A HOLISTIC STRATEGY FOR REHABILITATION
OF FATIGUE-CRACKS IN STEEL BRIDGES:
EVALUATION OF FATIGUE CRACKS IN THE
FIELD AND LABORATORY TESTING

by

Jordan Wynn

A thesis submitted to the Faculty of the University of Delaware in partial fulfillment of the requirements for the degree of Master of Civil Engineering

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ABSTRACT

Nationwide, a significant amount of bridges within the National Bridge Inventory (NBI) have been found to develop fatigue cracks. Typically section(s) in and surrounding welded connection details are particularly prone to the development of fatigue cracks as a result of the significant stress concentration. Existing flaws and/or discontinuities caused by notches, corners, welding toes etc., under the presence of tensile stresses will encourage the initiation and propagation of these cracks. Fatigue cracks found in bridge members can become potential areas of concern for bridge owners, as if not mitigated, continued cyclically applied loading to the bridge encourages the propagation of these cracks at these connection details. Ultimately, these cracks can lead to the possible gross deformation, loss of function or serviceability, or complete separation of the component known as fracture. Although fatigue inherently does not always lead to fracture of a structural component, it must be considered as potential consequence of the development of fatigue cracks if not diminished. As a result of this propagating issue, a holistic approach to the rehabilitation of cracks found in these steel bridge members has been deployed. This approach emphasizes the use of composite materials in addition to a carbon nanotube (CNT)-based sensing layer in order to create a structural patch to be applied over a fatigue crack. This patch not only rehabilitates damaged sections but also continuously monitors its progress through use of the CNT sensing layer. If employed properly, this new technique of utilizing a composite patch over the fatigue crack will inevitably decrease the overall stress concentration exhibited around the crack/crack mouth, which ultimately prolongs the general fatigue life of the structural member.
Chapter 1

INTRODUCTION

1.1 Background

When subjected to repetitive loading conditions, many materials are prone to the development of what is known as fatigue cracks. These series of cracks have been found to be detrimentally to the overall strength and integrity of the material. The phenomena identified as fatigue, is a condition whereby a material cracks or fails attributed to recurring stresses applied below the ultimate strength of the material when loaded monotonically. When examined in the Civil Engineering discipline, fatigue is one of the primary reasons for the failure of structural components, particularly bridge structures. Commonly, many fatigue conditions occur in or around welded details due to the introduction of microstructure imperfections inherent with the welding process. Additionally, these cracks can develop at the focal location of discontinuities, typically from microstructure flaws and defects in the structure. All fatigue cracks have an associated stress concentration as a consequence of notches, corners, welding toes, etc. Research has shown there is a correlation between these concentrations of stress and the fatigue life.

As a result of continued cyclical loading, cracks will continue to propagate, ultimately leading to the possible gross deformation, loss of function or serviceability, or complete separation of the component known as fracture. Although fatigue inherently does not always lead to fracture of a structural component, it must be
considered as a potential consequence of the development of fatigue cracks, if not mitigated.

1.2 Problem Statement/Objective

Currently, bridges all throughout the country contain connection details that are susceptible to the development of fatigue cracks. The Delaware Department of Transportation (DelDOT) has a number of steel bridges in its inventory that have fatigue cracks such as the one shown in Figure 1.1. However, in spite of these continued deteriorating structural elements, measures can be taken to retrofit and preserve existing cracked bridge members if fatigue cracks are detected in early stages of propagation.

Figure 1.1 Sample Fatigue Crack Developed on DelDOT Steel Bridge

This project is part of a larger effort to develop an overall strategy for the rehabilitation and monitoring of fatigue-prone and deteriorated steel members, with the goal to prolong the life and safety of those bridges. Currently, there are a number
of rehabilitation methods that are commonly employed in order to retard the propagation of these cracks. These techniques include but are not limited to: surface grinding of shallow cracks, drilling holes at the tips of cracks, welding of the cracks, metal reinforcements, reinforcements using composite materials, and/or modifying the connection details to decrease the stress concentration. Of these, the most commonly utilized procedure used by most bridge owners is that of drilling a 2 to 3 inch diameter hole at the end of the crack, a so-called crack-stop hole. The idea is that the stress concentration is decreased by introducing a larger smooth curved surface into the stress field. Once a crack-stop-hole is drilled, frequent inspections or monitoring are needed to ensure the procedure performs as intended. Even though this is common practice, there is currently no standard on how to precisely size these holes, how to maintain and inspect them, nor an understanding of the conditions under which the holes are or are not effective in stopping the fatigue crack from further propagating. This project aims at the development of an innovative holistic approach for the rehabilitation and monitoring of bridges members found to develop fatigue cracks.

The focus of this investigation is on improving the fatigue lives of steel bridge members. This project explored and evaluated the use of a novel integrated rehabilitation and monitoring techniques employing structural carbon-nanotube (CNT) infused sensing composites.

1.3 Overview of Approach

In order to develop a method to prolong the fatigue life of steel bridge members, this research was segmented into two individual phases. The initial phase involved the recording of strains using a wireless sensing network (WSN) from ambient traffic on an in-service bridge. Data taken from bridge was subsequently
analyzed in order to determine the current state of the structural member in question. The next phase involved laboratory testing incorporating the proposed holistic rehabilitation method. The overall purpose was to determine its effectiveness at prolonging the fatigue life of a damaged specimen.

In-service bridge monitoring was performed on DelDOT Bridge 1678-006. This structure was monitored for a period of just under two weeks. Four individual sampling sessions were carried out, each of which recorded about 48 hours of data. Conventional strain gauges were welded at various locations on the web near an existing fatigue crack. In addition, a neighboring girder exhibiting no fatigue damage was also instrumented at the same locations in order to compare damaged vs. undamaged condition. Upon culmination of sampling, data was analyzed in order to determine the fatigue life of the member. A number of cycle counting algorithms, including Rainflow and Simple Range counting methods, were employed as detailed in ASTM Standard E1049-85. Miner’s Rule was subsequently used in order to determine the effective stress range and tabulate the associated Miner’s Rule (Miner 1924). This value could then be used in order to estimate the cumulative damage (or remaining life) due to fatigue of the structure at each individual gauge location.

Laboratory fatigue testing was performed, adopting the approach set forth in the ASTM Standard E647. A standard single edge-notch compact (CT) specimen was employed as steel specimen for all laboratory testing. Per ASTM Standard, a short fatigue crack was initiated prior to actual fatigue testing. Several discrete tests were conducted; the first of which involved simple baseline testing of a CT specimen. The primary purpose of this test was to provide a reference against which to assess the progress and effectiveness of all other succeeding tests. Subsequent experiments
involved testing a specimen with a drilled crack-stop hole as displayed in the field, and the novel integrated structural patch. The final test integrated all elements previously studied and investigated, by means of fatigue testing, a rehabilitated specimen employing our proposed holistic methodology consisting of a structural composite material along with an integrated carbon nanotube sensing layer. The purpose of this final test was to evaluate the effectiveness of the proposed rehabilitation methodology at prolonging the fatigue life compared to the traditional method of drilling a crack-stop hole.

1.4 Outline of Thesis

This thesis is organized into five chapters. Chapter 1 provides a basic introduction to fatigue and fatigue cracking as it relates to steel bridge members. It also provides a statement of the problem that inevitably led to the subsequent research preformed. The justification as well as the objective of the study is described in this chapter.

Chapter 2 expounds upon the past work performed in reference to the topic. Specific details pertaining to current rehabilitation practices employed in the field of civil engineering can be found here. Moreover, the emerging innovative use on composite materials for the purpose of rehabilitation of steel bridges is discoursed. In addition, this chapter presents background information that has been previously researched related to carbon nanotube-based sensing and its applications.

Chapter 3 describes all aspects related to experimental research performed for this study. The initial portion of this chapter focuses on the field monitoring of an in-service bridge and the components involved in the wireless sensing network (WSN) system employed. The instrumentation plan used is presented as well as the actual
field installation of the strain gauges. The succeeding subchapters discuss the numerical data analysis executed as well as the laboratory testing of the ASTM steel specimens. The purpose of this chapter is to provide step-by-step procedures as to the methodology engaged in this study.

*Chapter 4* presents the findings from the aforementioned field monitoring, numerical data analysis and laboratory testing. Sample plots and tables of the data recorded throughout the length of the in-service monitoring period are also included herein. Additionally, this section initiates discussions based on the findings presented in this chapter.

*Chapter 5* summarizes the results for the entire project and provides overall conclusions drawn from the investigation. General recommendations are described in reference to rehabilitation as well as possible concentration for future research involving rehabilitation of fatigue cracks in steel in bridge members.
2.1 Introduction

2.1.1 Fatigue Cracking

The development cracks in fatigue-prone regions is an issue that plagues a significant number of steel bridges throughout the nation and is a major concern when considering the continued safe operation of a bridge. According to 2014 current data gathered by the Federal Highway Administration (FHWA), nearly one-fourth of total of the bridges listed in National Bridges Inventory (NBI) inventory have been designated as structural deficiency and/or functional obsolescence (http://www.fhwa.dot.gov). The development of fatigue cracks has greatly contributed to the deterioration of these structures. The fundamental phenomenon behind the development of fatigue cracks in steel bridges is a direct result of cyclical loading. Typically, these cracks originate from weld which may cause stress concentrations. These cracks also develop as a result of an existing flaw under the presence of tensile stresses (Mertz 2012). These flaws are typically poor weld details that eventually initiate a crack when tensile stresses are applied or with the introduction of abrupt cross-sectional changes. As a result of repetitive loading, these microscopic cracks develop into macro cracks as they are continually loaded, and may further propagate. Fatigue crack propagation may eventually lead to fracture of the member and potentially to catastrophic failure of the structure for the case of non-redundant structures. Bridges that are of this type or often referred to as fracture-critical. As stated in the Clarification of Requirements for Fracture Critical Members, “fracture critical” bridge/structure is defined by the FHWA as a steel member in tension whose
failure would result in possible section of or the entire bridge to collapse (Lwin 2012). This is due to lack of redundant elements in the structure. Although it is unlikely to cause collapse, any fracture is undesirable. Although some cracking may be tolerable, it is certainly not desired and requires close attention and action to be taken.

2.1.2 Rehabilitation

As fatigue cracking has the ability to lead to fracture or loss of serviceability if not mitigated, the cracks must be effectively rehabilitated in order to restore section to its full nominal properties. Numerous methods have been researched and employed to mitigate the propagation of fatigue cracking in steel bridge members. In some cases, particularly with redundant bridges, it has be found that the most effective method is to quite often simply monitor the crack rather than actually repair (Dexter 2013). This is mainly true for bridges with cracking due to distortion induced fatigue. However, for bridges that have developed fatigue cracks as a result of typical cyclical loading, a number of methods exist that can be employed to rehabilitate a member.

Sections 2 and 3 will present some commonly used rehabilitation methods. Additionally, a relatively new approach to repair fatigue cracks through the use of composite materials is discussed. These methods aim at extending the fatigue life for these structures. Some methods are strictly tailored to repairing fatigue details. While others’ intention is to improve fatigue resistance in order to prevent future fatigue cracking incidences. Nevertheless, each method has its own unique characteristics with a similar objective of improving the fatigue detail. Ultimately it is the responsibility of the individual to consider all methods and apply the procedure that will most effectively rehabilitate and extend the life of the structure for a given scenario.
2.2 Conventional Rehabilitation Methods

2.2.1 Current Rehabilitation Methods

The question of how to appropriately and effectively repair damaged girders/weld details/connections is a major concern for many bridge owners. There are currently various rehabilitation methods that are employed in different states that aim to reduce stress concentrations and/or cease the propagation of fatigue cracks, aiming to improve and increase the fatigue lives of these structures.

As discussed in the Federal Highway Administration’s Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges, rehabilitation methods can be divided into three major retrofit techniques categories: (1) Surface treatments, (2) repairs of through-thickness cracks and (3) modification of the connection or the global structure to reduce the cause of cracking (Dexter 2013). Surface treatments generally embody repair methods that can be described as "weld improvement" retrofits used to increase the fatigue strength of un-cracked welds. These include surface grinding, plasma re-melting of weld toe and impact treatments. Some examples of through-thickness cracks repairs are drilling holes, vee-and-weld, adding doubler/splice plates over crack and post-tensioning. Finally, Dexter’s third major category is that of modification of the actual connection detail. Connection can be retrofitted and altered using a few different methods but inevitably that detail is altered in some matter in order to reduce stress in the joint (Dexter 2013).
2.2.1.1 Surface Treatments

Surface treatments to welds plagued with fatigue cracking issues can lead to a reduction in stress concentration and the elimination of discontinuities from which fatigue cracks propagate. Weld improvement techniques, such as toe grinding, have been shown to increase the fatigue life by a factor of two or more, equivalent to an increase in the allowable stress range of 30% (Rutherford 2006). In scenarios where cracks are located at the edges of flanges or other plates, grinding can be employed in order to completely remove portions of a detail containing small cracks. (Dexter 2013). The issue associated with grinding is that much confidence is placed in the operator. If not careful, the operator can remove too much material and/or gouge the material. In addition, grinding can only be used in order to rehabilitate ‘shallow’ cracks found to be less than 3 mm (0.118 in.) in depth. Furthermore, extensive grinding can become an expensive repair method for bridges.

One of the easiest and least expensive surface treatments is hammer peening, which is a very effective and commonly used impact treatment. This method of impacting the surface allows for the introduction of compressive residual stress near the weld toe. These compressive residual stress “lower the effective tensile stress range locally on crack-like defect” (Dexter 2013). Peening is the process of working a metal’s surface to improve its material properties, usually by mechanical means such as hammer blows. It tends to expand the surface of the cold metal, thereby inducing compressive stresses or relieving tensile stresses already present. Peening can also encourage strain hardening of the surface metal. According to Branco (2004), air hammer peening is a reliable technique for repairing welds with shallow surface cracks up to 3 mm (0.118 in.) in depth. Generally speaking, the depth of the crack refers to the distance through the thickness of the material that the crack has
penetrated. Typically this is the smallest of the three dimensions. While crack length refers to the actual extension of crack growth from end to end; commonly the greatest of the three dimensions of a body. Referencing back to hammer peening, for cracks with depths beyond 3 mm (0.118 in.), only a very small increase in fatigue life was found. For crack depths above 5 mm (0.197 in.), there is relatively no benefit. Therefore hammer peening is generally found to only be effective for cracks with depths below 2.5 mm (0.098 in.). This study also ultimately found that a second hammer-peening treatment applied for repair of an existing crack does not provide any significant improvement in fatigue life.

Another quite effective impact treatment is Ultrasonic Impact Treatment (UIT). Dexter states that UIT is considered one of the most effective methods for improving the reliability and fatigue strengths of welded joints (Dexter 2013). Similar to that of hammer peening, UIT introduces beneficial compressive residual stresses at the weld toe by plastic deformation of the surface and reduces stress concentration by smoothening of the weld toe profile (Gunther 2005). In Gunther’s research this was performed by mechanical hammering in order to deform the weld toe at a frequency of around 200 Hz superimposed by ultrasonic treatment at a frequency of 27 kHz. In comparison to hammer peening, UIT is less noisy and equipment is easier to handle. But with required equipment licensing agreements it tends to be more expensive than conventional hammer peening (Gunther 2005). Overall, there is a significant enhancement of the fatigue resistance by UIT compared to the as-welded state. Therefore it can be considered as a viable rehabilitation method for shallow fatigue cracks particularly for high strength steels. Additional research into the effectiveness
of the method on full-scale welded bridge girders is needed as UIT technology was still proprietary (Dexter 2013).

### 2.2.1.2 Through-Thickness Crack Repairs

As previously stated, fatigue cracks initiate from locations of high stress concentrations. These cracks continue to propagate as a result of repetitive loading and ultimately can penetrate the steel plate in the thickness direction (through the depth), commonly known as through-thickness cracks. Through-thickness cracking can be a result of typical, cyclic loading or out-of-plane bending known also as distortion-induced fatigue. Limited research has explored this idea of out-of-plane bending in reference to crack propagation behavior (Ju 2012). Nevertheless, some research has been conducted on methods that can be applied in order to rehabilitate bridges plagued with through-thickness fatigue cracking issues.

