Accelerated Repair and Replacement of Expansion Joints

Final Report
April 2020

ABC UTC

Sponsored by
Accelerated Bridge Construction
University Transportation Center

U.S. Department of Transportation
Office of the Assistant Secretary for
Research and Technology

BRIDGE ENGINEERING CENTER
IOWA STATE UNIVERSITY
Institute for Transportation

About the ABC-UTC

The Accelerated Bridge Construction University Transportation Center (ABC-UTC) is a Tier 1 UTC sponsored by the U.S. Department of Transportation Office of the Assistant Secretary for Research and Technology (USDOT/OST-R). The mission of ABC-UTC is to reduce the societal costs of bridge construction by reducing the duration of work zones, focusing special attention on preservation, service life, construction costs, education of the profession, and development of a next-generation workforce fully equipped with ABC knowledge.

About the Bridge Engineering Center

The mission of the Bridge Engineering Center (BEC), which is part of the Institute for Transportation (InTrans) at Iowa State University, is to conduct research on bridge technologies to help bridge designers/owners design, build, and maintain long-lasting bridges. The mission of InTrans is to develop and implement innovative methods, materials, and technologies for improving transportation efficiency, safety, reliability, and sustainability while improving the learning environment of students, faculty, and staff in transportation-related fields.

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
ABC-UTC-2016-C1-ISU03-Final		

4. Title and Subtitle	5. Report Date
Accelerated Repair and Replacement of Expansion Joints	April 2020
	6. Performing Organization Code
7. Author(s)	**8. Performing Organization Report No.**
Brent M. Phares (orcid.org/0000-0003-0546-3760) and Kara Ruble (orcid.org/0000-0002-1603-9095)	
9. Performing Organization Name and Address	**10. Work Unit No. (TRAIS)**
Bridge Engineering Center Iowa State University 2711 South Loop Drive, Suite 4700 Ames, IA 50010-8664	**11. Contract or Grant No.** 69A3551747121
12. Sponsoring Organization Name and Address	**13. Type of Report and Period Covered**
Accelerated Bridge Construction University Transportation Center Florida International University 10555 W. Flagler Street, EC 3680 Miami, FL 33174 US Department of Transportation Office of the Assistant Secretary for Research and Technology and Federal Highway Administration 1200 New Jersey Avenue, SE Washington, DC 201590	Final Report (January 2018–September 2019) **14. Sponsoring Agency Code**

15. Supplementary Notes

Visit www.abc-utc.fiu.edu for other ABC reports.

16. Abstract

Bridge deck expansion joints allow for the movement of bridge decks exposed to thermal expansion and dynamic loading. These joints are often one of the first components of a bridge deck to fail, and there is a need for accelerated replacement options, especially in areas with high traffic volumes. Extensive research is being conducted on accelerated bridge construction (ABC) initiatives to reduce lane closure time. However, less attention has been devoted to accelerated repair and replacement of bridge deck expansion joints.

This project developed methods for the accelerated replacement of bridge deck expansion joints, beginning with a literature review. The combination of a stainless steel railing and ultra-high-performance concrete (UHPC) header with hydrodemolition was evaluated for its effectiveness as an accelerated option.

A life-cycle cost analysis with a sensitivity study compared the proposed replacement to current practices and two alternative methods. This analysis revealed that, for bridges with a life of greater than 50 years, the proposed replacement was the most cost-effective option.

The proposed replacement joint also underwent bonding, static, and fatigue testing. Hydrodemolition was also used in the replacement process of the testing. These tests indicated that the joint system utilizing hydrodemolition produces an excellent bond with the existing concrete. The static and fatigue testing revealed the joint system meets department of transportation (DOT) standards and would likely have a long service life.

17. Key Words	18. Distribution Statement
accelerated bridge construction—bridge expansion joints—bridge repair	No restrictions.

19. Security Classification (of this report)	20. Security Classification (of this page)	21. No. of Pages	22. Price
Unclassified.	Unclassified.	76	

Form DOT F 1700.7 (8-72) **Reproduction of completed page authorized**

(this page is intentionally left blank)

Accelerated Repair and Replacement of Expansion Joints

Final Report
April 2020

Principal Investigator: Brent M. Phares
Bridge Engineering Center
Iowa State University

Authors
Brent M. Phares and Kara Ruble

Sponsored by
Accelerated Bridge Construction University Transportation Center

ACCELERATED BRIDGE CONSTRUCTION
UNIVERSITY TRANSPORTATION CENTER

A report from
Bridge Engineering Center and Institute for Transportation
Iowa State University
2711 South Loop Drive, Suite 4700
Ames, IA 50010-8664
Phone: 515-294-8103 / Fax: 515-294-0467
www.intrans.iastate.edu

DISCLAIMER

TABLE OF CONTENTS

ACKNOWLEDGMENTS x
CHAPTER 1. INTRODUCTION 1
1.1. Project Motivation 1
1.2. Research, Objectives, and Tasks...1
CHAPTER 2. LITERATURE REVIEW 3
2.1. Typical Joints Used by the Iowa DOT...3
2.2. Bottlenecks in Construction...8
2.2.1. Demolition 9
2.2.2. Concrete 11
2.3. Elimination of Joints 13
2.3.1. Integral and Semi-Integral Abutments...14
2.3.2. Deck Over Backwall Concept...15
2.3.3. Link Slabs 16
CHAPTER 3. PROPOSED REPLACEMENT SYSTEM EVALUATION 17
3.1. Expansion Joint...17
3.2. Demolition 18
3.3. Concrete 18
3.4. Component Interaction...19
CHAPTER 4. COST ANALYSIS...20
4.1. Cost-Analysis Parameters 20
4.1.1. Bridge Geometry and Service Life 20
4.1.2. Service Life of Materials 21
4.1.3. Initial Costs 22
4.1.4. Interest Rate 23
4.2. Base Life-Cycle Cost Analysis 23
4.3. Sensitivity Study 24
4.3.1. Varying Parameters...24
4.3.2. Simulation Results and Discussion...25
4.4. Summary 32
CHAPTER 5. EXPERIMENTAL TESTING 34
5.1. Testing Objectives 34

5.2. Test Specimen 35
5.2.1. Geometry 35
5.2.2. Boundary Conditions 37
5.2.3. Reinforcing 38
5.2.4. Concrete 39
5.2.5. Instrumentation 40
5.3. Hydrodemolition 44
5.4. Bond Evaluation 48
5.4.1. Slant Shear Test 48
5.4.2. Split Cylinder Test 52
5.5. Static Testing 55
5.5.1. Vertical Loading 55
5.5.2. Thermal Loading 57
5.6. Fatigue Testing 59
5.7. Summary 62
CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS 64
REFERENCES 65

LIST OF FIGURES

Figure 2.1. Typical sliding plate joint 3
Figure 2.2. Sliding plate fused and separated from abutment 4
Figure 2.3. Cracked sliding plate joint 4
Figure 2.4. Typical strip seal joint 5
Figure 2.5. Strip seal joint with punctured gland 5
Figure 2.6. EMSEAL BEJS retrofit 6
Figure 2.7. EMSEAL BEJS retrofit installed 6
Figure 2.8. Typical compression joint 6
Figure 2.9. Compression joint with broken anchor 7
Figure 2.10. Typical finger joint 7
Figure 2.11. Typical modular joint 8
Figure 2.12. Modular joint with debris, preventing closure 8
Figure 2.13. Saw cutting 9
Figure 2.14. Pneumatic breaker 10
Figure 2.15. Hydrodemolition equipment 11
Figure 2.16. Magnesium phosphate cement being used for patching 12
Figure 2.17. Elastomeric concrete for expansion joint repair 12
Figure 2.18. UHPC being place into a joint 13
Figure 2.19. Typical integral abutment 14
Figure 2.20. Typical semi-integral abutment 15
Figure 2.21. Preliminary deck over backwall design 16
Figure 2.22. Typical link slab 16
Figure 3.1. D.S. Brown A2R-400 strip seal details 17
Figure 3.2. D.S. Brown steel railing for strip seal joint 18
Figure 4.1. Cost distribution for 30-ft bridge, 50-year bridge life 26
Figure 4.2. Cost distribution for 30-ft bridge, 75-year bridge life 28
Figure 4.3. Cost distribution for 30-ft bridge, 100-year bridge life 29
Figure 4.4. Cost distribution for 44-ft bridge, 50-year bridge life 30
Figure 4.5. Cost distribution for 44-ft bridge, 75-year bridge life 31
Figure 4.6. Cost distribution for 44-ft bridge, 100-year bridge life 32
Figure 5.1. D.S. Brown A2R-400 strip seal 35

Figure 5.2. Geometry of laboratory specimen 36
Figure 5.3. Shear studs on steel beams 37
Figure 5.4. Bridge deck roller support 38
Figure 5.5. Backwall fixed support 38
Figure 5.6. Reinforcing strain gauge 40
Figure 5.7. Embedded concrete strain gauge 41
Figure 5.8. Strain gauge layout 41
Figure 5.9. String potentiometer 42
Figure 5.10. String potentiometer layout 42
Figure 5.11. Direct current linear variable differential transformer 43
Figure 5.12. DCDT layout 44
Figure 5.13. Aquajet aqua cutter 710H 45
Figure 5.14. Water tank truck 45
Figure 5.15. Filter socks 46
Figure 5.16. Hydrodemolition progress 47
Figure 5.17. Cleaning the joint after hydrodemolition 47
Figure 5.18. Completed hydrodemolition 48
Figure 5.19. Slant shear cylinder test 49
Figure 5.20. Cylinder 1 slant shear failure 50
Figure 5.21. Cylinder 2 slant shear failure 51
Figure 5.22. Cylinder 3 slant shear failure 51
Figure 5.23. Split cylinder interface 52
Figure 5.24. Split cylinder test 52
Figure 5.25. Cylinder 1 split cylinder failure 54
Figure 5.26. Cylinder 2 split cylinder failure 54
Figure 5.27. Cylinder 3 split cylinder failure 55
Figure 5.28. HS-20-44 truck 56
Figure 5.29. Actuators used to mimic thermal loading 57
Figure 5.30. Average concrete strain, thermal performance test 58
Figure 5.31. Average reinforcing strain, thermal performance test 59
Figure 5.32. Fatigue loading setup 60
Figure 5.33. Average strain over time 61
Figure 5.34. Visual inspection of specimen 62

Figure 5.35. Visual inspection of joint....62

LIST OF TABLES

Table 4.1. Cost-analysis scenarios....20
Table 4.2. Material service lives....21
Table 4.3. Cost-analysis scenario replacement cycle....21
Table 4.4. Average costs....22
Table 4.5. Base life-cycle costs....23
Table 4.6. Statistical distributions of varying parameters....25
Table 4.7. Monte Carlo statistical summary for 30-ft bridge, 50-year life....27
Table 4.8. Monte Carlo statistical summary for 30-ft bridge, 75-year life....28
Table 4.9. Monte Carlo statistical summary for 30-ft bridge, 100-year life....29
Table 4.10. Monte Carlo statistical summary for 44-ft bridge, 50-year life....30
Table 4.11. Monte Carlo statistical summary for 44-ft bridge, 75-year life....31
Table 4.12. Monte Carlo statistical summary for 44-ft bridge, 100-year life....32
Table 5.1. Reinforcing bar list....39
Table 5.2. Concrete 28-day compressive strength....39
Table 5.3. Ductal JS1212 UHPC mix design....39
Table 5.4. UHPC cylinder compressive strength....40
Table 5.5. Slant shear test results....49
Table 5.6. Bond quality....53
Table 5.7. Split cylinder test results....53
Table 5.8. Vertical test results at impact load....56

ACKNOWLEDGMENTS

This project was supported by the Accelerated Bridge Construction University Transportation Center (ABC-UTC at https://abc-utc.fiu.edu/) at Florida International University (FIU) as the lead institution, with Iowa State University, the University of Nevada-Reno, the University of Oklahoma, and the University of Washington as partner institutions. The authors would like to acknowledge the support of the ABC-UTC and the U.S. Department of Transportation Office of the Assistant Secretary for Research and Technology.

Chapter 1. INTRODUCTION

1.1. Project Motivation

Bridge deck expansion joints are the components of the bridge that help to accommodate movement due to thermal expansion and, to a lesser extent, dynamic loading. Expansion joints may also serve to help prevent de-icing chemicals and other corrosives applied to bridge decks from penetrating and damaging bridge substructure components. Expansion joints are often one of the first components of a bridge deck to fail and may require multiple replacements throughout the life of a bridge. These replacements are seen as critical to extending bridge life and protecting the substructure components.

Currently, the replacement of an expansion joint can take anywhere from a few days to many weeks. These replacements typically involve extensive traffic interference and lane closures. Therefore, there is a need for accelerated replacement options and techniques, especially on bridges with high annual average daily traffic (AADT) and limited time for lane closures.

Accelerated bridge construction (ABC) has gained increasing momentum over recent years and is creating a permanent shift in how bridge construction is performed. ABC techniques focus on ways to reduce lane closures and many times utilize precast components. To date, however, there has been little research into using ABC techniques for expansion joint repair and replacement. The research summarized in this report focuses on developing these methods for accelerated expansion joint replacement.

1.2. Research, Objectives, and Tasks

The primary objectives of this research project were as follows:

- Conduct a literature review on replacement and even elimination of bridge deck expansion joints
- Develop methods for accelerated bridge expansion joint replacement and possible elimination
- Promote ABC for bridge deck expansion joint repair

The objectives were accomplished through the following research tasks:

- *Task 1 – Literature review:* Current means and methods of expansion joint maintenance, replacement, and elimination in Iowa and other states were explored with a focus on accelerated repair and replacement of bridge deck expansion joints.

- *Task 2 – Develop methods for accelerated expansion joint replacement:* Bottlenecks in expansion joint replacement were identified. Different methods and options for accelerated expansion joint replacement were developed by addressing these bottlenecks.

- *Task 3 – Perform experimental studies to confirm constructability:* To confirm the constructability of the methods developed through Task 2, experimental studies were conducted.

- *Task 4 – Provide accelerated options where expansion joint elimination is feasible:* Eliminating deck joints instead of replacement is a suitable option for bridges with moderate length. Accelerated options were explored for various elimination options.

- *Task 5 – Cost analysis of the various options:* A cost analysis with a sensitivity study was conducted to determine the economic viability of the various options.

- *Task 6 – Develop design guide/manual for bridge engineers:* A guide/manual for bridge engineers has been developed based on the outcomes of Tasks 1–5.

- *Task 7 – Final report*: This final report was prepared meeting the requirements for university transportation center (UTC) funded projects. This report contains a detailed summary of the results from the preceding tasks and a recommendation for future phases of the project.

Chapter 2. LITERATURE REVIEW

The researchers conducted a literature review to investigate typical joints used by the Iowa Department of Transportation (DOT), bottlenecks in construction of expansion joint replacements, and options for the elimination of expansion joints. These topics are summarized in the following sections.

2.1. Typical Joints Used by the Iowa DOT

Miller and Jahren (2014) conducted an extensive review of typical expansion joints used by the Iowa DOT as part of their research into the rapid replacement of expansion joints. They found that sliding plate, strip seal, compression, finger, and modular joints have all been used historically by the Iowa DOT. Integral abutments were also found to be used by the Iowa DOT, and these are addressed in more detail later in this chapter.

Sliding plate joints consist of one steel plate freely sliding over another steel plate embedded in the bridge deck. When the bridge expands and contracts, these plates are able to slide over one another to accommodate the movement. A typical sliding plate joint is shown in Figure 2.1.

Keith Smith/MoDOT 2012

Figure 2.1. Typical sliding plate joint

Sliding plate joints are no longer used by the Iowa DOT in replacements or new construction, but some older bridges still that have these joints in place. Sliding plate joints are not watertight. Over time, these joints can experience severe failure to the steel plates. Rust formation can cause the two plates to fuse together, preventing any movement to occur and reducing the strength.

