JOINT TRANSPORTATION RESEARCH PROGRAM

INDIANA DEPARTMENT OF TRANSPORTATION
AND PURDUE UNIVERSITY

Repair and Strengthening of Bridges in Indiana Using Fiber Reinforced Polymer Systems:

Volume 1—Review of Current FRP Repair Systems and Application Methodologies

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16. Abstract

For bridges that are experiencing deterioration, action is needed to ensure the structural performance is adequate for the demands imposed. Innovate repair and strengthening techniques can provide a cost-effective means to extend the service lives of bridges efficiently and safely. The use of fiber reinforced polymer (FRP) systems for the repair and strengthening of concrete bridges is increasing in popularity. Recognizing the potential benefits of the widespread use of FRP, a research project was initiated to determine the most appropriate applications of FRP in Indiana and provide recommendations for the use of FRP in the state for the repair and strengthening of bridges. The details of the research are presented in two volumes.

Volume 1 provides the details of a study conducted to (1) summarize the state-of-the-art methods for the application of FRP to concrete bridges, (2) identify successful examples of FRP implementation for concrete bridges in the literature and examine past applications of FRP in Indiana through case studies, and (3) better understand FRP usage and installation procedures in the Midwest and Indiana through industry surveys.

Volume 2 presents two experimental programs that were conducted to develop and evaluate various repair and strengthening methodologies used to restore the performance of deteriorated concrete bridge beams. The first program investigated FRP flexural strengthening methods, with a focus on adjacent box beam bridges. The second experimental program examined potential techniques for repairing deteriorated end regions of prestressed concrete bridge girders. Externally bonded FRP and near-surface-mounted (NSM) FRP were considered in both programs.

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EXECUTIVE SUMMARY

Introduction

Concrete bridge components experience damage and deterioration due to a variety of sources that range from environmental conditions to vehicle impacts. Such damage and deterioration can lead to reduced structural capacities, necessitating that action be taken to either repair or replace concrete bridge components. Innovative repair and strengthening techniques can provide a cost-effective means to lengthen the service life of bridges, providing cost savings compared to more traditional methods of repair or replacement. Fiber reinforced polymer (FRP) systems are a rapidly emerging solution for such applications. The need to investigate the use of FRP for the repair and strengthening of bridges in Indiana was identified, including the need to study specific applications to structural components that often experience deterioration in the field.

A research project was conducted to develop guidance for the application of FRP systems for the repair and strengthening of bridges in Indiana. To accomplish this objective, a study was first conducted to (1) summarize the current state-of-the-art for the application of FRP to concrete bridge components and (2) identify successful examples of FRP implementation for concrete bridges in the literature and examine past applications of FRP in Indiana through case studies, and (3) better understand FRP usage and installation procedures in the Mid west and Indiana through industry surveys. The details of this study are presented in volume 1. Two experimental programs were then performed to determine the most effective uses of FRP in Indiana for (1) flexural strengthening and (2) girder end region repair. The details of the experimental programs are presented in volume 2.

Findings

Volume 1

The primary findings of the literature review, case studies, and industry surveys include the following.

- Common FRP strengthening systems include near-surface-mounted (NSM) strips or bars and externally bonded sheets. Prestressed NSM or prestressed externally bonded systems can be implemented to achieve improved serviceability of beams.
- When using externally bonded FRP or near-surface-mounted FRP, proper anchorage or embedment should be provided. A common type of anchorage for externally bonded sheets is the FRP fan anchor, or spike anchor. Its major benefits include compatibility with the FRP strengthening system, ability to anchor flexural or shear strengthening systems, corrosion resistance, and ease of constructability.
- Issues encountered with previously conducted FRP repairs and retrofits in Indiana include inconsistent layering of FRP sheets, premature termination of FRP sheets near support locations or points of intersection of bridge elements, improper epoxy quantities, uneven distribution of epoxy, and inconsistent surface preparation.
- The dominant applications of FRP systems in Midwestern states have included beam shear strengthening and column confinement.

Volume 2

The primary findings of the flexural strengthening and end region repair experimental programs include the following.

Flexural Strengthening Experimental Program

- Both externally bonded FRP and NSM FRP are effective techniques for the strengthening of flexural members if properly designed and installed. Appropriate anchorage of the externally bonded FRP must be ensured.
- All FRP-strengthened specimens experienced reduced ductility compared to the specimens without FRP. Furthermore, while the FRP-strengthened specimens achieved post-cracking stiffnesses similar to that of the control specimens without FRP, all FRP-strengthened specimens exhibited significantly higher post-yielding stiffnesses relative to the control specimens.
- Considering the anchorage of externally bonded sheets, specimens with FRP spike anchors only at the ends of the primary FRP sheet consistently gained more capacity than specimens with spike anchors at multiple locations along the length of the primary sheet. The separation and redirection of fibers in the FRP sheet required for the installation of the spike anchors likely contributed to premature rupture at the anchor locations.
- The eccentricity of longitudinal steel reinforcement and the relative placement of NSM strips did not play a significant role in the effectiveness of the strengthening systems or the overall performance of the members.