#### 2.2.1.2.1 Crack-Stop Holes

The most common and widely used technique used to prevent the propagation of cracks in bridge girders is drilling crack-stop holes at the crack tips. The idea behind this technique is that by drilling the hole at the tip the stress concentration at the tip of the cracks are reduced. Thus the fatigue life of the bridge is extended. According to the *Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges*, in order for the hole to be sufficient enough to successfully arrest the crack the diameter needs to be 50.8 to 102 mm (2 to 4 in.). Theoretically, the diameter of the hole required to arrest crack growth can be calculated from a formula developed by Rolfe and Barsom (1977), Fisher et al (1980, 1990) and Dexter (2013). The
relationship to determine the diameter of a crack-stop hole is based on linear-elastic fracture-mechanic theory, and as follows:

\[
D = \frac{S_r \pi a}{8 \sigma_y} \geq 1.0 \text{ in} \quad \text{(Equation 2.1)}
\]

In this equation, \(S_r\) is the nominal stress range where the crack tip is located, \(\sigma_y\) (ksi) is the yield stress of the material and \(a\) (inches) is the half-crack length (the distance between crack tips is \(2a\)) (Dexter 2013). The issue associated with using the above equation is that quite often the equation generates a hole diameter that is larger than what can be practically drilled. In addition, many owners are not comfortable with placing large holes in members. These limitations often results in undersized holes used in the field to arrest cracks (Crain 2010).

A viable solution to this problematic restriction is to strengthen these undersized crack-stop holes. This can be done in a number of ways, the most common being the use of cold expansion. The use of cold expansion allows for the incorporation of compressive residual stress around the hole. After the hole is drilled, a hammer is used to drive a tapered mandrel (also referred as a drift pin) that is slightly larger than the hole through it. This creates a compressive field as a result of the hole plastically deforming (Dexter 2013). Research conducted at the University of Kansas investigated the use of piezoelectric impact compressive kinetics (PICK) technique on ‘mild’ steels in order to determine its possibility to enhancement undersized, drilled crack-stop holes. “The PICK tool is used to apply a compressive pre-stress coupled with grain refinement around holes used to arrest fatigue cracks in steel bridges” (Crain 2010).

Ultimately this study found that this tool has shown the ability to improve fatigue life of crack-stop holes by a factor of four than that of the untreated specimens. Further research in needed in order to design a new PICK tool for application in field
conditions in addition to testing this tool on scaled bridge members subjected to a realistic crack-stop hole’s out of plane stress environment (Crain 2010).

In a similar investigation the effectiveness of drilled stop holes was explored. An innovative method that has previously been employed by the aerospace industry, that radially expands a high interference bushing into the hole, was employed. This method is intended to induce residual compressive stresses around the hole (Reid 2014). The magnitude of the residual stress was approximately equal to $2/3$ of the tensile yield stress of the material (A36 steel with yield stress of 46.6 ksi and tensile strength of 70.1 ksi). Inducing these stresses around the hole theoretically is an effective way to prolong crack growth. Shown below in Figure 2.1 is this proprietary procedure known as StopCrackEx Process. Seven dogbone-type specimens composed of A36 steel with a small initial 0.38 mm (0.015 in.) starter notch for natural fatigue propagation were tested. Half the specimens were repaired employing the crack-stop hole drilling method and the other half with the new StopCrackEx process. Each specimen was cycled until a new crack initiated on the opposite side of the repair. Results showed at least a 12 times improvement in fatigue life for the new method compared to the conventional hole drilling method. In addition, an over 60 times fatigue life improvement was observed for one coupon specimen that was subsequently cycled to 20 million cycles with no evidence of any crack initiation. Ultimately both methods proved to be effective at retarding fatigue crack growth. But from the study, this new technique known as StopCrackEx demonstrated exceptional properties at completely discontinuing crack propagation in addition to long-term fatigue life improvements.
2.2.1.2.2 Doubler and Splice Plates

The addition of doubler or splice plates is another through-thickness cracks repair. Adding doubler plates for fatigue crack repair essentially allows for additional cross-sectional area, which in turn results in reduces stress ranges (Dexter 2013). These plates are intended to restore the full cross-sectional properties of the uncracked girder. This method can be used in conjunction with weld repairs and crack-stop hole drilling as shown in Figure 2.2. The philosophy behind this is that this will ensure the weld repair will have adequate fatigue resistance. Doubler plates can be added after the repair is made. Ideally this will decrease the stress range that originally contributed to the development of cracking, thus protecting the girder. An issue associated with the repair method is maintaining alignment of the two sides of the cracked section. “The cracked surface usually develops buckles, making alignment difficult” (Dexter 2013).
Nevertheless, the addition of a doubler plate has been shown to improve fatigue crack life, particularly for full-depth cracks in bridge girders.

![Bolted doubler plate repair. Dashed line represents crack beneath doubler plate and circle is the hole drilled to reduce the stress concentration at the crack tip. (Source: Dexter 2013)](image)

**Figure 2.2** Bolted doubler plate repair. Dashed line represents crack beneath doubler plate and circle is the hole drilled to reduce the stress concentration at the crack tip. (Source: Dexter 2013)

### 2.2.1.3 Connection Modification

Connection modification must be considered in cases where the rehabilitation method employed requires the stress ranges to first be decreased before the method can effectively repair the fatigue cracking issue. This could be done by through means of the addition of doubler plates or “introducing a soft-toe or radius” (Dexter 2013) in details compromised of sharp corners, as a means of decreasing local stresses. Another connection modification can even go as far as to redesigning the entire connection.
Although this can be extensive, it can result in a reduction of peak stresses in the joint and thus effectively correct the issue.

### 2.2.2 Conventional Methods Summary

Below is a Table 2.1 containing a comparison of a few of the previously discussed current rehabilitation methods. These are the most commonly employed technique for fatigue crack repairs in the Civil Engineering industry.

Table 2.1 Comparison Currently Fatigue Crack Rehabilitation Methods.

<table>
<thead>
<tr>
<th>Surface Treatment</th>
<th>Rehabilitation Method</th>
<th>Description/Process</th>
<th>Advantages</th>
<th>Limitations</th>
</tr>
</thead>
</table>
| Surface Grinding  | Use of a rotating abrasive wheel to smooth surface or totally remove portions of a detail containing small cracks | - Shown to increase fatigue life by factor of two or more  
- Could increase allowable stress ranges 30% | - Only applicable for shallow cracks (≤ 3mm)  
- Extensive grinding can be expensive  
- Only viable for crack penetration ≤30% of plate thickness | |
| Hammer Peening    | Use of mechanical means (hammer blows) to work a metal's surface to improve its material | - Easy  
- Inexpensive  
- Encourages strain hardening | - Only effective for shallow cracks (≤ 3mm) | |
| **Ultrasonic Impact Treatment (UIT)** | Cold mechanical treatment consisting of impacting surface to release tensions and add beneficial compressive stresses | - Automated  
- Effective  
- Less noisy than hammer blower  
- User friendly Equipment | - Requires equipment licensing agreements  
- More expensive than conventional hammer peening |
| **Drilling Crack-Stop Holes** | - Use of a circular drill bit in order to remove portion of material containing fatigue crack | - Quick and Simple  
- Removes stress concentration at crack tip | - Holes may be undersized  
- Hole must be precisely drilled |
| **Addition Doubler or Splice Plates** | - Addition of plate over damaged portion in order to increase cross sectional area | - Can be used in conjunction with other techniques  
- Effective for full-depth cracks | - Difficult to maintaining alignment of two sides of cracked section |

### 2.3 Composite Materials

A relatively new and innovated technique to address fatigue cracking issues in steel girders is the use of composite materials. These materials have mainly been used in the aerospace and the military industries but are beginning to emerge into applications in civil engineering. When bonded to a surface, composites have the ability to stiffen and possibly extend the service life of deficient bridge girders. Fiber-reinforced polymer (FRP) composites have the advantage of high stiffness-to-weight and strength-to-weight ratios, superior environmental durability, fatigue resistance, flexibility, high chemical corrosion resistance, and ease of handle and maintenance (Karbhari 1995). As a result they have been deemed a promising material to be used in structural rehabilitation. FRP composites consist of a polymer matrix material such as
epoxy or polyester plastic reinforced with fibers made of, e.g. glass, carbon, or Aramid. With their physical properties and endless conceivable ways in which to orient these fibers, composites have the ability to be tailored to specific material needs.

### 2.3.1 Fiber Reinforced Polymers (FRP)

Fiber reinforced polymers (FRP) (also known as fiber-reinforced plastic, fiber-reinforced thermoplastic and today as fiber reinforced composites) were not developed until the early 1940’s (Tang 1997). Early applications began after World War II in the production of fiberglass for use in composite boat hulls and radar cover. The navy later began using these materials for pressure vessels in mine sweeping vessels, crew boats and submarine parts. In addition, composites began to emerge as a result of consumer demand for recreation products such as composite fishing rods, tennis rackets, ski equipment and golf clubs. The aerospace industry began to use composites in pressure vessels, containers, and non-structural aircraft components (Tang 1997). In 1968, composites were introduced into the civil engineering field in the form of a dome structure built in Benghazi.

FRPs are anisotropic with strength dependency on the direction of the fibers. These fibers can be oriented in a variety on directions enabling composites with the ability to be tailored to the material/mechanical properties needed. The fibers are chopped, woven, stitched, and/or braided (Tang 1997) to form sheets or fabrics. In order to obtain the desired strength and/or stiffness, several sheets can be added on top of each other.

As discussed by Benjamin Tang in the FHWA’s article entitled *Fiber Reinforced Polymer Composite Applications in USA*, fiber reinforced polymer composites are composed of fiber reinforcements, resin, fillers, and additives. The
fibers occupy between 30% and 70% of the matrix volume and have the ability to provide increased stiffness and tensile capacity. Fibers are bound together in a stable matrix through the resin which offers high compressive strength. The purposes of the fillers are to reduce cost and shrinkage. The additives assist in improvement to mechanical and physical properties of the composite. In addition they help with workability (Tang 1997).

2.3.1 Fiber Options

The most common types of fibers used in advanced composites for structural applications are made of aramid, glass, and carbon. Composites based on these fibers have been used in numerous applications from planes and luxury vehicles to orthopedic products and fishing rods.

2.3.1.1 Aramid Fibers

Aramid fibers are strong synthetic fibers that are commonly used in aerospace and military applications, for ballistic rated body armor fabric and ballistic composites. In the civil engineering industry, the most common commercial grades used for structural applications are Kevlar 29 and Kevlar 49 (Tang 1997). These fibers have been found to have exceptional fatigue and creep resistance. Other properties include: abrasion resistance, non-conductivity, tremendously high melting point, low flammability, outstanding strength-to-weight ratio, high Young’s Modulus (see Table 2.2), and high ultimate tensile strength. As a result, aramid fibers are good candidates for retrofitting materials.
Table 2.2  Aramid Fiber Material Properties (Source: Zweben 1989)

<table>
<thead>
<tr>
<th>Typical Properties</th>
<th>Kevlar 29</th>
<th>Kevlar 49</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Density</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(g/cm³)</td>
<td>1.44</td>
<td>1.44</td>
</tr>
<tr>
<td>(lb/ft³)</td>
<td>89.90</td>
<td>89.90</td>
</tr>
<tr>
<td><strong>Young’s Modulus</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(GPa)</td>
<td>83/100</td>
<td>124</td>
</tr>
<tr>
<td>(ksi)</td>
<td>12040/14500</td>
<td>17984</td>
</tr>
<tr>
<td><strong>Tensile Strength</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(GPa)</td>
<td>2.27</td>
<td>2.27</td>
</tr>
<tr>
<td>(ksi)</td>
<td>329.24</td>
<td>329.24</td>
</tr>
<tr>
<td><strong>Tensile Elongation (%)</strong></td>
<td>2.80</td>
<td>1.80</td>
</tr>
</tbody>
</table>

2.3.1.1.2  Glass Fibers

Glass fibers have been found to be the least expensive of the three most commonly used fibers for civil engineering applications (Tang 1997). These fibers are subdivided into three classes: E-glass, S-glass and C-glass. The E-glass is elected for electrical uses while the S-glass for high strength. The C-glass is designated for high corrosion resistance, and it is generally uncommon for civil engineering application. Amongst these three glass fibers, the E-glass is the most common reinforcement material used in civil structures (Tang 1997). One of the drawbacks of glass fibers is that with increasing temperature, strength and stiffness decrease. When exposed to extremely high temperatures, particularly those above 80⁰C, the fiber tend to break. In contrast, when exposed to extremely cold temperature, those typically below -10⁰C, fiber become pulled out (Badawy, 2012).
Table 2.3  Glass Fiber Material Properties (Source: Zweben 1989)

<table>
<thead>
<tr>
<th>Typical Properties</th>
<th>E-Glass</th>
<th>S-Glass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (g/cm³)</td>
<td>2.60</td>
<td>2.50</td>
</tr>
<tr>
<td>Young’s Modulus (GPa)</td>
<td>72</td>
<td>87</td>
</tr>
<tr>
<td>Tensile Strength (GPa)</td>
<td>1.72</td>
<td>2.53</td>
</tr>
<tr>
<td>Tensile Elongation (%)</td>
<td>2.4</td>
<td>2.9</td>
</tr>
</tbody>
</table>

2.3.1.1.3 Carbon Fibers

As shown in Table 2.4, carbon fibers are divided into three main classifications: high strength, high modulus and ultra-high modulus. Similar to that of aramid, carbon fibers have very high fatigue and creep resistance. Some of these properties include: high strength-to-weight ratio, its ability to provide rigidity and strength, as well as stiffness. Additionally, of the previously discussed fiber options, carbon fiber is found to be the most expensive.

Table 2.4  Carbon Fiber Material Properties (Source: Zweben 1989)

<table>
<thead>
<tr>
<th>Typical Properties</th>
<th>High Strength</th>
<th>High Modulus</th>
<th>Ultra-High Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (g/cm³)</td>
<td>1.8</td>
<td>1.9</td>
<td>2.0 - 2.1</td>
</tr>
<tr>
<td>Young’s Modulus (GPa)</td>
<td>230</td>
<td>370</td>
<td>520 - 620</td>
</tr>
<tr>
<td>Tensile Strength (GPa)</td>
<td>2.48</td>
<td>1.79</td>
<td>1.03 - 1.31</td>
</tr>
<tr>
<td>Tensile Elongation (%)</td>
<td>1.1</td>
<td>0.5</td>
<td>0.2</td>
</tr>
</tbody>
</table>
2.3.1.2 Composite Action

As previously stated, when a polymer matrix material is reinforced with one of these fibers, such as carbon fiber, composites are formed. For instance, carbon fiber-reinforced polymers (CFRP) are a subcategory of FRP that is composed of carbon fibers. In the civil engineering industry CFRP is most renowned for its use as a rehabilitation method for bridges. Its low strength to weight ratio make it quite useful in applications such as, increasing the load capacity of older under-designed structures, seismic retrofitting, and repair of damaged structure resulting from fatigue cracking.

Many researchers have examined the use of carbon fiber composites to rehabilitate steel bridges. In a 2001 study conducted at the University of Delaware, CFRP plates were tested in order to determine effectiveness of strengthened steel bridge girders. The rehabilitation method developed consisted of bonding advanced composite material plates to the tension flange of steel bridge girders. The overall objective was to provide additional stiffness and strength (Miller 2001). Four full-scale 6.40m (21 ft.) long bridge girders were retrofitted and tested for effectiveness in the laboratory. These girders were rehabilitated with a single layer of CFRP bonded to inner and outer faces of tension flange and loaded in three-point bending to failure. In addition two field load test were performed (one before and one after CFRP rehabilitation) using loaded 3-axle dump trucks driving directly above the rehabilitated girder. Ultimately, this study found that the retrofit increased elastic stiffness from 10% to 37% in the lab study and an 11.6% increase was examined in the field testing. In addition it was estimated that strain decreased by 10% as a result of the increased strength and stiffness (Miller 2001).
From Table 2.4 and research using CFRP strips, ultra-high modulus carbon fiber composite is a suitable candidate for rehabilitation of steel structures. However, due to the electric conductivity of carbon fibers, it can be corrosive in nature due to galvanic interaction between steel and composite plates. In addition, composite are vulnerable when exposed to ultraviolet (UV) light, i.e. the sunlight, as well as moisture. The UV radiation, as well as exposure to moisture, have been shown to develop adverse effects on the mechanical properties of the polymeric epoxy matrix within the composite; resulting in the degradation of the composite. The actual carbon fibers however are not significantly affected by either environmental condition (Kumar 2002).

