	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 0.5 m (1.6 ft) from bottom of Abutment 2 77 mm (3.0 in) from the edge of flange.
5	9	SG-2E-1NW	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 0.5 m (1.6 ft) from bottom of Abutment 2 77 mm (3.0 in) from the edge of flange.
5	10	SG-2E-2NE	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 1.0 m (3.3 ft) from bottom of Abutment 2 77 mm (3.0 in) from the edge of flange.
5	11	SG-2E-2SE	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 1.0 m (3.3 ft) from bottom of Abutment 2 77 mm (3.0 in) from the edge of flange.
5	12	SG-2E-2SW	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 1.0 m (3.3 ft) from bottom of Abutment 2 77 mm (3.0 in) from the edge of flange.
5	13	SG-2E-3NE	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 1.5 m (4.9 ft) from bottom of Abutment 2 77 mm (3.0 in) from the edge of flange.
5	14	SG-2E-3SE	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 1.5 m (4.9 ft) from bottom of Abutment 2 77 mm (3.0 in) from the edge of flange.
5	15	SG-2E-3SW	Pile Strain Gage	4000	Upstream Pile (below Girder 1), 1.5 m (4.9 ft) from bottom of Abutment 2 77 mm (3.0 in) from the edge of flange.
5	16	TM-2M	Tiltmeter (Uniaxial)	6350	Abutment 2, Center of Abutment, 0.5 m (1.6 ft) from the bottom of middle girder.

b) East Montpelier Bridge

As-built gage information for the East Montpelier Bridge is given in Table B-0.3 and Table B-0.4 for Abutment 1 and Abutment 2 multiplexers, respectively.

Table B-0.3: Abutment 1 Multiplexer Channel List and Gage Locations

Mux	**Channel**	**Gage Label**	**Gage Type**	**Model**	**Location**
1	1	CM-1NT	Displacement Transducer	4420	Abutment 1, Upstream, Transverse, 1.7 m (5.6 ft) below construction joint.
1	2	CM-1N	Displacement Transducer	4420	Abutment 1, Upstream, Longitudinal, 1.7 m (5.6 ft) below construction joint.
1	3	CM-1S	Displacement Transducer	4420	Abutment 1, Downstream, Longitudinal, 1.7 m (5.6 ft) below construction joint.
1	4	P-1NT	Earth Pressure Cell	4815	Abutment 1, Upstream, 1.6 m (5.2 ft) above the bottom of abutment, 2.12 m (6.96 ft) away from abutment-wingwall connection.
1	5	P-1NM	Earth Pressure Cell	4815	Abutment 1, Upstream, 1.0 m (3.3 ft) above the bottom of abutment, 2.12 m (6.96 ft) away from abutment-wingwall connection.
1	6	P-1NB	Earth Pressure Cell	4815	Abutment 1, Upstream, 0.4 m (1.3 ft) above the bottom of abutment, 2.12 m (6.96 ft) away from abutment-wingwall connection.
1	7	P-1CM	Earth Pressure Cell	4815	Abutment 1, Center of Abutment, 1.0 m (3.3 ft) above the bottom of abutment.
1	8	P-1CB	Earth Pressure Cell	4815	Abutment 1, Center of Abutment, 0.4 m (1.3 ft) above the bottom of abutment.
1	9	P-1ST	Earth Pressure Cell	4815	Abutment 1, Downstream, 1.6 m (5.2 ft) above the bottom of abutment, 2.12 m (6.96 ft) away from abutment-wingwall connection.
1	10	P-1SM	Earth Pressure Cell	4815	Abutment 1, Downstream, 1.0 m (3.3 ft) above the bottom of abutment, 2.12 m (6.96 ft) away from abutment-wingwall connection
1	11	P-1SB	Earth Pressure Cell	4815	Abutment 1, Downstream, 0.4 m (1.3 ft) above the bottom of abutment, 2.12 m (6.96 ft) away from abutment-wingwall connection.
1	12	PW-1N	Earth Pressure Cell	4815	Wingwall 1 (Abutment 1, Upstream), 1.2 m (3.9 ft) above the bottom of abutment, 1 m (3 ft) away from abutment-wingwall connection.
1	13	TM-1M	Tiltmeter (Uniaxial)	6350	Abutment 1, Center of Abutment, 0.5 m (1.6 ft) from the bottom of middle girder.
1	14	P-R	Earth Pressure Cell	4815	Under Approach Slab on Abutment 1 side, Facing towards the abutment.

Table B-3: Abutment 1 Multiplexer Channel List and Gage Locations (cont.)

Mux	Channel	Gage Label	Gage Type	Model	Location
2	1	SGG-1N-TE[*]	Girder Strain Gage	4050	Upstream Girder, Downstream-Top Flange, 4.35 m (14.30 ft) from the end of steel girder section at Abutment 1.
2	2	SGG-1N-BE*	Girder Strain Gage	4050	Upstream Girder, Downstream-Bottom Flange, 4.35 m (14.30 ft) from the end of steel girder section at Abutment 1.
2	3	SGG-1N-TW*	Girder Strain Gage	4050	Upstream Girder, Upstream-Top Flange, 4.35 m (14.30 ft) from the end of steel girder section at Abutment 1.
2	4	SGG-1N-BW*	Girder Strain Gage	4050	Upstream Girder, Upstream-Bottom Flange, 4.35 m (14.30 ft) from the end of steel girder section at Abutment 1.
2	5	SGG-1S-TE*	Girder Strain Gage	4050	Downstream Girder, Downstream-Top Flange, 4.35 m (14.30 ft) from the end of steel girder section at Abutment 1.
2	6	SGG-1S-BE*	Girder Strain Gage	4050	Downstream Girder, Downstream-Bottom Flange, 4.35 m (14.30 ft) from the end of steel girder section at Abutment 1.
2	7	SGG-1S-TW*	Girder Strain Gage	4050	Downstream Girder, Upstream-Top Flange, 4.35 m (14.30 ft) from the end of steel girder section at Abutment 1.
2	8	SGG-1S-BW*	Girder Strain Gage	4050	Downstream Girder, Upstream-Bottom Flange, 4.35 m (14.30 ft) from the end of steel girder section at Abutment 1.
2	9	SG-1N-1NE	Pile Strain Gage	4000	Upstream Pile, 0.5 m (1.6 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
2	10	SG-1N-1NW	Pile Strain Gage	4000	Upstream Pile, 0.5 m (1.6 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
2	11	SG-1N-1SE	Pile Strain Gage	4000	Upstream Pile, 0.5 m (1.6 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
2	12	SG-1N-1SW	Pile Strain Gage	4000	Upstream Pile, 0.5 m (1.6 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
2	13	SG-1N-2NE	Pile Strain Gage	4000	Upstream Pile, 1.0 m (3.3 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
2	14	SG-1N-2NW	Pile Strain Gage	4000	Upstream Pile, 1.0 m (3.3 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
2	15	SG-1N-2SE	Pile Strain Gage	4000	Upstream Pile, 1.0 m (3.3 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
2	16	SG-1N-2SW	Pile Strain Gage	4000	Upstream Pile, 1.0 m (3.3 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.