This can lead to the plates separating from their attachments and the joints cracking (see Figure 2.2 and Figure 2.3).

Ohio DOT
(http://www.dot.state.oh.us/Divisions/Engineering/Structures/bridge%20operations%20and%20maintenance/PreventiveMaintenanceManual/BPMM/decks/deck9.htm)

Figure 2.2. Sliding plate fused and separated from abutment

Ohio DOT
(http://www.dot.state.oh.us/Divisions/Engineering/Structures/bridge%20operations%20and%20maintenance/PreventiveMaintenanceManual/BPMM/repairs/expjointrdefects10.htm)

Figure 2.3. Cracked sliding plate joint

This, in conjunction with extensive cyclic loading from traffic, leads to failure in sections of the joint.

Strip seal joints consist of two steel extrusions embedded in the approach slab and bridge deck. A neoprene gland is then placed in the gap, attached to the steel extrusions. This gland provides a water barrier to protect the structural components below the joint, and the flexible nature of the material allows it to move with the expansion and contraction of the bridge. A typical strip seal joint is shown in Figure 2.4.

ArchiExpo

Figure 2.4. Typical strip seal joint

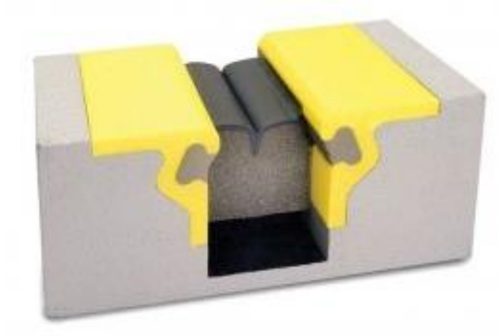
Strip seals are becoming the most common joint used in expansion joint replacements for the Iowa DOT. The neoprene gland is usually the first part of the joint to fail after about 15 years (see Figure 2.5).

Baker Engineering & Energy 2006

Figure 2.5. Strip seal joint with punctured gland

These glands can be removed and replaced with relative ease and do not compromise the structural integrity of the joint. A strip seal joint typically only needs full replacement if damage has occurred to the steel extrusion due to rust or snowplow damage, or if there is severe spalling of the concrete header surrounding the joint.

In the recent past, a strip seal retrofit designed by EMSEAL Joint Systems, Ltd. has become increasingly popular as an option with midwest DOTs. The EMSEAL bridge expansion joint system (BEJS) utilizes a compression seal that can be installed against the flat surface of the joint. This system is ideal for when a steel extrusion fails but there is no major damage to the surrounding concrete. The system can be installed with minimal, if any, demolition and bypasses the need for a steel extrusion. A typical profile of the EMSEAL BEJS is shown in Figure 2.6 and Figure 2.7



EMSEAL Joint Systems, Ltd.

Figure 2.6. EMSEAL BEJS retrofit

EMSEAL Joint Systems, Ltd.

Figure 2.7. EMSEAL BEJS retrofit installed

Compression joints consist of a compressible seal in the gap between the bridge deck and approach slab. A typical compression joint can be seen in Figure 2.8.

Keith Smith/MoDOT 2012

Figure 2.8. Typical compression joint

Although compression joints are being phased out of use by the Iowa DOT, many bridges in Iowa still use them. Compression joints have failure patterns similar to those of strip seal joints. The compression seal is typically the first thing to fail after about 10 years of service. With compression joints, it is optional to include steel armor with the joint. If the steel armor is included, it typically fails due to rust or the cyclic loading of traffic (see Figure 2.9).

Adam Miller/InTrans (Miller and Jahren 2014)

Figure 2.9. Compression joint with broken anchor

Finger joints and modular joints are typically used for larger bridges that require movement of more than 5 inches. Finger joints are comprised of interlocking "fingers," while modular joints are a series of connected strip seals. Typical views of these two joints can be seen in Figure 2.10 and Figure 2.11.

Keith Smith/MoDOT 2012

Figure 2.10. Typical finger joint

DOSHIN Rubber Engineering n.d.

Figure 2.11. Typical modular joint

Both of these joints are very effective for longer bridges. When a component of either joint is damaged, it can simply be replaced without affecting the rest of the joint system. Finger joints are not watertight, however, which can lead to erosion of the soil below and/or damage to substructure elements. Modular joints fail similarly to strip seals (see Figure 2.12) and can be repaired in a similar manner.

Adam Miller/InTrans (Miller and Jahren 2014)

Figure 2.12. Modular joint with debris, preventing closure

2.2. Bottlenecks in Construction

Miller and Jahren (2014) discovered in Phase I of their research that demolition of the existing joint and the placement and curing of the new concrete are the largest bottlenecks in construction. Various demolition methods and fast-setting concrete mixes were evaluated in order to develop methods for the accelerated replacement of expansion joints. The various methods evaluated are detailed in the following sections.

In addition to these methods, reducing the demolition area was also considered. A workshop was held by Miller and Jahren (2014) to brainstorm ways to speed up construction related to expansion joints. One group in the workshop suggested reducing the demolition area to the boundary of the steel extrusion itself. Typical demolition areas stretch 2 ft on both sides of the joint and 10–12 in. deep. However, the steel extrusion currently used by the Iowa DOT is only 1 ft wide on each side and 7 in. deep.

Reducing the demolition area to these dimensions significantly reduces the time required for demolition while still allowing the steel extrusion to be removed for replacement. This smaller area is also more economical, as it requires fewer hours of labor and less concrete in the replacement.

2.2.1. Demolition

Saw cutting, pneumatic breakers, and hydrodemolition were all evaluated as demolition methods for an accelerated joint replacement. Each of these methods are briefly described.

2.2.1.1. Saw Cutting

Saw cutting is the process of using a diamond-segmented blade to cut the concrete (see Figure 2.13).

EMSEAL Joint Systems, Ltd.

Figure 2.13. Saw cutting

This is a familiar demolition method for most contractors. It creates concrete blocks that are relatively easy to remove and has a low risk of damaging the remaining concrete. There are some concerns, however, with saw cutting in an accelerated context.

Saw cutting leaves a smooth surface on the remaining concrete. This has the risk of creating a poor bond with the new concrete and would likely need to be roughened before the placement of replacement concrete. This also means a potential loss of the existing reinforcing steel, requiring dowels to be installed. Saw cutting also requires a larger workforce and creates potential overcuts that need to be filled (Phares and Cronin 2015).

2.2.1.2. Pneumatic Breakers

Pneumatic breakers are handheld machines that come in a variety of weights and sizes. They have a narrow cutting edge that hammers out the concrete that needs removal. A typical pneumatic breaker can be seen in Figure 2.14.

EMSEAL Joint Systems, Ltd.

Figure 2.14. Pneumatic breaker

This is also a very common demolition method that is familiar to contractors. It does very well getting into confined areas and chiseling out concrete in narrow spaces below reinforcing steel (Vorster et al. 1992). Pneumatic breakers also create a rough surface that is potentially good for the bonding between new and old concrete.

The largest drawbacks to pneumatic breakers for accelerated removal is the larger workforce and the amount of time needed to complete demolition. The time needed is significantly longer than it is with the other two demolition methods evaluated in this study. There is also some risk associated with the chiseling action of the pneumatic breaker. This mode of demolition can potentially cause micro-fracturing in the remaining concrete and damage the existing reinforcing steel.

2.2.1.3. Hydrodemolition

Hydrodemolition is the process of directing pressurized water to demolish concrete. Generally, the hydrodemolition unit is programmed and controlled by a walk-behind operator (see Figure 2.15).

Kara Ruble/BEC

Figure 2.15. Hydrodemolition equipment

The operator controls the rate of the water pressure, speed of the nozzle as it performs passes, and width of the demolition area. With the semi-automated equipment, a very small workforce is needed for hydrodemolition. Similar to pneumatic breakers, hydrodemolition leaves a rough surface that is likely good for concrete bonding. Unlike pneumatic breakers, though, hydrodemolition does no damage to the existing reinforcing steel (except perhaps to the epoxy coating) and results in minimal residual cracking in the remaining concrete. Hydrodemolition is also the fastest of the three demolition methods evaluated.

Hydrodemolition is still gaining popularity with contractors, as it has two main obstacles. The first is the high mobilization costs associated with it. It can take up to a few hours to set up, depending on the site, and requires a tank truck to hold the water used in the demolition. The second obstacle is managing the wastewater slurry that is created as runoff from the hydrodemolition. This slurry needs to either be collected or filtered through filter socks before leaving the site.

2.2.2. Concrete

Magnesium phosphate cement (MPC), elastomeric concrete, and ultra-high-performance (UHPC) concrete were all evaluated for an accelerated joint replacement. In the following sections, various properties of these options are discussed.

2.2.2.1. Magnesium Phosphate Cement

MPC is a mixture utilizing dead burned magnesia and phosphate in a manner similar to that for portland cement. It has many advantages over portland cement, including fast set time, high early strength, and ability to be cast in cold weather. MPC can, on average, set in 20–30 minutes, and can be cast in temperatures ranging from -5°F to 86°F. If it is cast near the lower temperature limit, the set time increases to about 80 minutes. Once set, MPC can be opened to traffic after 3 hours (Li and Chen 2013).

Historically, MPC has only been used for smaller patch jobs (see Figure 2.16).

Tom Girolamo (https://eco-buildingandforestry.com/eco-friendly-concrete-repair/)

Figure 2.16. Magnesium phosphate cement being used for patching

With such a short working time, it may be difficult to use on larger expansion joint projects. MPC is documented to have been used by Alaska, Maryland, and Virginia DOTs (Burris et al. 2015).

2.2.2.2. Elastomeric Concrete

Elastomeric concrete is a mixture of a polymeric binder and aggregate that has been growing in popularity as an expansion joint repair material (see Figure 2.17).

©2020 D.S. Brown (https://dsbrown.com/concrete/delpatch-elastomeric-concrete-concreteparent/)

Figure 2.17. Elastomeric concrete for expansion joint repair

Elastomeric concrete bonds well to concrete and steel and, unlike portland cement concrete (PCC), has a very low spalling risk. Elastomeric concrete can be placed within 4–5 hours and opened to traffic roughly 3 hours after it sets (Gergely et al. 2009). The service life of elastomeric concrete is approximately 25 years before it needs to be replaced.

2.2.2.3. Ultra-High-Performance Concrete

UHPC is a relatively new class of fiber-reinforced, portland cement-based concrete. It utilizes a number of different admixtures to increase its strength and decrease its set time. According to the Federal Highway Administration (FHWA), UHPC is classified as having a minimum compressive strength of 21.6 ksi. UHPC has very low porosity, and, therefore, is very durable to chlorides and other de-icing chemicals, and it resists freeze-thaw cycles well (see Figure 2.18).

Source: New York State Department of Transportation

NYSDOT

Figure 2.18. UHPC being place into a joint

The set time of UHPC relies heavily on the ambient temperature. Most UHPC mixes can be opened to traffic 24–48 hours after they are placed. If the UHPC is heat-treated, this time can be reduced to as little as 12 hours (Graybeal 2006).

With being a newer type of concrete, proprietary mixes are still required for most applications of UHPC. These mixes have a much larger cost associated with them when compared to standard PCC and are not as familiar to contractors. As UHPC continues to grow in popularity, however, it is expected that non-proprietary mixes will be approved for use and costs will go down.

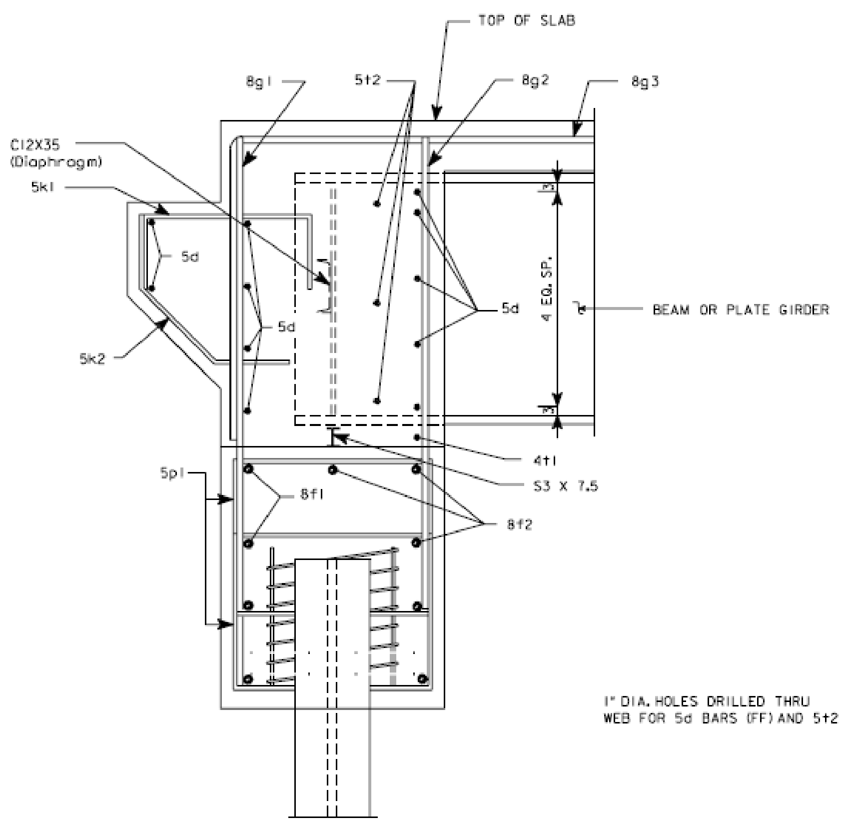
2.3. Elimination of Joints

While the focus of this report is the replacement of expansion joints, it is noteworthy to address the option of completely eliminating expansion joints. Across DOTs, the elimination of joints is the preferred option if possible for bridges of moderate length. Eliminating expansion joints prevents any damage to the substructure from water runoff, and it removes future maintenance of the eliminated joint. The use of integral abutments, semi-integral abutments, the deck over backwall concept, and link slabs have all been identified as options for accelerated joint elimination and are summarized in the following sections.

2.3.1. Integral and Semi-Integral Abutments

Integral and semi-integral abutments are similar to one another and are options for joint elimination at the bridge deck-abutment interface. Both integral and semi-integral abutments can be constructed in an accelerated context when the main slabs are precast and sealed with a cast-in-place UHPC connector.

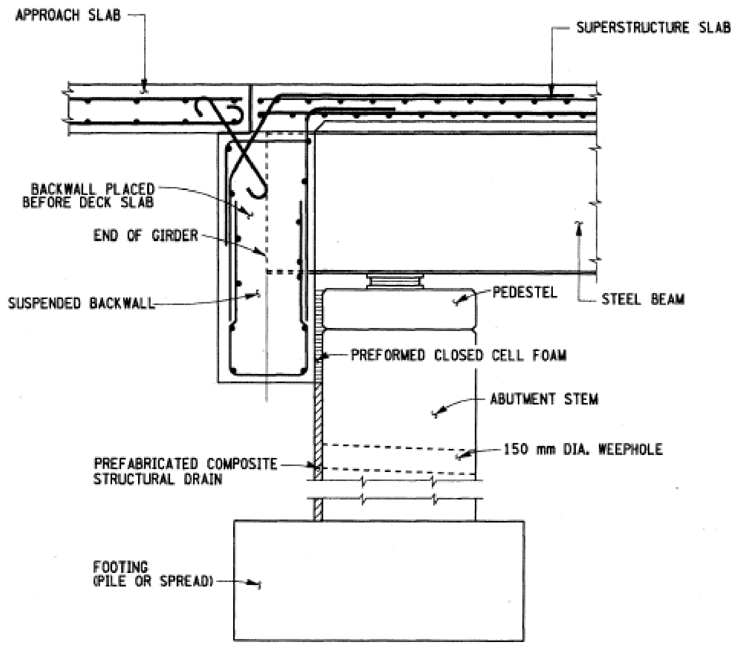
In an integral abutment, the bridge girder ends are encased in the backwall, and the abutment moves along with the thermal movement of the bridge deck. A typical profile of an integral abutment can be seen in Figure 2.19.



