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## Accelerated repair and replacement of expansion joints

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**Accelerated repair and replacement of expansion joints**

by

**Kara A. Ruble**

A thesis submitted to the graduate faculty  
in partial fulfillment of the requirements for the degree of

**MASTER OF SCIENCE**

Major: Civil Engineering (Structural Engineering)

Program of Study Committee:  
Brent M. Phares, Co-major Professor  
Katelyn Freeseaman, Co-major Professor  
Charles Jahren

The student author, whose presentation of the scholarship herein was approved by the program of study committee, is solely responsible for the content of this thesis. The Graduate College will ensure this thesis is globally accessible and will not permit alterations after a degree is conferred.

Iowa State University

Ames, Iowa

2020

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## ABSTRACT

Bridge deck expansion joints are the components of the bridge that help to accommodate for the movement of bridges due to thermal expansion and, to a lesser extent, dynamic loading. They may also serve to help prevent the passage of 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 the 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 multiple weeks. These replacements typically involve extensive traffic interference and lane closure. Therefore, there is a need for accelerated replacement options and techniques, especially in areas 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 herein focuses on developing such methods for accelerated joint replacements.

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 option.

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 greater than 50 years, the proposed replacement was the most cost effective option.

The proposed replacement joint also underwent bonding, static, and fatigue testing in the ISU structures 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.

## CHAPTER 1. INTRODUCTION

### 1.1 Problem Statement

Bridge deck expansion joints are the components of the bridge that help to accommodate for the movement of bridges due to thermal expansion and, to a lesser extent, dynamic loading. They may also serve to help prevent the passage of 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 the 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 multiple weeks. These replacements typically involve extensive traffic interference and lane closure. Therefore, there is a need for accelerated replacement options and techniques, especially in areas 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 herein focuses on developing such methods for accelerated joint replacements.

## 1.2 Research Objectives and Activities

The objectives of this research are to: (1) conduct a literature review on the repair and replacement of bridge deck expansion joints; (2) develop methods for accelerated bridge expansion joint replacement; and (3) promote ABC for bridge deck expansion joint repair.

These objectives will be achieved based on the following project activities.

- Literature review focused on current practices and options for accelerated joint replacement and elimination
- Development of an accelerated replacement method for bridge expansion joints
- Life cycle cost analysis to confirm economic feasibility of the proposed methods
- Experimental laboratory testing to confirm constructability and effectiveness of the proposed methods

## CHAPTER 2. LITERATURE REVIEW

A literature review was conducted to investigate typical joints used by the Iowa 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 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 will be addressed in more detail in a later section.

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 can be seen in Figure 2.1. Sliding plate joints are no longer used by the Iowa DOT in replacements or new construction, but there are still some older bridges that have these joints in place. Sliding plate joints are not water tight. 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 structural strength. This can lead to the plates separating from their attachments. This, in conjunction with extensive cyclic loading from traffic, leads to failure in sections of the joint.



**Figure 2.1** Typical sliding plate joint (MO DOT)



**Figure 2.2** Sliding plate fused and separated from abutment (Jahren & Miller)



**Figure 2.3** Cracked sliding plate joint (Ohio DOT)

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 can be seen in Figure 2.4. 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. These glands can be removed and replaced with relative ease, and does 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 snow plow damage, or if there is severe spalling of the concrete header surrounding the joint.



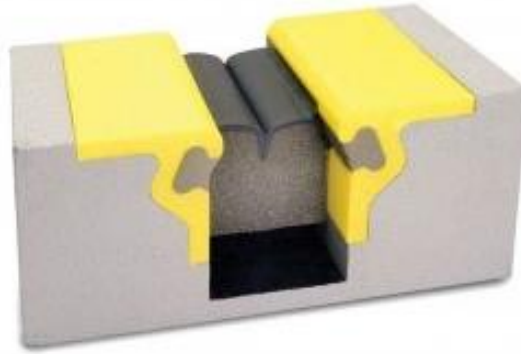


**Figure 2.4** Typical strip seal joint (ArchiExpo)



**Figure 2.5** Strip seal joint with punctured gland (Baker)

In recent years, a strip seal retrofit designed by EMSEAL 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. It can be installed with minimal, if any, demolition, and bypasses the need for a steel extrusion. A typical profile of the EMSEAL BEJS can be seen in Figure 2.6.



**Figure 2.6** EMSEAL BEJS retrofit schematic (EMSEAL)



**Figure 2.7** EMSEAL BEJS retrofit (EMSEAL)

Compression joints consist of a compressive seal in the gap between the bridge deck and abutment concrete. A typical compression joint can be seen in Figure 2.8. Although compression joints are being phased out of use by the Iowa DOT, there are still many bridges in Iowa that use them. Compression joints have similar failure patterns to strip seal joints. The compression seal is typically the first thing to fail after around 10 years. With compression joints, it is optional whether or not to include a steel armor with the joint. If the steel armor is included, it typically fails due to rust or the cyclic loading of traffic.



**Figure 2.8** Typical compression joint (MO DOT)



**Figure 2.9** Compression joint with broken anchor (Miller)

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. Both of these joints are very effective for larger 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 water tight, however, which can lead to erosion of the soil below and/or damage to substructure elements. Modular joints fail similarly to strip seals, and can be repaired in a similar manner.



**Figure 2.10** Typical finger joint (Haider)



**Figure 2.11** Typical modular joint (Mayer)



**Figure 2.12** Modular joint with debris, preventing closure (Miller)

## 2.2 Bottlenecks in Construction

Miller and Jahren (2014) discovered in phase one 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 feet on both sides of the joint, and 10-12” deep. However, the steel extrusion currently used by the Iowa DOT is only 1 foot wide on each side and 7” deep.

Reducing the demolition area to these dimensions significantly reduce 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 will require 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 (Figure 2.13). This is a familiar demolition method for most contractors. It creates concrete blocks that are relatively easy to remove and have 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 new 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 would need to be filled (Phares and Cronin).



**Figure 2.13** Saw cutting (EMSEAL)

### 2.2.1.2 Pneumatic breakers

Pneumatic breakers are hand held 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. 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). Pneumatic breakers also create a rough surface that is potentially good for the bonding between new and old concrete.

The largest drawback to pneumatic breakers for accelerated removal is the large workforce and time needed to complete the demolition. These are significantly longer than 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.



**Figure 2.14** Pneumatic breaker (EMSEAL)

### 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 (Figure 2.15). The operator controls the rate of the water pressure, speed of the nozzle as it performs its passes, and the 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.





**Figure 2.15** Hydrodemolition equipment (Equipment world)

### 2.2.2 Concrete

Magnesium phosphate cement, elastomeric concrete, and ultra-high performance 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

Magnesium phosphate cement (MPC) is a mixture utilizing dead burned magnesia and phosphate in a manner similar to Portland cement. It has many advantages over Portland cement, including fast set time, high early strength, and it can be cast in cold weather. MPC can, on average, set in 20-30 minutes, and can be cast in temperatures ranging from  $-5^{\circ}\text{F}$  to  $86^{\circ}\text{F}$ . If it is cast near the lower temperature limit, the set time will increase to around 80 minutes. Once set, MPC can be opened to traffic after 3 hours (Yue 2013).

Historically, MPC has only been used for smaller patch jobs (Figure 2.16). 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 2015).



**Figure 2.16** Magnesium phosphate cement (Burris)

#### 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. It bonds well to concrete and steel and, unlike Portland cement concrete, 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 2009). The service life of elastomeric concrete is approximately 25 years before it needs to be replaced.



**Figure 2.17** Elastomeric concrete (D.S. Brown)

### 2.2.2.3 Ultra-high performance concrete

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 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, other de-icing salts, and resists freeze-thaw cycles well. The set time of UHPC relies heavily on the ambient temperature. Most mixes of UHPC 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 Portland cement concrete, 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.



Source: New York State Department of Transportation

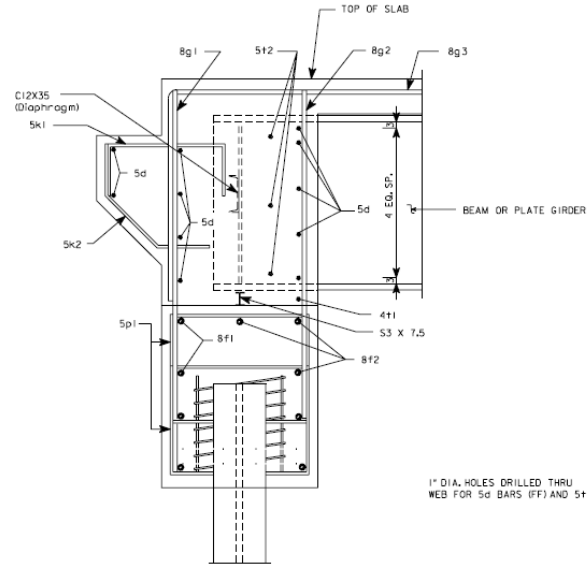
**Figure 2.18** Ultra high performance concrete (NY DOT)

## 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.

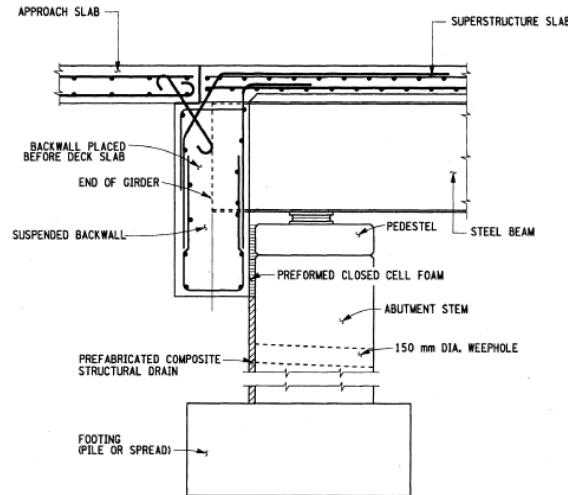
### 2.3.1 Integral and Semi-integral abutments

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.