[*] W and E in gage labels correspond to upstream (north) and downstream (south) flanges of the girders, respectively.

Table B-3: Abutment 1 Multiplexer Channel List and Gage Locations (cont.)

Mux	**Channel**	**Gage Label**	**Gage Type**	**Model**	**Location**
3	1	SG-1N-3NE	Pile Strain Gage	4000	Upstream Pile, 1.5 m (4.9 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
3	2	SG-1N-3NW	Pile Strain Gage	4000	Upstream Pile, 1.5 m (4.9 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
3	3	SG-1N-3SE	Pile Strain Gage	4000	Upstream Pile, 1.5 m (4.9 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
3	4	SG-1N-3SW	Pile Strain Gage	4000	Upstream Pile, 1.5 m (4.9 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
3	5	SG-1S-1NE	Pile Strain Gage	4000	Downstream Pile, 0.5 m (1.6 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
3	6	SG-1S-1NW	Pile Strain Gage	4000	Downstream Pile, 0.5 m (1.6 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
3	7	SG-1S-1SE	Pile Strain Gage	4000	Downstream Pile, 0.5 m (1.6 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
3	8	SG-1S-1SW	Pile Strain Gage	4000	Downstream Pile, 0.5 m (1.6 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
3	9	SG-1S-2NE	Pile Strain Gage	4000	Downstream Pile, 1.0 m (3.3 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
3	10	SG-1S-2NW	Pile Strain Gage	4000	Downstream Pile, 1.0 m (3.3 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
3	11	SG-1S-2SE	Pile Strain Gage	4000	Downstream Pile, 1.0 m (3.3 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
3	12	SG-1S-2SW	Pile Strain Gage	4000	Downstream Pile, 1.0 m (3.3 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
3	13	SG-1S-3NE	Pile Strain Gage	4000	Downstream Pile, 1.5 m (4.9 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
3	14	SG-1S-3NW	Pile Strain Gage	4000	Downstream Pile, 1.5 m (4.9 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
3	15	SG-1S-3SE	Pile Strain Gage	4000	Downstream Pile, 1.5 m (4.9 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.
3	16	SG-1S-3SW	Pile Strain Gage	4000	Downstream Pile, 1.5 m (4.9 ft) from bottom of Abutment 1 51 mm (2.0 in) from the edge of flange.

Table B-3: Abutment 1 Multiplexer Channel List and Gage Locations (cont.)

Mux	Channel	Gage Label	Gage Type	Model	Location
6	1	IN-1N-1	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 1, Upstream Pile, Flange facing the fascia of the bridge, 0.0-0.6 m (0-2 ft) below the bottom of Abutment 1.
6	2	IN-1N-1	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 1, Upstream Pile, Flange facing the fascia of the bridge, 0.0-0.6 m (0-2 ft) below the bottom of Abutment 1.
6	3	IN-1N-2	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 1, Upstream Pile, Flange facing the fascia of the bridge, 0.6-1.5 m (2-5 ft) below the bottom of Abutment 1.
6	4	IN-1N-2	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 1, Upstream Pile, Flange facing the fascia of the bridge, 0.6-1.5 m (2-5 ft) below the bottom of Abutment 1.
6	5	IN-1N-3	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 1, Upstream Pile, Flange facing the fascia of the bridge, 1.5-2.7 m (5-9 ft) below the bottom of Abutment 1.
6	6	IN-1N-3	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 1, Upstream Pile, Flange facing the fascia of the bridge, 1.5-2.7 m (5-9 ft) below the bottom of Abutment 1.
6	7	IN-1N-4	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 1, Upstream Pile, Flange facing the fascia of the bridge, 2.7-4.0 m (9-13 ft) below the bottom of Abutment 1.
6	8	IN-1N-4	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 1, Upstream Pile, Flange facing the fascia of the bridge, 2.7-4.0 m (9-13 ft) below the bottom of Abutment 1.
6	9	IN-1S-1	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 1, Downstream Pile, Flange facing the fascia of the bridge, 0.0-0.6 m (0-2 ft) below the bottom of Abutment 1.
6	10	IN-1S-1	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 1, Downstream Pile, Flange facing the fascia of the bridge, 0.0-0.6 m (0-2 ft) below the bottom of Abutment 1.
6	11	IN-1S-2	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 1, Downstream Pile, Flange facing the fascia of the bridge, 0.6-1.5 m (2-5 ft) below the bottom of Abutment 1.
6	12	IN-1S-2	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 1, Downstream Pile, Flange facing the fascia of the bridge, 0.6-1.5 m (2-5 ft) below the bottom of Abutment 1.
6	13	IN-1S-3	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 1, Downstream Pile, Flange facing the fascia of the bridge, 1.5-2.7 m (5-9 ft) below the bottom of Abutment 1.
6	14	IN-1S-3	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 1, Downstream Pile, Flange facing the fascia of the bridge, 1.5-2.7 m (5-9 ft) below the bottom of Abutment 1.
6	15	IN-1S-4	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 1, Downstream Pile, Flange facing the fascia of the bridge, 2.7-4.0 m (9-13 ft) below the bottom of Abutment 1.
6	16	IN-1S-4	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 1, Downstream Pile, Flange facing the fascia of the bridge, 2.7-4.0 m (9-13 ft) below the bottom of Abutment 1.