End Region Repair Experimental Program

- The deterioration of the end regions of prestressed concrete girders due to leaking expansion joints can result in significant reductions in strength.
- Restoring the tensile capacity lost due to deteriorated and ineffective prestressing strands in the bottom flange of prestressed concrete girders and ensuring adequate confinement of the repair region are critical factors when designing end region repair systems.
- An externally bonded FRP laminate system proved to be a viable technique for restoring the strength and stiffness of a bridge girder with end region deterioration.
- The use of NSM FRP strips for the repair of the deteriorated end region of a prestressed concrete girder did not provide adequate confinement of the repair region, and therefore, the strength and stiffness of the girder was not restored. If combined with externally bonded FRP laminate, the use of NSM strips may be a viable repair solution.
- Providing a supplemental diaphragm to repair a deteriorated end region of a girder and transfer load to new bearings did not restore the strength of the member. The use of a continuous diaphragm between adjacent girders may provide a more favorable result.

Implementation

Based on the findings of the research, updates to the *Indiana Design Manual* to allow the use of FRP for strengthening purposes is recommended. The experimental programs demonstrated that, if properly designed and detailed, FRP systems can be successfully used for flexural strengthening and for the repair and strengthening of deteriorated girder end regions. Past research has demonstrated other successful applications and provide guidance for proper anchorage. Current guidelines available for the design and implementation of FRP systems should be referenced within the *Indiana Design Manual*. Furthermore, special long-term considerations for the inspection of FRP systems are recommended.

Through performing the repairs on the deteriorated end regions during the research program and based on the test results, recommendations for end region repair were developed and delivered to INDOT. Design-related guidance based on the results of both experimental programs is also included in the final report.

To assist with the implementation of the research findings, an FRP guidebook has been developed and provided to INDOT. The document contains general FRP design guidance, key considerations when designing bridge repair systems, suggested language for the *Indiana Design Manual*, and recommendations for FRP installation procedures.

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1. INTRODUCTION

1.1 Background

There are over 600,000 bridges in the United States, about 56,000 of which are deemed structurally deficient according to ASCE (2017). The term “structurally deficient” simply means that the bridge received a low structural condition rating and that certain elements in the bridge require periodic monitoring or repair. The term does not imply that the structure is in imminent danger of collapse or that it is inherently unsafe. However, reparative action should be taken soon to bring the bridge back to satisfactory condition (ASCE, 2017). This information is further explained in the *ASCE Infrastructure Report Card: Bridges* report (ASCE, 2017), a document created every 4 years to deliver a performance evaluation of several aspects of the American infrastructure, such as bridges, roadways, rail, and transit systems. In the 2017 report card, the bridges in the United States scored a C+, which correlates to a considerable number of bridges that are in poor condition along with an average age of the bridges that is high and keeps increasing. The document also states that nearly 39% of US bridges are over the age of 50 years, where in most cases, the bridges were designed for a lifespan of 50 years. The simple fact holds true that “the average age of America’s bridges keeps going up and many of the nation’s bridges are approaching the end of their design life” (ASCE, 2017). With this information in mind, a simple, efficient, and economical method of repair and/or strengthening that can effectively extend the service lives of these bridges needs to be implemented.

Bridges in Indiana and the Midwest region are subject to harsh environmental conditions. Such conditions include high humidity, freeze-thaw cycles, and the widespread use of deicing salts. Together, these scenarios adversely affect the roadway structure by causing deterioration to either the concrete or other structural components. Aside from environmental effects, bridges can also experience damage from over-height truck impacts and exceedance of permissible truck loads. Once these damage or deterioration scenarios become prevalent, the design and implementation of a repair and/or strengthening system is necessary. These systems should fix the issue and maintain sustainability for years to come.

Fiber reinforced polymer (FRP) systems have become a widely used alternative for the repair and strengthening of bridges that experience such undesirable conditions. These systems are capable of repairing a wide range of minor to moderate damage that includes section loss, scraping, longitudinal cracking, and severing of strands from vehicle impact as well as reinforcement loss from corrosion or exceedance of design truck loads. Typical scenarios that may be appropriate for the application of FRP systems are shown in Figure 1.1. Deterioration of a concrete bridge girder end region in Indiana is shown in Figure 1.1(a). A concrete bridge girder that was struck by an over-height truck is presented in Figure 1.1(b).

When used for civil infrastructure repair, FRP systems incur additional benefits not experienced using conventional methods of repair. Cost savings can be expected when implementing FRP systems due to decreased construction time for installation (ACI Committee 440, 2007). Bridge closure times can be reduced or even eliminated, thus saving time and money on traffic rerouting and improving the safety of the traveling public. Finally, FRP systems are an excellent choice for temporary repair in cases when immediate reconstruction of a damaged bridge is not possible.