2.3.2 Debonding

Debonding and delamination are failure mechanisms that are associated with adhesives. Such failures, unless adequately considered, may significantly decrease the effectiveness of the strengthening approach. Debonding occurs when a reinforcing plate detaches from a parent member at the steel/adhesive interface shown in Figure 2.4. Typically, debonding of FRP strengthened members occurs in regions of high
stress concentrations (Buyukozturk 2004). It is often associated with the presence of cracks. Finite element modeling has shown that deep cracks have been found to have a significantly higher strain energy release. This energy, as defined by Aggelopoulos, is a “measure of crack driving force” (Aggelopoulos 2011). It was established that this force increases as the modulus and thickness of composite patch are increased. In a similar study done in Italy by Colombi of Milan Technical University, sensitivity of debond crack strain energy release rate in relation to parameters such as patch stiffness, adhesive thickness and pretension level were investigated (Colombi 2003). Ultimately one conclusion drawn was that strain energy release rate is not strongly dependent upon patch stiffness. This is mainly due to high stiffness of steel compared to CFRP patch. In addition this study found that a thin adhesive layer produces a larger debonded area, which in return reduces the effectiveness of the reinforcement. Finally, the research was able to confirm that there is a fundamental contribution to effectiveness of the patch when it is pretensioned. Pretensioning the patch causes a compressive load applied to steel plate. This in return reduces the crack strain energy release rate (Colombi 2003). Also, there was a reduction in load transferred from the steel plate to the composite strip. Due to the fact that presstressed specimen has been shown to stop crack propagation and extend the fatigue life it can be concluded that “pretensioning of the composite patch prior to bonding is strongly recommended in order to maximize effectiveness of the bonded patch on the steel section” (Colombi 2003).
2.3.3 Adhesive Selection

The matrix material (also referred to as adhesive or resin) selected plays a significant role in the effectiveness of the composite strip to rehabilitate a bridge plagued with fatigue cracking. As discussed in the Federal Highway Administration's Proceedings of the First Korea/U.S.A. Road Workshop, the resins most commonly employed in composites are the “unsaturated polyesters, epoxies, and vinyl esters” (Tang 1997). In contrast, the least common resins used are the polyurethanes and phenolics. As previously discussed, research conducted at the University of Delaware examined the used of carbon fiber-reinforced polymer (CRFP) plates to strengthen steel bridge girders. In this research two different adhesives were considered: a two-part high-strength epoxy (Araldite AV8113/HV811) and a methacrylate epoxy (ITW Plexus MA555). In order to select the appropriate adhesive, both shear strength and bond durability were considered. Bond durability must be accounted as these adhesives will be subjected to a number of environmental conditions that could inevitably affect the bond. These conditions include: deicing agents as well as chlorides from saltwater (Miller 2001).
These two structural adhesives, Ciba-Geigy AV8113/HV8113 and Plexus M4555, were used in this rehabilitation project at the University of Delaware to enable examination of the in-field application and long-term durability of both. Each adhesive is applied to half of the tested girder as shown in Figure 2.5 (Mertz 1996).

Figure 2.5  CFRP retrofit Scheme. (Source Mertz 1996)

Shear strength results are shown below in Table 2.5. Aside from higher shear strength, Ciba-Geigy's AV8113 adhesive also displayed good durability under several environmental conditions including immersion in 65°C water, a commercial deicing agent, and freeze-thaw conditions (Mertz 1996). Although previous research has shown that either epoxy is acceptable for the purposes intended in this research, the Plexus M4555 was found to be more beneficial because of the relatively short cure time and low sensitivity to thick bond lines. This was determined in spite of the findings that Plexus MA555 adhesives have lower shear strength in comparison to Ciba-Geigy AV8113. But ultimately a reduced cure time allows for the bridge to be opened to normal traffic sooner, which limits the impact and significant concern to the public. In reference to the low sensitivity to thick bond lines, this allows for the CFRP
plates to be placed over irregular surfaces, such as severely corroded girders, without sacrificing strength (1996).

<table>
<thead>
<tr>
<th>Adhesive</th>
<th>Shear Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Araldite AV8113/HV8113</td>
<td>13.8 - 17.2</td>
</tr>
<tr>
<td>Plexus MA555</td>
<td>8.6 - 10.3</td>
</tr>
</tbody>
</table>

### 2.4 Carbon Nanotube (CNT) Based Sensing

#### 2.4.1 Background Information

As explored by University of Delaware professor Erik Thostenson, carbon nanotubes have been found to possess properties that enable them to functionalize composites. For example, they have the ability to provide real-time monitoring of damages developed in structural members. “The formation of a carbon nanotube network around the structural reinforcement in fiber composites has enabled in situ monitoring of matrix damage accumulations” (Gao 2009). In spite of this new found knowledge, only a limited amount of research has been explored on the use of carbon nanotube networks for structural health monitoring (SHM) of structures.

Carbon nanotubes (CNTs) can be envisioned as a rolled sheet of graphite that is formed into a cylinder. These nanotubes can exist as either single-walled (SWCNT) or multi-walled (MWCNT) structures as shown in Figure 2.6. Multi-walled nanotubes are simply composed of a number single walled nanotubes held together with
relatively weak van der Waals forces (Thostenson 2001). These hexagonal networks of carbon atoms are approximately 1 nm diameter and 1 to 100 microns in length.

Figure 2.6  Carbon Nanotube Structure (Source: Martins-Júnior 2013)

2.4.2  Properties

In Thostenson’s 2001 study, research on several properties of CNTs including mechanical, electrical and thermal were reviewed. As a result of their symmetric structures, “these cage-like forms of carbon have been shown to exhibit exceptional material properties” (Thostenson 2001). Ultimately the review found that both theoretical and experimental results showed that CNTs have a particularly high Young’s modulus. Results displayed values greater than 1000 GPa (145,000 ksi) which is comparable (if not greater) to that of diamond which possess a Young’s modulus of about 1200 GPa (174,000 ksi). Furthermore it reported “strengths 10-100 times higher than the strongest steel at a fraction of the weight” (Thostenson 2001). Assuming a Young’s modulus of 1250 GPa (181,000 ksi), the yield strength of closely packed nanotubes was demined to be greater than 45±7 GPa (6527 ±1015 ksi) by Walters of Rice University (Walter 1999). When compared to that of typical high-
strength steels, this value is over 20 times greater. These findings further confirm the concept of carbon nanotube having astonishing mechanical properties.

In a 1999 study conducted by Jean-Paul Salvetat, elastic and shear moduli of single-walled carbon nanotubes (SWCNT) were explored. The inquiry concluded that as the diameter of the tube bundles increases, the axial and shear moduli drastically decrease (Salvetat 1999). This simply demonstrates the issue of slipping that can occur within the bundle of nanotubes.

In addition to the remarkably high Young’s modulus of carbon nanotubes, they also possess exceptionally large elastic strain and fracture strain sustaining capability. Aside from the mechanical properties Thostenson’s study also discussed the superior thermal and electric properties associated with CNTs. Thermal conductivity was found to be twice as high as diamond while the electric-current carrying capacity is more than 1000 times higher than copper wires (Thostenson 2001). As stated by Zhidong Han, the “unusually high thermal conductivity makes CNTs the best promising candidate material for thermally conductive composites” (Zhidong 2011). An example can be seen in the research conducted by Gao. In this 2010 research, a network of CNTs were applied for use in real-time monitoring of structural elements in order to detect damage development. CNT networks were formed around the structural reinforcements in fiber composites to enable this in situ monitoring. Ultimately it was discovered that “when electrically conductive carbon fibers are broken or when the formation of matrix cracks prevents intra-fiber contacts, bulk changes in composite electrical properties occur” (Gao 2009). This is due to the fact that the carbon fibers are conductive. Thus fracture of fibers will result in changes in electrical resistance.
The mechanical properties in combination with the electrical properties make CNTs excellent candidates for in situ sensing. As a result it has been reported that these nanotube-based materials have been employed as “electromechanical actuators and in a variety of sensing, applications, including mass sensors, humidity sensors, and strain sensors” (Thostenson 2006). But as previously stated, few studies have research the use of these sensors in applications of structural health monitoring. A study was conducted at the University of Delaware with the application of structural carbon nanotubes on concrete structures (Schumacher 2014). This research focused on the use of an integrated structural composite layer bonded to a concrete beam for distributed sensing. “This layer consisted of carbon nanotubes that are deposited on a carrier, which form a continuous conductive skin that is exceptionally sensitive to changes in strain and the formation of micro-damage and macro damage” (Schumacher 2014). The CNTs enables nerve-like distributed sensing capacities which allow for this damage detection. For this experiment a concrete beam measuring 533 mm (21 in.) in length, 152 mm (6 in.) in width and 152 mm (6 in.) in depth were loaded in three-point bending. Prior to loading, the reinforcing composite sensing layer was fabricated and bonded using an epoxy resin to the bottom portion of the concrete test specimen. When the load is applied in three-point bending, the layer will go into tension. Ultimately this investigation not only affirmed its hypothesis that structural CNTs can be applied to a concrete member for distributed sensing capabilities but also provided a new innovative approach that can be applied to SHM in the future. The use of these embedded nanoscale sensors allow for the detection of damage and deformations along the tensile face (2014). Thus allowing for real-time
feedback to changes in the structure such as strain and damage propagation/formation in order to evaluate structure’s integrity.

In Thostenson's 2006 exploration along with Schumacher's 2014 study they briefly articulated the multi-functionality of CNTs. Although CNTs generally do not offer increased material strength, when incorporated with composites, CNTs can serve as a platform allowing for sensing within the structural reinforcing (Thostenson 2006). As articulated by Schumacher, the patch act as both reinforcement, provided by the glass fibers, in addition to a sensor as a result of the conductive network formed by the CNTs. (Schumacher 2014). This dual purpose technology has the ability to usher in a new innovative, integrated methodology of addressing SHM in addition to the rehabilitation of fatigued structures.

The theory that CNT composites act as multifunctional reinforcement was further investigated in 2007 when Zhang established that CNT composites can actually play a role in suppressing fatigue crack growth. In this investigation both single walled (provided by Cheap Tube Inc.) and multi-walled (provided by Nanocyl) carbon nanotubes were dispersed in Exploy-2000 and used to preform fatigue crack propagation testing using a MTS-858 material testing system following ASTM standard E647-05 (Zhang 2007). This experiment ultimately observed a reduction in crack propagation rate by an order of magnitude. In the case of the epoxy/MWCNT composites a “~1000% reduction in the crack growth rate in the low stress intensity factor amplitude regime” was observed (Zhang 2007). This reduction in crack growth was further enhanced with increasing weight fraction of the nanotube additives. The same could be said for the epoxy/SWCNT composites. The weight faction used for the MWCNT was ~0.5% while the SWCNT was in the range of 0.1%-0.25%. The weight
fraction of the nanotubes as well as the applied stress intensity factor played a significant role in the reduction in crack growth rate that was witnessed (Zhang 2007). As a result of the addition of less than ~0.5 weight percentage of CNT additives, structure polymers fatigue performance potential can greatly develop. This study thus reaffirmed that carbon nanotube composites have the potential to be able to both suppress and detect the initiation of fatigue damage.

The use of CNTs has the potential for continued evaluation of the status of in-service SHM. The formation of these networks can enable real-time response to cracks changes in strain, temperature effects as well as the formation/propagation of damage as a result of the electrical response of CNTs. In addition, its abilities when combined with reinforcing fibers, to not only act as a sensor but also the multifunctional enabling the suppression of fatigue damage, allows this integrated patch to be an excellent candidate as composite reinforcement.

2.4.3 Fabrication

One method to fabricate CNT-epoxy composites is to first disperse the CNTs into an epoxy resin. In many cases for uses in SHM, a calendering approach is applied. “The calendering approach utilizes a three-roll mill to untangle the agglomerates by feeding the dispersion through high-precision rollers that impart high shear forces” (Schumacher 2014). As shown in a similar procedure in Figure 2.7, nanotubes are first mixed by hand into the epoxy resin. Typically a 0.5% weight concentration is added to the epoxy resin prior to processing through a three-roll mill (Thostenson 2006). As discussed previously by Zhang, the addition of less than ~0.5 wt.% of CNT additives has the potential to play a role in suppressing fatigue crack growth.
As employed in Gao’s 2009 experiment, nanotube/epoxy dispersions were infused and composite laminates were produced with “cross-ply constructions of [0/902/0], [0/903/0] and [0/904/0] using a vacuum-assisted resin transfer molding technique (VARTM)” (Gao 2009). Essentially, the vacuum-assisted resin transfer molding was used to fabricate the fiber–epoxy composites with embedded CNTs (Thostenson 2006).

A fiber sizing agent containing dispersed CNTs can be used alternatively to that of the traditional calendering approach. This inevitably will lead to the fabrication of a highly electrical conductive glass fiber/epoxy composite. Employing this method, “the distribution of CNTs in the composite can be controlled and substantial amount of CNTs agglomerates on the glass fiber surface which results in a 2 to 3 orders enhancement in the composite electrical conductivity in the axial, transverse and through-thickness directions” (Gao 2010).

CNTs can be also be fabricated as a long film. Employing this method, to make a sensor, network neural systems are developed in the form of a grid attached to the surface of a structure According to Rainieri, this method has potential applications in the field of structural monitoring of large civil structures (Rainieri 2007).
2.5 Field Instrumentation

2.5.1 Strain Gauge Placement

The placement of strain gauges can play a significant role in the determination of axial strain related to fatigue crack propagation. Strain gauges have been employed for decades in order to determine the applied stresses experienced by in-service bridges. Such as in the case of the monitoring system developed at the University of Delaware in 2000. The small battery operated system and foil strain gauges were able to capture peak stresses that ultimately assisted in load rating of the bridge, fatigue investigations, monitoring the bridge response as overloads crossed, and for general health monitoring of the structure (Shenton 2000). Stresses recorded from a strain gauge can then be plotted and related to AASHTO S-N fatigue curve or the actual design yield strengths of the material in order to draw conclusion about the current state of the structure. Strain gauge locations in reference to a fatigue crack can influence these values obtained. Therefore gauges should be place in a manner in which to capture the stimulating data experienced at the crack tip as well as along the projected crack path. Although there is no ‘optimal’ strain gauge location in which to capture all pertinent data, much consideration should be taken in order to employ gauges in an appropriate manner to accurately and appropriately monitor the structure, especially when considering fatigue and fatigue crack monitoring.

Ultimately in order to record the highest strains being experienced by a fatiguing girder, the strain gauge should be placed at the tip of the crack. This technique allows for evaluation of strains as the crack opens and closes. Moreover, gauges should not be placed too close the tip of the crack. Particularly when employing methodology of evaluating stress intensity factors using strain gauges,
placing gauges too close to crack tip should be circumvented “to avoid plasticity and three-dimensional effects” (Sarangi 2010). But employing gauges too far from the crack tip will result in the inability to accurately capture strains at the crack mouth opening. Thus in Sarangi’s 2010 research the optimal maximum radial distance ($r_{\text{max}}$) for gauge locations was examined. A finite element approach was implemented in order to determine the upper bound for radial location of the gauges. Ultimately this study found that $r_{\text{max}}$ increases as the crack length increases. In addition the result of $r_{\text{max}}$ as the ratio of crack length or half the crack length (denoted as $a$) to half the width (denoted as $b$) of the plate are shown below in Table 2.6 for a tested edge cracked plate shown in Figure 2.8.

![Figure 2.8 Edge Cracked Plate (Source: Sarangi 2010)](image)

Table 2.6 Variation of $r_{\text{max}}$ for edge cracked plate with $b=47.24$ in. (Source: Sarangi 2010)

<table>
<thead>
<tr>
<th>$a/b$</th>
<th>$a$ (in)</th>
<th>$r_{\text{max}}$ (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00625</td>
<td>0.30</td>
<td>0.07</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>---</td>
<td>---</td>
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</tr>
<tr>
<td>0.0125</td>
<td>0.59</td>
<td>0.16</td>
</tr>
<tr>
<td>0.025</td>
<td>1.18</td>
<td>0.30</td>
</tr>
<tr>
<td>0.05</td>
<td>2.36</td>
<td>0.53</td>
</tr>
<tr>
<td>0.1</td>
<td>4.72</td>
<td>1.01</td>
</tr>
<tr>
<td>0.15</td>
<td>7.09</td>
<td>1.62</td>
</tr>
<tr>
<td>0.2</td>
<td>9.45</td>
<td>2.02</td>
</tr>
<tr>
<td>0.25</td>
<td>11.81</td>
<td>2.66</td>
</tr>
<tr>
<td>0.3</td>
<td>14.17</td>
<td>3.61</td>
</tr>
<tr>
<td>0.35</td>
<td>16.54</td>
<td>5.39</td>
</tr>
<tr>
<td>0.4</td>
<td>18.90</td>
<td>8.23</td>
</tr>
<tr>
<td>0.45</td>
<td>21.26</td>
<td>19.90</td>
</tr>
<tr>
<td>0.5</td>
<td>23.62</td>
<td>13.68</td>
</tr>
<tr>
<td>0.55</td>
<td>25.98</td>
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</tr>
<tr>
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</tr>
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</tr>
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<td>1.36</td>
</tr>
<tr>
<td>0.8</td>
<td>37.80</td>
<td>0.70</td>
</tr>
</tbody>
</table>

As discussed by Sarangi, as the ratio of a/b increases, rmax increase initial but begins to decrease after reaching an a/b ratio of 0.45. The decrease in rmax values could be attributed to an influence by the bottom edge boundary condition (refer to Figure 2.8). This effect of the boundary condition is absent or insignificant at low values of a/b. This condition becomes more dominate at the ratio of a/b increase and crack length approaches the width b.