Table B-0.4: Abutment 2 Multiplexer Channel List and Gage Locations

Mux	Channel	Gage Label	Gage Type	Model	Location
4	1	CM-2NT	Displacement Transducer	4420	Abutment 2, Upstream, Transverse, 1.58 m (5.18 ft) below construction joint
4	2	CM-2N	Displacement Transducer	4420	Abutment 2, Upstream, Longitudinal 1.58 m (5.18 ft) below construction joint
4	3	P-2NT	Earth Pressure Cell	4815	Abutment 2, Upstream, 1.6 m (5.2 ft) above the bottom of abutment, 2.12 m (6.96 ft) away from abutment-wingwall connection.
4	4	P-2NB	Earth Pressure Cell	4815	Abutment 1, Upstream, 0.4 m (1.3 ft) above the bottom of abutment, 2.12 m (6.96 ft) away from abutment-wingwall connection.
4	5	P-2ST	Earth Pressure Cell	4815	Abutment 2, Downstream, 1.6 m (5.2 ft) above the bottom of abutment, 2.12 m (6.96 ft) away from abutment-wingwall connection.
4	6	P-2SB	Earth Pressure Cell	4815	Abutment 1, Downstream, 0.4 m (1.3 ft) above the bottom of abutment, 2.12 m (6.96 ft) away from abutment-wingwall connection.
4	7	SGG-2N-TW	Girder Strain Gage	4050	Upstream Girder, Upstream-Top Flange, 4.35 m (14.30 ft) from the end of steel girder section at Abutment 2.
4	8	SGG-2N-BW	Girder Strain Gage	4050	Upstream Girder, Upstream-Bottom Flange, 4.35 m (14.30 ft) from the end of steel girder section at Abutment 2.
4	9	SGG-2S-TW	Girder Strain Gage	4050	Downstream Girder, Downstream-Top Flange, 4.35 m (14.30 ft) from the end of steel girder section at Abutment 2.
4	10	SGG-2S-BW	Girder Strain Gage	4050	Downstream Girder, Downstream-Bottom Flange, 4.35 m (14.30 ft) from the end of steel girder section at Abutment 1.
4	11	TM-2M	Tiltmeter (Uniaxial)	6350	Abutment 2, Center of Abutment, 0.5 m (1.6 ft) from the bottom of middle girder.

Table B-4: Abutment 2 Multiplexer Channel List and Gage Locations (cont.)

Mux	Channel	Gage Label	Gage Type	Model	Location
5	1	SGG-0N-TE	Girder Strain Gage	4050	Upstream Girder, Downstream-Top Flange, 18.5 m (61 ft) from the end of steel girder section at Abutment 1.
5	2	SGG-0N-BE	Girder Strain Gage	4050	Upstream Girder, Downstream-Bottom Flange, 18.5 m (61 ft) from the end of steel girder section at Abutment 1.
5	3	SGG-0N-TW	Girder Strain Gage	4050	Upstream Girder, Upstream-Top Flange, 18.5 m (61 ft) from the end of steel girder section at Abutment 1.
5	4	SGG-0N-BW	Girder Strain Gage	4050	Upstream Girder, Upstream-Bottom Flange, 18.5 m (61 ft) from the end of steel girder section at Abutment 1.
5	5	SGG-0S-TE	Girder Strain Gage	4050	Downstream Girder, Downstream-Top Flange, 18.5 m (61 ft) from the end of steel girder section at Abutment 1.
5	6	SGG-0S-BE	Girder Strain Gage	4050	Downstream Girder, Downstream-Bottom Flange, 18.5 m (61 ft) from the end of steel girder section at Abutment 1.
5	7	SGG-0S-TW	Girder Strain Gage	4050	Downstream Girder, Upstream-Top Flange, 18.5 m (61 ft) from the end of steel girder section at Abutment 1.
5	8	SGG-0S-BW	Girder Strain Gage	4050	Downstream Girder, Upstream-Bottom Flange, 18.5 m (61 ft) from the end of steel girder section at Abutment 1.
5	9	SG-2N-1NE	Pile Strain Gage	4000	Upstream Pile, 0.5 m (1.6 ft) from bottom of Abutment 2. 51 mm (2.0 in) from the edge of flange.
5	10	SG-2N-1NW	Pile Strain Gage	4000	Upstream Pile, 0.5 m (1.6 ft) from bottom of Abutment 2. 51 mm (2.0 in) from the edge of flange.
5	11	SG-2N-1SE	Pile Strain Gage	4000	Upstream Pile, 0.5 m (1.6 ft) from bottom of Abutment 2. 51 mm (2.0 in) from the edge of flange.
5	12	SG-2N-1SW	Pile Strain Gage	4000	Upstream Pile, 0.5 m (1.6 ft) from bottom of Abutment 2. 51 mm (2.0 in) from the edge of flange.
5	13	SG-2N-3NE	Pile Strain Gage	4000	Upstream Pile, 1.5 m (4.9 ft) from bottom of Abutment 2. 51 mm (2.0 in) from the edge of flange.
5	14	SG-2N-3NW	Pile Strain Gage	4000	Upstream Pile, 1.5 m (4.9 ft) from bottom of Abutment 2. 51 mm (2.0 in) from the edge of flange.
5	15	SG-2N-3SE	Pile Strain Gage	4000	Upstream Pile, 1.5 m (4.9 ft) from bottom of Abutment 2. 51 mm (2.0 in) from the edge of flange.
5	16	SG-2N-3SW	Pile Strain Gage	4000	Upstream Pile, 1.5 m (4.9 ft) from bottom of Abutment 2. 51 mm (2.0 in) from the edge of flange.

Table B-4: Abutment 2 Multiplexer Channel List and Gage Locations (cont.)