Miller and Jahren 2014

Figure 2.19. Typical integral abutment

Semi-integral abutments differ from integral abutments in that semi-integral abutments still have the girders sitting on a bearing pad. The girder ends are still encased in the backwall, but the bearing pads allow the foundation to be fixed instead of moving with thermal effects. Semi-integral abutments are typically used when it is not possible to use an integral abutment (due to length, skew, or other factors). Semi-integral abutments are also more common when being considered for a joint retrofit. A typical profile of a semi-integral abutment can be seen in Figure 2.20.

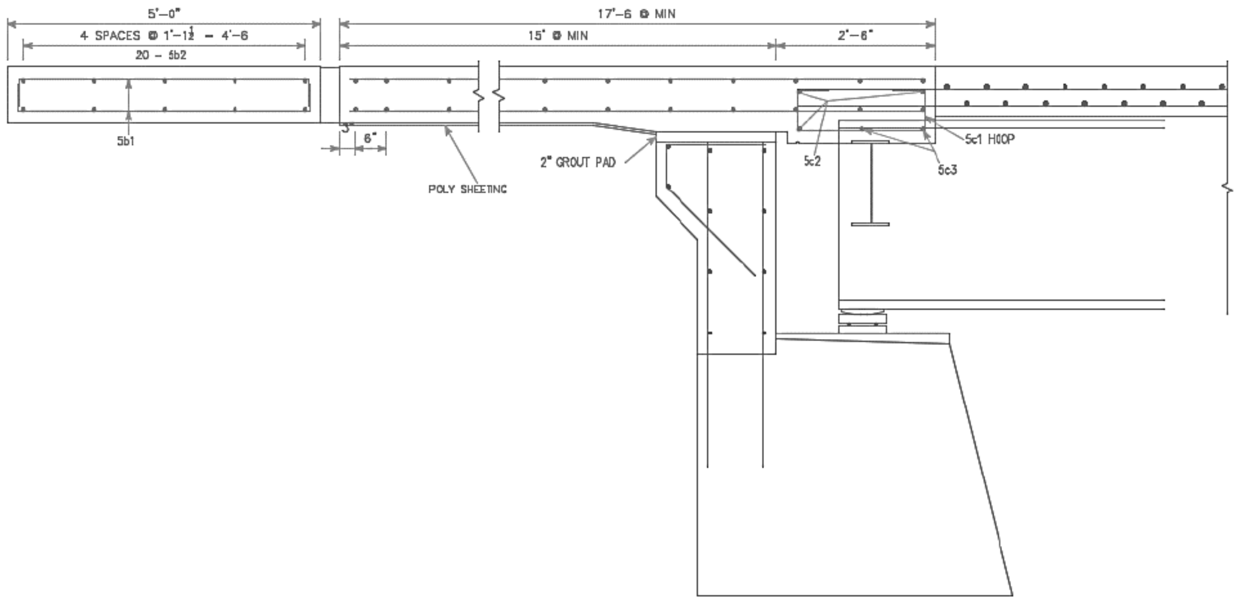


Miller and Jahren 2014

Figure 2.20. Typical semi-integral abutment

2.3.2. Deck Over Backwall Concept

The deck over backwall concept was developed in Phase III of Jahren's research into the accelerated repair of bridge expansion joints (Morandeira et al. 2019). This concept explores the idea of moving the expansion joint that is typically at the abutment into the bridge approach slab. This allows any water that seeps through the joint to flow into the soil beneath instead of damaging the substructure. Similar to integral and semi-integral abutments, the use of precast panels and cast-in-place UHPC connectors could be used to accelerate the construction of this system. A preliminary Iowa DOT detail of the deck over backwall concept can be seen in Figure 2.21.

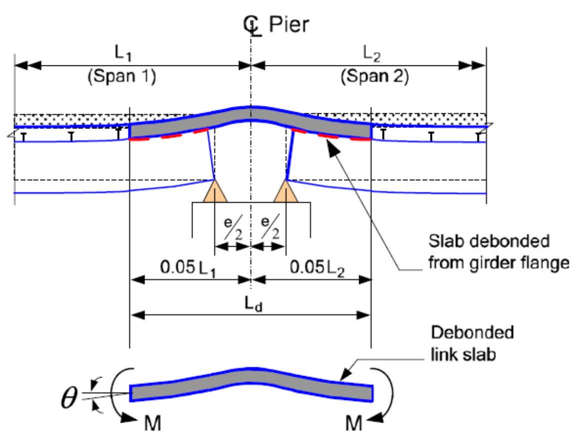


Iowa DOT

Figure 2.21. Preliminary deck over backwall design

2.3.3. Link Slabs

Link slabs are a common way to replace expansion joints over bridge piers. The slab is debonded from the girder to allow for rotation due to thermal movement of the bridge deck. UHPC has been used for link slabs previously and can be adapted for accelerated construction using one of many fast-setting concrete mixes. Ductal’s JS1212 mix of UHPC is one such option that cures quickly. A typical link slab can be seen in Figure 2.22.



Miller and Jahren 2015

Figure 2.22. Typical link slab

Chapter 3. PROPOSED REPLACEMENT SYSTEM EVALUATION

Based on the findings of the literature review, it was desired to select a single combination of available options to further investigate for the remainder of this study. This required selecting the type of expansion joint, the demolition method, and the concrete. It was also necessary to consider the interaction of the individual components as a holistic expansion joint system.

The process for selecting these components is summarized in the following sections. The goals for the performance of the proposed replacement are twofold. First, the replacement methods should require the minimum amount of lane closure time as possible. Second, the replacement methods should have as long a life as possible to minimize the total number of full replacements needed in a given bridge's life.

3.1. Expansion Joint

Of the types of expansion joints investigated, strip seal, finger, and modular joints were among the top rated joints by the Iowa DOT. For bridges of moderate length, strip seal joints are almost always used as the replacement joint. Finger joints and modular joints are reserved for longer bridges requiring greater than 5 in. of movement. Given this report focuses on bridges of moderate length, a strip seal joint was chosen as the expansion joint for the replacement to study further.

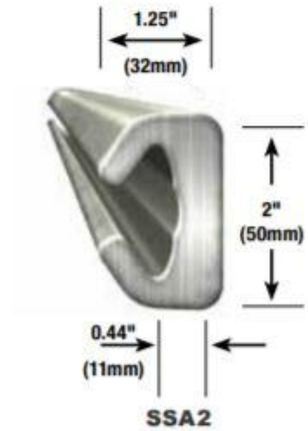
The Iowa DOT currently approves strip seal joints manufactured by two companies: Watson & Bowman and D.S. Brown. Their products are similar to each another.

After speaking to a representative from D.S. Brown, it was discovered that D.S. Brown has specifications for making a steel railing for a strip seal joint out of stainless steel. These steel railings are typically constructed with A36 steel, which commonly has a life of about 25 years for the railing. If stainless steel were used instead of A36 for the construction of the strip seal railings, the life of the railing could be extended almost indefinitely. To evaluate both the A36 and stainless steel railing options, the D.S. Brown A2R-400 strip seal (Figure 3.1 and Figure 3.2) was selected as the appropriate joint for this study.

Sealing Element Cross-Section	Sealing Element	Movement Range MR_L	MR_T	Joint Opening
 A2R	A2R – 400	4.0 (102)	±2.0 (51)	0.5 – 4.5 (13) (114)

D.S. Brown n.d.

Figure 3.1. D.S. Brown A2R-400 strip seal details



D.S. Brown n.d.

Figure 3.2. D.S. Brown steel railing for strip seal joint

3.2. Demolition

All of the demolition methods evaluated have advantages and disadvantages that must be considered if chosen as the demolition method for this study. Saw cutting is quick and allows for a smaller workforce, but it cuts the existing reinforcing steel. It also leaves a smooth surface that would need to be sandblasted before any new concrete is poured. Hydrodemolition is also quick and allows for a smaller workforce. It also creates a rough surface for excellent bonding with the new concrete. However, it has high mobilization costs and leaves a wastewater slurry that needs to be handled. Pneumatic breakers require a larger workforce and more time but create a rough bonding surface for the new concrete and can reach difficult areas.

Ultimately, saw cutting was eliminated as an option for a couple of reasons. To minimize the demolition time, it is undesirable to choose a method of demolition that requires a second phase of work to sandblast the surface of the existing concrete to create a proper bond with the new concrete. In addition to this, it is desired to maintain the cohesion of the bridge and joint by keeping the existing reinforcing steel in place. Saw cutting completely removes part of these reinforcing steel bars and requires dowels to be drilled and placed in the joint instead.

Both hydrodemolition and pneumatic breakers provide a good bonding surface for the new concrete. It was decided that hydrodemolition would be preferred over pneumatic breakers due to the significantly quicker demolition time and smaller workforce required. This preference is stipulated provided that the wastewater can be properly handled and dependent on the results of a life-cycle cost analysis.

3.3. Concrete

For the concrete options evaluated, magnesium phosphate cement was quickly eliminated as an option. It has not yet been used for a full replacement, and there are concerns about its feasibility for a project the size of a joint replacement with its extremely short working time. Both elastomeric concrete and UHPC are viable options for this application. Both are fast-setting and easy to mix.

Elastomeric concrete has become a common material for expansion joint headers. There are a few joint options that utilize it for various bridge joints and conditions, and there has been significant research into its performance.

UHPC is a newer material, still being investigated for all its possible applications. UHPC has increased strength and durability from conventional concrete to the order of 6 times the strength and 100 times the durability. Because of this, UHPC has an extremely long service life that would last until the end of the bridge's life.

For the purposes of this research, UHPC was selected as the proposed replacement concrete. It was selected over elastomeric concrete primarily due to its longer life. Elastomeric concrete typically only lasts 25–30 years before needing to be replaced. Similar to hydrodemolition, this selection is stipulated based on the results of a life-cycle cost analysis.

3.4. Component Interaction

In addition to their individual advantages, the components were chosen for the increased advantage of the components working together as a system. Stainless steel railings can last until the end of the bridge's life, so it makes sense to pick a concrete that will last just as long. UHPC is the only concrete evaluated that also can last until the end of the bridge's life.

UHPC requires a longer curing time than other fast-setting concretes, but placement and curing can still be accomplished over a weekend. With this in mind, UHPC and a stainless steel railing would only need one full replacement in the remaining life of a bridge. One weekend of construction for the replacement during the whole life of the bridge is more desirable than multiple days of construction 3–4 times throughout the life of the bridge with traffic disruptions.

For the UHPC and stainless steel system to last this long, an excellent bond is needed to avoid premature failure. In addition to being the fastest demolition option, hydrodemolition also provides the best bonding surface without requiring sandblasting after demolition.

Chapter 4. COST ANALYSIS

After the evaluation discussed in Chapter 3, it was decided to pursue a combination of hydrodemolition, UHPC, and stainless steel extrusion for the accelerated joint replacement procedure/concept. This combination was explored to extend the life of the replacement joint and further reduce life-cycle costs and the time associated with replacing the joint in the future. This chapter addresses the viability of this proposed replacement from a life-cycle cost perspective.

Four scenarios were compared in this cost analysis (see Table 4.1).

Table 4.1. Cost-analysis scenarios

Scenario	Demolition	Concrete	Steel extrusion	Notes
1	Pneumatic Breaker	PCC	A36	
2	Hydrodemolition	UHPC	Stainless	
3	Hydrodemolition	UHPC	A36	EMEAL BEJS retrofit used
4	Pneumatic Breaker	PCC	Stainless	

The first scenario represents the current procedures/materials included in a typical joint replacement. It uses PCC, A36 steel extrusions, and pneumatic breaker demolition. The second utilizes UHPC, stainless steel extrusions, and hydrodemolition in the replacement. The third represents if stainless steel extrusions are unavailable. It uses UHPC, hydrodemolition, and a typical A36 steel extrusion. However, when the A36 extrusion fails, the UHPC header will still be in good condition. Therefore, the use of the EMSEAL BEJS, discussed in the Literature Review, is recommended to extend the time between full replacements. This system is already used by DOTs as a joint retrofit when possible. The fourth represents if UHPC is unavailable. This uses the conventional PCC and pneumatic breaker demolition, but still uses stainless steel extrusions. The spalling of the PCC controls the time between replacements.

4.1. Cost-Analysis Parameters

4.1.1. Bridge Geometry and Service Life

Two bridges were used in this cost analysis. The first bridge had a width of 30 ft and the second had a width of 44 ft. These widths were chosen in accordance with typical bridge widths in Iowa. For each bridge, three different target bridge lives were evaluated: 50 years, 75 years, and 100 years.

For a full joint replacement, a demolition area of 1 ft on both sides of the joint and a depth of 7 in. was used. This results in a concrete quantity of 1.3 cubic yards for the 30-ft wide bridge and 1.9 cubic yards for the 44-ft wide bridge.

4.1.2. *Service Life of Materials*

The material service lives were determined to be the controlling factors for a joint replacement. The average service lives were used for the various concretes, steel extrusions, and expansion joint glands in this cost analysis. Table 4.2 summarizes the values used for the materials.

Table 4.2. Material service lives

Material	Average Service Life (yrs)
PCC	30
UHPC	100+
A36 Steel Extrusion	25
Stainless Steel Extrusion	100+
Neoprene Strip Seal Gland	15
EMSEAL BEJS	15

UHPC and stainless steel extrusions are expected to last the remainder of the bridge life. This idea is represented by a service life of 100+ years. The exact service life for these materials is still being determined as they become more popular for industry use.

A full joint replacement is necessary once the concrete and/or steel extrusion has failed. This places a full replacement every 25 years for systems using A36 steel extrusions, and every 30 years for systems using PCC. When a neoprene gland fails for a strip seal, it can easily be replaced in a matter of hours if the railing and concrete header are still in good condition. Given these service lives, an average replacement cycle was created for each scenario described earlier. These cycles continue to repeat until the end of the bridge service life. These cycles are summarized in Table 4.3.

Table 4.3. Cost-analysis scenario replacement cycle

Yrs	Scenario 1: Pneumatic Breaker, PCC, A36 Rail	Scenario 2: Hydrodemolition, UHPC, Stainless Rail	Scenario 3: Hydrodemolition, UHPC, A36 Rail	Scenario 4: Pneumatic Breaker, PCC, Stainless Rail
0	Full Replace	Full Replace	Full Replace	Full Replace
10				
15	Gland Replace	Gland Replace	Gland Replace	Gland Replace
20				
25	Full Replace		EMSEAL Retrofit	
30		Gland Replace		Full Replace
35				
40			Full Replace	

Yrs= number of years past the initial replacement of the joint

For scenario 2, the gland would continue to be replaced every 15 years. There would not be a second complete replacement throughout the bridge life.

4.1.3. Initial Costs

The costs included in this cost analysis are materials, mobilization, traffic control, and maintenance costs. The average costs were taken from historic bid tabs of the Iowa DOT. The bid tabs used included all full joint replacements and gland replacements conducted by the Iowa DOT in fiscal year (FY) 2018–2019. The costs for Ductal UHPC and EMSEAL BEJS were taken directly from the manufacturers. The average costs are listed in Table 4.4.