1.2 Scope and Objectives

Fiber reinforced polymers are becoming a widely-used alternative for reinforced and prestressed concrete repair and strengthening. There exists a vast knowledge of FRP use for multiple different applications across the country. However, further research is needed to determine the best practices for their use in the state of Indiana.

Through the literature review, case studies, and industry surveys, the goal of the research work summarized in this document is to establish FRP implementation strategies by determining the following.

- Appropriate scenarios for the use of FRP on Indiana bridges.

(a)

(b)

Figure 1.1 Damage scenarios appropriate for FRP repair (Figure(b) from Wipf et al., 2004).

- Proper application and anchorage techniques to effectively utilize the FRP systems.
- Best practices to ensure adequate performance of installed FRP systems.
- Issues with current FRP repairs in Indiana.

This study is part of a larger project with a final objective of producing a guidebook for the use of FRP systems in Indiana. To aid in the development of this guidebook, portions of this study described in this document (volume 1) will be used along with the results of an experimental program detailed in volume 2 of this report. Useful information that benefited the development of the experimental program are also included within this current document (volume 1).

2. STATE OF THE ART

2.1 Fiber Reinforced Polymer Overview

2.1.1 Background

A literature review was conducted to identify past research relating to the repair and/or strengthening of concrete bridges using fiber reinforced polymer technology. Research reports were gathered from state DOTs across the United States along with other documents from various other sources, including journal papers, standards/specifications, design guides, and international research reports. Research findings are presented to explore the state-of-the-art in FRP implementation for bridge rehabilitation. A synthesis study was conducted to achieve certain objectives before proceeding with further investigations related to the implementation of FRP in Indiana. The objectives of these tasks included the study of (1) materials and application techniques, (2) anchorage techniques, (3) flexural and shear behavior of members strengthened with FRP, and (4) strength and stiffness increases provided by FRP. Additional information related to durability of concrete structures that have received FRP systems as well as quality assurance/quality control procedures is provided in Pevey (2018).

2.1.2 Introduction to Fiber Reinforced Systems

Fiber reinforced polymer systems are rapidly becoming a popular method for repair and strengthening of damaged or degraded bridges. Fiber reinforced polymers are comprised of reinforcing fibers and a high strength resin. Once these fibers have become impregnated with the resin and cured, a valuable material for concrete infrastructure rehabilitation is created. Fiber types that are typically used include glass, carbon, and aramid (ACI Committee 440, 2007, 2017). FRP reinforcement used for repair and strengthening exists in many forms. External reinforcement includes bars, strips, sheets, and meshes. Externally bonded FRP systems utilize sheets and meshes, which are bonded to the concrete substrate by adhesives, typically a thermosetting epoxy (ACI Committee 440, 2007, 2017). For near-surface-mounted (NSM) FRP systems, bars or strips are placed inside grooves that are cut in the concrete substrate (ACI Committee 440, 2007, 2017). Examples of some commercially-available FRP products are shown in Figure 2.1.

2.1.3 Use in Civil Engineering Infrastructure

FRP materials can be used in structural applications such as the repair or strengthening of bridge girders that have experienced increased loadings beyond their initial intended use, impact damage from over-height vehicles, reinforcement loss due to corrosion, and deterioration of concrete. FRP systems can be externally applied to the concrete member, embedded into the concrete substrate, or prestressed and applied externally. The FRP system, depending on the location of application, can provide additional shear or flexural reinforcement to the damaged or deficient bridge member. In addition to flexural and shear improvements, FRP utilized in a complete wrap formation can provide confinement and strength improvements to concrete columns and pier caps of bridge substructures.

Aside from strengthening techniques, research has indicated that FRP systems provide improved corrosion resistance of concrete members (Jirsa et al., 2006;

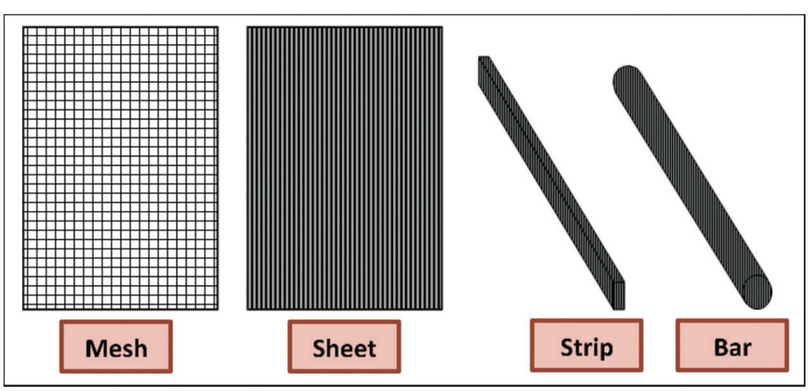


Figure 2.1 Common FRP product types.