Nevertheless, this finite element approach to establish rmax can be quite useful in the determination of proper strain gauge placement. When instrumenting a structure one must consider the upper radial distance for gauge placement. In order to record
accurate measurements, the radial location of strain gauge should be within this zone (Sarangi 2010).

In situations where a crack-stop hole has been drilled or a large section of the girder has been removed, such as in Figure 2.9, gauges are in many cases positioned at the free edge of the retrofit. But one thing to note is that when a strain gauge is placed near the edge of a hole (as shown in Figure 2.9), the strain gauge is also reading the local strain (Dexter 2013).

![Weldable Strain Gauges](image)

Figure 2.9  Weldable strain gauges applied near free edge of retrofit on Girard Point Bridge, Philadelphia, Pennsylvania. (Source Dexter 2013).

Lastly, in addition to the consideration of maximum radial distance from the crack tip to the gauge, the location and amount of remaining gauges to be place should also be deliberated. Aside from placement at the crack tip, strain gauges should also be placed along the projected path of the crack. This is typical practice and allows for local deformation to be recorded ahead of the crack tip during cycling. The use of a strain rosette can also be taken into account in order to estimate the principal strains at
that location. The orientation of gauges is significant as the electrical resistance (stress) of the gauges is sensed in the direction parallel to them. The use of a strain rosette arrangement will allow for the determination of principal stresses and ultimately the direction in which the crack is propagating, which can be quite useful for SHM of fatigue cracks.
Chapter 3

EXPERIMENTAL APPROACH

The methodology employed for this research project was broken down into three main components: field monitoring, numerical analysis and laboratory testing. Field monitoring allowed for initial in situ data from an in-service bridge structure found to have fatigue cracking issues. The numerical analysis was subsequently performed in order to better understand the structure. Using this knowledge, laboratory testing was executed with an intention of implementing a rehabilitation method that inevitably increase the fatigue life of these specimen.

3.1 Field Monitoring

In order to determine the stresses that bridge structures in this area experience on a daily basis the initial phase of this research project involved the instrumentation of DelDOT Bridge 1678-006. This structure as shown on the succeeding map is located in Newark, Delaware and carries two travel lanes of Route 141 headed from Newport, DE towards New Castle, DE. It spans over the northbound lanes of Interstate 95.
The instrumentation of DelDOT Bridge 1678-006 involved the use of typical strain gauges in order to record the stresses/strains experience by the structure at various portions of the day. This bridge was chosen as a result of investigation of structures in Delaware that have been found to have fatigue cracking issues. A time period of two weeks was selected as the duration of the monitoring and data collection of this bridge. The field study of this structure allowed for initial qualitative results of strains experienced throughout damaged area.

3.1.1 DelDOT Bridge 1678-006

DelDOT 1678-006 was originally constructed in the late 1960’s. As shown in the subsequent Google Earth image, this bridge consists of 5 spans with 4 piers located in the roadway of Interstate 95. Each span is comprised of six girders that carry the above roadway.
Upon inspection, both north and south ends of the abutments were found to have developed fatigue cracks. The south end abutment had a few minor surface cracks, while the north end abutment was discovered to have developed much more significant fatigue cracks. As shown in Figure 3.4, after a 2011 general bridge inspection, a crack was found in Beam 3 of Span 5 of this bridge. Although other cracks were discovered, the more noteworthy of these, shown below, initiated from the weld detail at the web-to-flange connection. After the 2011 inspection, a DelDOT crew drilled two 1-1/8” diameter crack stop holes in an attempt to cease the crack
propagation. In addition a timber block was placed under the beam. A further inspection in 2012 revealed a vertical crack propagating through the previously drilled hole. Most recently, in February of 2013, a larger 2” diameter arrest hole was drilled at the tip of the propagating crack in hopes of completely halting the continued growth of the fatigue crack.

Figure 3.4  Fatigue Crack on DelDOT Bridge 1678-006

As a result of its unrelenting fatigue cracking issue, this structure was a determined to be a prime candidate for the proposed research. This bridge was ultimately instrumented and monitored in order to determine the in situ conditions. Due to its connection type, it has been determined to be a fatigue Category E in accordance with the AASHTO LRFD Bridge Design Specification. In addition Fatigue Category E was chosen based on commentary discussed in FHWA’s Bridge Inspector’s Reference Manual as shown in Figure 3.5. This information along with the measured data, allowed for a baseline for our research. In addition this information jointly permitted the determination of the cumulative fatigue damage predictions and comparisons to that of the design life of the bridge. Thus permitting conclusion to be
drawn about the current state of the structure and determination of whether or not it exceeds allowable limits.

![Table showing Fatigue Category E and E']

Figure 3.5 Commentary for Fatigue Category E and E’ (Source: Bridge Inspector's Reference Manual – Section 8.1.33)

### 3.1.2 Strain Gauges

Typical CEA-06 series weldable strain gauges manufactured by Micro-Measurements (Division of Vishay Precision Group) were employed on DelDOT 1678-006 in order to measure the strain experienced by the structure throughout the day. An image of these typical gauges is depicted in Figure 3.6. Strain gauges were welded to web in the direction along the projected fatigue crack. The gauges were simple strain gauges used in order to record strain. Images taken during the strain gauge welding process of DelDOT Bridge 1678-006 are also presented below in Figures 3.7 and 3.8.
Figure 3.6 Sample Weldable Strain Gauge (Source: www.digikey.com)

Figure 3.7 Images Taken During Strain Gauge Welding Process of DelDOT 1678-006
In total, 12 strain gauges were used amongst two different girders. Five gauges were employed on the East Face of beam 3 on span 5, two on the West Face of that same girder and one on either side of the bottom flange (two in total on flange). A total of nine gauges were welded on this girder. All gauges welded to girder were wired back to the sensing network. Sketches of these gauge instrumentations are shown in Figure 3.9. As discussed in section 3.1.1, this girder has previously been drilled with several crack-stop holes after previous bridge inspections.
In addition to Beam 3, Beam 4 was also instrumented with strain gauges. This was done in order to compare the result of the beam with fatigue damage to that of a “healthy girder.” There were a total of three strain gauges welded to this girder as shown in the following Figure.
It should also be noted that gauges 4, 8 and 11 as well as 7, 9 and 12 are in the same position relative to the girder dimensions. But it should be detailed that these gauges are located on opposite sides of the girder or (8 & 9) or opposite girders (11 & 12). Please refer to Figures 3.9 and 3.10 for details regarding location of the gauges.

The theory behind placing the strain gauges in the locations shown in Figures 3.9 & 3.10 was to determine the stresses in the projected path of the fatigue crack. By the properties of fracture mechanics, the largest stresses should occur at the tip of the crack and decrease as you move away from the crack mouth opening. In addition
gauges 3, 4 and 5 were oriented similar to that of a strain rosette in order to determine the principal direction of the crack.

3.1.3 Potentiometer

In addition to the instrumentation of the strain gauge, potentiometers were also placed on the bridge. These gauges allowed for measurements of the displacements occurring as vehicles traveled over the girder. These displacements can be correlated to the distance the crack mouth is opening and closing. Two potentiometer gauges, presented in Figure 3.9, were placed on beam 3 of span 5.

3.1.4 Wireless Sensing Network

The wireless sensing network employed for this research was developed by LORD Corporation MicroStrain® Sensing Systems. This product known as V-Link®-LXRS®, shown in Figure 3.11, allows for the ability to record simultaneous measurements taken from the strain gauges and wirelessly sync them to a computer to store the data through use of Node Commander® software.

Figure 3.11 V-Link®-LXRS® (Source: www.microstrain.com)
As previously discussed, each strain gauge is wired to the wireless sensing network. This network is comprised of individual nodes that wirelessly send recorded strains to data logging element. Each node has four available channels that can be used for gauge inputs. As there are 12 strain gauges employed on the bridge, three nodes were utilized for the project. The identification numbers for the three nodes used in this study are Node 30343, Node 30344 and Node 33201. Figure 3.12 depicts two of the three nodes in which gauges were wired.

![Figure 3.12 Wireless Sensing Nodes](image)

### 3.1.5 Data Logging

Node Commander® 2.10.0 Software was employed in order to record and store all strain measurements taken during the two week process of monitoring.
DelDOT Bridge. Software can be set to continuously monitoring data or to do burst sampling of a specified time interval. A sampling rate of 32 hertz was implemented for this study. From the software, nodes can be configured and real-time synchronized sampling can be performed from any distance within the range of the individual sensor. In addition to the ability to remotely record strain data, system is also accessible to pressure sensor, displacement sensors, geophones, accelerometers, temperature sensors etc.

Figure 3.13  Node Commander Sampling
Figures 3.15, 3.16 and 3.17 depict a few of the photos taken during instrumentation process of the strain gauges. Girder was initially grinded to ensure contact with strain gauges. Gauges were then spot welded along the anticipated crack propagation and wires were run to monitoring system. The final step was to apply a protective coating over strain gauges. This coating acted as a safeguard to protect the gauges from the elements in addition to sealing gauge contacts.
Figure 3.15 Images Taken During Installation of Strain Gauges - West Face of Beam 3 on Span 5
Figure 3.16  Before Installation of Monitoring Network

Figure 3.17  After Installation of Monitoring Network
Overall, the monitoring of strain experience by this structure was a four part process. As depicted in Figure 3.18 initial phase involves the input sensors (strain gauges and potentiometers). These sensors are connected to nodes which wirelessly send recorded readings to data storage device by way of the wireless sensor data aggregator. Finally data is stored on a computer or other device for operator’s convenience.

Figure 3.18  Overall Monitoring Network Process (Source: www.microstrain.com)

3.2 Numerical Data Analysis

In order to better understand data taken in the field and transfigure it into more functional information that can be used in the second phase of research involving laboratory testing, numerical analysis was performed. Process included individual cycles counting, generation of histogram plots, statistical analysis, calculations of Miner’s Number and fatigue life calculations. Software such as MATLAB, DPlot and Microsoft Excel were instrumental in the undertaking of these procedures.
It should be duly noted that the recorded strain were initially converted to stresses employing the basic mechanics of materials formula and Young’s Modulus. This conversion allows for a better overall understanding of the data in addition to enabling the use of cycle counting algorithms and fatigue life calculations. Although it is assumed that there is a perfectly linear elastic relationship between stress and strain, this is not always the case; especially in scenarios with complex connections. Nevertheless for simplicity sake, this study modestly calculated the stresses using the basic mechanical principals in order to gain a rudimentary understanding of the stresses experienced by the structure.

### 3.2.1 Cycle Counting

The number of cycles a bridge experiences inevitably determine the fatigue life of that structure. The theory of cyclical applied tensile loading is a cumulative process that can produce fatigue damage including crack initiation and the propagation of the crack. These cracks initiate as a result of a fatigue prone conditions commonly originate from structural members experiencing high frequencies of repetitive loading, especially at detail connections. Microstructure flaws and defects present at points of discontinuity, such as fusion and junction locations, create focal points for crack initiation and subsequent development. Small voids or flaws develop initially sharp crack fronts, which concentrate stress at the point of growth. Therefore, even small tensile loadings can provide a sufficient mechanism for crack propagation. While the crack front becomes blunt after the application of load, subsequent unloading acts to again sharpen the edge, providing stress concentration for future stress range application. Stress range is also considered a deterministic parameter of fatigue damage due to high residual stresses located at areas of welded fusions. Many fatigue
conditions occur in or around welded details due to the introduction of microstructure imperfections inherent with the weld process. Additionally, thermal contraction of the weld site post cooling creates substantial residual tensile and compressive stresses across the face of the member, some instances applying pressures past the yield strength of the material. Consequently, stress range, rather than overall statically applied stress magnitude, is the predominate factor in inducing fatigue damage to structural steel members.

Stress range loading analysis was performed to compare observed loading magnitudes and frequencies to AASHTO LRFD fatigue resistance specifications. Using Table 6.6.1.2.3-1 of the AASHTO LRFD Bridge Design Specification, fatigue detail category of E was determined to be associated with the connection experimentally tested. Employing this knowledge with Table 2 of the FHWA Steel Design Handbook Design for Fatigue provided the detail category constant. This table taken from handbook is shown below as Figure 3.19. The curve displayed in Figure 3.20 represents a fatigue resistance threshold for stress range. The AASHTO LRFD fatigue resistance threshold specification provides a benchmark in which to compare our experimentally collected data to determine if extrapolated cumulative fatigue damage predictions for the design life of the bridge exceed allowable levels.
The horizontal region of the graph depicted in Figure 3.20 represents that of the infinite fatigue life. Within this section, theoretically a structure can withstand a boundless number of loading cycles without development of fatigue damage. For the
examined fatigue category E structure in this study, the associated stress range is 4.5 ksi for Fatigue I limit state. The sloped region demonstrates an inverse relationship between stress range and allowed frequency, indicating a finite design state. LRFD equation 6.6.1.2.5-2 defines the design finite-life equation as follows:

\[
(\Delta F_n)^{1/3} = (A/N)
\]

Equation 1  Finite-Life Design Definition (Source: LRFD Equation 6.6.1.2.5-2)

Cycle counting is an integral part in the determination of the fatigue life of the structure. As specified in The American Society of Testing Materials (ASTM), two main counting methods can be employed for the fatigue analysis. These include simple range counting and rainflow counting. Although both are viable cycle counting methods, ultimately rainflow counting has been found to be more accurate in the identifying stress ranges observed by a fatigue detail. Both methods however have been employed for analysis of DelDOT Bridge 1678-006.

3.2.1.1  Simple Range Counting

Simple range counting and its related methodologies essentially compare successive stress reversals to determine the applied stress range. A range represented by the difference between consecutive reversals is counted as a half cycle, with its corresponding negative range accounting for the second half of the full cycle. This counting is in accordance with the stress-strain response of the material. An example taken from ASTM E1049-85 is shown in Figure 3.21. Dependent upon the deviation
of simple range counting method selected, other quite often may only count positive (valley to peak) or negative (peak to valley) ranges.

![Graph showing simple range counting method](image)

<table>
<thead>
<tr>
<th>Range (units)</th>
<th>Cycle Counts</th>
<th>Events</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>0</td>
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</tr>
<tr>
<td>8</td>
<td>1.0</td>
<td>C-D, G-H</td>
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<td>7</td>
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<td>F-G</td>
</tr>
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<td>6</td>
<td>1.0</td>
<td>D-E, H-I</td>
</tr>
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<tr>
<td>4</td>
<td>1.0</td>
<td>B-C, E-F</td>
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<tr>
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<td>0.5</td>
<td>A-B</td>
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</tbody>
</table>

Figure 3.21 Cycle Counting Using Simple Ranging Counting (Source: ASTM E1049-85)

### 3.2.1.2 Rainflow Counting

Rainflow counting is quite more complex than that of simple range counting. This algorithm integrates a more realistic and accurate approach in order to identify stress ranges observed by a fatigue detail. Rainflow counting addresses shortcomings of simple range counting which may underestimate the actual fatigue damage imparted on a structure. One of the main detriments of the simple range counting method is that stress states that do not lie adjacent cannot be compared. Therefore, small oscillatory noise, or vibrations collected by instrumentation between primary stress reversals can hinder the algorithm from recording full amplitude stress cycles. However, rainflow algorithms are able to compare non adjacent stress states by relying on strain hysteresis loops. Small oscillations between major stress reversals are counted.
through linear-elastic loading and subsequent unloading within a closed, larger hysteresis loop. An example of rainflow counting is shown below in Figure 3.22. The actual notion involving hysteresis loops is illustrated in Figure 3.23.