**Figure 2.19** Typical integral abutment (Morandeira)

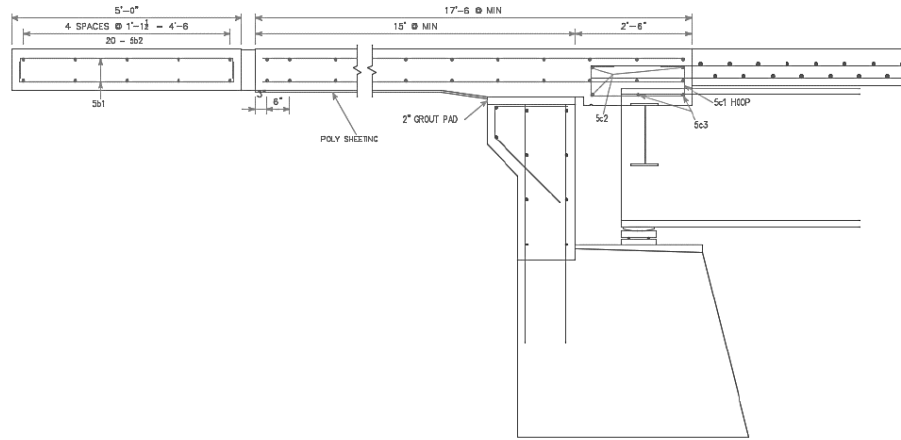
Semi-integral abutments differ from integral abutments in that semi-integral abutments still have the girders sit on a bearing pad. The girder ends are still encased in the backwall, but the bearing pads allow the foundation to be fixed instead of moving with thermal effects. Semi-integral abutments are typically used when it is not possible to use an integral abutment (due to length, skew, or other factors). They 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.



**Figure 2.20** Typical semi-integral abutment (Moranderia)

### 2.3.2 Deck over backwall concept

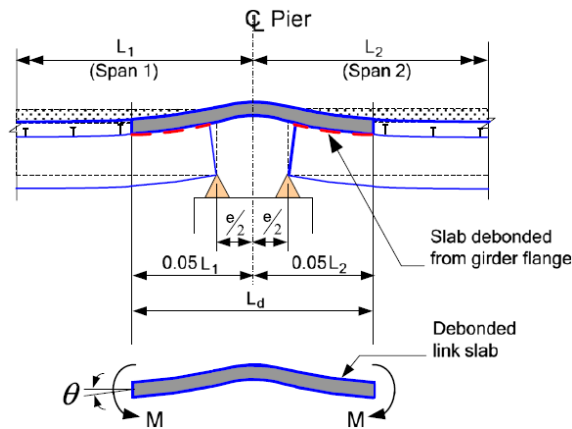
The deck over backwall concept was developed in phase three of Jahren's research into the accelerated repair of bridge expansion joints. This concept explores the idea of moving the expansion joint that is typically at the abutment into the bridge approach slab. This allows any water that seeps through 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.



**Figure 2.21** Preliminary deck over backwall design (Morandeira)

### 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 would cure quickly. A typical link slab can be seen in Figure 2.22.



**Figure 2.22** Typical link slab (Morandeira)

### CHAPTER 3. PROPOSED REPLACEMENT SYSTEM

Based on the findings of the literature review in chapter 2, 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, demolition method, and 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

Out 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 larger bridges requiring greater than 5 inches of movement. As, this report focuses on bridges of moderate length, a strip seal joint was chosen as the expansion joint for the replacement.

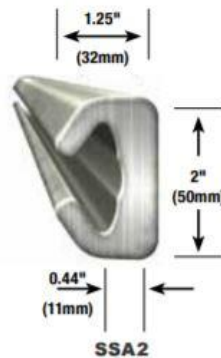
The Iowa DOT currently approves strip seal joints manufactured by two companies: Watson & Bowman and D.S. Brown. Both products are similar to one another. After speaking to a representative from D.S. Brown, it was discovered that D.S. Brown has specifications for making a steel railing for a strip seal joint out of stainless steel. These steel railings are typically constructed with A36 steel, which commonly has a life of



approximately 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. In order to evaluate both the A36 and stainless steel railing options, the D.S. Brown A2R-400 strip seal was selected as the appropriate joint for this study.

Sealing Element Cross-Section	Sealing Element	Movement Range		Joint Opening
		MR <sub>L</sub>	MR <sub>T</sub>	
 A2R	A2R - 400	4.0 (102)	±2.0 (51)	0.5 - 4.5 (13) (114)

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



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

### 3.2 Demolition

All the demolition methods evaluated have advantages and disadvantages that must be considered if chosen as the demolition method “of choice” 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 could be 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 would require 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 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 on the conditions that the wastewater can be properly handled and 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 concretes 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 on the results of a life cycle cost analysis.

### **3.4 Component Interaction**

In addition to their individual advantages, these components were chosen for the increased advantage of them working together as a system. Stainless steel railings can last until the end of the bridge 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 life. UHPC requires a longer curing time than other fast-setting concretes, but it can still be accomplished within a weekend. Even with this in mind, UHPC and a stainless steel railing would only need one full replacement in the remaining life of the bridge. One weekend of construction for the replacement in the whole life of the bridge is more desirable than multiple days of construction 3-4 times throughout the life of the bridge disrupting traffic. 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 afterwards.

## 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 is being explored to extend the life of the replacement joint and further reduce life-cycle costs and time associated with replacing the joint in the future. This chapter will address the viability of this proposed replacement from a life-cycle cost perspective.

Four scenarios were compared in this cost analysis (Table 4.1). The first represents the current procedures/materials included in a typical joint replacement. It uses PCC, A36 steel extrusions, and pneumatic breaker demolition. The second utilized 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 chapter 2, 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 typical PCC and pneumatic breaker demolition, but still uses stainless steel extrusions. The spalling of the PCC controls the time between replacements.

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	

## 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 feet, and the second had a width of 44 feet. 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' on both sides of the joint and a depth of 7" was used. This results in a concrete quantity of 1.3 cubic yards for the 30' wide bridge and 1.9 cubic yards for the 44' 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. 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.

<b>Material</b>	<b>Average Service Life (yrs)</b>
PCC	30
UHPC	100+
A36 Steel extrusion	25
Stainless Steel extrusion	100+
Neoprene Strip Seal Gland	15
EMSEAL BEJS	15

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 Portland cement concrete. When a neoprene gland fails for a strip seal, they can be easily 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. 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.

<b>N*</b>	<b>Scenario 1:</b> Pneumatic Breaker, PCC, A36 Rail	<b>Scenario 2:</b> Hydrodemolition, UHPC, Stainless Rail	<b>Scenario 3:</b> Hydrodemolition, UHPC, A36 Rail	<b>Scenario 4:</b> Pneumatic Breaker, PCC, Stainless Rail
0	Full Replace	Full Replace	Full Replace	Full Replace
5				
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	

\*N being the number of years past the initial replacement of the joint

#### 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 replacement conducted by

the Iowa DOT in FY18-19. The costs for Ductal UHPC and EMSEAL BEJS were taken directly from the manufactures. The average costs can be seen in Table 4.4. The geometry of the bridges have already been taken into account for these costs.

<b>Table 4.4 Average costs</b>		
<b>Item</b>	<b>Average Cost (\$)</b>	
	<b>30' Bridge</b>	<b>44' Bridge</b>
PCC	49.34	72.11
UHPC	2860.00	4180.00
Hydrodemolition	324.28	476.19
Pneumatic Breaker	87.25	128.12
EMSEAL BEJS	1350.00	1980.00
Steel extrusion w/ Neoprene	6476.70	9499.16
Stainless Steel extrusion w/ Neoprene	12953.40	18998.32
Neoprene Gland Installation & Testing	1465.80	2149.84
Neoprene Gland Install & Testing – Gland Replacement Only	3725.10	5463.48
Mobilization	7666.67	7666.67
Mobilization – Gland Replacement Only	24840.14	24840.14
Traffic Control	6312.50	6312.50
Traffic Control – Gland Replacement Only	6196.67	6196.67
Temporary Barrier Rail	296.40	434.72
Temporary Crash Cushion	1017.27	1017.27
Temporary Traffic Signals	4321.25	4321.25
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

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

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

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. For all bridge lives considered for each bridge widths 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 has the lowest costs of all scenarios. For the 50 year bridge life, scenario 3 had the lowest cost. This is 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.



<b>Table 4.5 Base life cycle costs</b>					
	<b>Scenario</b>	<b>Description</b>	<b>Bridge Life (years)</b>		
			<b>50</b>	<b>75</b>	<b>100</b>
<b>30' Bridge</b>	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
<b>44' Bridge</b>	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

### 4.3 Sensitivity Study

Since the parameters used in this analysis will vary with time and 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 is complete, the results of all the iterations can 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 variable parameters. Using the same Iowa DOT bid tabs as before, a statistical distribution was created for each item. The interest rate is also considered a varying parameter. The statistical distributions used are summarized in Table 4.6. 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 \$3,146.00 and a low value of \$286.00. A block distribution was used for UHPC to account for its relatively new use in industry. It is assumed that the price of UHPC will decrease significantly over the coming years as it is more widely used.