Kim & Bumadian, 2017; Tatar et al., 2016). However, research on the durability and long-term performance of FRP systems is ongoing with some mixed results (ACI Committee 440, 2007, 2017). In substructure elements like columns, FRP wraps can be used after properly repairing the concrete substrate to protect the concrete column and internal steel reinforcement from salt spray created by passing vehicles. FRP wraps are also employed to extend the life of beam end region repairs that are needed due to leaking expansion joints. The versatility of FRP systems for repairing or strengthening concrete bridge elements is quite extensive. This, along with its ease of constructability and improved aesthetic appeal, have brought FRP to the forefront of concrete infrastructure repair and strengthening techniques.

2.1.4 Type of FRP and Material Properties

To fully understand FRP systems, one must first become aware of the constituent materials that make up FRP and how they interact with each other. A matrix (formed from resin) is combined with the high-strength fibers to create the finished product. The fibers that are present within the polymer matrix are the primary load-carrying component and provide the high strength and stiffness commonly associated with FRP materials (ACI Committee 440, 2007). The common fiber materials (i.e., carbon, glass, or aramid) differ significantly in terms of their tensile properties. As shown in Table 2.1, the three types of fiber laminates are compared based on their modulus of elasticity, ultimate strength, and rupture strain. The term laminate refers to the material created once the fibers and resins are combined and the resin has cured (ACI Committee 440, 2007, 2017).

Glass fibers exist in four main forms: E-glass (most common), S2-glass (high-strength grade), ECR-glass (acidic resistant), and AR-glass (alkali resistant) (ACI Committee 440, 2007). Aramid fibers are organic fibers that offer good mechanical properties, specifically toughness and impact resistance. Aramid is more commonly known for its use as Kevlar in bulletproof vests (ACI Committee 440, 2007). Carbon fibers, which are a graphite material, have the highest elastic modulus and ultimate strength. FRP materials are a very brittle, exhibiting linear-elastic behavior up to failure (ACI Committee 440, 2017). It is important to note that most fibers used in FRP composites are long, continuous fibers and are not to be confused with the short fibers used in fiber-reinforced concrete mixtures.

Other types of fibers include steel and hybrids. An FRP composite called SFRP that incorporates steel fibers that have high strengths and demonstrate a linear elastic stress-strain behavior can be used for strengthening concrete members (ACI Committee 440, 2007). Choo et al. (2013) conducted repairs of reinforced concrete bridge girders in Kentucky using SFRP. Mechanical properties were presented in their report that showed the SFRP sheet having a tensile strength of 3,372 lbs/in. and an elastic modulus of 5.2×10^6 psi. The issue with SFRP is its inherent corrosion susceptibility.

Hybrid FRPs utilize two or more different fibers within the composite. A study conducted by Kang et al. (2012b) included the fabrication of a carbon-glass hybrid FRP sheet. It was anticipated that, despite the linear elastic behavior up to failure for sheets with only a single type of fiber, when fiber types are combined, pseudo-ductility can be achieved. This term refers to the phenomenon in which the load-carrying capacity is maintained as remaining fibers stretch after the first fiber failure, and the strain at ultimate failure is not less than the ultimate strain of any fiber. The authors combined high-strength carbon fibers with less stiff glass fibers at optimized glass fiber-carbon fiber ratios. These hybrid sheets were reported to cost only about 40% as much as typical carbon fiber reinforced polymer (CFRP) sheets of comparable strength, and they produce a desired pseudo-ductility as opposed to the inherent brittle nature commonly seen with typical CFRP sheets. It was found that the minimum ratio of glass fiber to carbon fiber to achieve pseudo-ductility was 6.8:1.0, and the optimal ratio was 8.8:1.0 (Kang et al., 2012b).

As detailed in ACI 440R-07, the type of FRP reinforcement that is created generally results from different processes of fiber manufacture and fabrication. Multi-end rovings consist of individual strands or bundles of filaments that are cut and placed into a resinous matrix. This process is typically used to create sheets. Sheets can be unidirectional, or, using a weaving process, can be made multi-directional. Another form of FRP manufacture is pultrusion, which utilizes a continuous molding process to create a constant cross-sectional profile (ACI Committee 440, 2007).

2.1.5 Resins and Adhesives

The resin forms the matrix of the laminate and serves the purpose of ensuring proper orientation of the fibers,

TABLE 2.1
FRP Laminate Material Properties (adapted from ACI Committee 440, 2017, as presented in Kim et al., 2012)

FRP System (with epoxy)	Young's Modulus (ksi)	Ultimate Strength (ksi)	Rupture Strain
Carbon (high-strength)	15,000–21,000	150–350	0.010–0.015
Glass (E-glass)	3,000–6,000	75–200	0.015–0.030
Aramid (high-performance)	7,000–10,000	100–250	0.020–0.03

Note: Fiber volume fraction of the laminates shown is about 40%–60%.