Table 4.4. Average costs

Item	Average Cost ($)	
	30-ft Bridge	44-ft Bridge
PCC	49.34	72.11
UHPC	2,860.00	4,180.00
Hydrodemolition	324.28	476.19
Pneumatic Breaker	87.25	128.12
EMSEAL BEJS	1,350.00	1,980.00
Steel extrusion w/ Neoprene	6,476.70	9,499.16
Stainless Steel extrusion w/ Neoprene	12,953.40	18,998.32
Neoprene Gland Installation & Testing	1,465.80	2,149.84
Neoprene Gland Install & Testing – Gland Replacement Only	3,725.10	5,463.48
Mobilization	7,666.67	7,666.67
Mobilization – Gland Replacement Only	24,840.14	24,840.14
Traffic Control	6,312.50	6,312.50
Traffic Control – Gland Replacement Only	6,196.67	6,196.67
Temporary Barrier Rail	296.40	434.72
Temporary Crash Cushion	1,017.27	1,017.27
Temporary Traffic Signals	4,321.25	43,21.25
Flaggers	465.60	465.60
Painted Pavement Markers	104.59	104.59
Wet Retroreflective Removal Tape Markings	115.00	115.00
Pavement Markers Removed	247.27	247.27

The geometry of the bridges has already been taken into account for these costs.

Equation (1) was used to project the future value (FV) of these costs for this cost analysis.

$$FV = PV \times (1 + i)^N \quad \textbf{(1)}$$

where:
PV = present value
i -=interest rate
N = number of years removed from the present

4.1.4. Interest Rate

Real interest rate was used in this cost analysis. Real interest rate accounts for the effect of inflation opposed to the nominal interest rate, which does not. The interest rate was determined with the average of the 30-year projections from Appendix C of Circular A-94 developed by the White House Office of Management and Budget. The circular states that, for cost estimates beyond 30 years, the 30-year projections should be used. The average 30-year real interest rate was calculated to be 3.80%.

4.2. Base Life-Cycle Cost Analysis

Using the average parameters, a base life-cycle cost analysis was conducted. The results of this base analysis are summarized in Table 4.5.

Table 4.5. Base life-cycle costs

	Scenario	Description	Bridge Life (years)		
			50	75	100
30-ft bridge	1	Pneumatic Breaker, PCC, A36 Rail	$193,974	$680,699	$2,499,211
	2	Hydrodemolition, UHPC, Stainless Rail	$165,960	$321,315	$1,409,272
	3	Hydrodemolition, UHPC, A36 Rail	$137,669	$625,682	$2,919,816
	4	Pneumatic Breaker, PCC, Stainless Rail	$250,684	$406,039	$1,867,421
44-ft bridge	1	Pneumatic Breaker, PCC, A36 Rail	$211,741	$742,691	$2,726,635
	2	Hydrodemolition, UHPC, Stainless Rail	$182,641	$350,065	$1,522,541
	3	Hydrodemolition, UHPC, A36 Rail	$147,093	$679,957	$3,191,089
	4	Pneumatic Breaker, PCC, Stainless Rail	$284,515	$451,939	$2,075,054

For all bridge lives considered for each bridge width, the proposed replacement (scenario 2) had a lower life-cycle cost than current practices (scenario 1). For the 75- and 100-year bridge lives, scenario 2 had the lowest costs of all scenarios. For the 50-year bridge life, scenario 3 had the lowest cost. This was primarily due to the use of the EMSEAL BEJS retrofit when the A36 steel extrusion fails at 30 years. The retrofit extends the time between full replacements, causing the break-even point between scenario 2 and 3 to be further in the future.

4.3. Sensitivity Study

Since the parameters used in this analysis vary with time and are based on the project, a sensitivity study was conducted to further evaluate the costs. A Monte Carlo simulation was used for the sensitivity study.

In a Monte Carlo simulation, each varying parameter is assigned a statistical distribution of cost. Then, the simulation runs iterations of the cost analysis with randomly selected values for each parameter using the assigned distribution.

This sensitivity study was conducted with 1,000,000 iterations. Once the simulation was complete, the results of all iterations could be used to create a histogram. These histograms help to show the range and likelihood of possible life-cycle costs for each scenario.

4.3.1. Varying Parameters

All material, mobilization, traffic control, and maintenance costs were considered as variable parameters. Using the same Iowa DOT bid tabs as before, a statistical distribution was created for each item. The interest rate was also considered as a varying parameter. The statistical distributions used are summarized in Table 4.6.

Table 4.6. Statistical distributions of varying parameters

Item	30-ft bridge		44-ft bridge	
	Avg. Cost ($)	Std. Dev.	Avg. Cost ($)	Std. Dev.
PCC	49.34	7.40	72.11	10.82
Hydrodemolition	324.28	48.64	476.19	71.43
Pneumatic Breaker	87.25	13.09	128.12	19.22
EMSEAL BEJS	1,350.00	202.50	1,980.00	297.00
Steel Extrusion w/Neoprene	6,476.70	882.90	9,499.16	12,94.92
Stainless Steel Extrusion w/Neoprene	12,953.40	1,765.80	18,998.32	2,589.84
Neoprene Gland Installation & Testing	1,465.80	388.50	2,149.84	569.80
Neoprene Gland Install & Testing – Gland Only	3,725.10	277.20	5,463.48	406.56
Mobilization	7,666.67	63.93	7,666.67	63.93
Mobilization – Gland Only	24,840.14	10,152.39	24,840.14	10,152.39
Traffic Control	6,312.50	2,784.55	6,312.50	2,784.55
Traffic Control – Gland Only	6,196.67	126.40	6,196.67	126.40
Temporary Barrier Rail	296.40	43.20	434.72	63.36
Temporary Crash Cushion	1,017.27	105.86	1,017.27	105.86
Temporary Traffic Signals	4,321.25	8.31	4,321.25	8.31
Flaggers	465.60	0.75	465.60	0.75
Painted Pavement Markers	104.59	118.52	104.59	118.52
Wet Retroreflective Removal Tape Markings	115.00	34.39	115.00	34.39
Pavement Markers Removed	247.27	546.58	247.27	546.58
Interest Rate	3.80%	1.69%	3.80%	1.69%

All but the UHPC costs utilize a normal distribution with an average value and standard deviation. The UHPC used a block distribution with a high value of $3,146 and a low value of $286. A block distribution was used for UHPC to account for its relatively new use in industry. It was assumed that the price of UHPC will decrease significantly over the coming years as it is more widely used.

4.3.2. Simulation Results and Discussion

Upon completion of the sensitivity study, the results were summarized in a series of histograms for each bridge width and life (see Figure 4.1 for one of these).

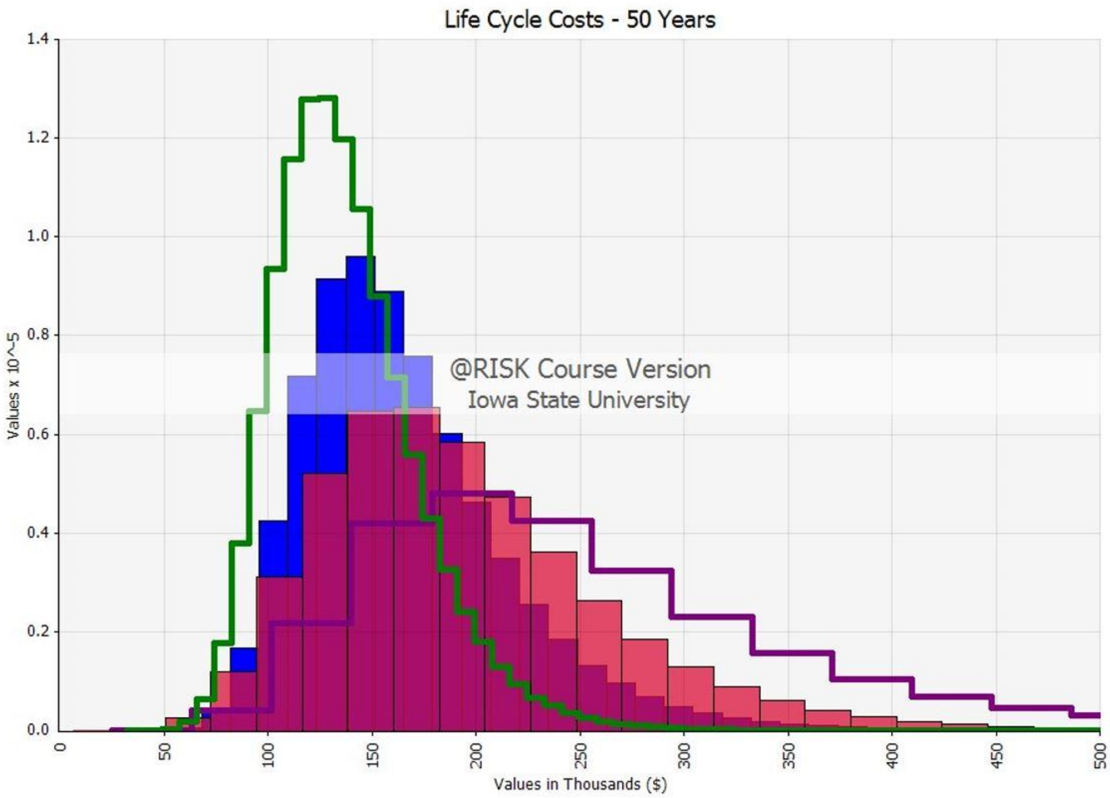


Figure 4.1. Cost distribution for 30-ft bridge, 50-year bridge life

Key to Histograms

On each histogram, the horizontal axis represents the total life-cycle costs and the vertical axis represents the number of 1,000,000 iterations that estimated the same cost. This allows a person to easily identify the most likely cost by the highest peak in the histogram. It also allows the distribution of likely costs to be identified to create the most holistic and accurate estimation. The histograms for the sensitivity study are overlaid for each option. The overlaid histograms allow a visual illustration of how different options compare with one another for each bridge width and lifetime. For example, if looking at a specific cost, the color with the higher peak at that cost has a higher probability of achieving that cost than the other color. The following colors represent each scenario in all of the histograms.

- Red – Scenario 1 (Pneumatic breaker, PCC, A36 rail)
- Blue – Scenario 2 (Hydrodemolition, UHPC, Stainless rail)
- Green – Scenario 3 (Hydrodemolition, UHPC, A36 rail)
- Purple – Scenario 4 (Pneumatic breaker, PCC, Stainless rail)

In addition to the histograms, a table is included for each bridge width and life summarizing the most likely, average, and 90th percentile cost (see Table 4.7).

Table 4.7. Monte Carlo statistical summary for 30-ft bridge, 50-year life

Scenario	Color	Life-Cycle Cost ($)		
		Most Likely	Average	90th Percentile
1	Red	167,732	193,975	285,793
2	Blue	143,840	165,960	230,907
3	Green	127,091	137,670	182,870
4	Purple	192,139	250,684	386,493

The 90th percentile represents the cost needed to achieve 90% of the iterations to be less than or equal to that value. Overall, the same patterns that were observed in the base life-cycle cost analysis were also seen in the Monte Carlo results. The proposed replacement (scenario 2) has a higher likelihood of a lower cost than current practices (scenario 1) for all bridge lives.

For a 30-ft bridge with a 50-year life, the most cost-effective option was scenario 3, utilizing hydrodemolition, UHPC, and an A36 railing. This is due to the use of the EMSEAL BEJS after the A36 railing. This delays the need for a full replacement of the expansion joint beyond the scope of a 50-year lifecycle. When comparing the proposed replacement (scenario 2) to current practices (scenario 1), the proposed replacement estimates a lower cost in all three cost categories.

For a 30-ft bridge with a 75-year life, the proposed replacement (scenario 2) surpassed scenario 3 for the lowest estimated life-cycle costs (Figure 4.2 and Table 4.8).

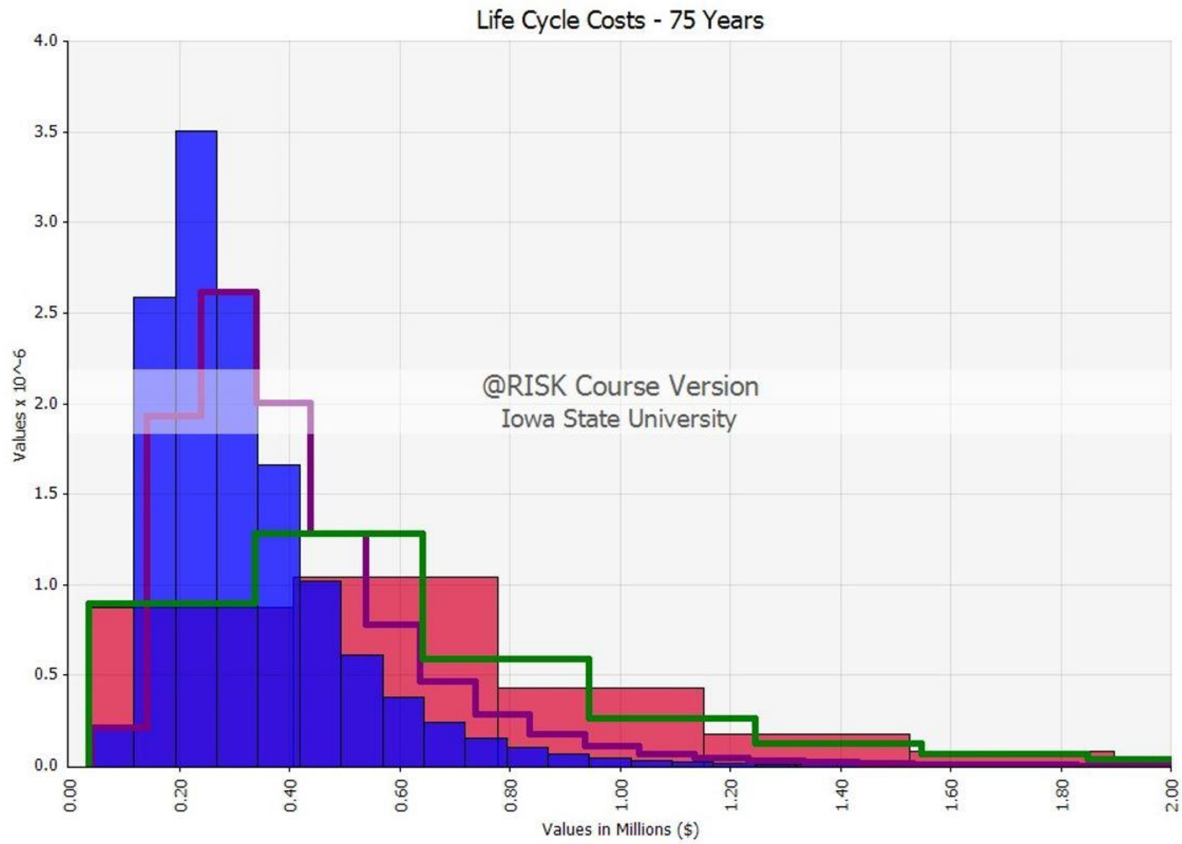


Figure 4.2. Cost distribution for 30-ft bridge, 75-year bridge life

Table 4.8. Monte Carlo statistical summary for 30-ft bridge, 75-year life

Scenario	Color	Life-Cycle Cost ($)		
		Most Likely	Average	90th Percentile
1	Red	352,407	680,699	1,262,211
2	Blue	216,442	321,316	533,555
3	Green	329,191	625,683	1,161,172
4	Purple	266,892	406,040	681,864

The most likely cost of the proposed replacement was about $125,000 less than current practices, with the current practices (scenario 1) becoming the highest estimated costs of all scenarios.

For a 30-ft bridge with a 100-year life, the proposed replacement (scenario 2), utilizing hydrodemolition, UHPC, and a stainless steel rail had the lowest estimated life-cycle costs (Figure 4.3 and Table 4.9).