The image above depicts the stress-time history plotted so that the time axis is vertically downward. The lines connecting the stress peaks and valleys are imagined to be a series of pagoda roofs. Several rules are imposed on rain dripping down these roofs so that cycles and half-cycles can be counted as explained in previous section.
Rainflow counting algorithms were first introduced in 1968 by Tatsuo Endo and M. Matsuishi in 1968. This theory has since been further research and developed during the subsequent years. It should be duly noted that S.D Downing and D.F. Socie’s study published in a 1982 article by the International Journal of Fatigue has henceforth been a major contributor to what we know today as rainflow counting algorithm. These counting algorithms along with Matsuishi & Endo and amongst other cycle counting algorithms have been incorporated into ASTM E 1049-85.

3.2.2  Miner’s Number

The cycle counts allow for the application of Miner’s Rule. In order to find a cumulative damage model for a spectrum of load range magnitudes, Palmgren-Miner’s rule was applied to the observed experimental stress ranges. Palmgren-Miner Rule is
an empirical approach for calculating the damage caused by stress signals of variable amplitudes such as in the data recorded for the field. Miner’s rule is expressed as:

$$\sum_{i=1}^{k} \frac{n_i}{N_i} = C$$

Equation 2  Miner’s Number

This expression is essentially a comparison of the experimental cycles to the allowable cycles. The theory of miner’s rule is based on each observed stress range proportionally incurring damage on the total fatigue life of the member. It essentially defines failure, where $n_i/N_i$ is the fractional damage received from the $i^{th}$ source. Failure ultimately occurs when the additive proportion of the spectrum of stresses ideally equal 1 ($C = 1$). The experimental cycles refers to the previously calculated cycles using both ASTM methods and that of rainflow analysis.

### 3.2.3 Fatigue Life

Upon calculation of Miner’s number for each of the 12 individual sensors used in order to monitor DelDOT 1678-006, the corresponding fatigue life can be tabulated. Assuming a design life of 25 years, fatigue life is calculated by dividing 25 years by Miner’s number. Prior to this tabulation stress range frequencies were extrapolated out to 25 years, thus normalizing the interval time for each channel as well as providing cumulative damage analysis through a likely design life timeframe. This arithmetic to calculate fatigue life is based on the theory of Miner’s rule proposed by A. Palmgren in 1924 and further developed by M. A. Miner in 1945. The overall steps involved for
fatigue life calculations are illustrated below in Figure 3.24. This same process has been outlined for the purposes of this research.

Figure 3.24  Steps in a Fatigue Life Calculation (Ariduru, 2004)

3.3 Laboratory Testing

Laboratory fatigue testing was performed adopting approach set forth in the American Society for Testing and Materials (ASTM) Standard E647. A standard single edge-notch, steel, compact (CT) specimen, as discussed in Section A1.1 of this standard, was employed as steel specimen for all laboratory testing. Diagram of geometry of the standard (CT) specimen used for experimental work is shown below and in Figure 3.25.
Figure 3.25  Standard Compact (CT) Specimen for Fatigue Crack Growth Rate Testing (Source: ASTM E647-08)

Figure 3.26  ASTM CT Specimen with 0.5 inch Thickness

Prior to loading specimen in tension for fatigue testing a 10.5 mm (0.41 in) crack was initiated by loading samples to about 70,000 cycles. Preceding this, all specimens were pre-cracked 1 mm (0.04 in) as dictated in ASTM E647-08. This was importance of this is to provide a sharpened fatigue crack of adequate size and
straightness to each specimen. The overall purpose of the laboratory testing is to demonstrate the extension of fatigue life using a composite material as a structural patch. This research aimed at the development of a patch that was composed of a sensing layer in addition to the actual structural layer. Thus allowing for in situ monitoring of a structural as fatigue crack is rehabilitated.

Much of the actual manufacturing of the composite structural patch was performed at the Center for Composite Material (CCM) here at the University of Delaware. This extensive process included: sandblasting, application of acetone, emersion of Carbon Nanotubes (CNTs), adhesive placement, vacuum bagging, “cooking” specimen etc. As a result of the rather elaborate procedure, the university’s CCM was entrusted in order to expedite the process. Please reference Appendix E for images taken during structural patch manufacturing process.

Due to its mechanical properties and overall provided stiffness, the structural composite employed for this study was M40J uni-directional ply laminate fiber with G94 resin system manufactured by Toray Composites (America), Inc. Referencing to the technical data sheet provided by Toraca Carbon Fibers America, Inc, this material’s fiber properties include a tensile strength of 640 ksi and a tensile modulus of 54.7 Msi. Taking into account the 35% ± 2% fiber/resin volume fraction, the tensile modulus of the composite patch is about 33.5 Msi - 35 Msi. As a result, this material was selected for our intended purposes for this project. Ideally, in order to precisely transfer the applied load from the steel specimen or member to the composite patch, in order to decrease the strain, patch must possess tensile modulus equal to or greater than that of the steel itself. Considering steel has a modulus of 29 Msi, M40J would appear to be an excellent candidate for our proposed research.
Material was also selected as a result of preliminary tests using dog-bone steel specimen and previous promising research that has demonstrated, when adhered to steel, material such as this has capability of decreasing the overall strain experienced in the steel by transferring the applied load. This reduction in strain concentration inevitably will lead to improved fatigue life for the steel and bridges when implemented.

Previous experiments have been conducted here at the University of Delaware solely utilizing the proposed sensing layer. As previously discussed, the sensing layer will allow for in-situ measurements in order to monitor structures when incorporated with structure patch. The overall objective of these experiments was to establish a point of reference for the subsequent holistic rehabilitation tests, which integrated both the sensing and the structured patch. Not to mention the simple validation of the accuracy of the proposed sensing layer; which these experiments in return did verify.

Referencing back to the current study, the general layout of laboratory testing was isolated into three individual phases. The initial phase involved a baseline test of the steel specimen previously discussed. This specimen was unaltered and unchanged from its original fabrication. Upon completion of this preliminary test, new specimen was drilled with a crack-stop hole, a fatigue crack rehabilitation method commonly used by bridge owners. Specimen #2 was then cyclically loaded to in order to simulated fatigue loading. The final experiment encompassed the rehabilitation of a steel specimen employing the composite patch and sensing layer. This test aimed at the demonstration of the effectiveness of our proposed innovative, rehabilitation method employing composite materials.
As a result of the information revealed from the field instrumentation process and some simple amplitude loading tests, a load with amplitude fluctuating between 20 kips and 28 kips was chosen for fatigue testing. Load was applied at 2.5 hertz. Although it unlikely that these experiments will exactly replicate the recorded strains experienced in the field monitoring phase of the project, load induced in laboratory testing will allow for accurate strain values that encourage fatigue damage. Field monitoring displayed that load induced on bridge is quite variable. Not only is the applied load fluctuating but also the strain measurements will continue to change as the crack continues to propagate. Strains recorded during a selected week will differ from measurements taken when the crack is longer. Therefore rather than attempting to recreate these unpredictable values, the aforementioned constant amplitude loading range was employed for all fatigue experiments.

Loading was performed utilizing a force transducer manufactured by MTS. Model 661.22c-01 was employed with maximum force capacity of 55 kips. With the given loading scenario, each fatigue test took at least 20 hours to complete to failure. For each test, loading was halted after every about 20,000-25,000 cycles in order to measure the crack length. Although ultimately the placement of back-face strain gauges will allow for calculation of crack length during testing, simple measurements will ensure consistency. Thus mechanical enabling the ability to draw comparisons amongst specimen and prove the rehabilitation method is working.

Ultimately, knowledge gained from these experiments will determine the effectiveness of proposed rehabilitation method and its possible application to be employed as viable solution to fatigue cracking issues in a real world scenario.
3.3.1 Baseline Testing

Initial E647 fatigue test was conducted in order to establish a baseline relationship between crack lengths and resistance changes. For this preliminary test, no structure patch was incorporated. A pre-cracked ASTM standard single edge-notch compact (CT) specimen was tested. Strain gauges were mounted to specimen prior to loading, as shown in Figure 3.27, in order to gain basic strain data experienced by the specimen as it is fatigued.

The philosophy behind the positioning and layout of strain gauges was quite simple. Referencing Figure 3.27, gauges 1, 2, 3 and 4 were to remain consistent amongst all lab specimens. Gauge 1 is located at the neutral axis of the specimen. Theoretically by the properties of basic mechanics, all fibers on one side of the neutral axis should be in a state of tension, while those on the opposite side are in compression. Therefore gauges were placed both above and below this location in order determine the strain response of specimen relative to the neutral axis. In addition, gauge 1 was situated in response to the subsequent tests employing the drilled hole. Although its dimensional location may not harmonize with the current baseline test, this gauge is position in order to capture the strain near the edge of the crack-stop hole that will be drilled in Specimen #2. Gauges 2 and 3 were placed above the neutral axis in order to later illustrate a strain profile. This profile should theoretically be linear. Lastly gauge 4 is located on the back face of the specimen. It positioning is imperative as reading taken from back-face strain gauges have the ability to be back-calculated in order to determine the actual incremental crack length. Newman’s 2011 report further expounds upon how to go about calculating the crack length from the back-face strain gauges. The process involves the use of inputting the strains into empirically based equations that in return allow the determination of a
ratio of crack length to specimen width (Newton 2011). Although these equations can be quite extensive, they do accurately permit determination of the crack length from simple measured strain.

Furthermore referencing back to strain gauge placement, in order to accurately compare and contrast data measured from different experimental phases, gauges locations have to match up. Therefore logically, strain gauges 1, 2, 3 and 4 shown in the figure below are all unwavering with gauges 1, 2, 3 and 4 on all lab specimens.

The anomaly for this specimen was Gauge 5, which for baseline testing was located 0.50 inches from the crack mouth. This gauge assisted in pre-cracking strain measurements as well as measuring strains occurring at the crack mouth.

![Figure 3.27 E647 Fatigue Test Specimen #1 Instrumented with Strain Gauges](image_url)
Specimen was fatigue loaded to failure employing the previously described loading conditions at 2.5 hertz. This experiment allowed for a general understanding of the overall fatigue limits of the steel specimen. This threshold will be used in order to draw comparisons to subsequent test results.

Figure 3.28  Baseline Testing Experiment #1 Setup
3.3.2 Drilled Crack-Stop Hole Rehabilitated Plate

In order to simulate the scenario experienced in the field on DelDOT 1678-006, a crack-stop hole was drilled into a (CT) specimen prior to fatigue testing. This experiment essentially had dual purposes. The initial goal of this testing was to ensure that a drilled crack-stop hole will actually reduce the overall stress concentration. In addition this investigation aimed at demonstrating that the suggested minimum diameter of the hole drilled (1 inch) is sufficient enough to stop the propagation of a fatigue crack. Preliminary calculations have been computed employing formula developed by Rolfe and Barsom in 1977. This simple formula discussed in Chapter 2 and utilized in Appendix D, theoretically, determines the diameter of the crack-stop hole required to arrest crack growth by means of the nominal stress range, crack length and yield stress. Although this formula is quite simplified, it does provide initial insight into an estimated initial hole diameter that should be employed in order to arrest propagation of fatigue crack.

As briefly articulated in Chapter 2, the rehabilitation method of drilling a crack-stop hole has been shown to extent the fatigue life of damaged sections. However this fatigue improvement is based on the proper drilling of the hole. Figure 3.29, found in the Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges, illustrates how the drill bit should be positioned relative to the crack tip in order to capture the plastic zone region (Dexter 2013). If crack tip is not fully captured, the crack will continue to propagate. There are two viable methods in which to drill the hole. The first technique involves the centering of the drill bit over the crack tip. The second practice requires user to drill in such a manner that the outer diameter of the drill bit intersects with the crack tip. This method of drilling ahead of the anticipated crack ideally will ensure the entire crack is fully encompassed (Dexter 2013). This
methodology outlined in Figure 3.29 was employed for specimen tested in this study. Hole was drilled in Specimen #2 about 1.5 mm (0.0015 in) from where crack tip was estimated to be. Rationale for this was to ensure entire crack tip was encompassed in drilled hole. A drill press with a twist bit was employed in order to drill hole. As discussed by FHWA there are two viable bits that may be utilized: an annular cutter (or auger hole saw) and twist bits. However, twist bits should only be used for holes up to 25.4 mm (1.0 in) in diameter (Dexter 2013). Considering the size of the hole to be drilled was one inch, a twist bit was employed.

![Figure 3.29 Crack tip identification with red dye penetrant and proper drill bit placement (Dexter, 2013)](image)

Figure 3.29 Crack tip identification with red dye penetrant and proper drill bit placement (Dexter, 2013)
Initially it was suggested to replicate the diameter of the crack-stop hole, along with the gauge instrumentation plan employed in the field in order to best connect the field and lab testing for this project. This however was found to be an issue is several aspects of the research as lab investigations involved small scale testing. A one inch diameter drilled was selected for lab specimen as it is the smallest hole that the FHWA recommends for the purpose of fatigue cracks rehabilitation in steel bridges (Dexter 2013). The bridge monitored in the initial phase of the project utilized a two inch diameter hole. Relative to the depth of the girder two inches is a reasonable size hold diameter for the field. In addition simple calculation shown in Appendix D suggest a 28.6 mm (1.125 in.) diameter hole should theoretically be sufficient enough to cease crack propagation based on the current loading and the given crack length in the bridge member. However for the sake of small scale laboratory testing, the initial crack length is only 10.5 mm (0.41 in.) and the plate length is 254 mm (10 in.). This is in comparison to in the field which possesses a 216 mm (8.5 in) crack length and 727 mm (28.625 in) web depth. Simple calculations utilized this same equation demonstrated in Appendix D using the parameters of lab specimen would clearly validate a 1 inch hole is sufficient; especially when considering nominal stress range is only 8 ksi and crack the full crack length is less than 2 inches. Therefore it is more appropriate to employ a one inch diameter hole as opposed to the scenario depicted in the field.

In addition to the drilled hole, strain gauges were also mounted to the specimen as shown in the subsequent figure. Gauge placement, as previously discussed in Section 3.3.1, was based on maintaining consistency amongst all specimens. Therefore
strain gauges in Figure 3.29 are all located consistent with gauges 1, 2, 3, 4 on the previous and forthcoming lab specimens.

Upon completion of hole drilling and mounting of strain gauges, specimen 2 was cyclically loaded to failure as previous specimen. Experimental setup is depicted in Figure 3.31.

Figure 3.30  Fatigue Test Specimen #2 with Drilled Crack-Stop Hole and Strain Gauges
Figure 3.31 Drilled Crack-Stop Hole Experiment #2 Setup
3.3.3 Composite Patch Rehabilitated Plate

As stated earlier in this section, the composite patch material selected for application in this research was M40J manufactured by Toray Composites (America), Inc. Due to its properties this material has shown quite promising properties that enable it to be a good candidate for adherence to steel as a rehabilitation method.

The overall objective of this final experiment was to demonstrate the effectiveness of the initially proposed rehabilitation method. With the incorporation of the carbon nanotube (CNT) sensing layer, it aimed at the validation of the possibility for a holistic approach to be employed in future applications in order to address fatigue cracking issues while continuously monitoring damaged sections.

Preliminary setup for testing involved the application of a CNT infused sensing layer, followed by the adherence of the structural composite patch to the steel specimen. The manufacturing of the sensing layer first involved the dispersion and coating of CNTs onto a 34 g/m² aramid fiber. This process encompasses the complete bath impregnation of the fabric into a CNT solution. Images taken during this process are depicted in Appendix E. After CNT coating process, sheet was trimmed into two sheets measuring 203mm by 90mm for application to our steel specimen.

Composite patch was simultaneously fabricated employing the previously stated M40J material. Structural patch was comprised of a total of 22 ply. Each ply was 0.09mm in thickness. Once each ply was combined into a single patch, an autoclave was utilized in order to apply pressure and uniform temperature to the sheets. The goal was to maintain a constant adhesive thickness of 1.75mm throughout patch. Final step comprised the cutting the completed material into two individual patches measuring 190mm by 90mm by 2mm thickness. The rational of the fabrication of two CNT sensing layers and two structural patches was in order to apply this
rehabilitation patch to both sides of the cracked steel specimen in order to completely bridge fatigued specimen.