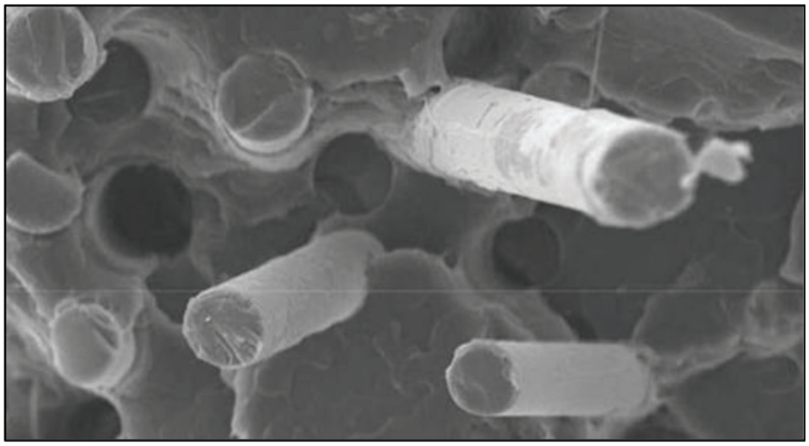


Figure 2.2 Fiber reinforced polymer microstructure (Czigány et al., 2013).

acts as a load transfer medium for the fibers, and protects the fibers from the environment (Orton, 2007). A visual of the microstructure of a fiber reinforced polymer is shown in Figure 2.2, where the rod-like structures are the fibers and the surrounding material is the matrix.

As explained in ACI 440R-07 and *fib* Bulletin 14 (2001), resins used for FRP reinforcement can be grouped into thermoplastics and thermosets. Thermoplastic bonding resins are solid at room temperature and are heated to a liquid. Thermoplastic bonding resins can be reformed once heated (ACI Committee 440, 2007; *fib* Task Group 9.3, 2001). For externally bonded FRP, thermosets are used (ACI Committee 440, 2017). Thermoset bonding resins are liquid at room temperature and are impregnated into the fibers of FRP fabrics and cured by heating and/or the use of a catalyst. After curing, the resin is solid and cannot be reformed. Thermoset resins include polyesters, vinyl esters, phenolics, and the most widely used resin, epoxies (ACI Committee 440, 2007, *fib* Task Group 9.3, 2001). Epoxies typically have better mechanical properties than other thermoset resins, while vinyl esters and polyesters cost less (*fib* Task Group 9.3, 2001).

Once the matrix is combined with the high-strength fibers, the materials then become what is known as a composite, which can be used for various civil infrastructure repair or strengthening applications. The matrix can have a significant effect on the mechanical properties of the composite (*fib* Task Group 9.3, 2001). The composite is an anisotropic material since its mechanical properties are present in only one direction. These mechanical properties are dependent on the amount of fiber per volume in a particular direction (ACI Committee 440, 2007).

Bonding adhesives are used to connect externally bonded FRP laminates to the concrete surface (ACI Committee 440, 2007, 2017). These adhesives are typically like the bonding matrix used to hold the fibers in a laminate together. Like the matrix, a common adhesive used to attach a composite to a concrete structure is two-part epoxy (ACI Committee 440, 2007). The adhesive provides the means for shear to be transferred between the concrete surface and the FRP laminate (ACI Committee 440, 2017). Proper surface preparation of the concrete surface is imperative to a successful bond between the FRP and concrete. Sufficient bond of these two materials ensures composite action under loading and full utilization of the FRP laminate.

2.1.6 Failure Mechanisms

Failure in an FRP-strengthened system typically occurs as a result from high stress concentrations within the system. These may result from the presence of cracks in the concrete substrate (Kang et al., 2012a). Stress concentrations can also result from poor surface conditions prior to FRP installation or sharp bends in the FRP material (Quinn, 2009). Furthermore, according to Quinn (2009), premature failure can occur due to improper mixing ratios when preparing the epoxy resin. This in turn can produce "weak points" in the epoxy adhesive that lead to failures initiating at these locations. Multiple failure mechanisms can occur in bonded FRP reinforcement systems and, depending on the installation procedure chosen, different mechanisms will govern. It is important to note that strengthening systems will either be bond critical or contact critical. The bond between the FRP system and the concrete substrate is imperative to the success of bond-critical applications. The use of FRP for shear and flexural strengthening is generally bond critical (ACI Committee 440, 2007, 2017). Contact-critical applications do not require bond between the FRP system and the concrete and depend only on contact between these two components. The use of FRP for column confinement is a contact-critical application (ACI Committee 440, 2007, 2017). Generally, failure mechanisms include FRP debonding and FRP rupture. FRP rupture is a more desirable failure, as the FRP reinforcement reaches its full capacity before failure, utilizing the material more efficiently (Orton et al., 2008; Quinn, 2009; Shekarchi, 2016). An important concept to understand is that this failure is abrupt, causing a sudden loss in load-carrying capacity that may be detrimental to structural integrity (Chennareddy & Taha, 2017). The designer should consider this phenomenon when implementing FRP systems for the strengthening of concrete bridge elements. Debonding failure of the FRP is a common occurrence and exists in various forms.