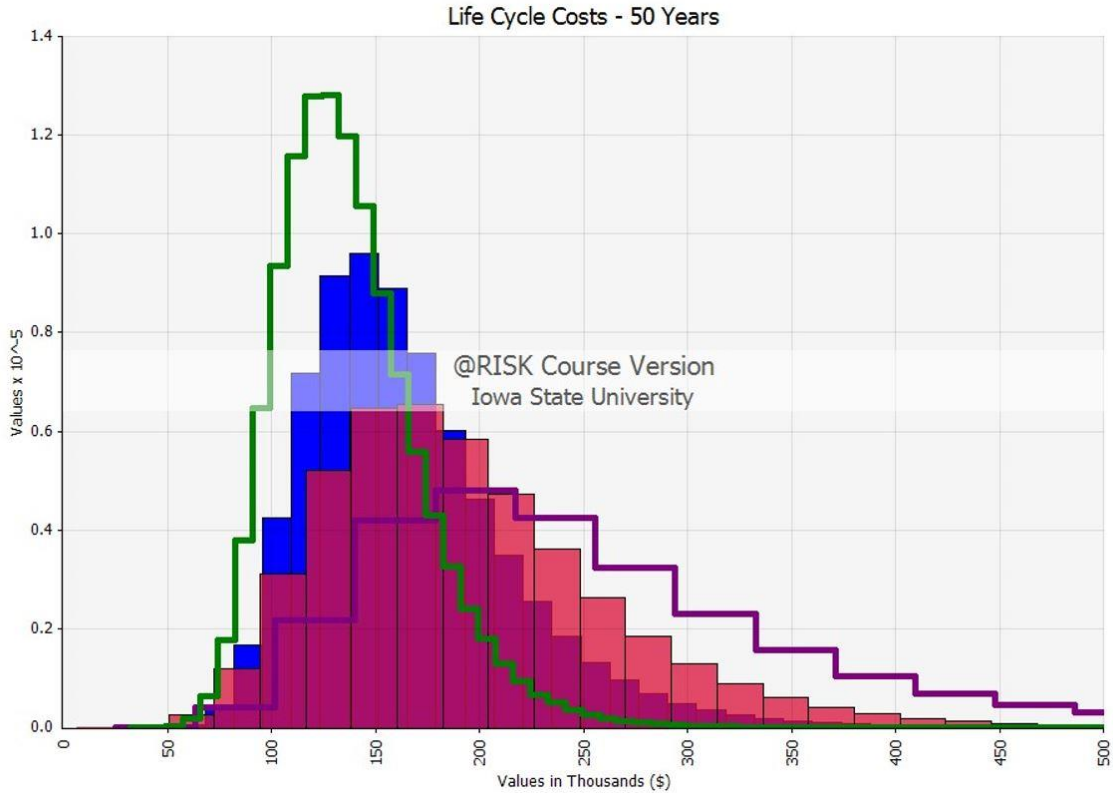
<b>Table 4.6</b> Statistical distributions of varying parameters				
<b>Item</b>	<b>30' Bridge</b>		<b>44' Bridge</b>	
	<b>Avg. Cost (\$)</b>	<b>Sta. Dev.</b>	<b>Avg. Cost (\$)</b>	<b>Sta. Dev.</b>
PCC	49.34	7.40	72.11	10.82
Hydrodemolition	324.28	48.64	476.19	71.43
Pneumatic Breaker	87.25	13.09	128.12	19.22
EMSEAL BEJS	1350.00	202.50	1980.00	297.00
Steel extrusion w/ Neoprene	6476.70	882.90	9499.16	1294.92
Stainless Steel extrusion w/ Neoprene	12953.40	1765.80	18998.32	2589.84
Neoprene Gland Installation & Testing	1465.80	388.50	2149.84	569.80
Neoprene Gland Install & Testing – Gland Only	3725.10	277.20	5463.48	406.56
Mobilization	7666.67	63.93	7666.67	63.93
Mobilization – Gland Only	24840.14	10152.39	24840.14	10152.39
Traffic Control	6312.50	2784.55	6312.50	2784.55
Traffic Control – Gland Only	6196.67	126.40	6196.67	126.40
Temporary Barrier Rail	296.40	43.20	434.72	63.36
Temporary Crash Cushion	1017.27	105.86	1017.27	105.86
Temporary Traffic Signals	4321.25	8.31	4321.25	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%

### 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. On each histogram, the x-axis represents the total life cycle costs and the y-axis represents the number of the 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 life. 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 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)

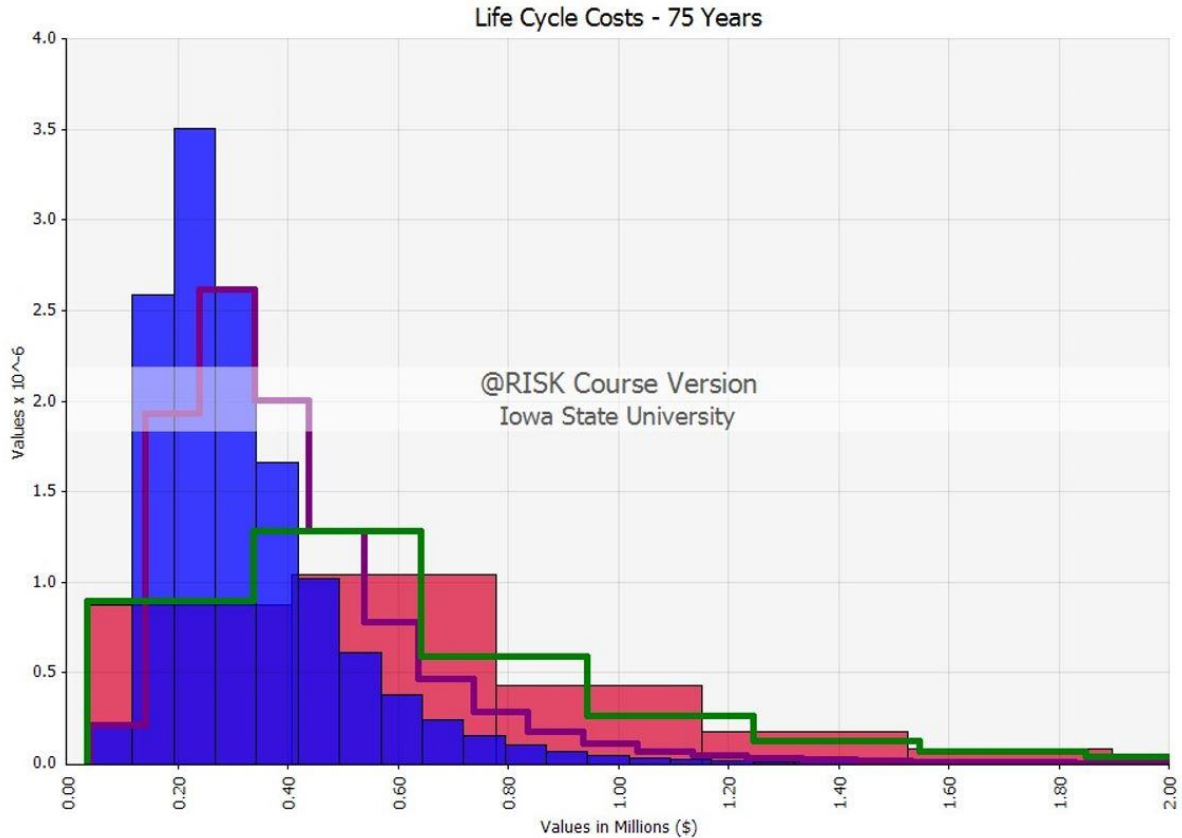
In addition to the histograms, a table is included for each bridge width and life summarizing the most likely, average, and 90<sup>th</sup> percentile cost. The 90<sup>th</sup> percentile represents the cost needed to achieve 90% of the iterations to be less than or equal to that value. Overall, the 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.



**Figure 4.1** Cost distribution for 30' bridge, 50-year bridge life

<b>Table 4.7</b> Monte carlo statistical summary for 30' bridge, 50-year life				
<b>Scenario</b>	<b>Color</b>	<b>Life Cycle Costs (\$)</b>		
		<b>Most Likely</b>	<b>Average</b>	<b>90<sup>th</sup> Percentile</b>
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

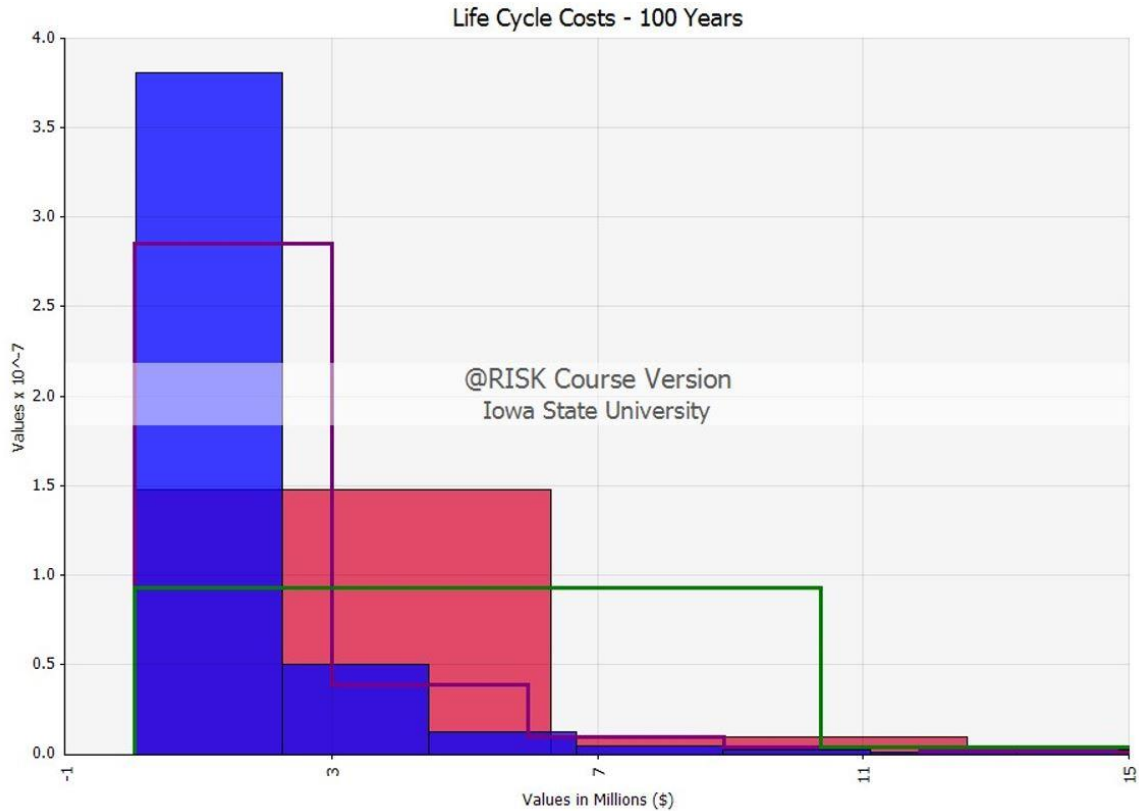
For a 30' bridge with a 50 year life, the most cost effective option is scenario 3, utilizing hydrodemolition, UHPC, and an A36 railing. This is due to the use of the EMSEAL BEJS after the A36 railing. This delays the need for a full replacement of the expansion joint beyond the scope of a 50 year life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



**Figure 4.2** Cost distribution for 30' bridge, 75-year bridge life

<b>Table 4.8</b> Monte carlo statistical summary for 30' bridge, 75-year life				
<b>Scenario</b>	<b>Color</b>	<b>Life Cycle Costs (\$)</b>		
		<b>Most Likely</b>	<b>Average</b>	<b>90<sup>th</sup> Percentile</b>
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