Hysol epoxy paste adhesive manufactured by Henkel was employed for application of the sensing layer in addition to the composite patch to steel. This two-part epoxy forms a strong bond and is said to be one of the toughened epoxy paste adhesive for bonding metal, wood, plastics and glass. The bonds that are formed are flexible and resist water, salt spray and most common fluids. Epoxy paste was applied to the previously depicted sandblasted ASTM plate in Figure 3.26. Sensing layer was then adhered to specimen, followed by the structure patch. In addition, similar to the aforementioned experiments, pre-cracked specimen was also mounted with strain gauges prior to experiment as shown in Figure 3.32.
Patch was initially placed in a manner which was hypothesized to most effectively transfer the load from steel to the composite in order to ultimately increase the overall fatigue life. Theoretically, in order to determine which patch orientation would be most effective, numerous experiments would need to be performed. However for the sake of time and the confinements of the research objective to simply demonstrate that this proposed rehabilitation method is viable, a single patch orientation was selected as shown in Figure 3.32. Numerous elements had to be considered in order to determine the patch location and orientation. Some of these
included the location of the neutral axis prior to addition of patch, required bond area, debonding issues, plate dimensions, electrode positioning and location of crack mouth relative to patch edge. An example of this consideration is if patch was placed too close to crack mouth, where displacements are greatest, debonding of the patch will inevitably occur. In the instance were patch is positioned too far from crack, the neutral axis of specimen can be caused to shift up. As a result of these and numerous other scenarios, patch was positioned and placed as shown in Figure 3.32. Ultimately all concerns could not be addressed without running a series of tests solely on the effect of the patch location. However considering the extent of this research, a single alignment was proposed and tested.

As in the case on experiment 1 and 2, specimen #3 was mounted and cyclically loaded to failure at 2.5 hertz. Experimental setup is shown below in Figure 3.33.
Figure 3.33  Composite Patch Specimen Experiment #3 Setup
Chapter 4
EXPERIMENTAL RESULTS & DISCUSSION

4.1 In-Service Bridge Monitoring

The following section summarizes the results from the in-service monitoring of DelDOT 1678-006. As previously discussed in Section 3.1, this bridge was instrumented with twelve strain gauges in various locations and strains readings were taken for a period of two weeks. Upon completion of this data collection, plots were generated and a fatigue life analysis was performed in order to better understand the structure and the effect that fatigue loading has played on the remaining fatigue life of the bridge.

4.1.1 Strain Readings

In order to condense the file sizes created by the recorded data, data collection was divided into individual recording sessions. Each session sampled for about 48 hours employing burst sampling settings. Using the Node Commander software, burst samples were recorded for ten minute intervals and then ceased recording for twenty minutes before beginning to sample data again. Session 1 was initiated May 5th, followed by session 2 on May 7th, session 3 began on May 9th and session 4 started on May 13th. A sampling rate of 32 Hz was implemented for this study. In addition as detailed in Section 3.1.4, each individual node was responsible for recording four of the 12 strain gauges mounted on the structure. The identification numbers for the three nodes used in this study are Node 30343, Node 30344 and Node 33201. Plots were generated for each of the strain gauges for each node recorded during each of the sampling sessions. Figures 4.1 to 4.4 illustrate a sample of the strain readings recorded
from the Channel 1 of Node 30343. Referencing Figure 3.9, this is strain gauge #4 located perpendicular to the tip of the crack tip.

Figure 4.1  Node 30343 – Channel 1 - Session 1

Figure 4.2  Node 30343 – Channel 1 - Session 2
Plots are displayed as strain (µε) vs. time (s) on the y- and x-axis, respectively. The red horizontal line symbolizes 0 microstrain (µε), though this does not depict the actual origin by which negative and positive strain are determined. It is a result of the inability to stop traffic on the bridge prior to calibrating software for strain reading. Although the origin may be close to zero, it cannot be assumed to be exactly zero.
Observation of the plots generally illustrates higher amplitude strain recorded during certain times of the day, particularly daytime travel times. This is consistent with the ideology that more traffic occurs during peak hours of the early morning commute and then repeats for the evening rush hour. As shown in Figures 4.1 to 4.4, there is increased amplitude strain ranges beginning around morning rush hour and extending through the daylight hours to the evening commute. Night hours showed the lowest amplitude in strain range throughout the sampling sessions. The increase traffic flow over the bridge during these peak travel periods increases applied live load strains on the bridge, creating high frequencies of fatigue inducing stress ranges.

As shown in the previous sample graphs of the experimental data, over time the recorded strain began to deviate from its origin; particularly for Sessions 3 and 4. Sampling sessions 1 and 2 appear to oscillate through a constant mean, of a similar magnitude. Sessions 3 and 4, however, displayed significant variation in the mean center of oscillation, at times completely entering the compressive range. The periods of divergent oscillation do not seem to correlate to time of day, reducing the probabilistic odds of correlation with increased traffic flow, or daily temperature changes due to time of day. This deviation for the mean was observed for all Nodes not just Node 30343 as shown in previous graphs. Please reference Appendix B for strain gauge plots remaining eleven gauges. There are several possible explanations for the apparent incongruities in the data, including thermal expansion and contraction causing variation in strain, or simply accuracy malfunctions in the sensors themselves. However, further evaluation should be performed to determine the root cause of wandering strain readings.
In addition to the divergence of strain readings from the mean, there were also discrepancies found in other plots. An example is shown in Figure 4.5, which is Channel 4 of Node 30343. This graph depicts a jump in sampling data between session 3 and session 4 of approximately 600 με. The data collected in session 4 of channel 4 node 343 showed extremely higher values of strain than the first three sessions. This type of irregularity in the data was the only case amongst all other plots besides channel 2 of Node 33201. As depicted in Figure 4.6, this channel also experiences a jump between 3 and 4, but this abnormality was not as extreme as in the case of Node 30343. Although other channels did experience variations in mean oscillation between and within sessions, this specific extreme event was not observed other than in these two anomalies.

Figure 4.5 Data Irregularity in Node 30343 – Channel 4
It was also witnessed for many of the plots of a repetitive, irregular oscillatory motion. Examples are illustrated in Figures 4.7 and 4.8. These two worst case instances reference strain gauge #5 (of the strain rosette on East Face of Beam 3) and gauge #12 (West Face of Beam 4 on Span 5). These deviations are believed to be a result of temperature variations throughout the day. Each strain gauge produces a temperature-dependent measurement signal. Due to the sensitivity of strain gauges, thermal expansion and contraction of the bridge girder consequently led to the drift in strain readings as shown below. The data can be corrected by measuring the temperature and employing a thermal correction curve available through the manufacturer. But for the purpose of this study, these variations were ignored. Ultimately we are just interested in determining the actual stress cycles and not the absolute values. The overall differences between maximum and minimum values do not change as a result of the slow-varying temperature.
The full data views in Figures 4.1 to 4.8 were helpful in the determination of the general trend of the strain diagrams. However, in order to draw distinct conclusions of the individual strain gauges, the plot had to be further inspected. A sample examination of a generated recorded strain graphs over a concise two second
interval is shown below. Note brackets in legend indicate the actual gauge number. Please reference Figure 3.9 in chapter 3 for specific gauge locations.

![Sample Strain Readings - East Face of Beam 3 on Span 5](image)

This example is taken from the east face of beam 3 and was recorded during sampling session 1 at 8:00 pm. It illustrates the readings of all strain gauges located on this particular face of beam 3. As expected for this time of day, gauges located below the neutral axis are in tension (gauge 1) and every gauge above it is in compression (gauges 3-7). In addition, the initial hypothesis was that the strain experienced by any individual gauge should be greatest near the crack tip and decrease moving away from crack. This premise is confirmed in Figure 4.9. Across this two second time interval, gauge 4 experienced the largest strain values. Referencing the previous strain gauge instrumentation plan displayed in Figure 3.9, this strain gauge is positioned 0.5” from
the crack-stop hole and projected crack tip. Furthermore, gauge 7 sensed the least
strain and, coincidentally recalling Figure 3.9 again, this gauge is located the furthest
from the crack tip. These observations, for the most part, carry throughout all plots
when examined over incremental intervals. Therefore it can be concluded from this
study in addition to a previous laboratory testing conducted at the University of
Delaware that compared calculating stresses of a steel beam subjected to applied
loading to values taken from the data acquisition system, that this monitoring system
is accurately capturing the actually strains experienced by this structure throughout the
day.

Figures 4.10 and 4.11 apply this same technique displaying the raw strains
experience over a period of a condensed time interval at different times of the day.
These plots however, illustrate the differences between recorded strains for gauges
positioned at the same location on opposite girders and opposite faces of the girders.
Essentially one can begin to draw conclusions about the strains experienced by the
damaged girder opposed to that of the “healthy girder.”
Figure 4.10  Sample Strain Readings Comparisons – Session 1- 7:40 am

Figure 4.11  Sample Strain Readings – Session 1- 7:40 am
Figures 4.10 and 4.11 are from session 1 at 7:40am. The time interval essentially corresponds to that of the morning rush hour. Figure 4.10 displays gauges 4, 8 and 11 centrally located 7 ¾ inches from the bottom flange. Gauges 4 and 8 are welded on opposite faces to beam 3, which has been found to have multiple fatigue cracks. Gauge 11 is located on beam 4, the healthier girder, exhibiting no visible fatigue cracking issues. At a glance is duly noted that gauges 4 and 8 exhibit much higher strains than that of gauge 11. This is to be expected as gauge 11 is on the “healthy girder.” In addition, the strain gauges positioned on Beam 3 both follow the same trend and display similar strain readings. This observation coincides with initial expectations of these gauges experiencing similar strains as they are on opposite sides of the same girder.

Referencing Figure 4.11, this diagram comprises strain gauges 7, 9 and 12, which are all located 14 ½ inches from the bottom flange. Although equivalent inclinations are displayed, these tendencies are not as dominate as in the case of Figure 4.10. As in the previous case, the “healthier girder” (gauges 12) has been shown to sense less strain. However, when comparing the gauges on opposite faces of beam 3, for this instantaneous point in time, gauge 7 and 9 do not appear to experience the same strains. At first glance, this result deviates from original expectations as they are located in the same position from the of bottom flange. But one must also considered that as these gauges are located further from the crack tip, thus the strain readings are lower. Therefore comparing the actual difference between strains of parallel gauges, the results are actually quite similar to previously discussed. In addition one must consider these are very small strains; only slightly above noise level readings.
Therefore these deviations are minuscule. Thus the initial theory for both cases is confirmed.

Overall, although there were some discrepancies in the recorded data, the monitoring system was generally able to accurately record the strains experienced by DelDOT 1678-006. From this data it was possible to infer information in reference to the current state of the structure. Recorded data allowed for a foundation in which to begin building our laboratory study.

4.1.2 Potentiometers

As discussed in Section 3.1.3, two potentiometers were employed in the field monitoring of DelDOT 1678-006. These gauges allowed for displacement measurements to be taken as the crack opened and closed due to the overhead traffic. Similar to that of the strain readings, Figure 4.12 depicts the readings taken by the gauges plotted as displacement (mm) vs. sampling ticks.
Both potentiometers were wired to Node 33201. Referencing Figure 3.9 of Chapter 3, channel 5 of this node corresponds to the gauge located near the middle of the crack. Channel 7 is located near the site of the crack initiation.

Comparing the plots of the displacement data for channels 5 and 7, channel 5 experienced larger ranges of displacements as cycles occurred. In addition this channel experienced an overall average mean displacement of 0.30 mm (0.012 in), while channel 7 saw a mean of 0.308 mm (0.012 in). In the case of both potentiometers, displacement measurements generally did not deviate from its average. These values generally correlate to the initial offset of the individual potentiometers. Therefore based on this understanding that the origins for both gauges are the same, simply examination of the ranges solely, channel 5 displayed greater amount of deviation from this origin.
Examining channel 5, the periods of larger displacements appear to be consistent with the daily traffic. The greater motions occur during morning commuting times and evening rush hour. This observation does not waiver from what was displayed from the strain readings. Referencing channel 7, it appears that the displacement measurements tend to decrease over time. It starts with a rather large displacement range of about 0.12 mm (0.005 in) occurring at the beginning of the data record. Ranges drastically drop off towards the end of the recording session. More research is required in order to further draw conclusions related to results of these potentiometer measurements. But ultimately these gauges were simply employed in order to gain a general understanding of the amount of crack mouth opening displacement (CMOD) for this in-service structure.

4.1.3 Fatigue Life Analysis

Fatigue crack development and propagation occurs as a result of repetitive cyclical loading. In order to determine the fatigue life associated with this structure, the number of cycles had to first be calculated for this specific recording data. Assuming this number remains uniform throughout the year, the number of cycles experienced per year could ultimately be estimated. Interpolating this value out to the actual design life of the structure would thus allow for comparison of the remaining fatigue life. As previously mentioned in Section 3.2, the numerical analysis employed for this research incorporated two of the cycle counting strategies presented by ASTM E1049-85 in order to achieve diversity and redundancy in cumulative fatigue damage conclusions. Data collected from 12 different sensors monitoring the bridge were analyzed using the rainflow counting algorithm, simple range counting (full cycle counting), and simple range counting (half cycle counting) methods. The
corresponding Miner’s number for each channel was also calculated to find the fatigue life of the bridge. For this study, a design life of 25 years was assumed. Consequently stress range frequencies were extrapolated out to 25 years, thus normalizing the interval time for each channel as well as providing cumulative damage analysis through a likely design life timeframe. In order to organize finding, histogram plots and data tables were generated for all strain sensors employed in this study. It should also be noted that bin sizes selected for production of histogram plots were determined employing Scott’s Rule (Scott 1979).

In order to provide a visual representation of the stress ranges experienced by a particular gauge, a sample histogram depicting the stress ranges and the frequencies at which they occur for Channel 1 of Node 30343 is displayed in Figure 4.13. This plot shows the results of each counting method used; rainflow counting, simple range counting (full cycle counting), and simple range counting (half cycle counting) methods extrapolated out to 25 years. Only stress ranges greater or equal to 4.5 ksi were examined, as previously stated in Section 3.2.1, for the connection in question 4.5 ksi is the threshold corresponding to infinite fatigue life. Any value below this ultimately will not contribute to the development of fatigue damage.
The rainflow counting and full cycle strategies defined in the ASTM standards weighed more consideration in this evaluation, with the half cycle analysis included as a highly conservative reference. As presented in Figure 4.13, half cycle counting does not truly represent fatigue damage. This is due to a lack of achieving a full stress cycle when counting. As far as the other two methods, rainflow and full cycle stress range counting in Figure 4.13 display a relatively strong correlation. As anticipated, the rainflow counting method displays a slightly higher damage model prediction than the full cycle simple counting method. However this is only the case when considering stress ranges greater than 4.5ksi. Although not depicted on the graph, for ranges less than 4.5 ksi, full cycle counting method obtained generally higher frequencies than that of rainflow counting method. Oscillatory noise and low level stress applications will result in the simple counting method recording higher relative low level stress
frequencies, while underestimating larger stress range applications. The general overall exponentially decaying trend is consistent amongst all counting methods.

Examination of histogram plots more thoroughly reveal that the majority of cycles occurring at stress levels less than 4.5 ksi. While for the case of channel 1 of Node 30343, Figure 4.14 declares the overall maximum to be 25.29 ksi. This plot is of the actual undisturbed raw data for the all recording sessions. Although the maximum stress can be considered to be quite large, one must consider the fact that this value occurs possibly once or twice. When interpolated out to 25 years the number of these cycles compared to that of the smaller stresses can be rendered insignificant. The more substantial risk to fatigue crack propagation is due to the smaller repetitive loads as opposed to a few heavy loads. Referencing Figure 4.14 once again, although cycles are exponentially decreasing, it should be noted that there are still thousands of cycles greater than 4.5 ksi. This may be detrimental to the structure considering that this data was recorded for less than 200 hours. Please reference appendix C for remaining histogram plot for all strain gauges.
As previously discussed in this chapter, in order to provide numerical results, each channel (strain sensor) employed on DelDOT 1678-006 was analyzed using rainflow counting, simple range counting (full-cycles) and simple range counting (half-cycles). In addition the maximum stress range in ksi was tabulated. Using Miner’s equation as discussed in Section 3.2.2, Miner’s number and the corresponding anticipated fatigue life was calculated for each channel. As a fatigue design life of 25 years was assumed, all stress ranges were extrapolated out to 25 years. These formulated values have been organized and are shown in the subsequent Table 4.1.