According to Article C3.4.3.2 of AASHTO (2012), debonding at the termination points of externally bonded FRP systems applied to the tension face of a member under a combination of shear and flexure can occur in three different modes. The first mode listed in AASHTO (2012) is critical diagonal crack debonding with or without concrete cover separation (Oehlers & Seracino, 2004; Yao & Teng, 2007). When the end of the FRP reinforcement is located in a region of high shear stress and low internal shear reinforcement, a significant diagonal shear crack may intersect the FRP and then propagate toward the end of the repair system (AASHTO, 2012). The second debonding failure mode listed in AASHTO (2012) is concrete cover separation (Teng et al., 2002). This mode of failure occurs when a crack initiates at the end of the FRP reinforcement and travels along the longitudinal steel reinforcement (AASHTO, 2012). Finally, the third failure mode listed is plate-end interfacial debonding (Teng et al., 2002). Interfacial debonding between the various layers of the FRP strengthening system can occur based on high interfacial shear stresses and the "weakest element" in the system (AASHTO, 2012). The weakest element, according to AASHTO (2012), is typically the concrete substrate.

Kang et al. (2012a) explain that for externally bonded systems, debonding can propagate through the various layers depicted in Figure 2.3 and will take the "path of least resistance." The illustration in Figure 2.3 is also in agreement with *ASTM D7522, Standard Test Method for Pull-Off Strength for FRP Laminate Systems Bonded to Concrete Substrate* (ASTM, 2015). This standard test method describes the many forms of debonding failure that occur in testing the pull-off strength of externally bonded FRP systems. It describes either an adhesive or cohesive failure, where adhesive failures occur at the various interfaces within the system and cohesive failures occur within the element in the system (i.e., within the adhesive, FRP laminate, or concrete) (ASTM, 2015). Kang et al. (2012a) note that thinner adhesive layers will decrease the likelihood of a concrete-adhesive interfacial failure. Therefore, thick and stiff bond layers are not ideal. It is important to note that FRP debonding is a brittle failure mode that can occur at relatively light applied loads (Kang et al., 2012a). Various experimental tests have proven the issue of debonding, and methods of mitigating this issue have been presented. One of the most common ways to preclude this mode of failure and utilize the FRP system more efficiently is the use of FRP system anchorage, which is discussed in Section 2.1.7.

Near-surface-mounted (NSM) strengthening systems are an alternative to externally bonded systems that offer increased material efficiency as well as decreased risk of debonding failure (De Lorenzis & Teng, 2006). The general failure modes of FRP rupture and debonding, however, still exist for near-surface-mounted systems. Debonding has been shown to occur due to high stress concentrations at the near-surface-mounted cutoff points (i.e., at the ends of bars/tapes/strips) and in regions of flexural cracks (Hassan & Rizkalla, 2003). Tests by Hassan and Rizkalla (2003) showed that rupture of the near-surface-mounted FRP is possible if a minimum embedment length is provided (shorter embedment lengths failed due to debonding). Flexural behavior of beams strengthened with NSM FRP and the effect of embedment length on failure mode is discussed in more detail in Section 2.2.3.

2.1.7 Anchorage Techniques

Anchorage is necessary in an FRP system primarily to ensure the full capacity of the laminate is utilized before an undesired debonding failure occurs. As explained by Grelle and Sneed (2013), anchorage systems not only help to preclude or delay debonding of the FRP material from the concrete, but they also act as a load-transfer mechanism at critical locations of the structural member. Moreover, the anchorage provides, in some cases, a ductile failure mode instead of the brittle and sudden failure associated with FRP debonding or rupture (Grelle & Sneed, 2013). Aside from

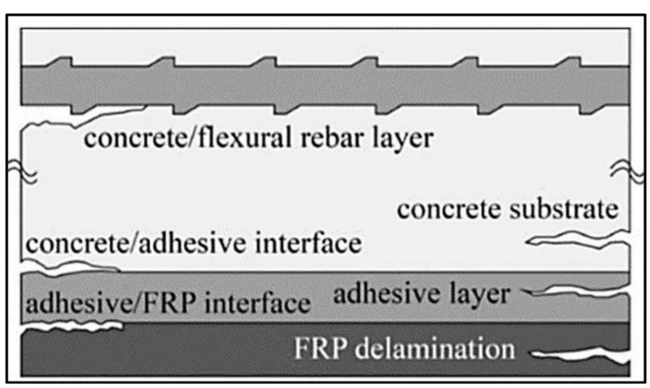


Figure 2.3 Possible paths for debonding (Kang et al., 2012a).

the benefits of the anchorage systems, there are some possible unwanted scenarios that can occur from an anchored-FRP system. Global anchorage failure or FRP rupture are sudden and brittle failures that can occur in anchored systems due to localized stress concentrations imposed by the anchor (Grelle & Sneed, 2013). Overall, the key to a successful FRP system is the achievement of full composite action between the concrete and the FRP. To ensure this behavior exists up to failure of the repaired member, external anchorage is required at both ends of the bonded FRP and can be placed along the length of the repaired section based on the internal reinforcement of the member (Spadea et al., 2000).