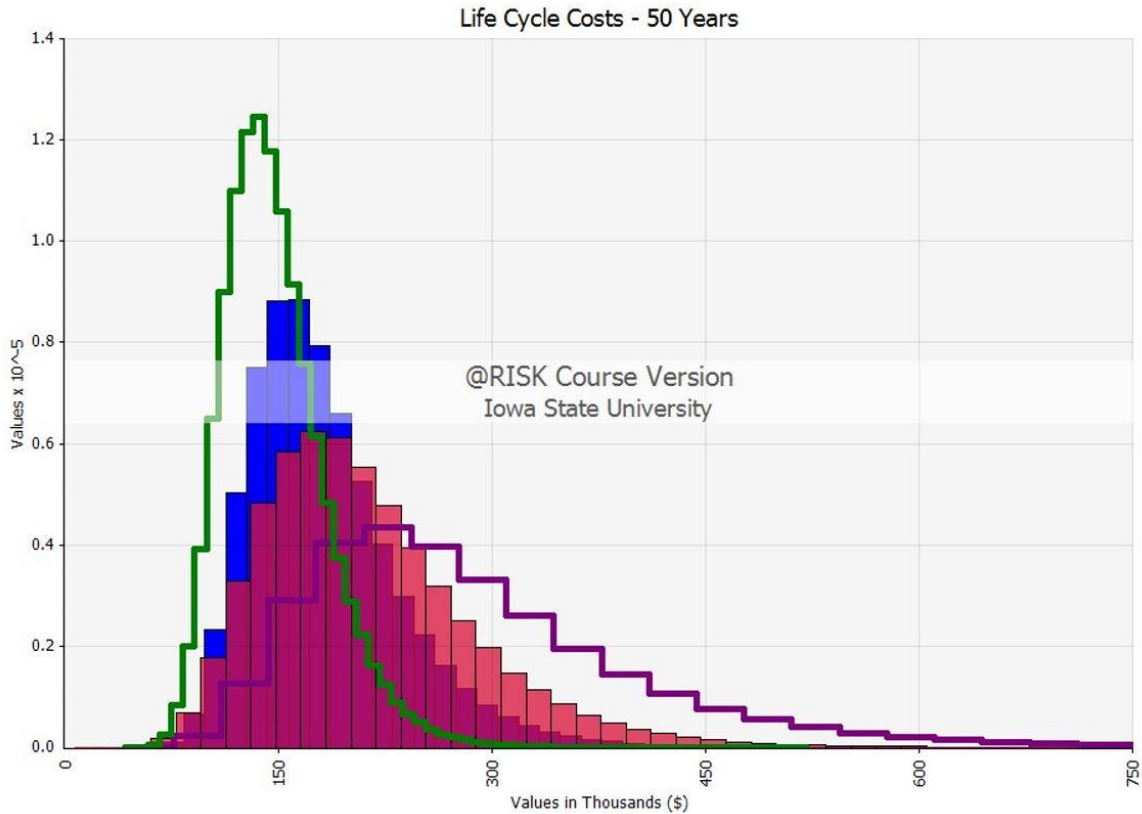
For a 30' bridge with a 75 year life, the proposed replacement (scenario 2) surpasses scenario 3 for the lowest estimated life cycle costs. The most likely cost of the proposed replacement is about \$125,000 less than current practices, with the current practices (scenario 1) becoming the highest estimated costs of all the scenarios.



**Figure 4.3** Cost distribution for 30' bridge, 100-year bridge life

<b>Table 4.9</b> Monte carlo statistical summary for 30' bridge, 100-year life				
<b>Scenario</b>	<b>Color</b>	<b>Life Cycle Costs (\$)</b>		
		<b>Most Likely</b>	<b>Average</b>	<b>90<sup>th</sup> Percentile</b>
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

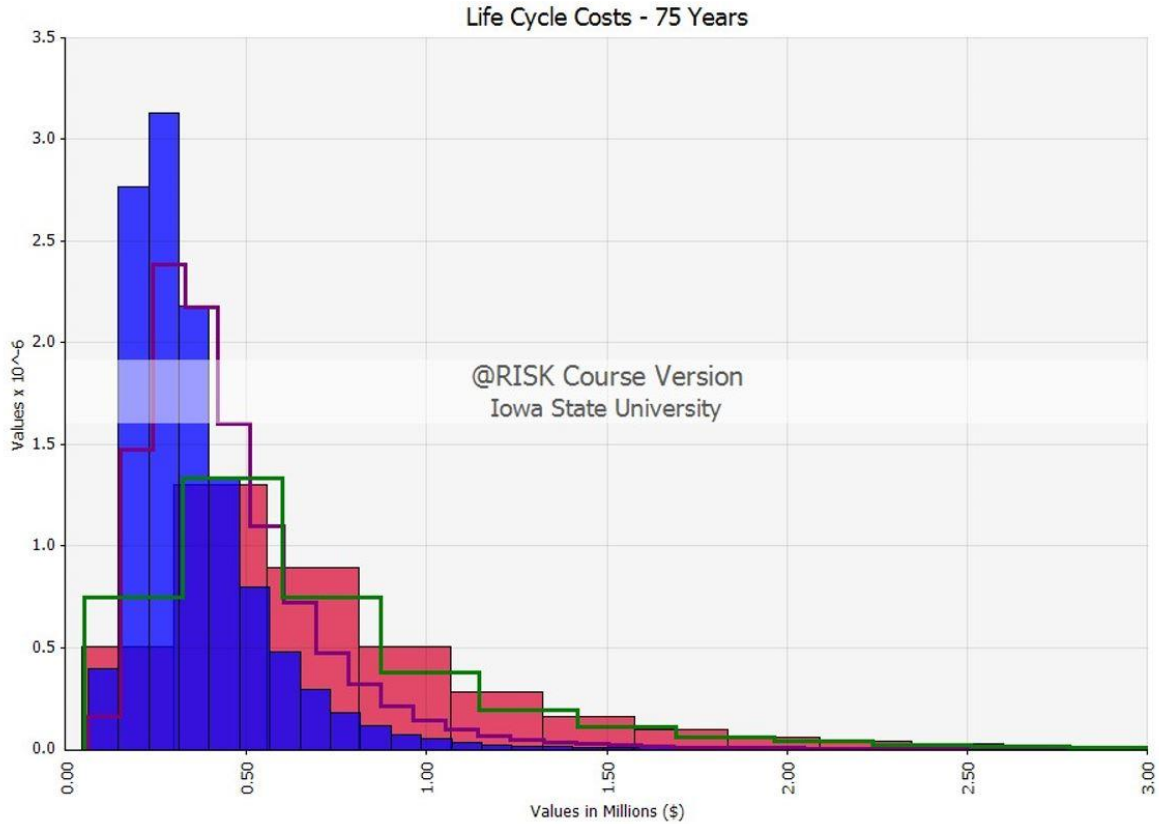
For a 30' bridge with a 100 year life, the proposed replacement (scenario 2), utilizing hydrodemolition, UHPC, and a stainless steel rail, has the lowest estimated life cycle costs. The current practices (scenario 1) and scenario 3 have similar estimations for the highest life cycle costs. Scenario 4, utilizing a pneumatic breaker, PCC, and a stainless railing, has a considerably lower cost than scenarios 1 and 3. This is mostly due to the extended joint life the stainless steel railing provides.



**Figure 4.4** Cost distribution for 44' bridge, 50-year bridge life

<b>Table 4.10</b> Monte carlo statistical summary for 44' bridge, 50-year life				
<b>Scenario</b>	<b>Color</b>	<b>Life Cycle Costs (\$)</b>		
		<b>Most Likely</b>	<b>Average</b>	<b>90<sup>th</sup> Percentile</b>
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 trends identified in the 30' bridge analysis are also present for the 44' bridge. For a 50 year life, the lowest estimated cost is, again, for scenario 3 with the use of the EMSEAL BEJS retrofit. The proposed replacement (scenario 2) also still has a lower life cycle cost than the current practices (scenario 1).

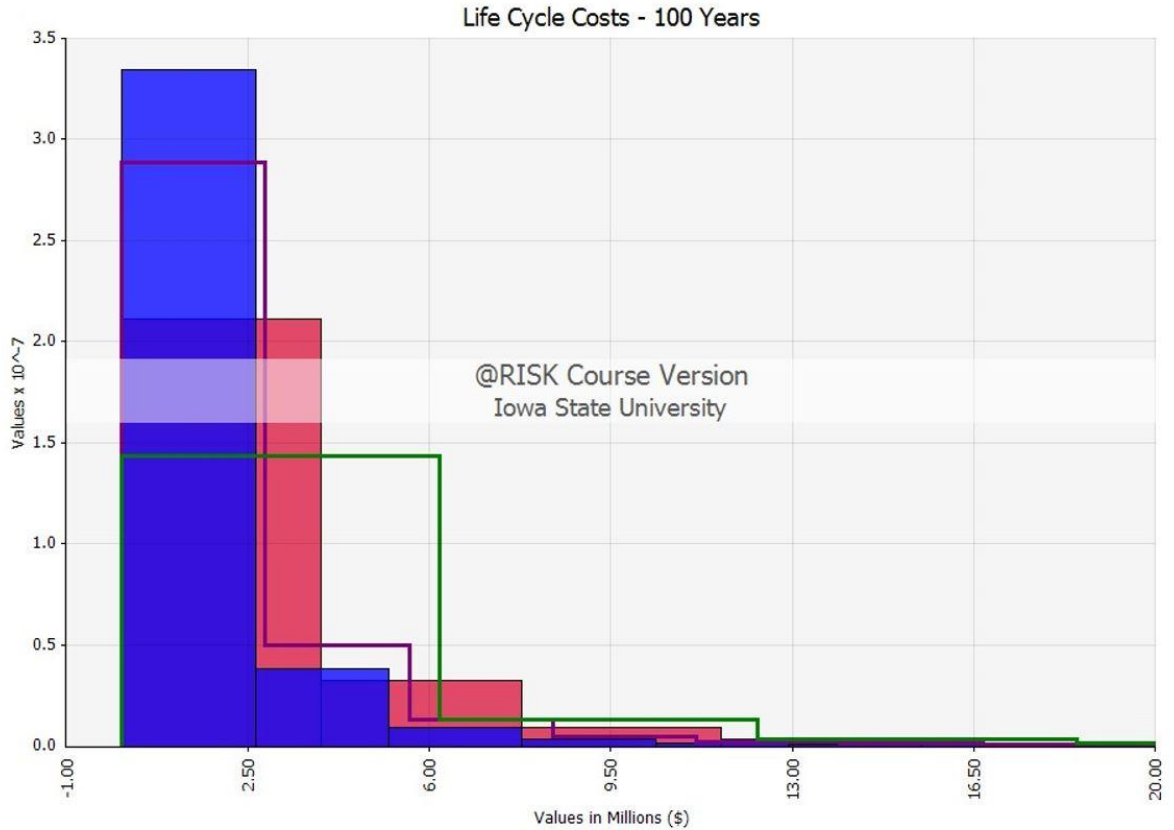


**Figure 4.5** Cost distribution for 44' bridge, 75-year bridge life

<b>Table 4.11</b> Monte carlo statistical summary for 44' bridge, 75-year life				
<b>Scenario</b>	<b>Color</b>	<b>Life Cycle Costs (\$)</b>		
		<b>Most Likely</b>	<b>Average</b>	<b>90<sup>th</sup> Percentile</b>
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

For a 44' bridge with a 75 year life, the proposed replacement (scenario 2) surpasses scenario 3 for the lowest estimated life cycle costs like the 30' bridge. The most likely cost of the proposed replacement is about \$170,000 less than current practices, with the current practices (scenario 1) becoming the highest estimated costs of all the scenarios.