Mux	Channel	Gage Label	Gage Type	Model	Location
7	1	1N-1[*]	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 2, Upstream Pile, Flange facing the centerline of the roadway, 0.0-0.6 m (0-2 ft) below the bottom of Abutment 2.
7	2	1N-1*	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 2, Upstream Pile, Flange facing the centerline of the roadway, 0.0-0.6 m (0-2 ft) below the bottom of Abutment 2.
7	3	1N-2*	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 2, Upstream Pile, Flange facing the centerline of the roadway, 0.6-1.5 m (2-5 ft) below the bottom of Abutment 2.
7	4	1N-2*	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 2, Upstream Pile, Flange facing the centerline of the roadway, 0.6-1.5 m (2-5 ft) below the bottom of Abutment 2.
7	5	1N-3*	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 2, Upstream Pile, Flange facing the centerline of the roadway, 1.5-2.7 m (5-9 ft) below the bottom of Abutment 2.
7	6	1N-3*	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 2, Upstream Pile, Flange facing the centerline of the roadway, 1.5-2.7 m (5-9 ft) below the bottom of Abutment 2.
7	7	1N-4*	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 2, Upstream Pile, Flange facing the centerline of the roadway, 2.7-4.0 m (9-13 ft) below the bottom of Abutment 2.
7	8	1N-4*	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 2, Upstream Pile, Flange facing the centerline of the roadway, 2.7-4.0 m (9-13 ft) below the bottom of Abutment 2.
7	9	1S-1*	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 2, Downstream Pile, Flange facing the centerline of the roadway, 0.0-0.6 m (0-2 ft) below the bottom of Abutment 2.
7	10	1S-1*	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 2, Downstream Pile, Flange facing the centerline of the roadway, 0.0-0.6 m (0-2 ft) below the bottom of Abutment 2.
7	11	1S-2*	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 2, Downstream Pile, Flange facing the centerline of the roadway, 0.6-1.5 m (2-5 ft) below the bottom of Abutment 2.
7	12	1S-2*	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 2, Downstream Pile, Flange facing the centerline of the roadway, 0.6-1.5 m (2-5 ft) below the bottom of Abutment 2.
7	13	1S-3*	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 2, Downstream Pile, Flange facing the centerline of the roadway, 1.5-2.7 m (5-9 ft) below the bottom of Abutment 2.
7	14	1S-3*	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 2, Downstream Pile, Flange facing the centerline of the roadway, 1.5-2.7 m (5-9 ft) below the bottom of Abutment 2.

[*] Originally intended for use at Stockbridge. Abutment numbers on gage labels don't correspond to East Montpelier callout.

7	15	1S-4*	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 2, Downstream Pile, Flange facing the centerline of the roadway, 2.7-3.7 m (9-12 ft) below the bottom of Abutment 2.
7	16	1S-4*	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 2, Downstream Pile, Flange facing the centerline of the roadway, 2.7-3.7 m (9-12 ft) below the bottom of Abutment 2.

c) Stockbridge Bridge

As-built gage information for the Stockbridge Bridge is given in Table B-0.5 and Table B-0.6 for Abutment 1 and Abutment 2 (as determined by structural drawings) multiplexers, respectively.

Table B-0.5: Abutment 1 (per Structural Drawings) Multiplexer Channel Allocation and Gage Locations

Mux	**Channel**	**Gage Label**	**Gage Type**	**Model**	**Location**
6	1	CM-2NT	Displacement Transducer	4420-50mm	Abutment 1, Upstream, Transverse, 1.28 m (4.18 ft) below construction joint
6	2	CM-2N	Displacement Transducer	4420-100mm	Abutment 1, Upstream, Longitudinal, 1.28 m (4.18 ft) below construction joint
6	3	CM-2S	Displacement Transducer	4420-100mm	Abutment 1, Downstream, Longitudinal, 3.55 m (11.65 ft) below construction joint
6	4	P-2NT	Earth Pressure Cell	4810	Abutment 1, Upstream, 3.6 m (12.0 ft) above the bottom of abutment, 1 m (3 ft) away from abutment-wingwall connection
6	5	P-2NM	Earth Pressure Cell	4810	Abutment 1, Upstream, 2.1 m (7.0 ft) above the bottom of abutment, 1 m (3 ft) away from abutment-wingwall connection
6	6	P-2NB	Earth Pressure Cell	4810	Abutment 1, Upstream, 0.6 m (2.0 ft) above the bottom of abutment, 1 m (3 ft) away from abutment-wingwall connection
6	7	P-2CT	Earth Pressure Cell	4810	Abutment 1, Center of Abutment, 3.6 m (12.0 ft) above the bottom of abutment,
6	8	P-2CB	Earth Pressure Cell	4810	Abutment 1, Center of Abutment, 0.6 m (2.0 ft) above the bottom of abutment,
6	9	P-2ST	Earth Pressure Cell	4810	Abutment 1, Downstream, 3.6 m (12.0 ft) above the bottom of abutment, 1 m (3 ft) away from abutment-wingwall connection
6	10	P-2SM	Earth Pressure Cell	4810	Abutment 1, Downstream, 2.1 m (7.0 ft) above the bottom of abutment, 1 m (3 ft) away from abutment-wingwall connection
6	11	P-2SB	Earth Pressure Cell	4810	Abutment 1, Downstream, 0.6 m (2.0 ft) above the bottom of abutment, 1 m (3 ft) away from abutment-wingwall connection
6	12	PW-2N	Earth Pressure Cell	4815	Wingwall 1, 1.2 m (4.0 ft) above the bottom of wingwall, 1 m (3 ft) away from abutment-wingwall connection
6	13	PW-2S	Earth Pressure Cell	4815	Wingwall 2, 1.2 m (4.0 ft) above the bottom of wingwall,

					1 m (3 ft) away from abutment-wingwall connection
6	14	P-R	Earth Pressure Cell	4810	Under Approach Slab on Abutment 1 Side, 1.5 m (4.9 ft) away from the bridge end, At the centerline of roadway, 1.0 m (3.0 ft) below approach slab

Table B-5: Abutment 1 (per Structural Drawings) Multiplexer Channel Allocation and Gage Locations (cont.)

Mux	**Channel**	**Gage Label**	**Gage Type**	**Model**	**Location**
7	1	SGG-2N-TN	Girder Strain Gage	4050	Upstream Girder, Upstream-Top Flange, 17.2 m (56.4 ft) from the end of steel girder section at Abutment 1.
7	2	SGG-2N-TS	Girder Strain Gage	4050	Upstream Girder, Downstream-Top Flange, 17.2 m (56.4 ft) from the end of steel girder section at Abutment 1.
7	3	SGG-2N-BN	Girder Strain Gage	4050	Upstream Girder, Upstream-Bottom Flange, 17.2 m (56.4 ft) from the end of steel girder section at Abutment 1.
7	4	SGG-2N-BS	Girder Strain Gage	4050	Upstream Girder, Downstream-Bottom Flange, 17.2 m (56.4 ft) from the end of steel girder section at Abutment 1.
7	5	SGG-2S-TN	Girder Strain Gage	4050	Downstream Girder, Upstream-Top Flange, 16.0 m (52.5 ft) from the end of steel girder section at Abutment 1.
7	6	SGG-2S-TS	Girder Strain Gage	4050	Downstream Girder, Downstream-Top Flange, 16.0 m (52.5 ft) from the end of steel girder section at Abutment 1.
7	7	SGG-2S-BN	Girder Strain Gage	4050	Downstream Girder, Upstream-Bottom Flange, 16.0 m (52.5 ft) from the end of steel girder section at Abutment 1.
7	8	SGG-2S-BS	Girder Strain Gage	4050	Downstream Girder, Downstream-Bottom Flange, 16.0 m (52.5 ft) from the end of steel girder section at Abutment 1.