Types of anchorage can be grouped into either metallic or non-metallic, with the latter being the more popular choice with respect to corrosion resistance and ease of installation. Some metallic anchor systems include threaded anchor rods and modified anchor bolt systems (Quinn, 2009). Threaded anchor rod systems involve the use of steel plates and angles acting as a clamp for the FRP sheet, along with a steel rod securing this assembly to the concrete section (Deifalla & Ghobarah, 2006, Quinn, 2009). An example of the threaded anchor rod system on a T-beam section is shown in Figure 2.4. A modified anchor bolt system that has been studied makes use of two discontinuous FRP plates which the FRP sheet used for strengthening is wrapped around. This assembly is then anchored onto the concrete using wedge anchors (Bae & Belarbi, 2013). An example of this modified anchor bolt system is shown in Figure 2.5. Other metallic anchor systems include those found in externally-prestressed FRP sheet systems that incorporate steel anchorages at the fixed and jacking ends of the system as shown in Figure 2.6 (Wight et al., 2001). Non-metallic systems typically involve the use of the same FRP used for the strengthening application at hand (e.g., a beam strengthened longitudinally with CFRP sheets could be anchored with CFRP anchors made from the same fibers as the sheets). Some types of non-metallic systems include FRP fan anchors (spike anchors), L-shaped FRP plates, and FRP U-wraps (which can also be used for shear strengthening applications) (Kim et al., 2012). Many studies have been completed utilizing the FRP fan, or spike, anchorage system, sometimes referred to as simply "FRP anchors," in different strengthening systems, and it has proven to be effective in precluding debonding failures in FRP-retrofitted members (Huaco Cárdenas, 2009; Orton, 2007; Pudleiner, 2016; Quinn, 2009; Wang, 2013). The use of U-wraps for anchorage is further addressed in volume 2 of this report.

In Wang (2013), the performance of glass fiber reinforced polymer (GFRP) anchors were experimentally investigated. The results from testing were used to verify the use of GFRP as an alternative to CFRP, and a means for quality control of the GFRP anchors was presented. All beam specimens were strengthened using CFRP sheets. Failure modes experienced during testing included rupture of the CFRP flexural reinforcing sheet and pull-out of the GFRP anchor. It was concluded that pull-out failure was undesirable, as the anchor did not have sufficient capacity to develop the full strength of the sheet in this case. However, this phenomenon occurred in unbonded CFRP sheet specimens, as the specimens with bonded sheets were able to develop the full capacity of the flexural sheet reinforcement. The advantages of the GFRP anchors over CFRP anchors were determined to be larger deformation

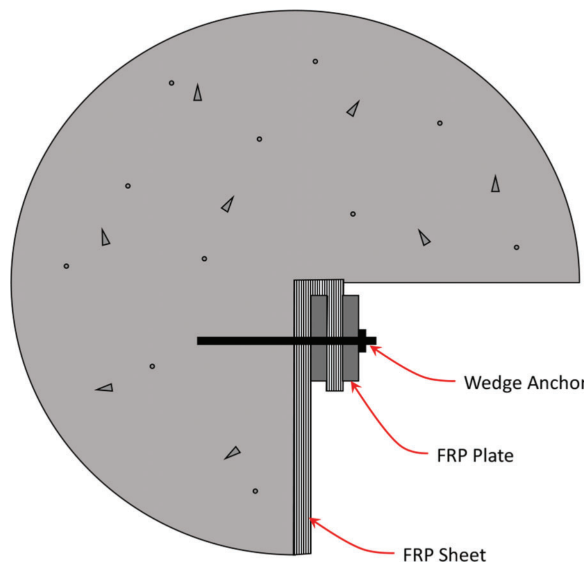


Figure 2.5 Modified anchor bolt system (adapted from Bae & Belarbi, 2013).