**Figure 4.6** Cost distribution for 44' bridge, 100-year bridge life

Table 4.12 Monte carlo statistical summary for 44' bridge, 100-year life				
Scenario	Color	Life Cycle Costs (\$)		
		Most Likely	Average	90 <sup>th</sup> 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

For a 44' bridge with a 100 year life, the proposed replacement (scenario 2) has the lowest estimated life cycle costs. The current practices (scenario 1) and scenario 3 once again have the highest life cycle costs. Scenario 4 still has lower costs than scenarios 1 and 3, though not as considerably as the 30' bridge.

#### 4.4 Summary

Both a base life cycle cost analysis and sensitivity study was 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 the options.

## CHAPTER 5. EXPERIMENTAL TESTING

### 5.1 Testing Objectives

Experimental testing was conducted in the ISU structures lab in order to address several objectives related to the proposed accelerated joint replacement method. These objectives are listed below.

1. Evaluate the constructability of using hydrodemolition and UHPC in the replacement of expansion joints
2. Understand the performance of the bonding interface between the existing concrete and newly poured UHPC
3. 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. In order to satisfy the objectives listed above, it was decided to conduct a series of tests on a single lab specimen. First, the specimen was constructed with a joint mimicking a standard Iowa DOT joint with traditional concrete. This joint was tested to create a baseline for an existing joint. Then the proposed replacement procedure was conducted and the new joint 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” x 7” 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 new joint
- Conduct fatigue testing on new joint

The proposed solution for this research recommends the use of a stainless steel extrusion in conjunction with UHPC. This would allow 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. Each of these are described separately and holistically.

### 5.2.1 Geometry

Overall dimensions of the cast-in-place specimen can be seen in Figure 5.2. The entire lab specimen has an 8' x 8' footprint. The length encompasses two distinct sections. In these different sections, the thickness varies. The existing bridge deck section has a thickness of 10" and is 5' long. The backwall section has a thickness of 2' and is 3' long. Both sections are 8' wide.

An expansion joint exists between the existing bridge deck and backwall sections. Both the existing joint and replacement joint was a D.S. Brown A2R-400 strip seal (Figure 5.1). The steel extrusion for both joints was comprised of A36 steel. The minimum installation gap for the expansion joint is 2" in accordance with the manufacturer's recommendations.

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



**Figure 5.1** D.S. Brown A2R-400 strip seal (Ruble)

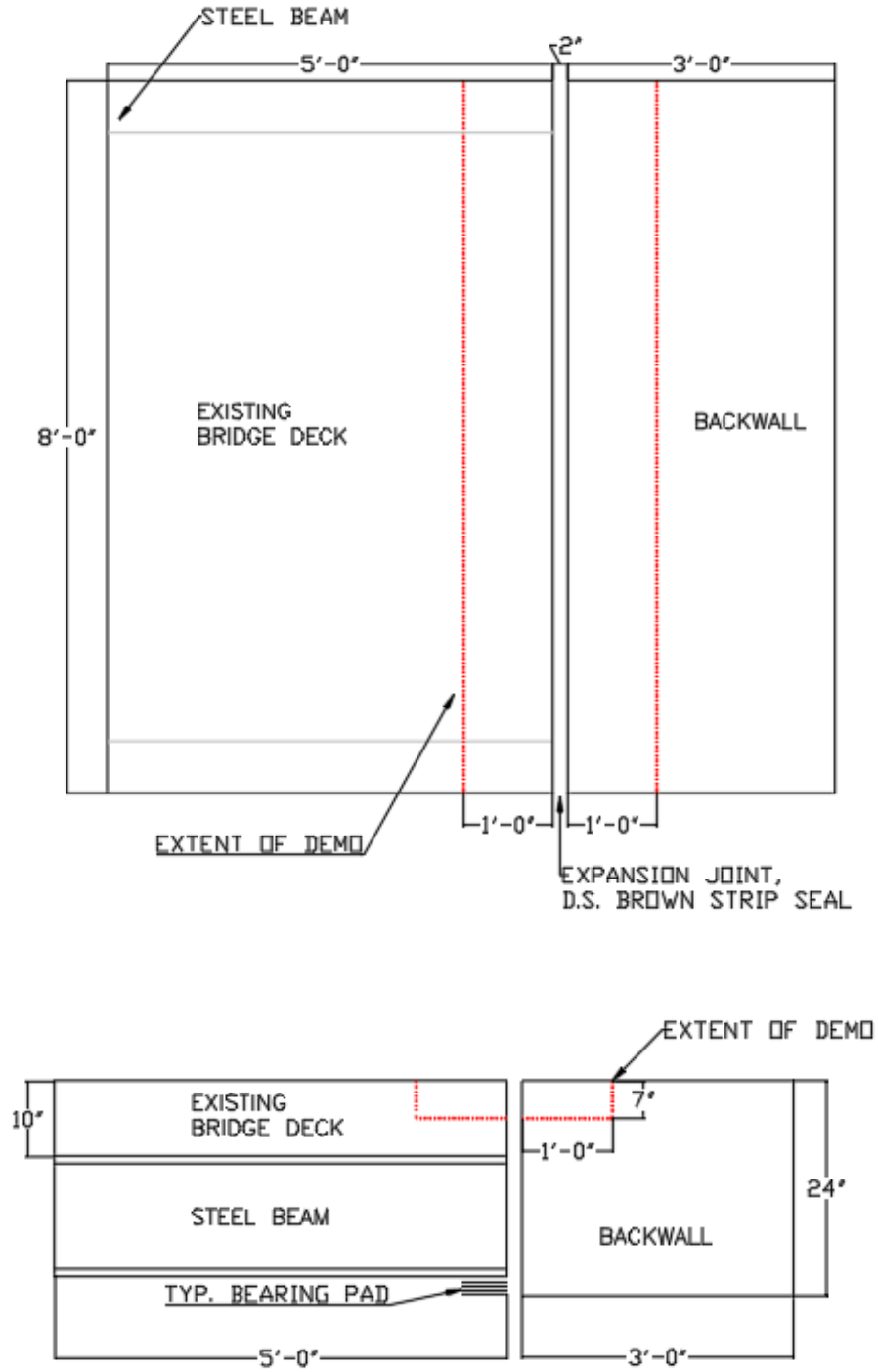
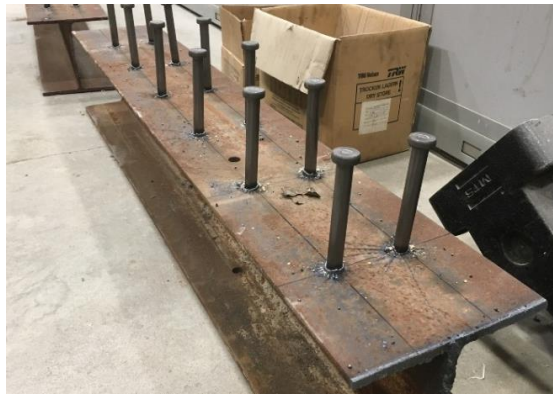


Figure 5.2 Geometry of lab specimen (Ruble)

### 5.2.2 Boundary conditions

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



**Figure 5.3** Shear studs on steel beams (Ruble)

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



**Figure 5.4** Bridge deck roller support (Ruble)

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



**Figure 5.5** Backwall fixed support (Ruble)

### 5.2.3 Reinforcing

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



<b>Part</b>	<b>Bar</b>	<b>Location/Type</b>	<b>Number</b>	<b>Length</b>	<b>Spacing</b>
Existing Bridge Deck	#6	Top, Long	15	4'-5"	6"
	#6	Bottom, Long	15	4'-5"	6"
	#7	Top, Transverse	8	7'-6"	7.5"
	#7	Bottom, Transverse	8	7'-6"	7.5"
Backwall	#3	Hoop	7		1'
	#5	Top, Transverse	2	7'-6"	30"
	#7	Bottom, Transverse	5	7'-6"	7.5"

### 5.2.4 Concrete

Typical Portland cement concrete (standard Iowa DOT C4 mix design) was used for the pour of the initial specimen. The entire initial specimen was allowed to fully cure for 28 days after placement. Cylinders were made to test the compressive strength of the concrete during the curing process. The results of the 28-day compression strength test can be seen in Table 5.2.

<b>Cylinder #</b>	<b>Strength (psi)</b>
1	5699
2	5551
3	5619
<b>Average</b>	<b>5623</b>

Ductal JS1212 proprietary mix was used for the replacement UHPC. The mix design for this is detailed in Table 5.3 below. 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 can be seen in Table 5.4.