Figure 4.14  Raw Data Histogram Plot
Table 4.1  Data Analysis Results

<table>
<thead>
<tr>
<th>Gauge Number</th>
<th>Node Channel</th>
<th>RAINFLOW COUNTING</th>
<th>SIMPLE RANGE COUNTING (Full Cycles)</th>
<th>SIMPLE RANGE COUNTING (Half Cycles)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Max Stress Range (ksi)</td>
<td>Miner’s Number</td>
<td>Fatigue Life (Years)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10.194</td>
<td>0.085</td>
<td>293.772</td>
</tr>
<tr>
<td>(4)</td>
<td>Ch1</td>
<td>27.125</td>
<td>26.443</td>
<td>0.945</td>
</tr>
<tr>
<td>(5)</td>
<td>Ch2</td>
<td>4.126</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>(7)</td>
<td>Ch3</td>
<td>2.758</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>(7)</td>
<td>Ch4</td>
<td>10.194</td>
<td>0.085</td>
<td>293.772</td>
</tr>
<tr>
<td></td>
<td></td>
<td>25.102</td>
<td>22.670</td>
<td>1.103</td>
</tr>
<tr>
<td>(8)</td>
<td>Ch1</td>
<td>5.498</td>
<td>0</td>
<td>29002.657</td>
</tr>
<tr>
<td>(9)</td>
<td>Ch2</td>
<td>18.805</td>
<td>6.681</td>
<td>36.732</td>
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<tr>
<td>(10)</td>
<td>Ch3</td>
<td>6.309</td>
<td>0.019</td>
<td>1315.789</td>
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<tr>
<td>(11)</td>
<td>Ch4</td>
<td>2.694</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

As shown in the table, in cases where the stress ranges are less than 4.5 ksi, the fatigue life is infinite in accordance with AASHTO LRFD Bridge Design Specifications (AASHTO 2010). In addition, Miner’s number is essentially equivalent to zero for these gauges as the comparison of experimental stresses to the allowable stresses extrapolated over 25 year is significantly low.

The results are consistent with initial hypothesis that the stress decreases as you move away from the fatigue crack mouth. The highest of these values occurring directly at the tip of the crack. In addition, the calculated fatigue life is lowest that this location. This is ultimately the worst case scenario. As presented in Table 4.1, the value associated with the lowest fatigue life for this section employing rainflow counting method is 0.945 years. Even employing the other counting methods the fatigue life is still relatively low. As a crack has already been discovered in beam 3, it is logical for fatigue life values to be significantly low for this structure, especially for gauge 4 located near the crack tip.

Comparing values obtained from strain gauges located in the same location on different girders/faces, the fatigue life values are about the same. Referencing
instrumentation plans in Figures 3.9 & 3.10, strain gauges 7, 9 and 12 are all located 14 \( \frac{1}{2} \) inches from the bottom flange. Therefore theoretically one would expect for these locations to have the same fatigue life. As denoted in Table 4.1, this philosophy holds true as the tabulated fatigue life is \( \infty \). In addition maximum stresses ranges only deviate 0.24 from their mean. Similarly gauges 4, 8 and 11 are all located 7 \( \frac{3}{4} \) inches from the bottom flange. Gauges 4 & 8, which are located near the crack tip, values parallel each other; while gauge 11’s values are a bit lower. This is attributed to gauge 11’s location on the “healthy girder” while the remaining two are positioned near the tip of the crack. This comparison illustrates the differences the stress ranges experienced and ultimately the fatigue life of a “healthy girder” vs a “damaged girder.”

Overall, cumulative fatigue analysis through Miner’s number showed varying fatigue life and stress range distributions throughout the channels. However, observation shows a direct correlation between location of the sensor in terms of distance from the crack mouth and ensuing fatigue life. A parallel can be drawn, as increased distance from the crack tip yielded reduced fatigue damage. The highest stress range (and therefore lowest fatigue life) occurred, as predicted and previously discussed, at strain gauge 4. This gauge was the closest to the crack mouth. Additional comparing the beam 3 (girder with fatigue crack) to beam 4 (“healthy girder” without fatigue damage) early predictions held true. For each gauge placed on beam 4, all results illustrated an infinite fatigue life by failing to incur maximum stress ranges of more than the 4.5 ksi threshold. Results demonstrate that is girder is in fact a “healthy girder,” with no reasonably suspicion of development of any fatigue related damage in the near future. Similarly result of beam 3 can be perceived as unsurprising. As
formerly stated, it is reasonable expectation for such low tabulated fatigue life values for this bridge as a fatigue crack has initiated and continues to propagate, especially considering the large stress concentration now present at the tip of the crack.

Therefore, in spite of relatively low fatigue life and the high max stress values, results are consistent with the overall philosophy of fatigue cracking and related stresses.

In addition to analyzing the cumulative damage experience by each channel, each session was also analyzed individually as a result of discrepancies discussed in Section 4.1.1. Ideally, the cumulative fatigue calculations for the entire recorded dataset should parallel that of the individually analyzed sessions. Results of this investigation are presented in Table 4.2.

Table 4.2  Data Analysis Results by Individual Sampling Session

<table>
<thead>
<tr>
<th>Sampling Session</th>
<th>Max Stress Range (ksi)</th>
<th>Miner’s Number</th>
<th>Fatigue Life (Years)</th>
<th>Max Stress Range (ksi)</th>
<th>Miner’s Number</th>
<th>Fatigue Life (Years)</th>
<th>Max Stress Range (ksi)</th>
<th>Miner’s Number</th>
<th>Fatigue Life (Years)</th>
<th>Max Stress Range (ksi)</th>
<th>Miner’s Number</th>
<th>Fatigue Life (Years)</th>
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<tr>
<td></td>
<td>Ch1</td>
<td>Ch2</td>
<td>Ch3</td>
<td>Ch4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td>1.907</td>
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<td>35.314</td>
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<td>1.740</td>
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<td>8.455</td>
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<tr>
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<tr>
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<td>AVG</td>
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<td>1.957</td>
<td>12.346</td>
<td>0.619</td>
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<td>0.016</td>
<td>6208.979</td>
<td>1.753</td>
<td>0</td>
<td>--</td>
</tr>
</tbody>
</table>
Upon computation of the average amongst each individual sampling session, results signify that Miner’s numbers are actually quite comparable to that of the compounded data set containing all four sessions. However when further analyzing each session’s tabulated values some inconsistencies do arise. As data was generally recorded over the similar time period, it is hypothesized that values between sessions should remain relatively constant. The most notable change in fatigue life occurs during session 3 for the majority of the channels, exhibiting a substantially higher fatigue life than other data sessions. The variation may however be attributed its diverged sampling time frame. Unlike the others, session 3 was set to record from Friday afternoon until early Sunday morning; times when truck traffic volumes are expected to be substantially lower than during working hours. Also, of the data collected during session 3, \(~58\%~\) was attained between the hours of 7:00 p.m. and 7:00 a.m., resulting in an unrepresentative, and underestimated, sampling of stress conditions for extrapolation. The remaining night collection proportions for session 1, 2 and 4 are \(~52\%~, \sim62\%~, \sim41\%~\) respectively. These slight disproportionalities in sampling conditions are exacerbated when multiplied over a 25 year period. Collection timing as well as day of the week plays a substantial role in determining fatigue life over a large extrapolated period, lending to a possible inadequacy experienced.

Recalling the large incongruities shown in Figures 4.5 and 4.6, session were additionally re-analyzed employing only sampling sessions 1-3 in order to provided diversity in calculation comparisons in which to draw conclusions. Referencing these diagrams, there was a large spike in the strain data that was experience between session 3 and session 4. Consequently, in order to determine the magnitude this
session played in the overall data, it was detached from this particular analysis. Results of the re-evaluated data are displayed below in Table 4.3.

Table 4.3  Data Analysis Results of Sessions 1-3

<table>
<thead>
<tr>
<th>Gauge Number</th>
<th>Node Channel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rainflow Counting</td>
<td>Simple Range Counting (Full Cycles)</td>
</tr>
<tr>
<td>Max Stress Range (kst)</td>
<td>Miner's Number</td>
</tr>
<tr>
<td>4</td>
<td>Ch1</td>
</tr>
<tr>
<td>6</td>
<td>Ch2</td>
</tr>
<tr>
<td>7</td>
<td>Ch3</td>
</tr>
<tr>
<td>3</td>
<td>Ch4</td>
</tr>
</tbody>
</table>

Comparing Table 4.3 to the original Table 4.1, only slight variations in the projected fatigue damage are revealed. Due to the aforementioned variations in actual sampling times, session 4 generally exhibited lower rates of fatigue damage. The exclusion of session 4 resulted in an increase in average projected cumulative fatigue damage in comparison to Table 4.1. Discarding data from session 4 lends to a more accurate data set, as unexplained irregularities in the data created severe discontinuities as well as divergence from previously observed results. However, discarding session 4 may also lead to skewed damage predictions considerably, as the weight of weekend collection times will become disproportionally high. Weekend traffic conditions will be represented by one out of three sessions, compared to an actual two out of seven day distribution. Therefore it can be concluded that Table 4.1
actually presents the most accurate and proportionally precise results to the analysis of the in-service monitoring of DelDOT 1678-006. Nevertheless, considering such a small segment of sampling data was extrapolated out to a 25 year period, there is a significant amount of uncertainties as well as possible inaccuracies. Greater periods of sampling are required in order to accurately estimate remaining fatigue life of this member. However considering the results do indicate an estimated remaining fatigue life of less than a year there is a significant rise for concern.

4.2 Laboratory Testing

The subsequent sections present and discuss results obtained from the laboratory experimentation of the ASTM E647 compact steel specimen as presented in section 3.3. As discussed previously discussed within the section, three different tests were run: a baseline plate, a rehabilitated plate with drilled crack-stop hole, and a plate rehabilitated with a composite patch. Referencing procedures discussed in section 3.3 all specimens were pre-crack and cyclically loaded at 2.5 Hz to failure. Upon completion of testing, comparison each of individual’s crack growth propagation relative to the number of cycles completed can be examined in order to determine the overall effectiveness of the proposed rehabilitation method at prolonging the fatigue life of the damaged specimen.

4.2.1 Baseline Testing

The initial specimen tested was a standard, unaltered ASTM E647 compact steel specimen as presented in section 3.3. Ultimately it was discovered that utilizing the previously discussed fatigue loading, the steel specimen employed in baseline testing took a total of about 176,000 cycles before failure occurred. An image taken of
the specimen after failure is shown in Figure 4.15. As this was our baseline specimen, this value will serve as a point of reference for all remaining fatigue tests. In order to demonstrate the prolonging of fatigue life all succeeded tests should surpass this value.

![Figure 4.15 Specimen #1 after 176,000 load cycles (failure)](image)

The propagation of the crack relative to the number of cycles is revealed in Figure 4.16. This plot illustrates the length of the crack utilizing the back-face strain measurements as well as using the beach marks of the specimen. By examining the cross-section of the failed specimen along the fatigue crack, small discontinuities known as beach mark can be discovered. Beach marks occurred as a result of ceased loading for a period of time. A good example of specimen beach marks will be further discussed in the succeeding section. Using these beach marks, the crack length at a distinct number of fatigue cycles can be determined. Further analysis of Figure 4.16
divulges that these beach marks correlate quite well to that of the back-face strain measurements, reaffirming its use as a means of measuring crack length. It should be duly noted as discussed in section 3.3.1 crack length was calculated using the back-face strain gauges is determined utilizing the procedure set forth by Newman in 2011 (Newman 2011). Referencing Figure 4.16 once again, it should also be noted that in that green line indicates fatigue crack measurements calculated utilizing this method. Line is displayed up until point in which gauge went off-scale from data acquisition system.

Figure 4.16  Propagation of Crack Length (Specimen #1)
As stated in Section 3.3, for each test loading was halted after every about 20,000 cycles in order to measure the crack length. This mechanical means of measurement was a simple method to reaffirming the readings taken. Figures 4.17 and 4.18 illustrate some photos taken during one of these periods, more specifically after the specimen had completed 130,000 cycles. From these images, particularly Figure 4.18, it can be seen that the crack does not actually grow completely straight.

Figure 4.17  Specimen 1 after 130,000 Cycles (view of front side, instrumented)
Examining the data a little more closely, a profile of the strains experienced throughout the specimen can be developed. Figure 4.19 illustrates sample strain profiles taken during loading of specimen. These plots exemplify the relationship between strain readings while moving away from the crack tip. Strain gauges placed on the face of specimen were examined to order to generate a strain profile; the back-face gauge was not included. Theoretically, by mechanical principals considering the stress concentration at the crack tip, as the distance from the crack mouth increase, the strain should exponentially decrease as it approaches the neutral axis then linearly reduce after that point. These plots generally follow that trend.
Figure 4.19  Sample Instantaneous Strain Profile after about 40,000 Cycles
The overall trend of the strain data is shown in Figure 4.20. This plot is only depicts about 10,000 cycles but gives a good overall representation of the dataset. It should be noted that the readings for strain gauge #5 are not depicted in any plots as this gauge was located 12.7 mm (0.5 in) from the notch tip. Consequently, shortly after crack initiation of 10.5 mm (0.41 in) and the commencement the fatigue loading, the crack propagated though this gauge. Therefore only a minimal amount of strain readings were able to be captured after pre-cracking of the specimen.

Considering gauge #1, located 61 mm (2.40 in) from the notch tip where crack is originating, as specimen is continually loaded and the crack continues to propagate
towards this gauge, strain readings persistently increase. Although not shown in the Figure 4.20, gauge #5 followed this similar pattern. Strain readings exponentially increased until they reached a point in which they exceeded the system’s allowable readable range. Moving further away to gauge #2, strain readings generally do not deviate too much, which is to be expected. Recalling from section 3, it should be noted that gauge #4 is the back-face strain gauge and therefore doesn’t significantly correlate with any of the other gauges and is not a part of the strain profile.

Figure 4.21 presents the same data as the previous Figure 4.20 on a different time scale. This plot displays a few seconds of data, thus the actual strain readings that are experienced for an individual cycle can be determined. As displayed, all gauge readings consistently follow a sinusoidal pattern which is consistent with the pre-set fatigue loading configuration. For this incremental portion in time, gauge #1 is experiencing strains ranging between about 20 μƐ and -15 μƐ; gauge #2 between about -190 μƐ and -260 μƐ; gauge #3 between about -330 μƐ and -445 μƐ; gauge #4 between about -700 μƐ and -950 μƐ. Gauge #5 is once again not depicted as values near the crack tip exceeded the allowable limits.
4.2.2 Drilled Crack-Stop Hole Rehabilitated Plate

Specimen #2 was a steel plate that utilized the crack-stop hole drilling rehabilitation method once fatigue crack had been initialized. This specimen reached a total of about 195,000 fatigue cycles before failure occurred. The failed specimen is shown in Figure 4.22.
It is apparent that this value exceeds that of the previous specimen. Specimen #1 ultimately failed after a total of 176,000 cycles. Subtracting out the cycles to initiate crack and pre-crack length, as in Figure 4.23, this specimen was subject to about 101,000 fatigue cycles before failure occurred. Correspondingly, Specimen 2 experienced 120,000 fatigue cycles prior it failing. A 20% increase in the allowable fatigue cycles was demonstrated through this experiment. Therefore it can be confirmed that the act of drilling a crack-stop hole will effectively increase the tolerable number of fatigue cycles that a specimen can take before failure occurs.
As for Specimen 1, strain profiles were plotted at a random instantaneous point in time during the fatigue testing. Figure 4.24 illustrate these graphs for both ends of the loading spectrum; about 20 kips and 28 kips. From examination of these figures it can be established that the neutral axis has shifted up, when compared to Specimen 1, as a result of drilling a crack-stop hole, which was to be expected. This radial distance of section loss is essentially equivalent to distance neutral axis has shifted, about 25.4 mm (1 in). In addition, it appears that each strain profile follows a seemingly exponentially decreasing manner as distance from the notch tip is increased.

Figure 4.23  Propagation of Crack Length (Specimen #1 & #2)
Figure 4.24  Sample Instantaneous Strain Profile after about 125,000 Cycles
Similar to Specimen #1, it is clear to see from the above image taken after about 160,000 fatigue cycles that the crack is not growing straight. This is most likely attributed to the irregularity in the manufacturing of the sharp notch of the steel plate.