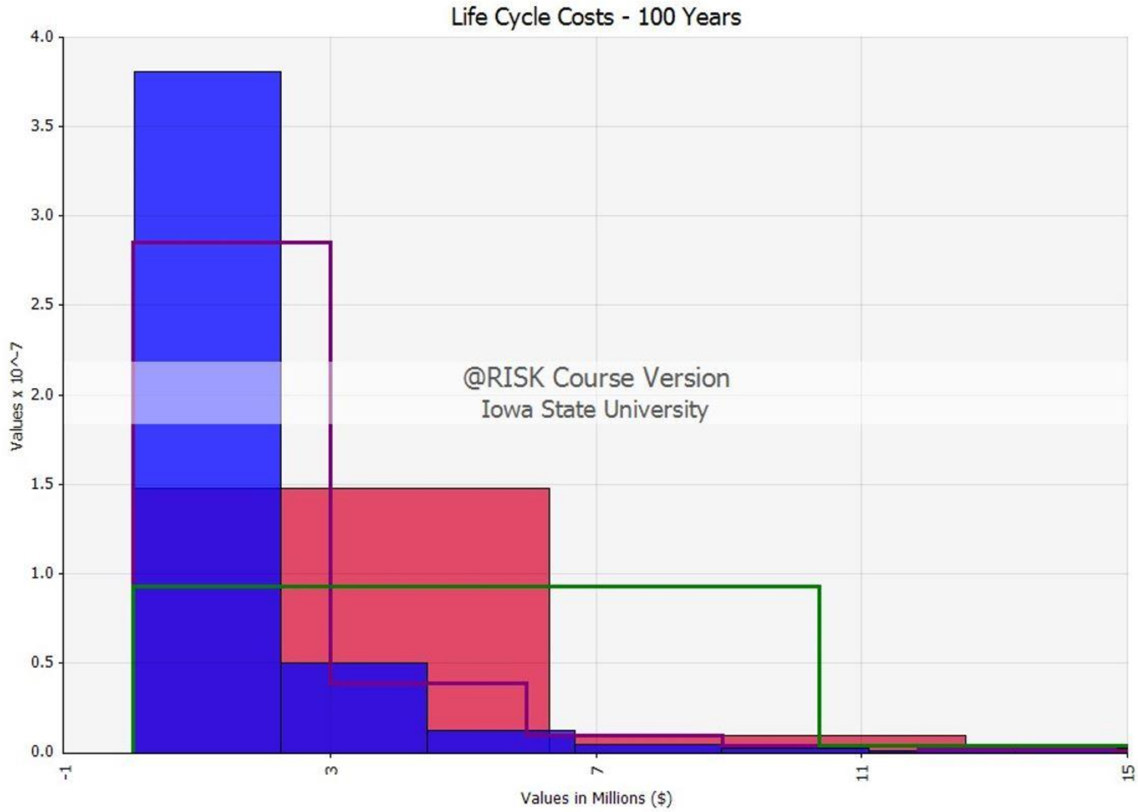


Figure 4.3. Cost distribution for 30-ft bridge, 100-year bridge life

Table 4.9. Monte Carlo statistical summary for 30-ft bridge, 100-year life

Scenario	Color	Life-Cycle Cost ($)		
		Most Likely	Average	90th Percentile
1	Red	542,237	2,499,211	5,390,829
2	Blue	371,405	1,409,272	2,982,472
3	Green	533,071	2,919,817	6,395,665
4	Purple	482,739	1,867,421	3,950,279

The current practices (scenario 1) and scenario 3 had similar estimations for the highest life-cycle costs. Scenario 4, utilizing a pneumatic breaker, PCC, and a stainless railing, had a considerably lower cost than scenarios 1 or 3. This was mostly due to the extended joint life that the stainless steel railing provides.

The trends identified in the 30-ft bridge analysis are also present for the 44-ft bridge. For a 50-year life, the lowest estimated cost was, again, for scenario 3, with the use of the EMSEAL BEJS retrofit (Figure 4.5 and Table 4.10).

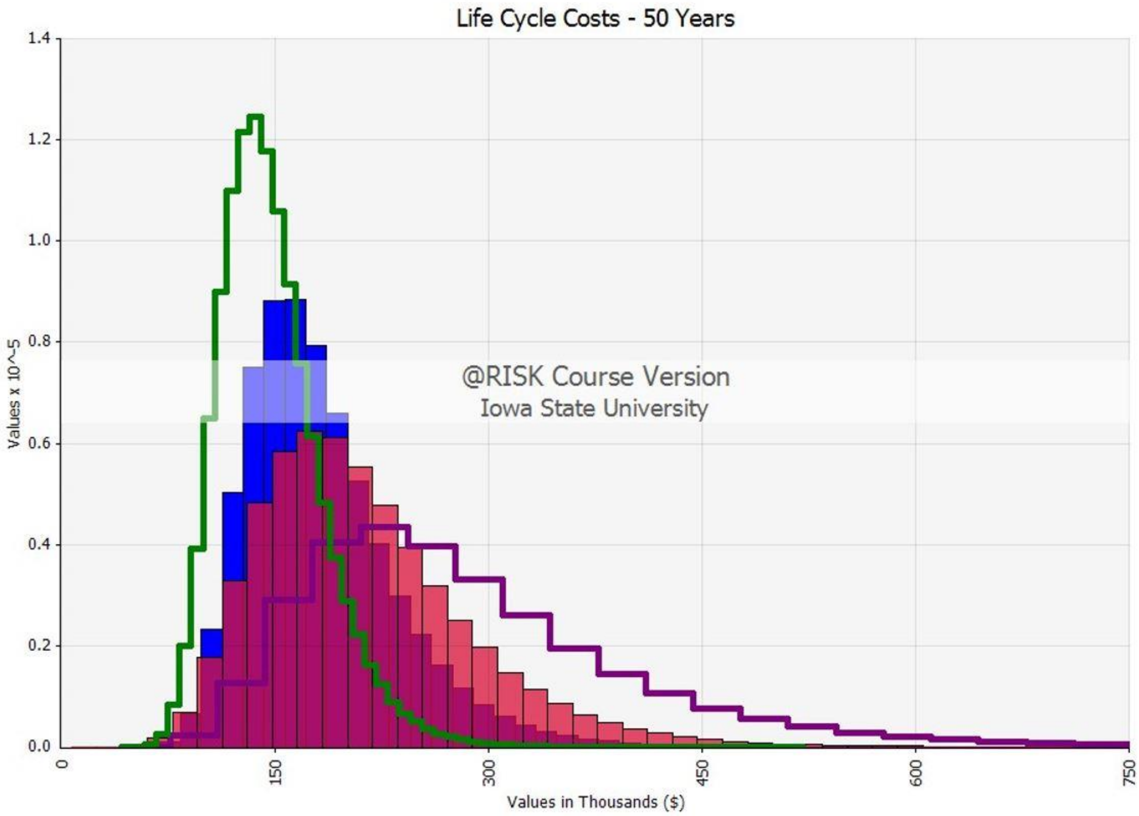


Figure 4.4. Cost distribution for 44-ft bridge, 50-year bridge life

Table 4.10. Monte Carlo statistical summary for 44-ft bridge, 50-year life

Scenario	Color	Life-Cycle Cost ($)		
		Most Likely	Average	90th Percentile
1	Red	174,218	211,741	309,468
2	Blue	156,560	182,641	252,529
3	Green	134,438	147,094	194,257
4	Purple	217,024	284,515	435,698

The proposed replacement (scenario 2) also still had a lower life-cycle cost than the current practices (scenario 1).

For a 44-ft bridge with a 75-year life, the proposed replacement (scenario 2) surpassed scenario 3 for the lowest estimated life-cycle costs, like the 30-ft bridge costs (Figure 4.5 and Table 4.11).

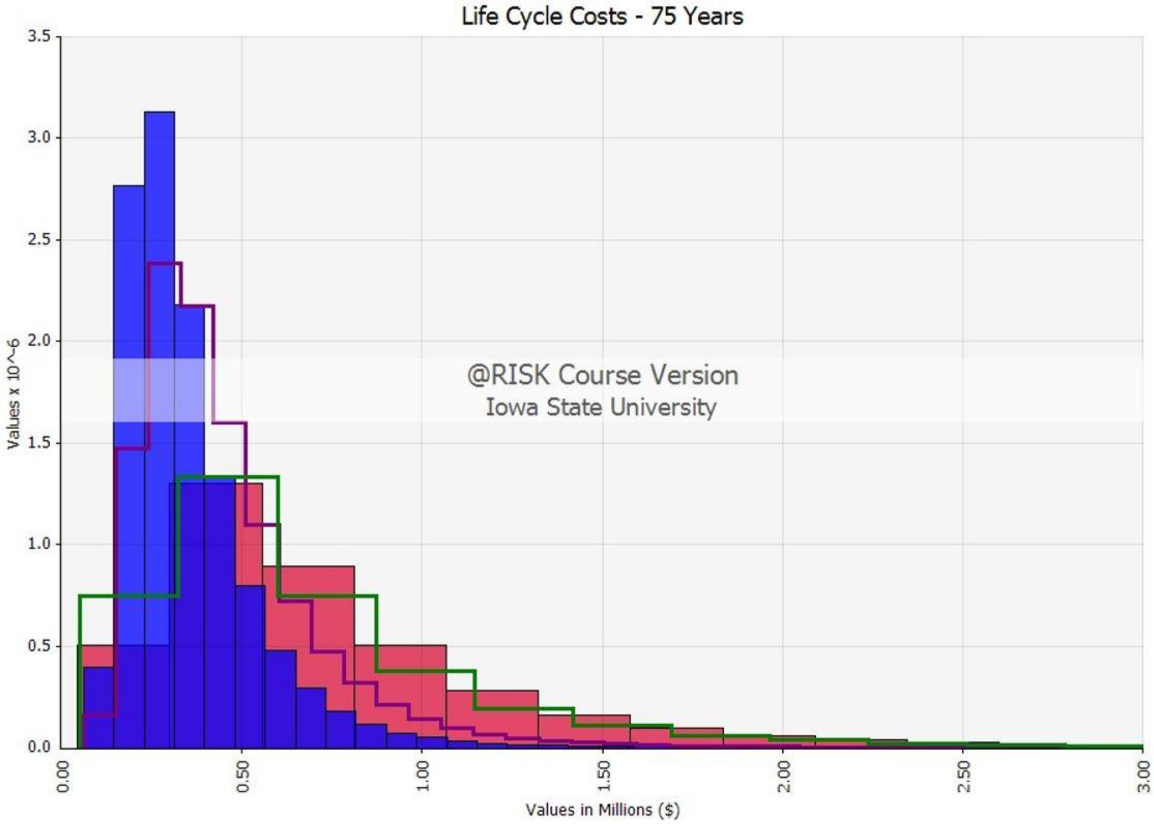


Figure 4.5. Cost distribution for 44-ft bridge, 75-year bridge life

Table 4.11. Monte Carlo statistical summary for 44-ft bridge, 75-year life

Scenario	Color	Life-Cycle Cost ($)		
		Most Likely	Average	90th Percentile
1	Red	393,724	742,692	1,374,751
2	Blue	225,146	350,066	578,822
3	Green	325,286	679,957	1,260,598
4	Purple	290,723	451,940	755,655

The most likely cost of the proposed replacement was about $170,000 less than current practices, with the current practices (scenario 1) becoming the highest estimated costs of all scenarios.

For a 44-ft bridge with a 100-year life, the proposed replacement (scenario 2) had the lowest estimated life-cycle costs (Figure 4.6 and Table 4.12).

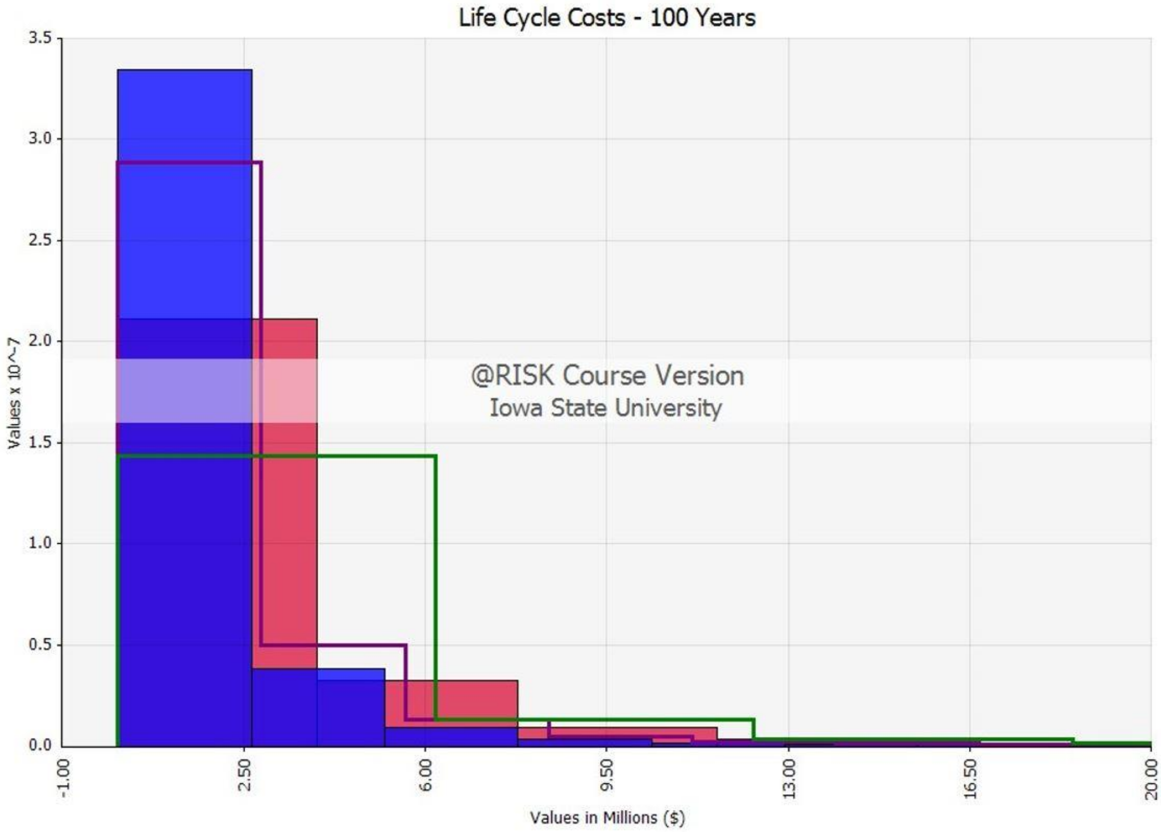


Figure 4.6. Cost distribution for 44-ft bridge, 100-year bridge life

Table 4.12. Monte Carlo statistical summary for 44-ft bridge, 100-year life

Scenario	Color	Life-Cycle Cost ($)		
		Most Likely	Average	90th Percentile
1	Red	571,706	2,726,636	5,883,011
2	Blue	403,909	1,522,541	3,218,814
3	Green	535,015	3,191,090	6,997,887
4	Purple	542,021	2,075,054	4,382,615

The current practices (scenario 1) and scenario 3, once again, had the highest life-cycle costs. Scenario 4 still had lower costs than scenarios 1 or 3, although not as considerably as with the 30-ft bridge.

4.4. Summary

Both a base life-cycle cost analysis and sensitivity study were conducted to evaluate the economic viability of the proposed replacement as outlined in Chapter 3. The proposed replacement, current practices, and two alternative options were considered in this analysis. From these economic studies, the following conclusions can be made:

- For a bridge life of 50 years, the proposed replacement utilizing hydrodemolition, UHPC, and a stainless railing has a lower estimated life-cycle cost than current practices.

- However, the proposed replacement is not the lowest estimated life-cycle cost for a bridge life of 50 years. It is more cost effective to use a retrofitted joint for an A36 railing near the end of the bridge's life instead of using a stainless steel railing for this bridge life.

- For bridge lives of greater than 50 years, the proposed replacement has the overall lowest estimated life-cycle cost of all of the options.

Chapter 5. EXPERIMENTAL TESTING

5.1. Testing Objectives

Experimental testing was conducted in the Iowa State University Structural Engineering Research Laboratory to address several objectives related to the proposed accelerated joint replacement method, as follows:

- Evaluate the constructability of using hydrodemolition and UHPC in the replacement of expansion joints

- Understand the performance of the bonding interface between the existing concrete and newly poured UHPC

- Understand the performance of the proposed replacement as compared to typical DOT joint standards

This chapter details the setup, procedure, and results of all testing conducted for this research. To satisfy the objectives, it was decided to conduct a series of tests on a single laboratory specimen.