<b>Table 5.3 Ductal JS1212 UHPC mix design</b>	
<b>Item</b>	<b>Mass (lbs/cu.ft)</b>
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

<b>Table 5.4 UHPC cylinder compressive strength</b>			
<b>Cylinder #</b>	<b>12-Hrs (psi)</b>	<b>24-Hrs (psi)</b>	<b>28-Day (psi)</b>
1	7853	12831	20379
2	7823	12703	19449
3	9251	13723	18202
<b>Average</b>	<b>8309</b>	<b>13086</b>	<b>19343</b>

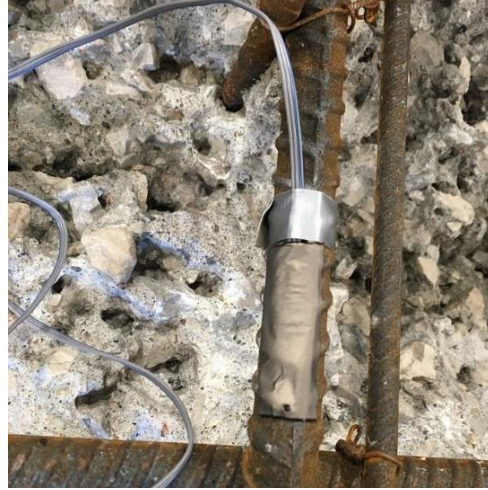
### 5.2.5 Instrumentation

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

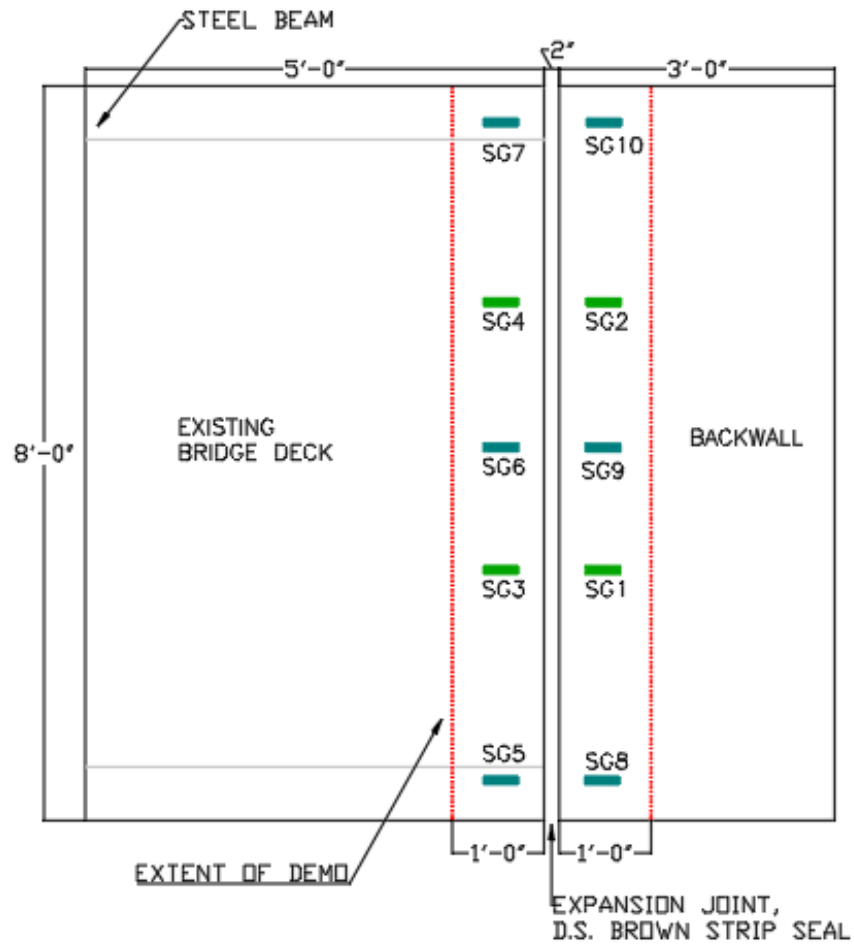
Strain gauges were used to measure the strain throughout the expansion joint. Four reinforcing steel strain gauges were used (SG1-4), two on each side of the joint within the demolition area. They were placed one-third and two-third the length of the joint. Six embedded concrete strain gauges were used (SG5-10). They were placed on the end and middle of the joint on both sides. These locations and labels can be seen in Figure 5.6.



**Figure 5.6** Embedded concrete strain gauge (Ruble)

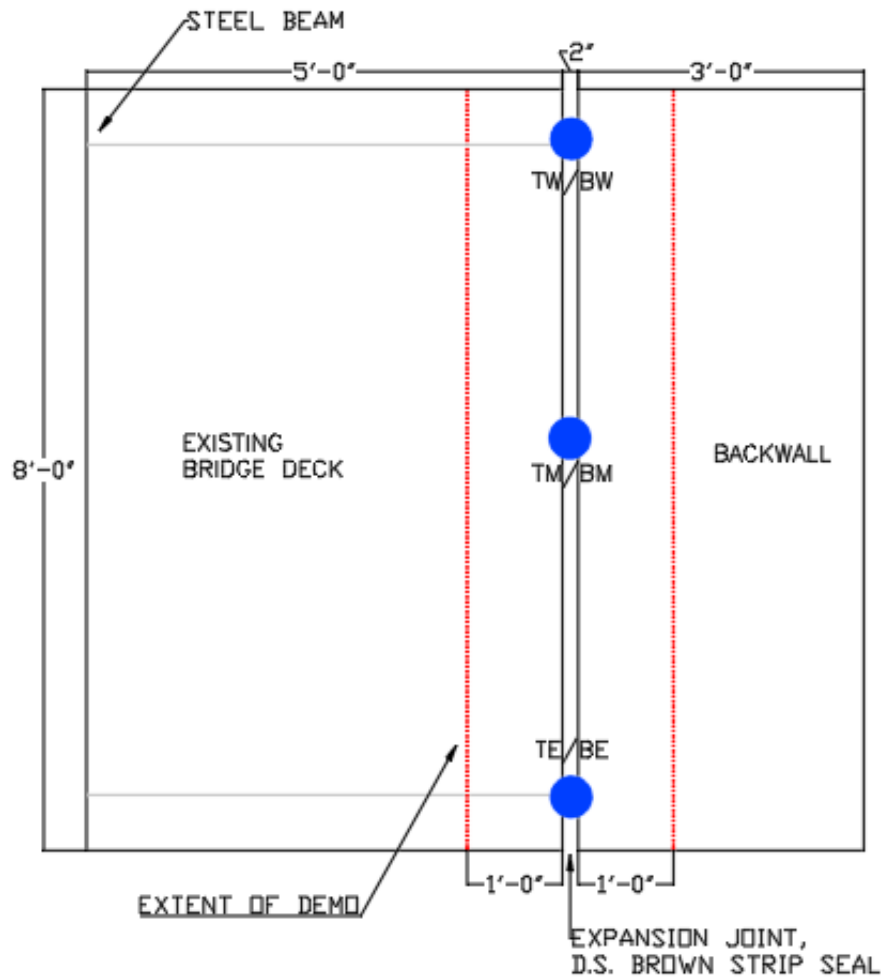


**Figure 5.7** Reinforcing strain gauge (Ruble)



**Figure 5.6** Strain gauge layout (Ruble)

String potentiometers were used to measure any displacement that occurred during testing. Three string potentiometers were placed over the joint to measure horizontal displacement and three string potentiometers were placed below the joint to measure vertical displacement. The locations of the string potentiometers can be seen in Figure 5.. 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.”

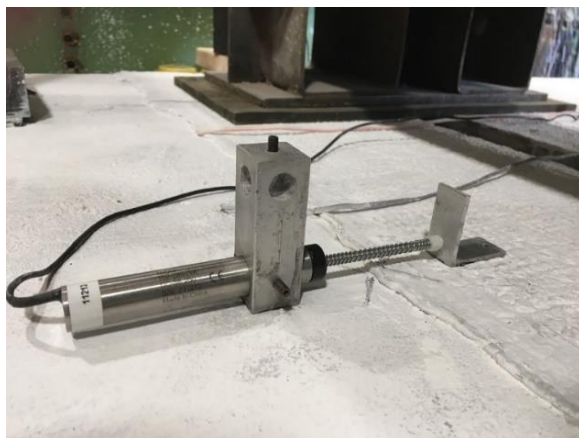


**Figure 5.9** String potentiometer layout (Ruble)

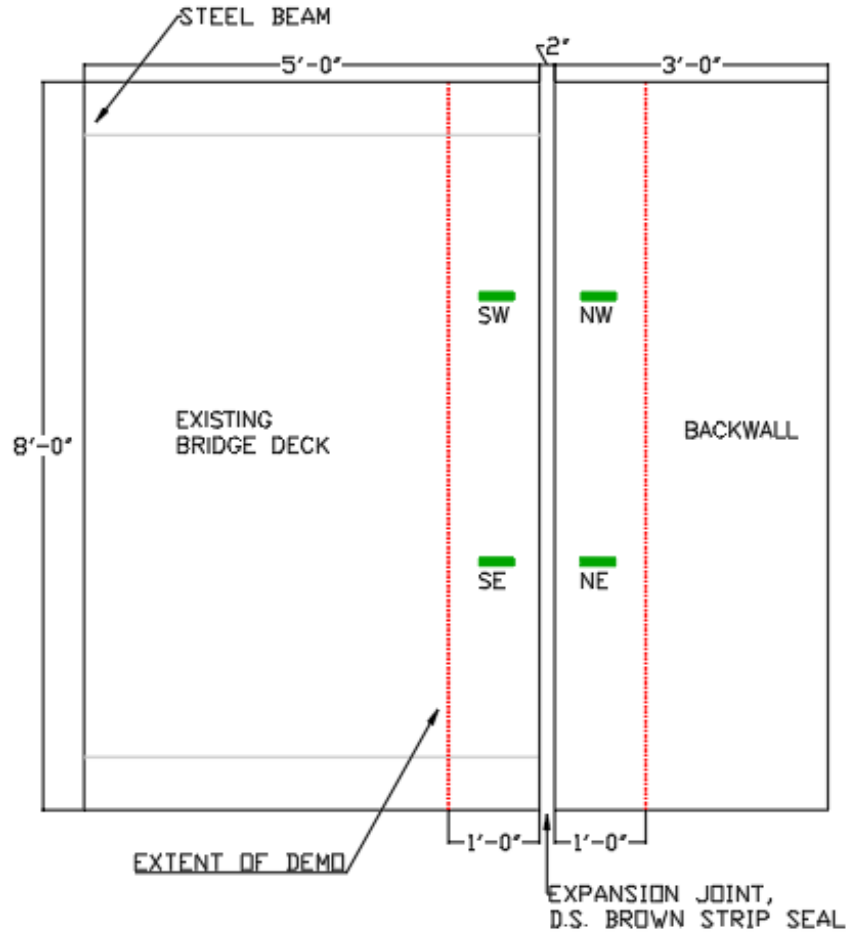


**Figure 5.10** String potentiometer (Ruble)

DCDTs were used to measure any displacement that occurred at the interface between the new UHPC and the existing concrete. Four DCDTs were used, two on either side of the joint. They were placed at one-third and two-third the length of the joint. The locations and labels of the DCDTs can be seen in Figure 5.8.