Section 4.2.1 briefly discussed the “beach marks” which were used as one of the methods by which to reaffirm the length of the crack after a certain number of fatigue cycles. An example of such marks is shown in Figure 4.26. These beach marks correlated to fatigue loading occurring prior to the drilling of the crack-stop hole; during the crack initiation phase. Initially the testing machine was set to load the specimen 40,000 cycles to the initial crack. In order to reach a total initial crack length of 10.5 mm (0.41 inch) as indicated in the ASTM E647 Specifications, an additional
35,000 cycles were applied. This was broken up into intervals of 10,000 cycles for Specimen #2. An additional 4,000-5,000 cycles were run as a result of machine errors, thus five beach marks are shown in this figure rather than four. Not only do these marks allow for confirmation of loading but additionally crack length in relation to number of cycles can also be examined due to these markings. Although considerable confidence should not be placed in these measurements, inevitably they can be used and compared against the back face strain gauge measurements in addition to simple mechanical measurements with a scale, in order to further reaffirm crack length propagation during fatigue testing.
4.2.3 Composite Patch Rehabilitated Plate

Specimen #3 was pre-cracked and rehabilitated using the proposed composite patch. This patch composed of M40J fibers was placed on the specimen after the addition of a Carbon nanotube sensing layer as introduced in Section 3.3. Our research aims at addressing the current issue of effectively rehabilitating fatigue cracks found on steel structures through means of this proposed holistic approach.
As in the case with all previous experiments, Specimen #3 was cyclically loaded at 2.5 Hz with an applied load fluctuation between 20 and 28 kips. Upon completion of fatigue testing, it was determined that Specimen #3 finally failed after a total of about 315,000 cycles. The failed specimen is shown below in Figure 4.27.

![Image of Specimen #3 after failure](image)

**Figure 4.27** Specimen #3 after 315,000 cycles (failure)

It is clear to see from the previously depicted image that the composite patch itself did not fail, but rather the adhesion between the steel and composite material was lost. This phenomenon known as debonding was introduced in Section 2.3.2. Once debonding occurred within this experiment, the patch essentially was deemed futile. With the loss of adhesion, the load cannot effectively be transferred from the steel to the patch anymore. The full loading is then placed back onto the further damaged steel
section causing crack propagation to reinitiate towards failure. This emphasizes the significance of the bond for effective rehabilitation of fatigue cracks.

Figure 4.28 as well as Figure 4.29 depicts data recorded as a result of the incorporation carbon nanotube (CNT) infused sensing layer of the patch. These graphs simply demonstrate the real-time monitoring capability the CNT sensing layer possesses. The utilization of this layer allowed for the continual monitoring and ability to measure the crack length although the crack is actually now covered by a patch. These results of crack length determined from CNT sensing layer remain consistent with simple crack length measurements taken throughout the experiment. This capability of the layer addresses many concerns individuals may have in regards to the ability to continue to monitor crack once it is covered by the composite rehabilitation patch.

![Crack Lengths vs. Resistance Comparison](image)

Figure 4.28  Crack Lengths vs. Resistance Comparison
Results of our investigation in relation to crack propagation lengths for all specimens tested are presented in Figure 4.30. It depicts the crack growth with respect to the number of fatigue cycles. Much attention should be drawn towards the compositely rehabilitated plate employed in experiment #3. As shown in this figure, comparing baseline testing (Specimen #1) and the specimen rehabilitated with the composite patch (Specimen #3), there is a considerably noticeable difference in the crack length at any given number of cycles. This difference inevitable can be linked to the increased fatigue life as a result of the fatigue crack repair method implemented. It is apparent from the graph that crack growth has been impeded as a result of the composite material. Numerically speaking, a nearly 135% increase in the allowable number of cycles before failure occurred was demonstrated by this experiment in comparison to that of the baseline specimen. Neglecting the 75,000 cycles required for crack initiation and pre-cracking, Specimen #1 was subjected to an additional 101,000
cycles before failure occurred while specimen #3 experience 239,000 cycles; a total difference of 138,000 fatigue cycles. Looking at Specimen #2 vs. Specimen #3, a near 100% increase was shown.

Figure 4.30  Comparison of all Specimen Results with Respect to Crack Length

Employing the composite patch also allowed for the crack to propagate beyond length in which it typically failed. For Specimens #1 and #2, failure ultimately occurred once a critical crack length of about 3.40 in (87 mm) was reached. From the aforementioned figure it is shown and can be concluded that the structurally reinforced (utilizing composite patch) specimen allows for the crack to develop beyond the critical crack length (87 mm) that previous specimen failed. Specimen #3 did not fracture until crack length extended to about 97 mm. Culminated results indicate that
the application of such a rehabilitation method, will in fact prolong the fatigue life of damaged specimen.
Chapter 5

CONCLUSIONS & RECOMMENDATIONS

5.1 Conclusions

From the results of the initial field instrumentation and subsequent fatigue analysis, it can be concluded that the girder to abutment transition present in DelDOT Bridge 1678-006 possesses a significant fatigue prone detail that will continue to see considerable fatigue damage in the future. Stress range analysis shows substantial cumulative fatigue damage during short interval collection time frames that surmount to severe damage when extrapolated to 25 year frequencies.

Although by the simple calculations presented in Appendix D, a 1.154 inches (29.312 mm) in diameter should be sufficient to cease crack propagation, it is my belief that due to the complex loading experienced at the location in question, the crack will eventually continue to grow. This is drawn from the large displacements in addition to the strain readings taken adjacent to the edge of the drilled hole. In order to improve and guarantee the rehabilitation of this member, stresses ultimately need to be decreased. Whether this is done employing one of the methods discussed in Chapter 2 or by our proposed holistic strategy, stresses adjacent to crack ultimately need to be diminished.

This research aimed at addressing the issue currently present in a number of DelDOT bridges. Quite often when a fatigue crack is discovered, the immediate response is to drill a crack-stop hole. As discussed in results in Chapter 4, a drilled hole does in fact contribute to the prolonging of fatigue life. In our laboratory experiments, a controlled environment with a constant load amplitude, a 20% increase in the allowable fatigue cycles before complete section failure occurs was shown
employing this commonly used method of repair. Therefore it may be assumed that this as a viable method for the rehabilitation of fatigue cracks discovered in steel bridges that will have some benefit.

However, as denoted in Section 4.2.3, a significant increase in fatigue life was revealed as a result of our proposed rehabilitation method. By employing this holistic approach, the allowable fatigue cycles experience prior to complete failure, neglecting cycles necessary for crack initiation, was increased by 135%. Measurements taken from the sensing layer additionally demonstrated a drastic decrease in overall stresses experienced in the steel specimen. It follows that this improvement in the fatigue life of the damaged specimen is due to properties of the composite material. The chosen composite, M40J possesses a modulus of elasticity of 230 GPa, according to the manufacturer, making it a high strength and relatively high modulus material. The particular sample employed for research was found to have a modulus of closer to 203 GPa by means of ASTM D-3039. However A36 Steel retains a tensile modulus of 200 GPa. As a result of the composite material having a modulus greater than that of the steel, stiffness is adequate and the load can successfully be transferred from the steel to the patch, thus alleviating a portion of the load contributing to the fatigue propagation. Hence it can be concluded that the proposed holistic rehabilitation method employing a composite patch will inevitably extend the fatigue life of steel bridges plagued with fatigue crack issues. In addition, the sensing capabilities of the integrated CNT sensing layer allows for tracking the crack long-term and in real-time.

5.2 Future Work

More research is required in the area of the debonding that occurs between the composite and the material it is adhered to. Specimen #3 failed as a result of...
debonding. More understanding is needed as to when this issue occurs and how it affects load transfer capacities and capabilities. Once debonding occurs, the patch essentially becomes useless as it can no longer transfer the necessary loads. The ability to transfer the loads from the steel member to the patch is a crucial element in order to decrease the stresses at the crack and inevitably prolong the fatigue life. Therefore much effort must be placed in addressing this issue. This is particularly important as these systems will be operating under field conditions, potentially exposed to severe temperature and humidity conditions. As a result, environmental testing to ensure the overall system, i.e. composite and sensing patch as well as the bond line are durable.

Generally speaking, one way to avert debonding is in the consideration of the actual patch placement. Location, size and orientation of the patch relative to the crack mouth opening can significantly affect where and when deboning occurs. Therefore, the “most effective” composite patch location must also be taken into account. As discussed in Section 3.3.3, there are many patch configurations that may be selected compared to what was presented in this research. Obviously, placing patch too close to crack mouth will allow debonding to immediately occur. However, placing it too far from crack mouth, relative to the specimens’ section dimensions, could cause a neutral axis to shift. One thought discussed during this research process was the possibility of placing structural patches as strips in a series next to each other. Then, as one fails, others will remain intact until failure. Nevertheless, a greater amount of exploration is needed in this area in order to combat debonding issues that may occur.

In addition, it would be noteworthy to consider the use of an even higher strength and modulus composite fiber, such as M60J. For this study, the actual amount
of fatigue life that is prolonged employing this solution was not investigated. Although our holistic solution has been determined to increase the overall fatigue life, no examination into the magnitude of this increase related to time has been prompted. The use of an even higher strength and higher modulus fiber is an area of interest that could demonstrate an even more significant improvement. Not only would this be proven to be more effective at fatigue crack retardation but it could also possibly be employed as a permanent solution to fatigue cracking that may in fact be considered equivalent to the restoration of the damaged section to its original design capacities. Nevertheless, the University of Delaware will continue research in the field and explore different fiber options in order to prolong fatigue life.

Future research will also explore the incorporation of the patch into a real world scenario, i.e. an in-service bridge, in order to evaluate the composite patch’s ability to work under more complex stress scenarios. Our laboratory experiments were simplified in that they only produced pure tensile stresses in the vicinity of the crack tip with no contribution from shear. Reference the Delaware Bridge instrumented, from the displacement and strain readings it is evident that there are other contributing factors to the readings. It is my belief that this structure was not experiencing pure tension. By the theory of basic static forces, it can be proven that there is shear experienced as the fatigue crack developed at the abutment location. Therefore much consideration must be taken in order to further validate that this proposed solution can not only improve the fatigue life of a simple laboratory specimen by also prove viable to withstand complex stress conditions experienced in an actual real-world scenario.

Furthermore, the application of our proposed solution under field conditions must be addressed. Our study was conducted in a controlled environment with all
technology necessary to accurately develop, mount and apply this rehabilitation patch. Much of that is not available in the field. Not to mention, in some cases, the lack of required bond area in the field needed in order for the patch to properly be mounted. Consequently more research in necessary in order to begin to transition our proposed method into the field. DelDOT Bridge 1678-006 will continue to be an excellent candidate for this future exploration.

Inevitably our goal was to simply demonstrate the capabilities of this holistic rehabilitation solution under controlled conditions in the laboratory. This objective and theory regarding this innovative proposed rehabilitation method’s ability to retard fatigue crack growth was validated through this research. With the current data and more research, the hope is to eventually propose this solution for incorporation into the civil engineering industries. Fatigue cracking is a persisting problem and without research addressing such issues, the problem will endure to propagate. Thus it is the hope that our exploration will significantly contribute to how this issue is confronted in the future, in addition to further stimulating the philosophy and thinking behind how we can most effectively address fatigue cracking issues in steel bridges.
REFERENCES


43. Torayca M40J Data Sheet, Technical Data Sheet No. CFA-014, http://www.toraycfa.com/pdfs/M40JDataSheet.pdf
44. Torayca M60J Data Sheet, Technical Data Sheet No. CFA-018, http://www.toraycfa.com/pdfs/M60JDataSheet.pdf


Appendix A
DelDOT BRIDGE INSTRUMENTATION

Figure A.1  Strain Gauge Instrumentation Plan – East Face of Beam 3 on Span 5
Figure A.2  Strain Gauge Instrumentation Plan – West Face of Beam 3 on Span 5
Figure A.3  Strain Gauge Instrumentation Plan – West Face of Beam 4 on Span 5
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Figure A.13 Approach Roadway
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Figure B.3  Raw Strain Recordings – Node 30343 – Channel 3 – (Gauge 7)
Figure B.4  Raw Strain Recordings – Node 30343 – Channel 4 – (Gauge 5)
Figure B.5  Raw Strain Recordings – Node 30343 – All Channels
Figure B.6  Raw Strain Recordings – Node 30344 – Channel 1 – (Gauge 8)
Figure B.7  Raw Strain Recordings – Node 30344 – Channel 2 – (Gauge 10)
Figure B.8  Raw Strain Recordings – Node 30344 – Channel 3 – (Gauge 11)
Figure B.9  Raw Strain Recordings – Node 30344 – Channel 4 – (Gauge 12)
Figure B.10 Raw Strain Recordings – Node 30344 – All Channels
Figure B.11 Raw Strain Recordings – Node 33201 – Channel 1 – (Gauge 3)
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Figure B.14 Raw Strain Recordings – Node 33201 – Channel 4 – (Gauge 9)
Figure B.15 Raw Strain Recordings – Node 33201 – All Channels
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Figure B.17 Sample Strain Readings Comparisons – Session 1 - 8:00 pm
Figure B.18 Sample Strain Readings Comparisons – Session 1 - 8:00 pm

Figure B.19 Sample Strain Readings Comparison – Session 2 - 1:00 am
Figure B.20  Sample Strain Readings Comparisons – Session 2- 1:00 am

Figure B.21  Sample Strain Readings Comparison – Session 2- 7:40 am
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Appendix C
FIELD INSTRUMENTATION HISTOGRAM PLOTS

Figure C.1  Strain Recordings Histogram – Node 30343 – Channel 1 – (Gauge 4)

Figure C.2  Strain Recordings Histogram – Node 30343 – Channel 2 – (Gauge 6)
Figure C.3  Strain Recordings Histogram – Node 30343 – Channel 3 – (Gauge 7)

Figure C.4  Strain Recordings Histogram – Node 30343 – Channel 4 – (Gauge 5)
Figure C.5  Strain Recordings Histogram – Node 30344 – Channel 1 – (Gauge 8)

Figure C.6  Strain Recordings Histogram – Node 30344 – Channel 2 – (Gauge 10)
Figure C.7  Strain Recordings Histogram – Node 30344 – Channel 3 – (Gauge 11)

Figure C.8  Strain Recordings Histogram – Node 30344 – Channel 4 – (Gauge 12)
Figure C.9  Strain Recordings Histogram – Node 33201 – Channel 1 – (Gauge 3)

Figure C.10  Strain Recordings Histogram – Node 33201 – Channel 2 – (Gauge 2)
Figure C.11 Strain Recordings Histogram – Node 33201 – Channel 3 – (Gauge 1)

Figure C.12 Strain Recordings Histogram – Node 33201 – Channel 4 – (Gauge 9)
Appendix D

SAMPLE CALCULATIONS

As a result of crack re-initiation on DelDOT 1678-00, multiple crack stop holes have been drilled. As discussed in literature review undersized drilled hole may cause in the continued propagation of fatigue crack. The overall purpose of drilling the hole is to remove the stress concentration permitting the crack to grow. Theoretically, the diameter of the hole required to arrest crack growth can be calculated from a formula developed by Rolfe and Barsom (1977), and later redeveloped by Fisher et al (1980, 1990) and Dexter (2013). For this purposes of this study this formula shown below was utilized in order to calculate the minimum diameter hole required to cease crack growth. This formula was developed based on empirical data by Rolfe and Barsom.

\[ D = \frac{S_r \pi a}{8 \sigma_y} \geq 1.0 \text{ in} \quad (Source: Dexter 2013) \]

Where:

- \( S_r \) = nominal stress range at crack tip - [24.9 ksi]
- \( a \) = the half-crack length (inches) - [8.5/2]
- \( \sigma_y \) = the yield stress of the material (ksi) – [36 ksi assumed for this study]

Substituting in the values determined for this study, reveals that the minimum hole diameter to arrest hole is 1.154 inches in diameter. The original crack stop hole diameter drilled in 2011 was 1.125 inches; thus is it is not surprising that the crack re-
initiated. The subsequent hole drilled in 2013 measures 2 inches in diameter and by these simple calculations it should be sufficient to stop crack growth without implementing the proposed rehabilitation method.

It should be noted that the equation presented above is based on assumptions of a simplified stress field. Therefore much weight should not be placed on the results as stress concentration may be much more distorted and complicated on an actual bridge.
Appendix E

ADDITIONAL LABORATORY TESTING PHOTOS

Figure E.1  Fatigue Testing Setup of Specimen #1

Figure E.2  Strain Gauge Instrumentation of Specimen #1
Figure E.3  Specimen 1 Measurement after 130,000 Cycles

Figure E.4  Specimen #2
Figure E.5  Fatigue Testing Setup of Specimen #2
Figure E.6  Specimen 2 after about 160,000-165,000 Cycles (assuming 40-45k for pre crack)
Figure E.7  Specimen 2 Measurement after about 160,000-165,000 Cycles (assuming 40-45k for pre crack)
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Figure E.14  Single Ply of M40J Composite
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Figure E.16  Two-Part Epoxy Paste Adhesive
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Figure E.18  Specimen after Application of Adhesive
Figure E.19 Placement of Patch onto Steel Specimen

Figure E.20 Specimen in Vacuum Prior to Placing in Oven
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Figure E.23  Previous E647 Fatigue Test Specimen Instrumented with Sensing Patch

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