First, the specimen was constructed with a joint mimicking a standard Iowa DOT joint with conventional concrete. This joint was tested to create a baseline for an existing joint.

Then, the proposed replacement procedure was conducted and the new joint was tested. This allowed the new procedure and joint to be evaluated for both constructability and effectiveness in structural performance. An outline procedure for the experimental testing is listed below. In addition to this testing program, the concrete bond was evaluated with a series of slant shear and split cylinder tests.

- Construct the test specimen in accordance with Iowa DOT standards
- Conduct vertical load and thermal load performance testing
- Perform the joint replacement
  - Remove 12 in. × 7 in. of concrete along both sides of the joint using hydrodemolition
  - Replace the expansion joint and pour the new UHPC header
- Conduct vertical load and thermal load static testing on the new joint
- Conduct fatigue testing on the new joint

The proposed solution for this this research recommends the use of a stainless steel extrusion in conjunction with UHPC. This allows the whole system to stay in place for the remainder of the bridge life. Due to manufacturing limitations at this time, it was not possible to get a single stainless steel extrusion for the experimental testing. Therefore, a typical A36 steel extrusion was used. Both steel types will perform similarly in terms of mechanical performance. The biggest difference is the increased corrosion resistance and life of the stainless steel extrusion.

5.2. Test Specimen

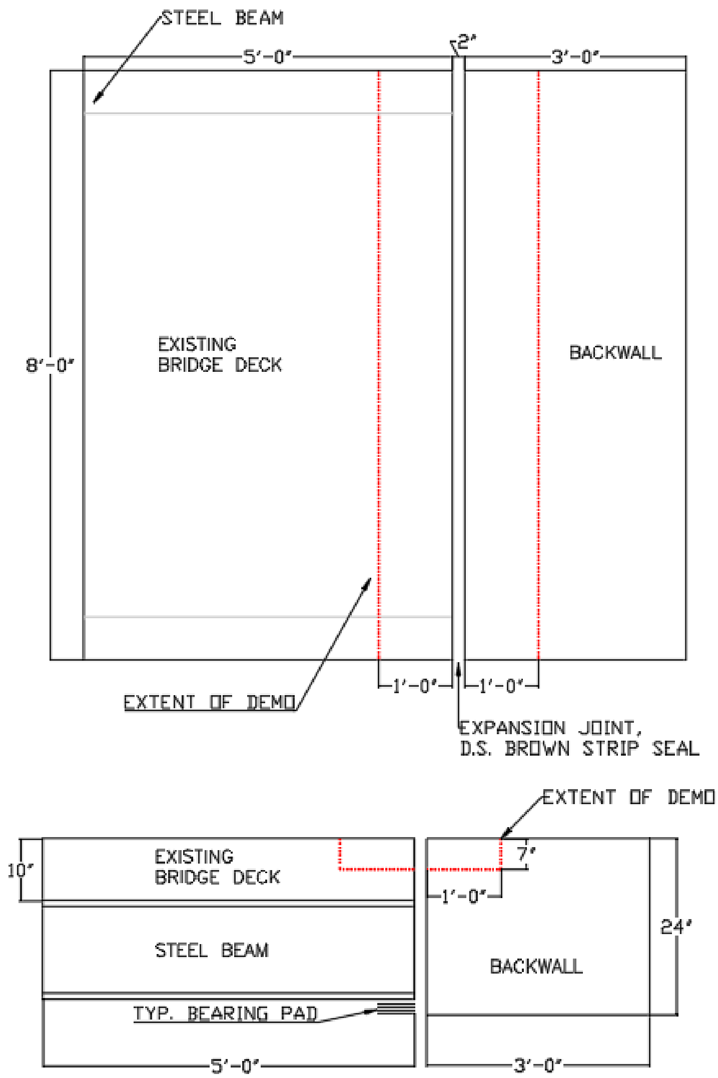
The following sections outline the specific geometry, boundary conditions, steel reinforcing, concrete, and instrumentation used for the test specimen (see Figure 5.1). Each of these are described separately and holistically.

Kara Ruble/BEC

Figure 5.1. D.S. Brown A2R-400 strip seal

5.2.1. Geometry

Overall dimensions of the cast-in-place specimen can be seen in Figure 5.2.



Kara Ruble/BEC

Figure 5.2. Geometry of laboratory specimen

The entire laboratory specimen has an 8 ft × 8 ft footprint. The length encompasses two distinct sections. In these different sections, the thickness varies. The existing bridge deck section has a thickness of 10 in. and is 5 ft long. The backwall section has a thickness of 2 ft and is 3 ft long. Both sections are 8 ft wide.

An expansion joint exists between the existing bridge deck and backwall sections. Both the existing joint and the replacement joint were a D.S. Brown A2R-400 strip seal (see previous Figure 5.1).

The steel extrusion for both joints was comprised of A36 steel. The minimum installation gap for the expansion joint was 2 in., in accordance with the manufacturer's recommendations.

When replacing the expansion joint, a 1-ft strip of concrete was removed with hydrodemolition on both sides of the joint to a depth of 7 in. This demolition area allowed the existing steel extrusion to be removed and replaced with a new steel extrusion. This concrete was replaced with UHPC.

5.2.2. Boundary Conditions

The existing bridge deck section was supported by two 11-in. deep steel beams. The two steel beams were on the outer ends of the width of the section, which mimics the spacing of typical girders on a DOT bridge. The beams had shear studs in the top flanges, embedded into the concrete of the bridge deck (Figure 5.3). The beams were 5-ft long.

Kara Ruble/BEC

Figure 5.3. Shear studs on steel beams

The end of the existing bridge deck near the joint was sitting on 3 in. bearing pads. The other end of the existing bridge deck was supported by a steel pipe/roller filled with concrete. This filled pipe acted as a roller, allowing the specimen to move freely horizontally. The roller was supported by a steel section to maintain a level surface on the specimen (Figure 5.4).

Kara Ruble/BEC

Figure 5.4. Bridge deck roller support

The backwall section of the specimen sat directly on the floor. The end of this section was tied to the strong floor of the laboratory, allowing it to act as a fixed support (Figure 5.5).

Kara Ruble/BEC

Figure 5.5. Backwall fixed support

5.2.3. Reinforcing

Reinforcing for the laboratory specimen followed that of a typical Iowa DOT project with black/uncoated reinforcing steel. The longitudinal reinforcing for the existing bridge deck was #6 bars with 6 in. spacing. The bottom longitudinal bars had 2 in. cover. The transverse reinforcing for the existing bridge deck was #7 bars with 7.5 in. spacing. The top transverse bars had 3.5 in. cover. The backwall section utilized #3 hoop reinforcing with two #5 bars at the top corners spaced 1 ft apart. The hoop reinforcing had 3.5 in. cover. The transverse reinforcing for the backwall was #7 bars with 7.5 in. spacing and 2 in. cover. Table 5.1 summarizes all reinforcing bars, their lengths, and the characteristics for the laboratory specimen.

Table 5.1. Reinforcing bar list

Part	Bar	Location/Type	Number	Length	Spacing
Existing Bridge Deck	#6	Top, Long	15	4 ft-5 in.	6 in.
	#6	Bottom, Long	15	4 ft-5 in.	6 in.
	#7	Top, Transverse	8	7 ft-6 in.	7.5 in.
	#7	Bottom, Transverse	8	7 ft-6 in.	7.5 in.
Backwall	#3	Hoop	7		1 ft
	#5	Top, Transverse	2	7 ft-6 in.	30 in.
	#7	Bottom, Transverse	5	7 ft-6 in.	7.5 in.

5.2.4. Concrete

Typical PCC (standard Iowa DOT C4 mix design) was used for the initial specimen. The entire initial specimen was allowed to fully cure for 28 days after placement. Cylinders were cast to test the compressive strength of the concrete during the curing process. The results of the 28-day compression strength test are show in Table 5.2.

Table 5.2. Concrete 28-day compressive strength

Cylinder #	Strength (psi)
1	5,699
2	5,551
3	5,619
Average	**5,623**

Ductal JS1212 proprietary mix was used for the replacement UHPC. The mix design for this is detailed in Table 5.3.

Table 5.3. Ductal JS1212 UHPC mix design

Item	Mass (lbs/ft^3)
Premix	137.04
Water	7.80
Premia 150 (SP*)	1.12
Optima 100 (Rt*)	0.75
Turbocast 650 (A*)	1.44
Steel fiber @ 2%	9.74

*SP – superplasticiser, Rt – Retarder, A – Accelerant

This concrete was allowed to cure without heat treatment until it reached a compressive strength of 14 ksi as specified by the DOT. UHPC cylinders were also cast to test the compressive strength of this concrete. The compressive strength was tested after 12 hours, 24 hours, and 28 days for the UHPC. The results of these tests are shown in Table 5.4.

Table 5.4. UHPC cylinder compressive strength

Cylinder #	12-Hrs (psi)	24-Hrs (psi)	28-Day (psi)
1	7,853	12,831	20,379
2	7,823	12,703	19,449
3	9,251	13,723	18,202
Average	**8,309**	**13,086**	**19,343**

5.2.5. Instrumentation

Three forms of instrumentation were used over the course of experimental testing: strain gauges, string potentiometer, and direct current displacement transducers (DCDTs).

Strain gauges were used to measure the strain throughout the expansion joint. Four reinforcing steel strain gauges (SG1-4) were used: two on each side of the joint within the demolition area (Figure 5.6).

Kara Ruble/BEC

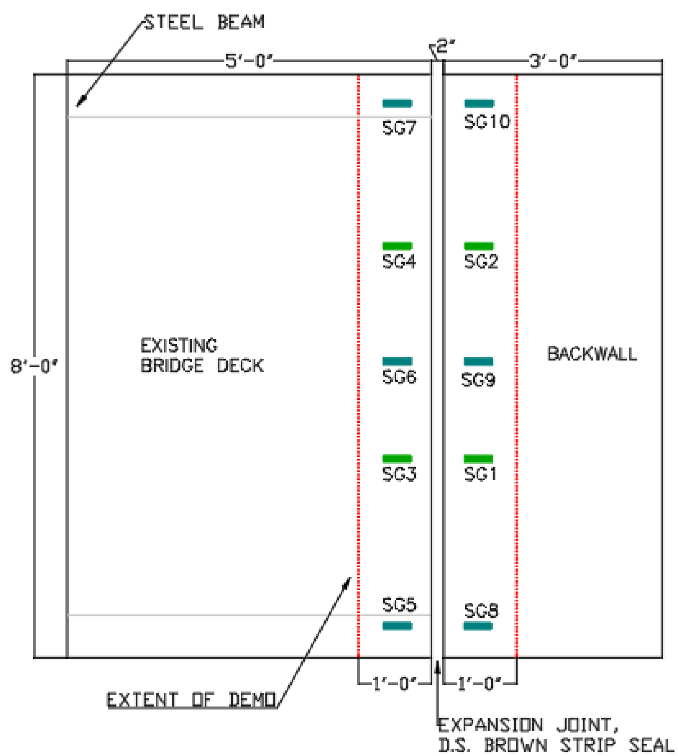
Figure 5.6. Reinforcing strain gauge

The gauges were placed one-third and two-thirds the length of the joint. Six embedded concrete gauges strain (SG5-10) were used (Figure 5.7).

Kara Ruble/BEC

Figure 5.7. Embedded concrete strain gauge

These gauges were placed on the end and middle of the joint on both sides. These locations and labels are shown in Figure 5.8.



Kara Ruble/BEC

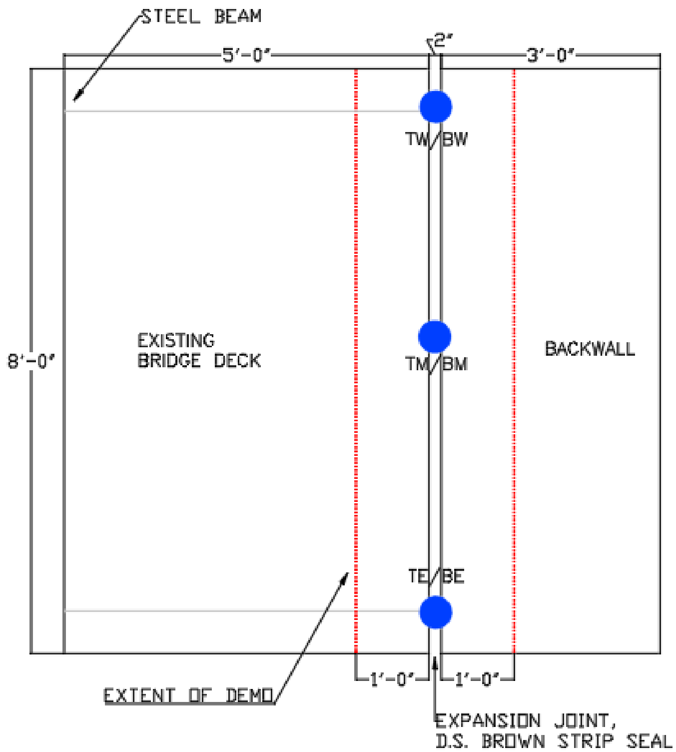
Figure 5.8. Strain gauge layout

String potentiometers (Figure 5.9) were used to measure any displacement that occurred during testing.

Kara Ruble/BEC

Figure 5.9. String potentiometer

Three string potentiometers were placed over the joint to measure horizontal displacement, and three were placed below the joint to measure vertical displacement. The locations of the string potentiometers are shown in Figure 5.10.

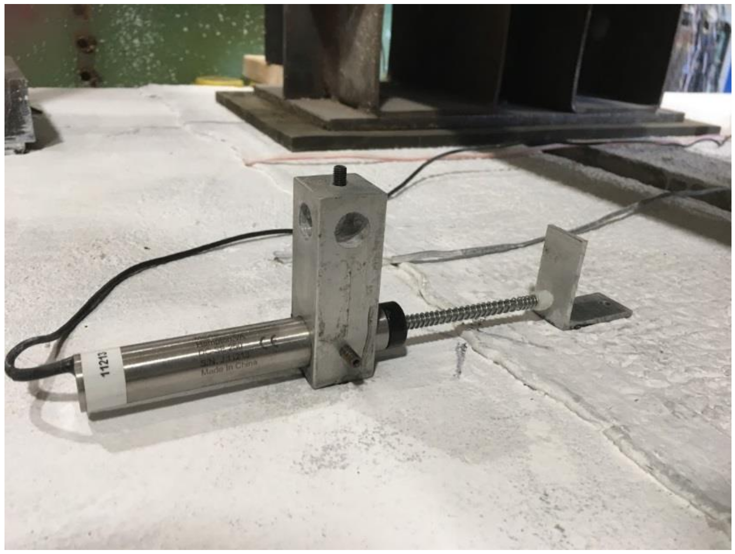


Kara Ruble/BEC

Figure 5.10. String potentiometer layout

The string potentiometers on top of the specimen have labels beginning with T, while the string potentiometers below the specimen have labels beginning with B.