**Figure 5.7** Direct current linear variable differential transformer (Ruble)



**Figure 5.8** DCDT layout (Ruble)

### 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. In order to accommodate for the water runoff and the hydrodemolition equipment, the demolition took place outside in the loading dock area of the ISU structural engineering laboratory. CLC Hydro Services used an Aquajet Systems Aqua Cutter 710H (Figure 5.9) to perform the demolition. The Aqua Cutter was supplied with water from a water tank truck CLC Hydro Services provided (Figure 5.10).



**Figure 5.9** Aquajet aqua cutter 710H (Ruble)



**Figure 5.10** Water tank truck (Ruble)

The mobilization for the hydrodemolition took approximately 3 hours. This included filling up 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 were used to filter the concrete particulates out of the water before it entered the storm drain (Figure 5.11).



**Figure 5.11** Filter socks (Ruble)

When it was time for the demolition to begin, the width of demolition and water velocity was programmed into the Aqua Cutter using a hand-held remote. The depth of demolition is 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 is reached. Once the necessary number of passes is determined, the demolition speeds up considerably.

Typically with hydrodemolition, the jet of water runs parallel to the length of the expansion joint. This allows the best access to underneath 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 configuration of it in the loading dock, it was necessary to orient the Aqua Cutter perpendicular with 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 was necessary to reach the depth of 7". This was then programmed into the equipment, and the Aqua Cutter automatically moved down the joint when 5 passes were complete at one



section. It took the Aqua Cutter approximately 10 minutes to complete one foot 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. The demolition took approximately 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 5.12** Hydrodemolition progress (Ruble)



**Figure 5.13** Cleaning the joint after hydrodemolition (Ruble)



**Figure 5.14** Completed hydrodemolition (Ruble)

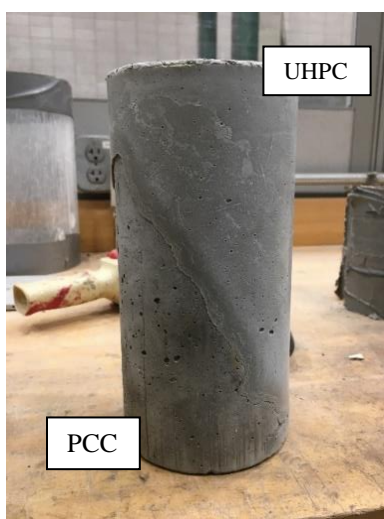
If the rate of this hydrodemolition is extrapolated for a 30' bridge, it would take approximately 5 hours to demolish the necessary concrete after mobilization. This is likely considerably faster than the time necessary for demolition with a pneumatic breaker. Furthermore, the demolition of the test specimen required approximately 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. In order 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 in order 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. 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 (Figure 5.15).



**Figure 5.15** Slant shear cylinder test (Ruble)

The cylinders have a diameter of four inches and a length of eight inches. The bonding interface is the shape of an oval, resulting in a bonding interface area of 25.13 in<sup>2</sup>.

The shear strength (S) can be calculated from this test using Equation (2).

$$S = P / A \quad (2)$$

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 ACI Concrete Repair Guide (Tayeh et al. 2012). The results of the slant shear test are summarized in Table 5.5. 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.

<b>Table 5.5</b> Slant shear test results			
<b>Cylinder #</b>	<b>P (lbs)</b>	<b>S (psi)</b>	<b>Acceptable?</b>
1	44,334	1,764	No
2	51,773	2,060	Yes
3	55,373	2,203	Yes
<b>Average</b>	<b>50,493</b>	<b>2,009</b>	<b>Yes</b>



**Figure 5.16** Cylinder 1 slant shear failure (Ruble)



**Figure 5.17** Cylinder 2 slant shear failure (Ruble)



**Figure 5.18** Cylinder 3 slant shear failure (Ruble)

#### 5.4.2 Split cylinder test

The split cylinder test utilizes three cylinders made with half UHPC and half traditional concrete. The two materials are joined down the center of the cylinder vertically (Figure 5.19). The cylinders were then tested on their side in order to determine the indirect tensile strength of the bond (Figure 5.20).



**Figure 5.19** Split cylinder interface (Ruble)



**Figure 5.20** Split cylinder test (Ruble)

The cylinders have a diameter of four inches and a length of eight inches. This results in a bonding surface area of 32 in<sup>2</sup>. The splitting tensile strength (T) can be calculated from this test using Equation (3).

$$T = (2 P) / (\pi A) \quad (3)$$

where

P – Experimental maximum load

A – Area of the bond interface

According to Tayeh et al. (2012), the quality of a bond can be evaluated based on the criteria in Table 5.6. The values have been converted into empirical units from the original source.

<b>Table 5.6 Bond quality</b>	
<b>Quality</b>	<b>Strength (psi)</b>
Excellent	$\geq 305$
Very Good	247 – 305
Good	203 – 247
Fair	102 – 203
Poor	0 – 102

The results of the split cylinder test are summarized in Table 5.7. Cylinder 1 experienced a primary failure along the bonding interface with some failure in the PCC (Figure 5.21). Cylinders 2 and 3 both experienced failure in the PCC substrate (Figure 5.22 and Figure 5.23). 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.

<b>Table 5.7 Split cylinder test results</b>			
<b>Cylinder #</b>	<b>P (lbs)</b>	<b>T (psi)</b>	<b>Bond Quality</b>
1	34,436	681.10	Excellent
2	42,153	838.61	Excellent
3	42,162	838.79	Excellent
<b>Average</b>	<b>39,517</b>	<b>786.17</b>	<b>Excellent</b>



**Figure 5.21** Cylinder 1 split cylinder failure (Ruble)



**Figure 5.22** Cylinder 2 split cylinder failure (Ruble)



**Figure 5.23** Cylinder 3 split cylinder failure (Ruble)

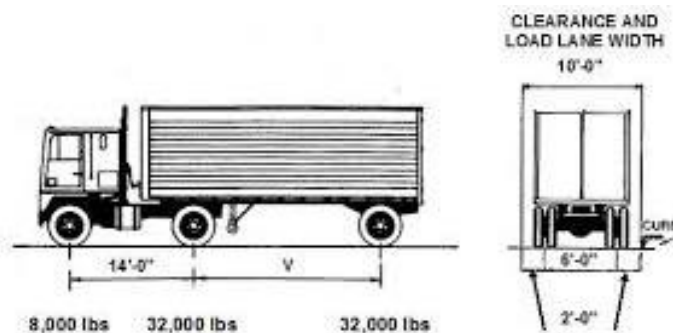


## 5.5 Static Testing

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. This static testing was used to confirm 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 specimen 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 5.24). This load was applied to the joint area on two 10" x 20" loading pads, spaced 6' apart. An actuator and spreader beam were used to apply the load evenly to both loading areas. The specimen was loaded to 16 kips on each loading area initially. This load was then increased to 21.3 kips on each loading area. This is equivalent to the impact load of the truck.



**Figure 5.24** HS-20-44 truck (Morandeira)

Table 5.8 summarizes the results of the vertical test. Both the reinforcing strain and concrete strain is minimal for the original and replacement joints. The maximum average

displacement experienced in the two joints are 0.063 inches for the original joint and 0.077 inches for the replacement joint. These results indicate that the replacement joint performs similarly to the Iowa DOT standard expansion joint.

<b>Table 5.8 Vertical test results at impact load</b>		
	<b>Original Joint</b>	<b>Replacement Joint</b>
SG1	-1	1
SG2	-3	2
SG3	1	1
SG4	2	1
<b>Avg. Reinforcing Strain</b>	<b>-0.25</b>	<b>1.25</b>
SG5	4	0
SG6	-2	0
SG7	0	-4
SG8	0	1
SG9	-2	0
SG10	0	1
<b>Avg. Concrete Strain</b>	<b>0</b>	<b>-0.33</b>
TW	-0.05	-0.043
TM	-0.049	-0.046
TE	-0.042	-0.046
<b>Avg. Horiz. Displacement (inch)</b>	<b>-0.047</b>	<b>-0.045</b>
BW	-0.049	-0.067
BM	-0.067	-0.073
BE	-0.072	-0.092
<b>Avg. Vert. Displacement (inch)</b>	<b>-0.063</b>	<b>-0.077</b>

### 5.5.2 Thermal loading

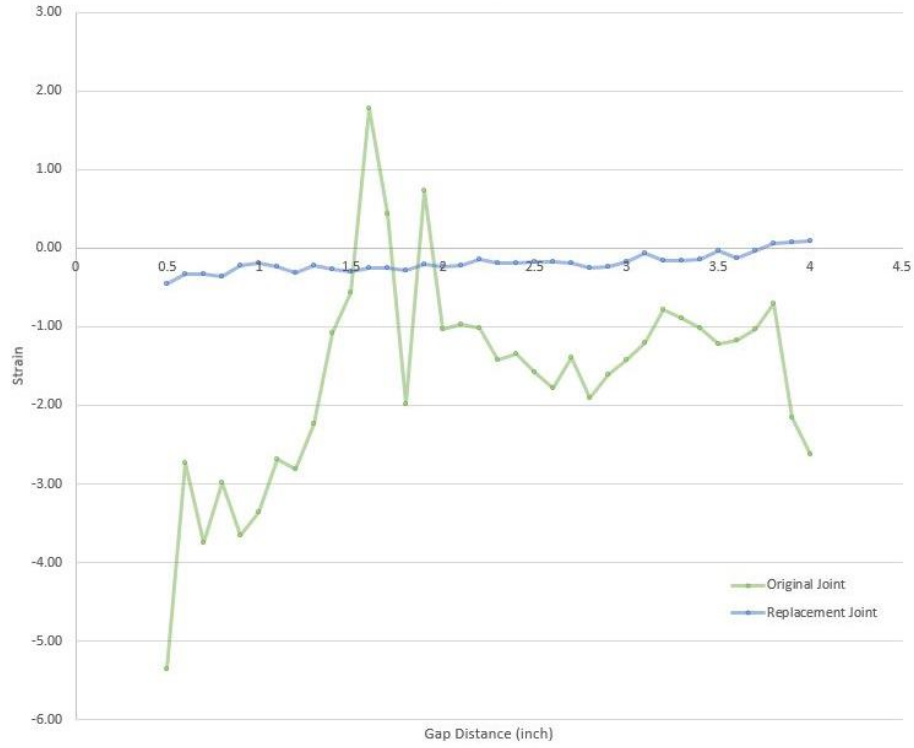
Thermal loading was applied to the original and replacement specimen to compare the performance of the two joints under thermal expansion and contraction.