Table B-5: Abutment 1 (per Structural Drawings) Multiplexer Channel Allocation and Gage Locations (cont.)

Mux	Channel	Gage Label	Gage Type	Model	Location
8	1	SG-2N-1NE	Pile Strain Gage	4000	Upstream Pile, 0.3 m (1.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
8	2	SG-2N-1NW	Pile Strain Gage	4000	Upstream Pile, 0.3 m (1.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
8	3	SG-2N-1SE	Pile Strain Gage	4000	Upstream Pile, 0.3 m (1.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
8	4	SG-2N-1SW	Pile Strain Gage	4000	Upstream Pile, 0.3 m (1.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
8	5	SG-2N-2NE	Pile Strain Gage	4000	Upstream Pile, 1.2 m (4.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
8	6	SG-2N-2NW	Pile Strain Gage	4000	Upstream Pile, 1.2 m (4.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
8	7	SG-2N-2SE	Pile Strain Gage	4000	Upstream Pile, 1.2 m (4.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
8	8	SG-2N-2SW	Pile Strain Gage	4000	Upstream Pile, 1.2 m (4.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
8	9	SG-2N-3NE	Pile Strain Gage	4000	Upstream Pile, 2.1 m (7.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
8	10	SG-2N-3NW	Pile Strain Gage	4000	Upstream Pile, 2.1 m (7.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
8	11	SG-2N-3SW	Pile Strain Gage	4000	Upstream Pile, 2.1 m (7.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
8	12	SG-2N-4NE	Pile Strain Gage	4000	Upstream Pile, 3.0 m (10.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
8	13	SG-2N-4NW	Pile Strain Gage	4000	Upstream Pile, 3.0 m (10.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
8	14	SG-2N-4SW	Pile Strain Gage	4000	Upstream Pile, 3.0 m (10.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.

Table B-5: Abutment 1 (per Structural Drawings) Multiplexer Channel Allocation and Gage Locations (cont.)

Mux	Channel	Gage Label	Gage Type	Model	Location
9	1	SG-2S-1NE	Pile Strain Gage	4000	Downstream Pile, 0.3 m (1.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
9	2	SG-2S-1NW	Pile Strain Gage	4000	Downstream Pile, 0.3 m (1.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
9	3	SG-2S-1SE	Pile Strain Gage	4000	Downstream Pile, 0.3 m (1.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
9	4	SG-2S-1SW	Pile Strain Gage	4000	Downstream Pile, 0.3 m (1.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
9	5	SG-2S-2NE	Pile Strain Gage	4000	Downstream Pile, 1.2 m (4.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
9	6	SG-2S-2NW	Pile Strain Gage	4000	Downstream Pile, 1.2 m (4.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
9	7	SG-2S-2SE	Pile Strain Gage	4000	Downstream Pile, 1.2 m (4.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
9	8	SG-2S-2SW	Pile Strain Gage	4000	Downstream Pile, 1.2 m (4.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
9	9	SG-2S-3NE	Pile Strain Gage	4000	Downstream Pile, 2.1 m (7.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
9	10	SG-2S-3NW	Pile Strain Gage	4000	Downstream Pile, 2.1 m (7.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
9	11	SG-2S-3SW	Pile Strain Gage	4000	Downstream Pile, 2.1 m (7.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
9	12	SG-2S-4NE	Pile Strain Gage	4000	Downstream Pile, 3.0 m (10.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
9	13	SG-2S-4NW	Pile Strain Gage	4000	Downstream Pile, 3.0 m (10.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.
9	14	SG-2S-4SW	Pile Strain Gage	4000	Downstream Pile, 3.0 m (10.0 ft) from bottom of Abutment 1 64 mm (2.5 in) from the edge of flange.

Table B-5: Abutment 1 (per Structural Drawings) Multiplexer Channel Allocation and Gage Locations (cont.)

Mux	**Channel**	**Gage Label**	**Gage Type**	**Model**	**Location**
11	1	TM-2M	Tiltmeter (Biaxial) - Ch-1 (Longitudinal)	6160	Abutment 1, Center of Abutment, 1.0 m (3.0ft) below the bottom of middle girder
11	2	TM-2M	Tiltmeter (Biaxial) - Ch-2 (Transverse)	6160	Abutment 1, Center of Abutment, 1.0 m (3.0ft) below the bottom of middle girder
11	3	IN-2N-1	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 1, Upstream Pile, Flange facing the roadway 0.0-0.6 m (0-2 ft) below the bottom of Abutment 1
11	4	IN-2N-1	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 1, Upstream Pile, Flange facing the roadway 0.0-0.6 m (0-2 ft) below the bottom of Abutment 1
11	5	IN-2N-2	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 1, Upstream Pile, Flange facing the roadway 0.6-1.5 m (2-5 ft) below the bottom of Abutment 1
11	6	IN-2N-2	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 1, Upstream Pile, Flange facing the roadway 0.6-1.5 m (2-5 ft) below the bottom of Abutment 1
11	7	IN-2N-3	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 1, Upstream Pile, Flange facing the roadway 1.5-2.7 m (5-9 ft) below the bottom of Abutment 1
11	8	IN-2N-3	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 1, Upstream Pile, Flange facing the roadway 1.5-2.7 m (5-9 ft) below the bottom of Abutment 1
11	9	IN-2N-4	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 1, Upstream Pile, Flange facing the roadway 2.7-4.0 m (9-13 ft) below the bottom of Abutment 1
11	10	IN-2N-4	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 1, Upstream Pile, Flange facing the roadway 2.7-4.0 m (9-13 ft) below the bottom of Abutment 1
11	11	IN-2N-5	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 1, Upstream Pile, Flange facing the roadway 4.0-5.2 m (13-17 ft) below the bottom of Abutment 1
11	12	IN-2N-5	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 1, Upstream Pile, Flange facing the roadway 4.0-5.2 m (13-17 ft) below the bottom of Abutment 1

Table B-5: Abutment 1 (per Structural Drawings) Multiplexer Channel Allocation and Gage Locations (cont.)