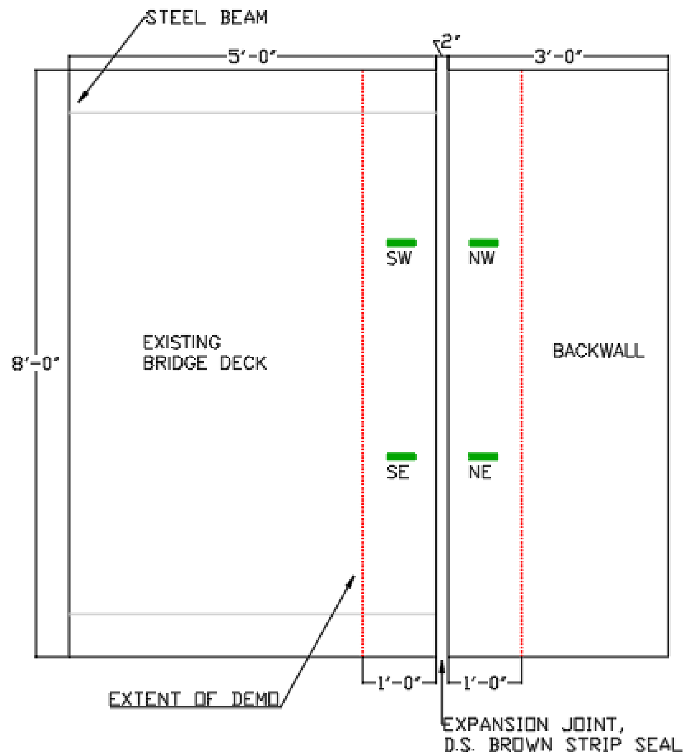
DCDTs were used to measure any displacement that occurred at the interface between the new UHPC and the existing concrete (Figure 5.11).



Kara Ruble/BEC

Figure 5.11. Direct current linear variable differential transformer

Four DCDTs were used, two on either side of the joint. The DCDTs were placed at one-third and two-third the length of the joint. The locations and labels of the DCDTs are shown in Figure 5.12.



Kara Ruble/BEC

Figure 5.12. DCDT layout

5.3. Hydrodemolition

Hydrodemolition was used to remove the necessary concrete for the expansion joint replacement. CLC Hydro Services was contracted to perform the demolition. To accommodate for the water runoff and the hydrodemolition equipment, the demolition took place outside in the loading dock area of the laboratory. CLC Hydro Services used an Aquajet Systems Aqua Cutter 710H (Figure 5.13) to perform the demolition.

Kara Ruble/BEC

Figure 5.13. Aquajet aqua cutter 710H

The Aqua Cutter was supplied with water from a water tank truck that CLC Hydro Services provided (Figure 5.14).

Kara Ruble/BEC

Figure 5.14. Water tank truck

The mobilization for the hydrodemolition took about 3 hours. This included filling the water tank from the fire hydrant, positioning the Aqua Cutter onto the test specimen, hooking up the Aqua Cutter to the water tank, and setting up the wastewater filtering systems. For this project, a series of filter socks and pea gravel was used to filter the concrete particulates out of the water before it entered the storm drain (Figure 5.15).

Kara Ruble/BEC

Figure 5.15. Filter socks

When it was time for the demolition to begin, the width of the demolition and water velocity was programmed into the Aqua Cutter using a hand-held remote. The depth of demolition was set using a trial-and-error process by taking a few passes with the water, checking the depth, and doing another pass, until the appropriate depth was reached. Once the necessary number of passes was determined, the demolition sped up considerably.

Typically, with hydrodemolition, the jet of water runs parallel to the length of the expansion joint. This allows the best access under the reinforcing steel, leaving a clean demolition area and preventing excessive use of a pneumatic breaker afterwards. Due to the small geometry of the test specimen and the configuration of it in the loading dock, it was necessary to orient the Aqua Cutter perpendicular to the joint instead of parallel. This caused the demolition area to be less clean than it could have been, but otherwise had no impact on how the hydrodemolition was conducted.

Through the trial-and-error process, it was determined that five passes of the water jet were necessary to reach the depth of 7 in. This was then programmed into the equipment, and the Aqua Cutter automatically moved down the joint when five passes were complete at one section. It took the Aqua Cutter about 10 minutes to complete 1 ft of demolition. Periodically, CLC Hydro Services would pause the Aqua Cutter to check on the progress of the demolition and make sure no unwanted damage was occurring (Figure 5.16).

Kara Ruble/BEC

Figure 5.16. Hydrodemolition progress

The demolition took about two hours from the start of the trial-and-error process to the completion of demolition. When the Aqua Cutter completed the demolition of the expansion joint, CLC Hydro Services cleaned the demolition area and removed the excess water from the joint (Figure 5.17 and Figure 5.18).

Kara Ruble/BEC

Figure 5.17. Cleaning the joint after hydrodemolition


Kara Ruble/BEC

Figure 5.18. Completed hydrodemolition

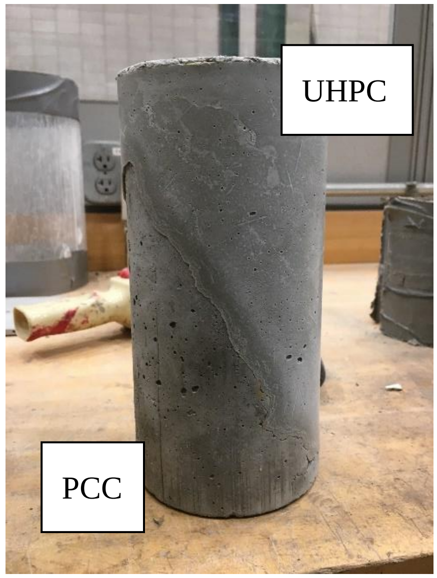
If the rate of this hydrodemolition is extrapolotated for a 30-ft bridge, it would take about 5 hours to demolish the necessary concrete after mobilization. This is likely considerably faster than the time necessary for demolition with a pneumatic breaker. Furthermore, the demolition of the test specimen required about 1 hour of work with a pneumatic breaker to get the remaining concrete underneath the longitudinal reinforcing steel. However, if the Aqua Cutter is allowed to perform the demolition parallel to the expansion joint, this concrete under the reinforcing steel will have been demolished with the Aqua Cutter.

5.4. Bond Evaluation

The bonding interface between the new UHPC and the existing concrete was identified as a potential weak point in the replacement procedure. For the replacement to have an extremely long service life, the bond strength must also be strong. Therefore, slant shear and split cylinder tests were performed to quantify this bond strength in shear and indirect tensile strength, respectfully.

5.4.1. Slant Shear Test

The slant shear test utilizes three cylinders made with half UHPC and half traditional concrete (Figure 5.19).



Kara Ruble/BEC

Figure 5.19. Slant shear cylinder test

The two materials are joined at a 30-degree angle at the halfway point in the cylinders. When a compressive force is applied to these cylinders, they will eventually fail along the 30-degree joint line

The cylinders had a diameter of 4 in. and a length of 8 in. The bonding interface was the shape of an oval, resulting in a bonding interface area of 25.13 in^2. The shear strength (S) can be calculated from this test using Equation (2).

$$S = P / A \qquad \textbf{(2)}$$

where:
P – Experimental maximum load
A – Area of the bond interface

An acceptable shear strength in a bonding surface ranges from 2,000–3,000 psi according to the American Concrete Institute (ACI) Guide to Concrete Repair (Tayeh et al. 2013). The results of the slant shear test are summarized in Table 5.5.

Table 5.5. Slant shear test results

Cylinder #	P (lbs)	S (psi)	Acceptable?
1	44,334	1,764	No
2	51,773	2,060	Yes
3	55,373	2,203	Yes
Average	**50,493**	**2,009**	**Yes**

The shear strength of cylinder 1 does not meet the acceptable strength as specified. However, the average value of shear strength does meet this requirement. All three cylinders experienced failure along the bonding interface, along with fractures in the PCC substrate (Figure 5.20 through Figure 5.22).

Kara Ruble/BEC

Figure 5.20. Cylinder 1 slant shear failure

Kara Ruble/BEC

Figure 5.21. Cylinder 2 slant shear failure

Kara Ruble/BEC

Figure 5.22. Cylinder 3 slant shear failure

5.4.2. Split Cylinder Test

The split cylinder test utilized three cylinders made with half UHPC and half conventional concrete. The two materials were joined down the center of the cylinders vertically (Figure 5.23).

Kara Ruble/BEC

Figure 5.23. Split cylinder interface

The cylinders were then tested on their sides to determine the indirect tensile strength of the bond (Figure 5.24).

Kara Ruble/BEC

Figure 5.24. Split cylinder test

The cylinders had a diameter of 4 in. and a length of 8 in. This resulted in a bonding surface area of 32 in^2. The splitting tensile strength (T) can be calculated from this test using Equation (3).

$$T = (2\ P) / (\pi\ A) \qquad \textbf{(3)}$$

where:
P – Experimental maximum load
A – Area of the bond interface

According to Tayeh et al. (2013), the quality of a bond can be evaluated based on the criteria in Table 5.6.

Table 5.6. Bond quality

Quality	Strength (psi)
Excellent	≥305
Very Good	247–305
Good	203–247
Fair	102–203
Poor	0–102

Values to empirical units from the original source

The results of the split cylinder test are summarized in Table 5.7.

Table 5.7. Split cylinder test results

Cylinder #	P (lbs)	T (psi)	Bond Quality
1	34,436	681.10	Excellent
2	42,153	838.61	Excellent
3	42,162	838.79	Excellent
Average	**39,517**	**786.17**	**Excellent**

Cylinder 1 experienced a primary failure along the bonding interface with some failure in the PCC (Figure 5.25).

Kara Ruble/BEC

Figure 5.25. Cylinder 1 split cylinder failure

Cylinders 2 and 3 both experienced failure in the PCC substrate (Figure 5.26 and Figure 5.27).

Kara Ruble/BEC

Figure 5.26. Cylinder 2 split cylinder failure

Kara Ruble/BEC

Figure 5.27. Cylinder 3 split cylinder failure

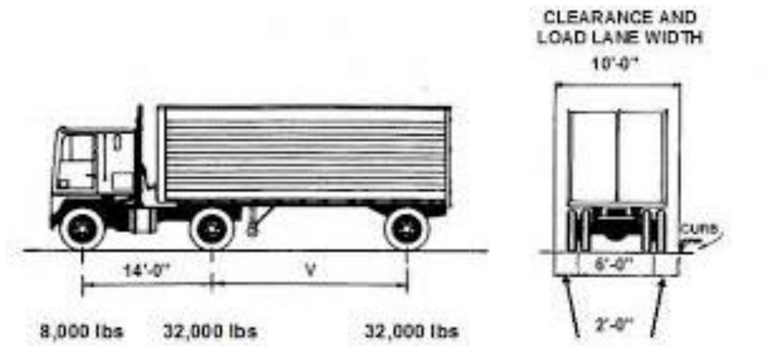
All three cylinders had an indirect tensile strength that far exceeded the threshold for an excellent bond quality. The failure in the PCC substrate, instead of the bonding interface, in cylinders 2 and 3 is further evidence of this excellent bond quality.

5.5. Static Testing

Static testing was applied to both the initial specimen and the specimen with the replacement joint. Vertical loading of an HS-20-44 truck and thermal loading were both considered for this static testing. The same process was used for both cases. The static loading test results were used to confirm that the replacement joint maintains the performance standards required by the Iowa DOT.

5.5.1. Vertical Loading

Vertical loading was applied to the original and replacement specimens to compare the performance of the two joint systems. The loading was designed to mimic the rear axle of an HS-20-44 truck (Figure 5.28).



Ryan et al. 2012/FHWA and NHI

Figure 5.28. HS-20-44 truck

This load was applied to the joint area on two 10 in. × 20 in. loading pads spaced 6 ft apart. An actuator and spreader beam were used to apply the load evenly to the two loading areas. The specimen was initially loaded to 16 kips on each loading area. This load was then increased to 21.3 kips on each loading area. This is equivalent to the impact load of the truck.

Table 5.8 summarizes the results of the vertical load test.

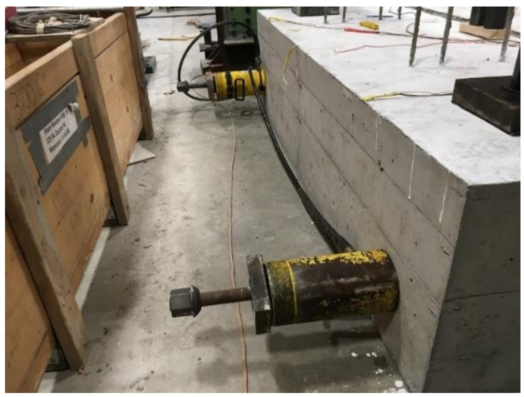
Table 5.8. Vertical test results at impact load

	Original Joint	**Replacement Joint**
SG1	-1	1
SG2	-3	2
SG3	1	1
SG4	2	1
Avg. Reinforcing Strain	**-0.25**	**1.25**
SG5	4	0
SG6	-2	0
SG7	0	-4
SG8	0	1
SG9	-2	0
SG10	0	1
Avg. Concrete Strain	**0**	**-0.33**
TW	-0.05	-0.043
TM	-0.049	-0.046
TE	-0.042	-0.046
Avg. Horiz. Displacement (inch)	**-0.047**	**-0.045**
BW	-0.049	-0.067
BM	-0.067	-0.073
BE	-0.072	-0.092
Avg. Vert. Displacement (inch)	**-0.063**	**-0.077**

Both the reinforcing strain and concrete strain was minimal for the original and replacement joints. The maximum average displacement experienced in the two joints was 0.063 in. for the original joint and 0.077 in. for the replacement joint. These results indicate that the replacement joint performs similarly to the Iowa DOT standard expansion joint.

5.5.2. Thermal Loading

Thermal loading was applied to the original and replacement specimens to compare the performance of the two joints under thermal expansion and contraction. To mimic thermal loading, actuators were used to expand and contract the expansion joint gap in the specimens (Figure 5.29).

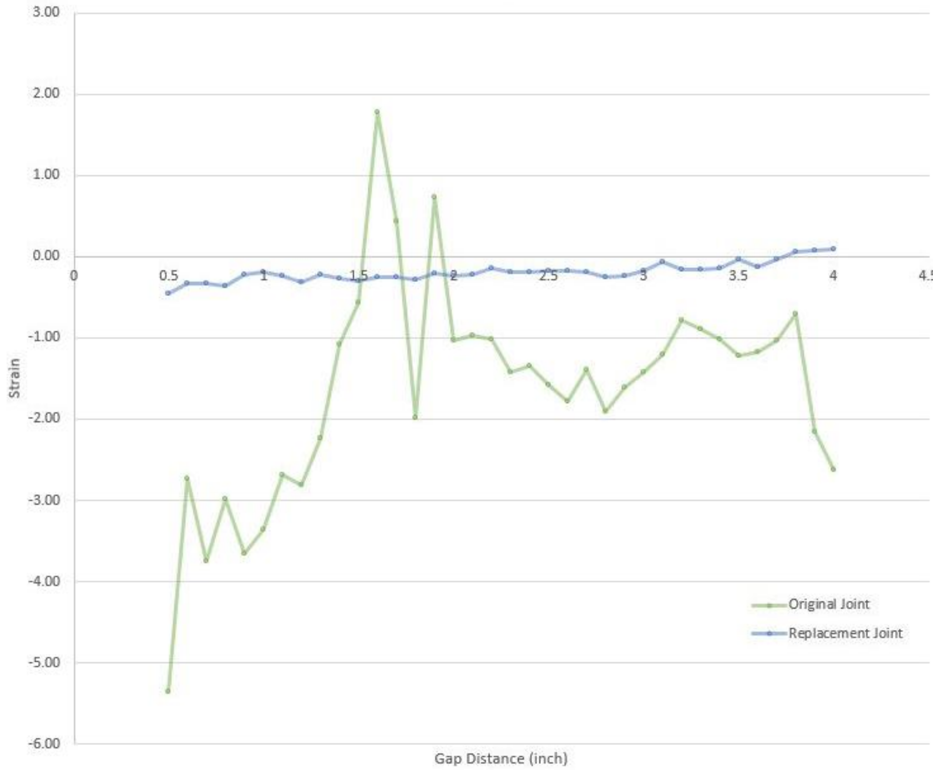


Kara Ruble/BEC

Figure 5.29. Actuators used to mimic thermal loading

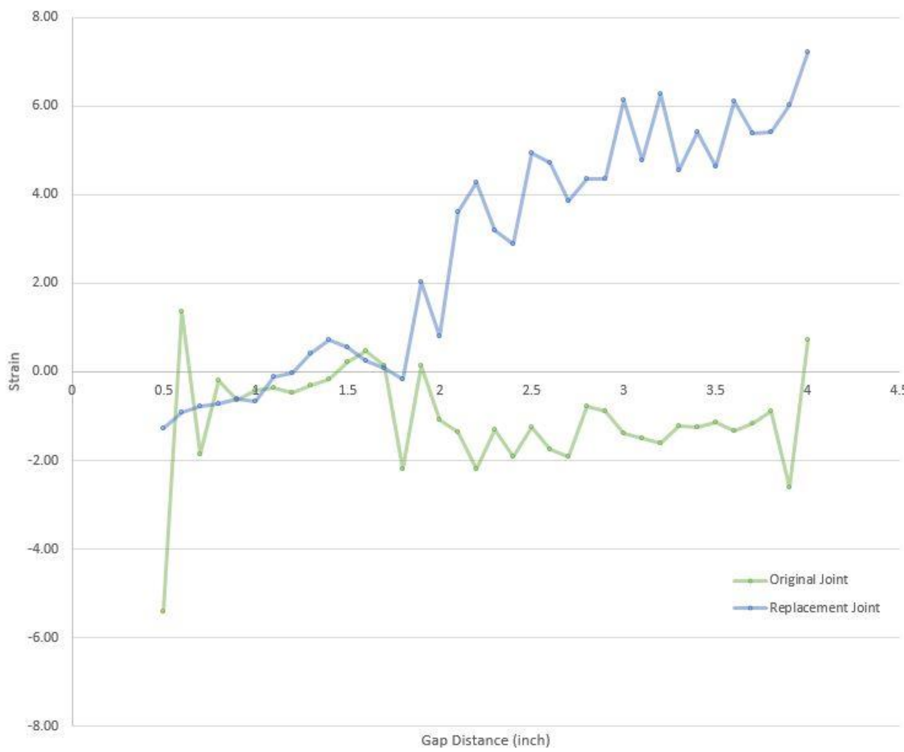
This allowed the specimens to undergo simulated tensile and compressive forces due to thermal expansion and contraction. The full range of the strip seal was explored.