In order to mimic thermal loading, actuators were used to expand and contract the expansion joint gap in the specimen (Figure 5.25). This allowed the specimen to undergo simulated tensile and compressive forces due to thermal expansion and contraction. The full range of the strip seal was explored.

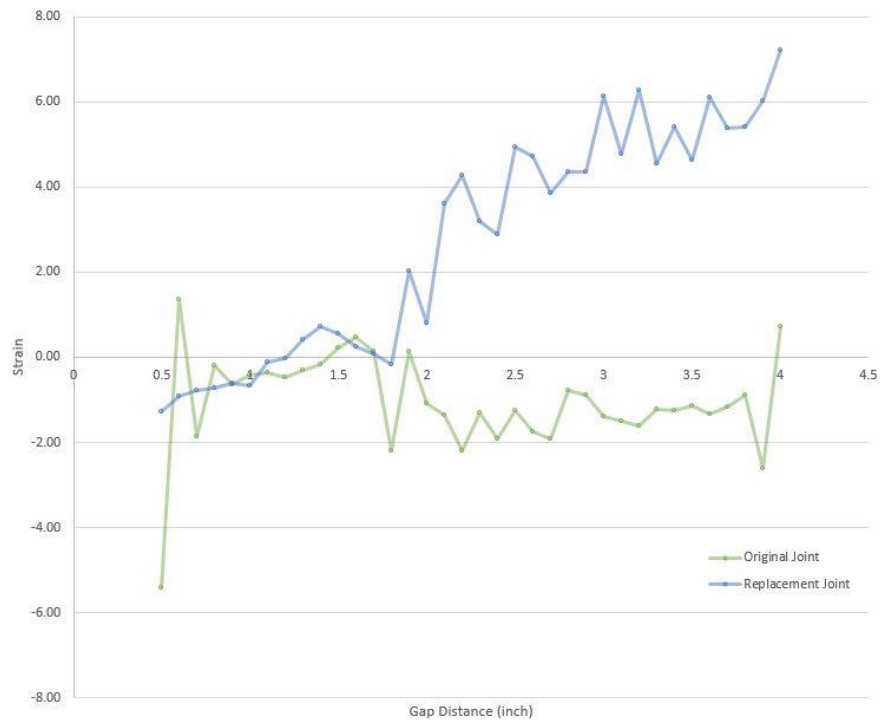


**Figure 5.25** Actuators used to mimic thermal loading (Ruble)

Figure 5.26 and Figure 5.27 summarize the average concrete and reinforcing strain, respectively, experienced in the two joint systems across the range of gap distances. For all cases, the strain values are relatively minor, similar to what was seen with the vertical loading. For the concrete strain, the original joint experienced a greater range of strain values, 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 can be 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 would meet Iowa DOT standards.



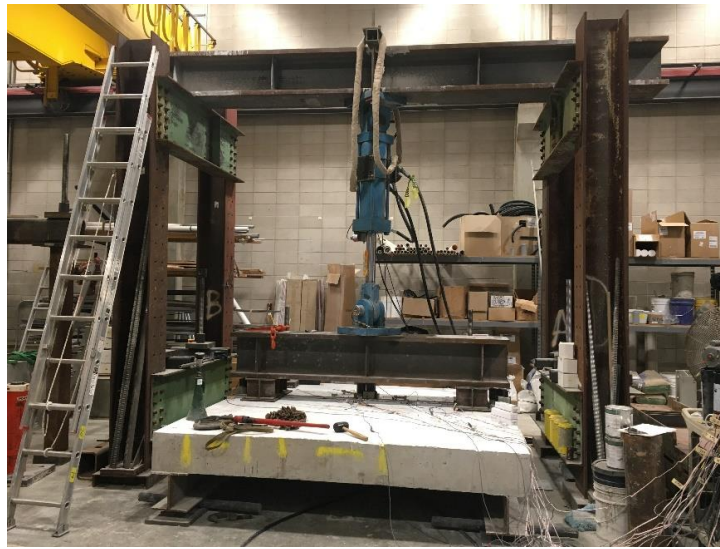
**Figure 5.26** Average concrete strain, thermal performance test (Ruble)



**Figure 5.27** Average reinforcing strain, thermal performance test (Ruble)

## 5.6 Fatigue Testing

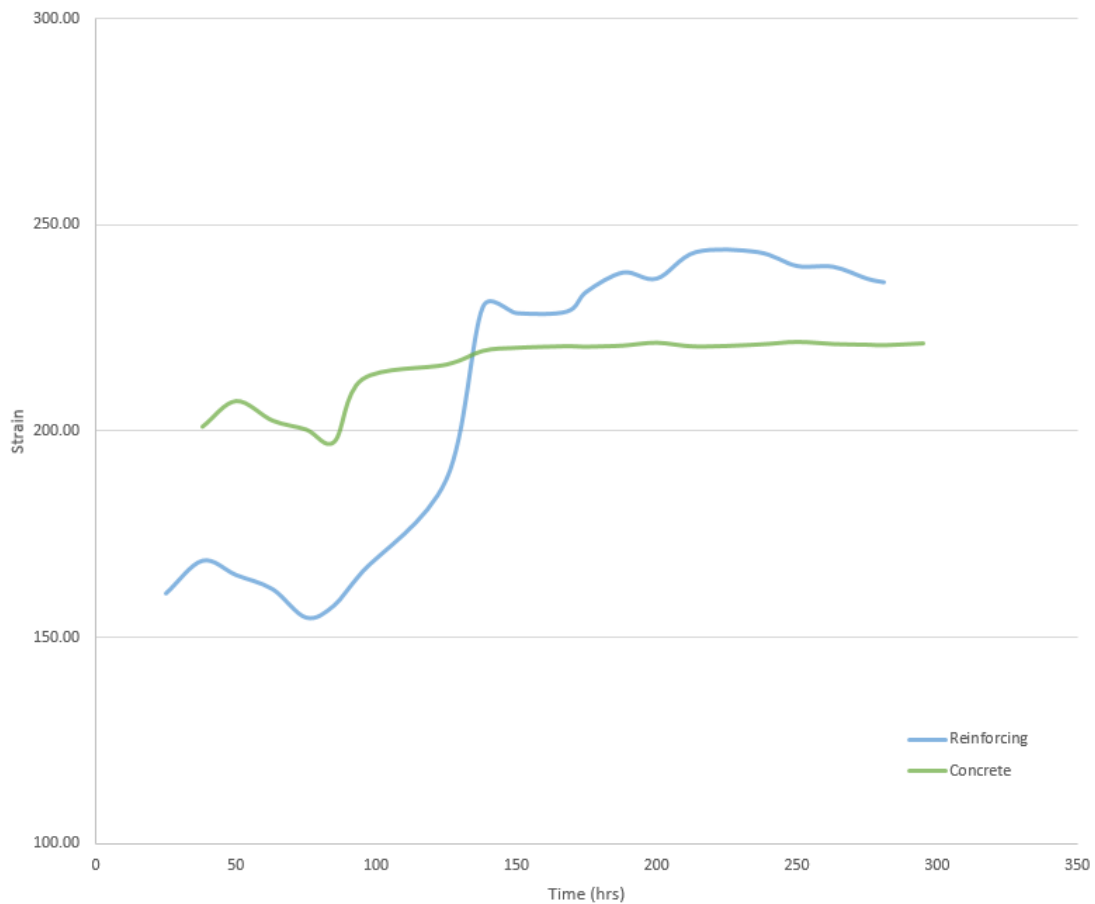
After all static testing was completed, the UHPC replacement joint underwent fatigue testing. This was used to evaluate the durability of UHPC/joint over time. Cyclic loading was applied on the specimen at service level conditions. 1,000,000 cycles were applied at one Hertz. It took approximately 12 days to complete. Visual inspection for cracking was conducted and documented every 250,000 cycles.



**Figure 5.28** Fatigue loading set up (Ruble)

Figure 5.29 summarizes the average concrete and reinforcing strain experienced in the specimen throughout the length of the fatigue test. 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. These both suggest that 1,000,000 cycles did very little damage to the specimen, and it could potentially withstand many more cycles. Time and budget constraints prevented further fatigue testing on the specimen. Instead, a brief literature review was conducted to explore possible timelines. Benjamin Graybeal conducted static and fatigue

tests on UHPC joints in his research *Behavior of Field-Cast Ultra-High Performance Concrete Bridge Deck Connections Under Cyclic and Static Structural Loading*. He found that the UHPC joints were able to withstand at least 5,000,000 cycles at the impact load of 21.3 kips without being significantly affected. While a strip seal joint varies from a UHPC connection joint, these results are promising in regards to the longevity of the proposed joint of this study.



**Figure 5.29** Average strain over time (Ruble)



**Figure 5.30** Visual inspection of specimen (Ruble)



**Figure 5.31** Visual inspection of joint (Ruble)

### 5.7 Summary

Bond testing, static testing, and fatigue testing were conducted in order 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 approximately 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 show the bond between UHPC and traditional concrete to be excellent. The shear strength of the bond is 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 the replacement joint. Both meet 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 & FUTURE WORK

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 greater than 50 years, the proposed replacement was the most cost effective option. For bridges with a life of 50 years or less, it is more cost effective to consider a joint retrofit near the end of the bridge life.

The proposed replacement joint also underwent bond, static, and fatigue testing in the ISU structural engineering 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 testing showed that the proposed joint would meet current DOT standards. The fatigue testing suggests that the proposed joint would be resilient to traffic loads for a long period of time.

Based on the results of this study, the following suggestions are made for future work in the accelerated repair and replacement of expansion joints.

- Further study into the differences between stainless steel and A36 railings
- Fatigue testing of the joint until failure
- Long-term observation of the proposed replacement joint installed in a real bridge

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