Mux	Channel	Gage Label	Gage Type	Model	Location
12	1	IN-2S-1	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 1, Downstream Pile, Flange facing the roadway 0.0-0.6 m (0-2 ft) below the bottom of Abutment 1
12	2	IN-2S-1	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 1, Downstream Pile, Flange facing the roadway 0.0-0.6 m (0-2 ft) below the bottom of Abutment 1
12	3	IN-2S-2	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 1, Downstream Pile, Flange facing the roadway 0.6-1.5 m (2-5 ft) below the bottom of Abutment 1
12	4	IN-2S-2	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 1, Downstream Pile, Flange facing the roadway 0.6-1.5 m (2-5 ft) below the bottom of Abutment 1
12	5	IN-2S-3	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 1, Downstream Pile, Flange facing the roadway 1.5-2.7 m (5-9 ft) below the bottom of Abutment 1
12	6	IN-2S-3	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 1, Downstream Pile, Flange facing the roadway 1.5-2.7 m (5-9 ft) below the bottom of Abutment 1
12	7	IN-2S-4	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 1, Downstream Pile, Flange facing the roadway 2.7-4.0 m (9-13 ft) below the bottom of Abutment 1
12	8	IN-2S-4	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 1, Downstream Pile, Flange facing the roadway 2.7-4.0 m (9-13 ft) below the bottom of Abutment 1.
12	9	IN-2S-5	Inclinometer (Biaxial) - Ch-1 (Longitudinal)	6150	Abutment 1, Downstream Pile, Flange facing the roadway 4.0-5.2 m (13-17 ft) below the bottom of Abutment 1
12	10	IN-2S-5	Inclinometer (Biaxial) - Ch-2 (Transverse)	6150	Abutment 1, Downstream Pile, Flange facing the roadway 4.05.2 m (13-17 ft) below the bottom of Abutment 1

Table B-0.6: Abutment 2 (per Structural Drawings) Multiplexer Channel Allocation and Gage Locations

Mux	Channel	Gage Label	Gage Type	Model	Location
1	1	CM-1NT	Displacement Transducer	4420-50mm	Abutment 2, Upstream, Transverse, 2.68 m (8.78 ft) below construction joint.
1	2	CM-1N	Displacement Transducer	4420-100mm	Abutment 2, Upstream, Longitudinal, 2.68 m (8.78 ft) below construction joint.
1	3	CM-1S	Displacement Transducer	4420-100mm	Abutment 2, Downstream, Longitudinal, 1.58 m (5.17 ft) below construction joint.
1	4	P-1NT	Earth Pressure Cell	4810	Abutment 2, Upstream, 3.6 m (12.0 ft) above the bottom of abutment, 1 m (3 ft) away from abutment-wingwall connection.
1	5	P-1NM	Earth Pressure Cell	4810	Abutment 2, Upstream, 2.1 m (7.0 ft) above the bottom of abutment, 1 m (3 ft) away from abutment-wingwall connection.
1	6	P-1NB	Earth Pressure Cell	4810	Abutment 2, Upstream, 0.6 m (2.0 ft) above the bottom of abutment, 1 m (3 ft) away from abutment-wingwall connection.
1	7	P-1CT	Earth Pressure Cell	4810	Abutment 2, Center of Abutment 3.6 m (12.0 ft) above the bottom of abutment.
1	8	P-1CB	Earth Pressure Cell	4810	Abutment 2, Center of Abutment 0.6 m (2.0 ft) above the bottom of abutment.
1	9	P-1ST	Earth Pressure Cell	4810	Abutment 2, Downstream, 3.6 m (12.0 ft) above the bottom of abutment, 1 m (3 ft) away from abutment-wingwall connection.
1	10	P-1SM	Earth Pressure Cell	4810	Abutment 2, Downstream, 2.1 m (7.0 ft) above the bottom of abutment, 1 m (3 ft) away from abutment-wingwall connection.
1	11	P-1SB	Earth Pressure Cell	4810	Abutment 2, Downstream, 0.6 m (2.0 ft) above the bottom of abutment, 1 m (3 ft) away from abutment-wingwall connection.
1	12	Empty Channel			
1	13	PW-1N	Earth Pressure Cell	4815	Wingwall 3, 1.2 m (4.0 ft) above the bottom of wingwall, 1 m (3 ft) away from abutment-wingwall connection.
1	14	PW-1S	Earth Pressure Cell	4815	Wingwall 4, 1.8 m (6.0 ft) above the bottom of wingwall, 1 m (3 ft) away from abutment-wingwall connection.

Table B-6: Abutment 2 (per Structural Drawings) Multiplexer Channel Allocation and Gage Locations (cont.)