Figure 5.30 and Figure 5.31 summarize the average concrete and reinforcing strain, respectfully, experienced in the two joint systems across the range of gap distances.



Kara Ruble/BEC

Figure 5.30. Average concrete strain, thermal performance test



Kara Ruble/BEC

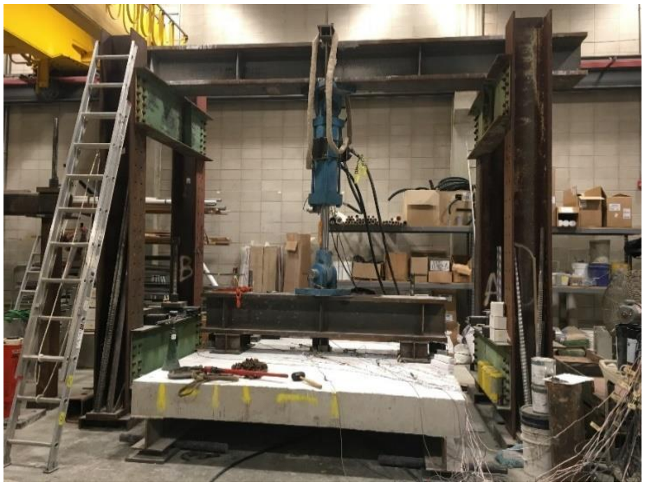
Figure 5.31. Average reinforcing strain, thermal performance test

For all cases, the strain values were relatively minor, similar to what was seen with the vertical load test results. For the concrete strain, the original joint experienced a greater range of strain values, and the replacement joint strain values stayed consistently near zero. This indicates that the UHPC was virtually unaffected by the effects of thermal loading.

For the reinforcing strain, the opposite pattern was seen. The replacement joint experienced the greater strain, while the original joint's strain values stayed near zero. Nevertheless, both joint systems performed well under thermal loading and met Iowa DOT standards.

5.6. Fatigue Testing

After all static testing was completed, the UHPC replacement joint underwent fatigue testing (Figure 5.32).

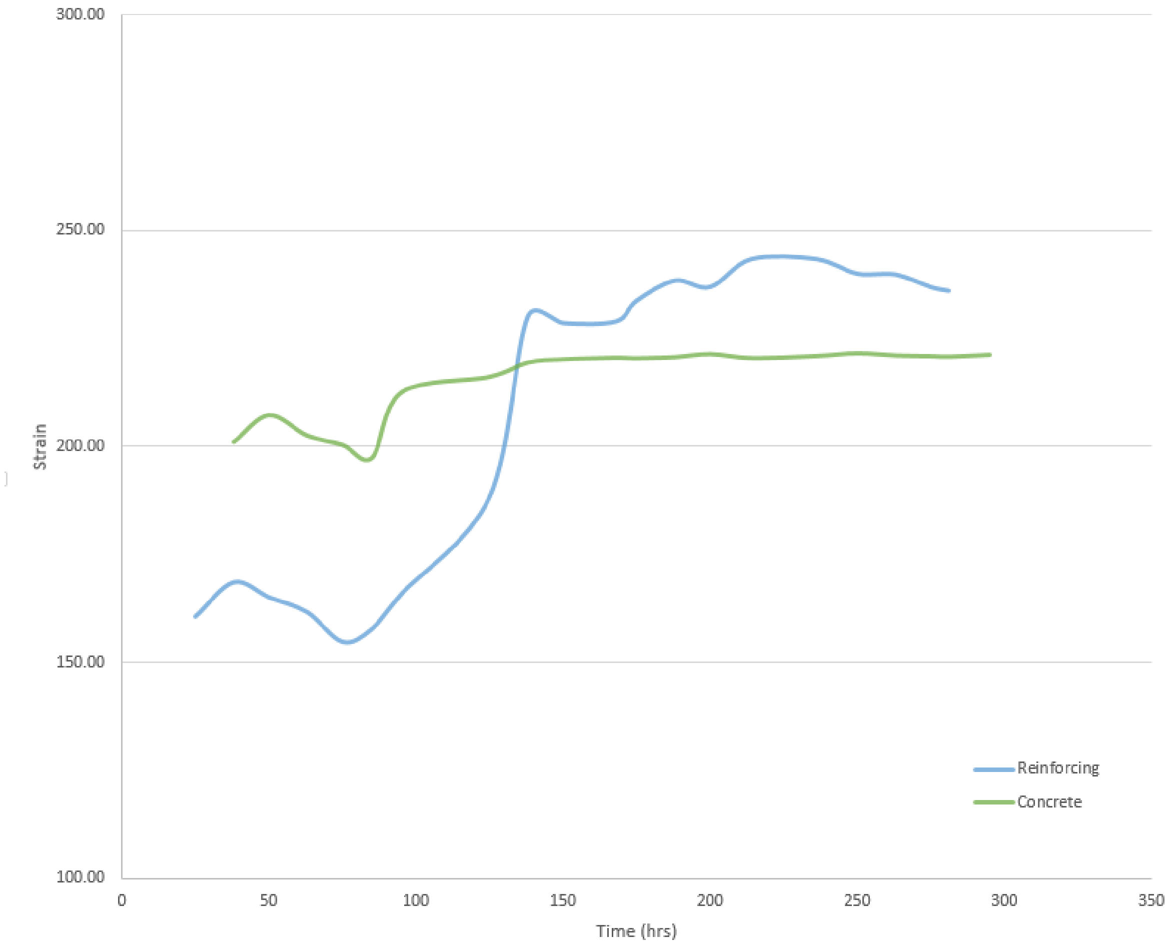


Kara Ruble/BEC

Figure 5.32. Fatigue loading setup

This was used to evaluate the durability of UHPC/joint over time. Cyclic loading was applied on the specimen at service level conditions. 1,000,000 cycles were applied at 1 Hz. It took about 2 days to complete. Visual inspection for cracking was conducted and documented every 250,000 cycles.

Figure 5.33 summarizes the average concrete and reinforcing strain experienced in the specimen throughout the length of the fatigue test.



Kara Ruble/BEC

Figure 5.33. Average strain over time

The strain in both the concrete and reinforcing varied very little throughout the test. Visual inspection of the specimen showed no signs of cracking or wear (Figure 5.34 and Figure 5.35).

Kara Ruble/BEC

Figure 5.34. Visual inspection of specimen

Kara Ruble/BEC

Figure 5.35. Visual inspection of joint

5.7. Summary

Bond testing, static testing, and fatigue testing were conducted to compare the performance of the proposed joint replacement to current DOT expansion joints. Hydrodemolition was used to perform the concrete removal portion of the joint replacement as proposed in Chapter 3 in order to evaluate the effectiveness and speed of the demolition. From these tests, the following conclusions can be made:

- For CLC Hydro Services, 3 hours were needed to complete the mobilization for hydrodemolition. After that, it took about 10 minutes per foot of expansion joint to complete the demolition. This indicates that hydrodemolition is an effective option for the accelerated demolition of concrete in an expansion joint replacement.

- Both the slant shear and split cylinder tests showed the bond between UHPC and traditional concrete to be excellent. The shear strength of the bond was within the acceptable range, and the indirect tensile strength can be categorized as excellent. This indicates that the existing concrete will fail before the bond interface in the new expansion joint system.

- There were no meaningful changes from the static testing of the original joint to those from the replacement joint. Both met the requirements of the Iowa DOT standards and would perform adequately.

- Fatigue testing up to 1,000,000 cycles had very little effect on the specimen. This indicates that the new expansion joint system would be resilient to traffic loads over long periods of time.

Chapter 6. CONCLUSIONS AND RECOMMENDATIONS

Through the course of this research, a literature review was conducted and methods for an accelerated expansion joint replacement were developed. The combination of a stainless steel railing and UHPC header with hydrodemolition was evaluated for its effectiveness as an accelerated construction option. This combination would provide a replacement option that could last the remainder of the bridge life. Instead of multiple full replacements in the bridge life, this would only require the replacement of the neoprene strip seal gland approximately every 15 years. This gland replacement is routinely performed by DOT maintenance crews.

The proposed replacement method involves high initial costs and required evaluation of its economic viability. A life-cycle cost analysis with a sensitivity study compared the proposed replacement to current practices and two alternative methods. This analysis revealed that, for bridges with a life of longer than 50 years, the proposed replacement was the most cost-effective option. For bridges with a life of 50 years or fewer, it is more cost effective to consider a joint retrofit near the end of the bridge's life.

The proposed replacement joint also underwent bonding, static, and fatigue testing in the Iowa State University Structural Engineering Research Laboratory. Hydrodemolition was also used in the replacement process of the testing. These tests indicated that the joint system utilizing hydrodemolition produces an excellent bond with the existing concrete. The static and fatigue testing revealed the joint system meets DOT standards and would likely have a long service life.

REFERENCES

Baker Engineering & Energy. 2006. *Evaluation of Various Types of Bridge Deck Joints*. Arizona Department of Transportation, Phoenix, AZ.

Burris, L., K. Kurtis, and T. Morton. 2015. *Novel Alternative Cementitious Materials for Development of the Next Generation of Sustainable Transportation Infrastructure***.** Federal Highway Administration, McLean, VA.

Cronin, M. and B. Phares. 2016. Economic Viability of Implementing Empirical Bridge Deck Design with Stainless Reinforcing Steel. *International Journal of Civil and Structural Engineering Research,* Vol. 3, No. 2, pp. 11–23.

DOSHIN® Rubber Engineering. n.d. *DOSHIN 3-DOF Modular Expansion Joint*. DOSHIN® Rubber Engineering, Selangor Darul Ehsan, Malaysia. https://doshinrubber.com/wp-content/uploads/2015/05/3-DOF-Modular-System-Rev.-2.pdf.

D.S. Brown. n.d. *Steelflex® Strip Seal Expansion Joint Systems*. D.S. Brown, North Baltimore, OH. https://dsbrown.com/wp-content/uploads/2018/11/B_EJS_NewConstrucEJS_BRO00-5723_v065-WEB_StripSeal_Pages.pdf.

EMSEAL. n.d. *BEJS: Bridge Expansion Joint System*. EMSEAL Joint Systems, Ltd. Westborough, MA. https://www.emseal.com/bridge-expansion-joint-division/wp-content/uploads/sites/3/2018/10/emseal-bejs-information-guide_bridges-roadways.pdf.

Gergely, J., V. Ogunro, and M. Manus. 2009. *Material Property and Quality Control Specifications for Elastomeric Concrete Used at Bridge Deck Joints*. University of North Carolina at Charlotte, NC.

Graybeal, B. A. 2006. *Material Property Characterization of Ultra-High Performance Concrete*. Federal Highway Administration, McLean, VA.

Iowa DOT. 2018. *Iowa DOT LRFD Bridge Design Manual*. Iowa Department of Transportation Office of Bridges and Structures, Ames, IA.

Iowa DOT. Contracts: Plans and Estimating Proposals. https://iowadot.gov/contracts/Plans-and-estimation-proposals.

Iowa DOT. Bid Tabulations. https://iowadot.gov/contracts/Historical-Completed-lettings/Bid-Tabs.

Li, Y. and B. Chen. 2013. Factors that Affect the Properties of Magnesium Phosphate Cement. *Construction and Building Materials,* Vol. 47, pp. 977–983.

Miller, A. M. and C. T. Jahren. 2014. *Rapid Replacement of Bridge Deck Expansion Joints Study – Phase I.* Institute for Transportation, Construction Management and Technology Program, Iowa State University, Ames, IA. https://intrans.iastate.edu/app/uploads/2018/03/bridge_deck_expansion_joint_repair_phase_I_w_cvr.pdf.

Miller, A. M. and C. T. Jahren. 2015. *Rapid Bridge Deck Joint Repair Investigation – Phase II.* Draft Final Report. Institute for Transportation, Construction Management and Technology Program, Iowa State University, Ames, IA.

Morandeira, D., E. Miller, and C. T. Jahren. 2019. *Rapid Bridge Deck Joint Repair Investigation – Phase III.* Draft Final Report. Institute for Transportation, Construction Management and Technology Program, Iowa State University, Ames, IA.

Phares, B. and M. Cronin. 2015. *Synthesis on the Use of Accelerated Bridge Construction Approaches for Bridge Rehabilitation*. Accelerated Bridge Construction-University Transportation Center (ABC-UTC), Florida International University, Miami, FL.

Ryan, T. W., J. E. Mann, Z. M. Chill, and B. T. Ott. 2012. *Bridge Inspector's Reference Manual*. FHWA NHI 12-049. Federal Highway Administration and National Highway Institute, Arlington, VA.

Smith, Keith/MoDOT. 2012. 751.13 Expansion Joint Systems. In MoDOT Engineering Policy Guide. https://epg.modot.org/index.php/751.13_Expansion_Joint_Systems.

Tayah, B. A., B. H. Abu Bakar, M. A. Megat Johari, and Yen Lei Voo. 2013. Evaluation of Bond Strength between Normal Concrete Substrate and Ultra High Performance Fiber Concrete as a Repair Material. *Procedia Engineering*, Vol. 54, pp. 554–563.

Vorster, M. C., J. P. Merrigan, R. W. Lewis, and R. E. Weyers. 1992. *SHRP-S-336: Techniques for Concrete Removal and Bar Cleaning on Bridge Rehabilitation Projects*. Strategic Highway Research Program, Washington, DC.

White House Office of Management and Budget. 2017. Appendix C: Discount Rates for Cost-Effectiveness, Lease Purchase, and Related Analyses. *Guidelines and Discount Rates for Benefit-Cost Analysis of Federal Programs.* OMB Circular No. A-94. Washington, DC.