Mux	**Channel**	**Gage Label**	**Gage Type**	**Model**	**Location**
2	1	SGG-1N-TN	Girder Strain Gage	4050	Upstream Girder, Upstream-Top Flange, 17.6 m (57.7 ft) from the end of steel girder section at Abutment 2.
2	2	SGG-1N-TS	Girder Strain Gage	4050	Upstream Girder, Downstream-Top Flange, 17.6 m (57.7 ft) from the end of steel girder section at Abutment 2.
2	3	SGG-1N-BN	Girder Strain Gage	4050	Upstream Girder, Upstream-Bottom Flange, 17.6 m (57.7 ft) from the end of steel girder section at Abutment 2.
2	4	SGG-1N-BS	Girder Strain Gage	4050	Upstream Girder, Downstream-Bottom Flange, 17.6 m (57.7 ft) from the end of steel girder section at Abutment 2.
2	5	SGG-1S-TN	Girder Strain Gage	4050	Downstream Girder, Upstream-Top Flange, 16.4 m (53.8 ft) from the end of steel girder section at Abutment 2.
2	6	SGG-1S-TS	Girder Strain Gage	4050	Downstream Girder, Downstream-Top Flange, 16.4 m (53.8 ft) from the end of steel girder section at Abutment 2.
2	7	SGG-1S-BN	Girder Strain Gage	4050	Downstream Girder, Upstream-Bottom Flange, 16.4 m (53.8 ft) from the end of steel girder section at Abutment 2.
2	8	SGG-1S-BS	Girder Strain Gage	4050	Downstream Girder, Downstream-Bottom Flange, 16.4 m (53.8 ft) from the end of steel girder section at Abutment 2.
2	9	SGG-0N-TN	Girder Strain Gage	4050	Upstream Girder, Upstream-Top Flange, 34.5 m (113.2 ft) from the end of steel girder section at Abutment 1.
2	10	SGG-0N-TS	Girder Strain Gage	4050	Upstream Girder, Downstream-Top Flange, 34.5 m (113.2 ft) from the end of steel girder section at Abutment 1.
2	11	SGG-0N-BN	Girder Strain Gage	4050	Upstream Girder, Upstream-Bottom Flange, 34.5 m (113.2 ft) from the end of steel girder section at Abutment 1.
2	12	SGG-0N-BS	Girder Strain Gage	4050	Upstream Girder, Downstream-Bottom Flange, 34.5 m (113.2 ft) from the end of steel girder section at Abutment 1.
2	13	SGG-0S-TN	Girder Strain Gage	4050	Downstream Girder, Upstream-Top Flange, 32.0 m (105.0 ft) from the end of steel girder section at Abutment 1.
2	14	SGG-0S-TS	Girder Strain Gage	4050	Downstream Girder, Downstream-Top Flange, 32.0 m (105.0 ft) from the end of steel girder section at Abutment 1.
2	15	SGG-0S-BN	Girder Strain Gage	4050	Downstream Girder, Upstream-Bottom Flange, 32.0 m (105.0 ft) from the end of steel girder section at Abutment 1.
2	16	SGG-0S-BS	Girder Strain Gage	4050	Downstream Girder, Downstream-Bottom Flange, 32.0 m (105.0 ft) from the end of steel girder section at Abutment 1.

Table B-6: Abutment 2 (per Structural Drawings) Multiplexer Channel Allocation and Gage Locations (cont.)

Mux	Channel	Gage Label	Gage Type	Model	Location
3	1	SG-1N-1NE	Pile Strain Gage	4000	Upstream Pile, 0.3 m (1.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
3	2	SG-1N-1NW	Pile Strain Gage	4000	Upstream Pile, 0.3 m (1.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
3	3	SG-1N-1SE	Pile Strain Gage	4000	Upstream Pile, 0.3 m (1.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
3	4	SG-1N-1SW	Pile Strain Gage	4000	Upstream Pile, 0.3 m (1.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
3	5	SG-1N-2NE	Pile Strain Gage	4000	Upstream Pile, 1.2 m (4.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
3	6	SG-1N-2NW	Pile Strain Gage	4000	Upstream Pile, 1.2 m (4.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
3	7	SG-1N-2SE	Pile Strain Gage	4000	Upstream Pile, 1.2 m (4.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
3	8	SG-1N-2SW	Pile Strain Gage	4000	Upstream Pile, 1.2 m (4.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
3	9	SG-1N-3NE	Pile Strain Gage	4000	Upstream Pile, 2.1 m (7.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
3	10	SG-1N-3NW	Pile Strain Gage	4000	Upstream Pile, 2.1 m (7.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
3	11	SG-1N-3SE	Pile Strain Gage	4000	Upstream Pile, 2.1 m (7.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
3	12	SG-1N-3SW	Pile Strain Gage	4000	Upstream Pile, 2.1 m (7.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
3	13	SG-1N-4NE	Pile Strain Gage	4000	Upstream Pile, 3.0 m (10.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
3	14	SG-1N-4NW	Pile Strain Gage	4000	Upstream Pile, 3.0 m (10.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
3	15	SG-1N-4SE	Pile Strain Gage	4000	Upstream Pile, 3.0 m (10.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
3	16	SG-1N-4SW	Pile Strain Gage	4000	Upstream Pile, 3.0 m (10.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.

Table B-6: Abutment 2 (per Structural Drawings) Multiplexer Channel Allocation and Gage Locations (cont.)

Mux	Channel	Gage Label	Gage Type	Model	Location
4	1	SG-1S-1NE	Pile Strain Gage	4000	Downstream Pile, 0.3 m (1.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
4	2	SG-1S-1NW	Pile Strain Gage	4000	Downstream Pile, 0.3 m (1.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
4	3	SG-1S-1SE	Pile Strain Gage	4000	Downstream Pile, 0.3 m (1.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
4	4	SG-1S-1SW	Pile Strain Gage	4000	Downstream Pile, 0.3 m (1.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
4	5	SG-1S-2NE	Pile Strain Gage	4000	Downstream Pile, 1.2 m (4.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
4	6	SG-1S-2NW	Pile Strain Gage	4000	Downstream Pile, 1.2 m (4.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
4	7	SG-1S-2SE	Pile Strain Gage	4000	Downstream Pile, 1.2 m (4.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
4	8	SG-1S-2SW	Pile Strain Gage	4000	Downstream Pile, 1.2 m (4.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
4	9	SG-1S-3NE	Pile Strain Gage	4000	Downstream Pile, 2.1 m (7.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
4	10	SG-1S-3NW	Pile Strain Gage	4000	Downstream Pile, 2.1 m (7.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
4	11	SG-1S-3SE	Pile Strain Gage	4000	Downstream Pile, 2.1 m (7.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
4	12	SG-1S-3SW	Pile Strain Gage	4000	Downstream Pile, 2.1 m (7.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
4	13	SG-1S-4NE	Pile Strain Gage	4000	Downstream Pile, 3.0 m (10.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
4	14	SG-1S-4NW	Pile Strain Gage	4000	Downstream Pile, 3.0 m (10.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
4	15	SG-1S-4SE	Pile Strain Gage	4000	Downstream Pile, 3.0 m (10.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.
4	16	SG-1S-4SW	Pile Strain Gage	4000	Upstream Pile, 3.0 m (10.0 ft) from bottom of Abutment 2 64 mm (2.5 in) from the edge of flange.

Table B-6: Abutment 2 (per Structural Drawings) Multiplexer Channel Allocation and Gage Locations (cont.)

Mux	Channel	Gage Label	Gage Type	Model	Location
5	1	SGP-TN	Pier Strain Gage	4200	0.3 m (1 ft) below top of interior pier column, upstream
5	2	SGP-TS	Pier Strain Gage	4200	0.3 m (1 ft) below top of interior pier column, downstream
5	3	SGP-TE	Pier Strain Gage	4200	0.3 m (1 ft) below top of interior pier column, towards Abutment 2
5	4	SGP-TW	Pier Strain Gage	4200	0.3 m (1 ft) below top of interior pier column, towards Abutment 1
5	5	SGP-BN	Pier Strain Gage	4200	0.3 m (1 ft) above bottom of interior pier column, upstream
5	6	SGP-BS	Pier Strain Gage	4200	0.3 m (1 ft) above bottom of interior pier column, downstream
5	7	SGP-BE	Pier Strain Gage	4200	0.3 m (1 ft) above bottom of interior pier column, towards Abutment 2
5	8	SGP-BW	Pier Strain Gage	4200	0.3 m (1 ft) above bottom of interior pier column, towards Abutment 1

Table B-6: Abutment 2 (per Structural Drawings) Multiplexer Channel Allocation and Gage Locations (cont.)*[4]

Mux	Channel	Gage Label	Gage Type	Model	Location
10	1	TM-1M	Tiltmeter (Biaxial) - Ch-1 (Longitudinal)	6160	Abutment 2, Center of Abutment, 1.0 m (3.0ft) below the bottom of middle girder
10	2	TM-1M	Tiltmeter (Biaxial) - Ch-2 (Transverse)	6160	Abutment 2, Center of Abutment, 1.0 m (3.0ft) below the bottom of middle girder

BIBLIOGRAPHY

API. (1993). *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms - Working Stress Design*. Washington, DC: American Petroleum Institute.

Arsoy, S., Barker, R.M., and Duncan, J.M. (1999). *The Behavior of Integral Abutment Bridges*. (VTRC 00-CR3). Virginia Transportation Research Council, Va.

AASHTO. American Association of State Highway and Transportation Officials.(2010). *AASHTO LRFD Bridge Design Specifications*, Washington D.C.

AASHTO American Association of State Highway and Transportation Officials, (2007). *Maintenance Manual for Roadways and Bridges*, Washington, D.C.

Barker, R.M., Duncan, J.M., Rojiani, K.B., Ooi, P.S.K., Tan, C.K., Kim, S.G. (1991). *National Cooperative Highway Research Program (NCHRP) Manuals for the Design of Bridge Foundations.* Transportation Research Board, Washington.

Bogard, D., Matlock, H. (1980). *Simplified Calculation of p-y Curves for Laterally Loaded Piles.* Unpublished Report, Ertec Inc.

Civjan, Scott A., Kalayci, Emre, Quinn, Brooke H., Breña, Sergio F., and Allen, Chad A. (2013)."Observed Integral Abutment Bridge Substructure Response." Engineering Structures 56: 1177-191.

Civjan, S., Kalayci, E., Brena, S., Quinn, B. and Allen, C.. (2014). *Performance Monitoring of Jointless Bridges- Phase III- Final Report Part I.* (Report No. 2014-07). Vermont Agency of Transportation and Federal Highway Administration.

Civjan, S., Quinn, B., Brena S., Kalayci, E., Allen, C. (2014). *Performance Monitoring of Jointless Bridges- Phase III- Final Report Part II*. (Report No. 2014-07). Vermont Agency of Transportation and Federal Highway Administration.

Conboy, D., Stoothoff, E. (2005). "Integral Abutment Design and Construction: The New England Experience." *Proceedings of the 2005 FHWA Conference: Integral Abutment and Jointless Bridges.* Baltimore, Maryland, pp. 50-60.

Crafco, Retrieved from: http://www.crafco.com/PDF%20Files/Matrix%20502/Matrix_501%20PDS.pdf

D.S.Brown, Manufacturer of V-Seal Expansion Joint Systems. Retrieved from: http://www.dsbrown.com/Bridges/ExpansionJointSystems/VSeal.aspx

EM-SEAL Manufacturer. Retrieved from: http://www.emseal.com/Products/Infrastructure/BridgeJointSeals

Hassiotis, S., Khodair, Y., Roman, E., and Dehne, Y. (2006). *Evaluation of Integral Abutments.* (FHWA-NJ-2005-025). Stevens Institute of Technology, Hoboken. NJ.

Kalayci, Emre. (2012). *Performance Monitoring and Analysis of Integral Abutment Bridges*. (Doctoral Dissertation) Retrieved from: http://scholarworks.umass.edu/dissertations_1/387

Koch, Gerhardus H., Brongers, Michael P.H., Thompson, Neil G., Virmani, Y. Paul, Payer, J.H., (2001). *Corrosion Costs and Preventive Maintenance Strategies in the United States*. (FHWA Publication No. FHWA-RD-01-156). NACE Corrosion Society.

Kunin, J., Alampalli, S. (2000). "Integral Abutment Bridges: Current Practice in United States and Canada." *Journal of Performance of Constructed Facilities* (ASCE) 14, no. 3: 104-111.

Olson, Scott M. (2013). *Thermal Behavior of IDOT Integral Abutment Bridges and Proposed Design Modifications*. (FHWA Publication No. FHWA-ICT-12-022), Illinois Department of Transportation.

Maruri, R., Petro, S. (2005). Integral abutments and jointless bridges (IAJB) 2004 survey summary. *Proceedings of the 2005 FHWA conference: integral abutments and jointless bridges*, Baltimore, MD.

MassDOT, Massachusetts Dept. of Transportation, (2013). *LRFD Bridge Manual Part II*.

Milner, Micah H., Shenton III, Harry W., (2014).Survey of Past Experience and State-of-the-Practice in the Design and Maintenance of Small Movement Expansion Joints in the Northeast. *AASHTO Transportation System Preservation Technical Services Program* (*TSP2*) (Report no. 242). Newark, Delaware

Mistry, V.C. (2005). Integral Abutment and Jointless Bridges. *Proceedings of the 2005 FHWA conference; integral abutments and jointless bridges*. Baltimore, MD.

NACE Corrosion Society, (2012). *Corrosion Control Plan for Bridges.*

Purvis, Ron, (2003). Bridge Deck Joint Performance. *Transportation Research Board, National Research Council* (NCHRP Report Synthesis of Highway Practice 319). Washington, DC .

Sherafati, Ardalan. (2013). *Expanding the Length of Jointless Bridges by Providing Rotational Capacity over the Pile Head*. Diss. Univeristy of Nebraska-Lincoln. DigitalCommons@University of Nebraska - Lincoln.