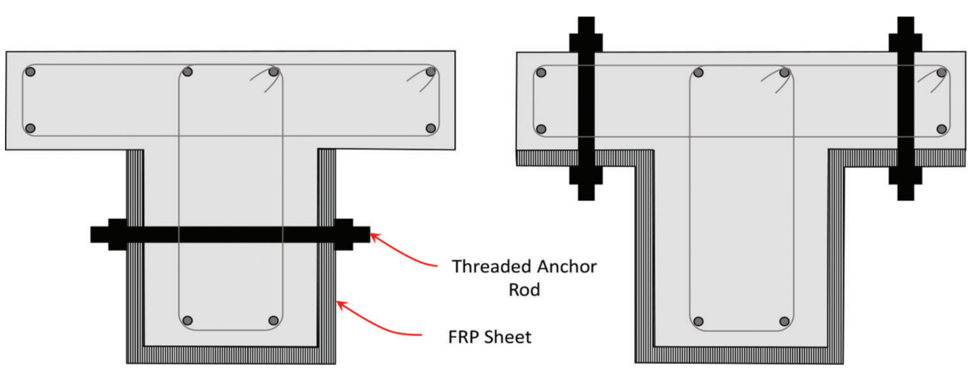


Figure 2.4 Threaded anchor rod system (adapted from Kim et al. 2012, originally from Deifalla & Ghobarah, 2006).

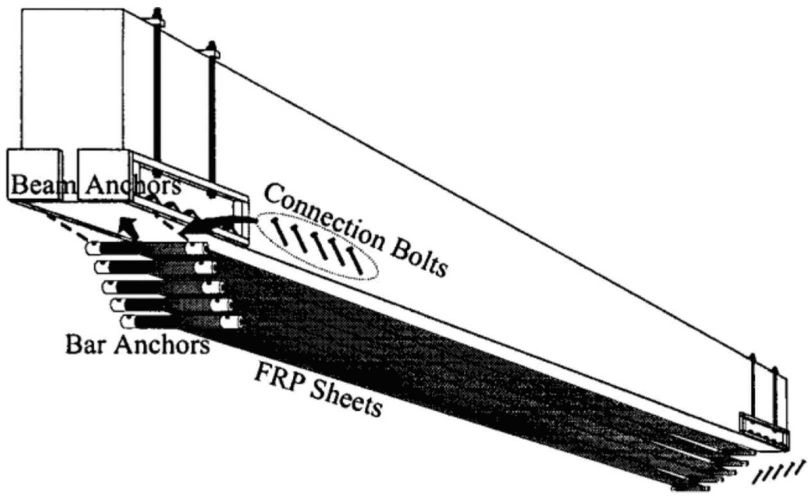


Figure 2.6 Externally-prestressed FRP system (Wight et al., 2001)

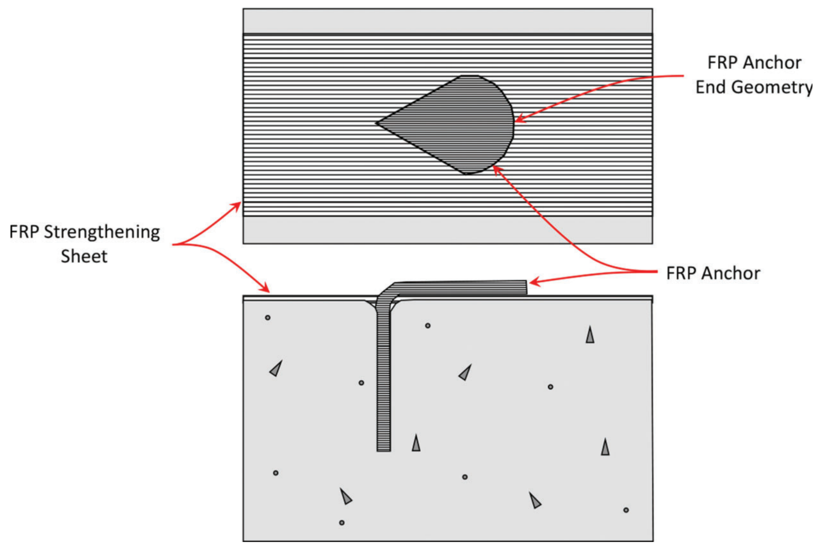


Figure 2.7 FRP fan anchorage system (adapted from Huaco Cárdenas, 2009).

capacity and easier handling during installation as well as the failure of the GFRP anchor being less abrupt. Disadvantages of the GFRP anchors included lower tensile strength as compared to CFRP, resulting in "bulkier" anchors due to the need for more material. Overall, the GFRP anchors were successful in developing the capacity of the CFRP reinforcement when the CFRP reinforcement was bonded to the concrete substrate (Wang, 2013). A schematic of the fan anchorage system is shown in Figure 2.7. Note that this illustration is taken from another author (Huaco Cárdenas, 2009). However, the overall geometry and layout of the anchor itself is similar.

Detailing of the FRP fan anchorage system was investigated by Quinn (2009). In this study, reinforced concrete inverted T-beams were strengthened in shear using a three-sided CFRP wraps (i.e., U-wraps). CFRP anchors were used on the sides of the beam stem to anchor the sheets at their ends, as shown in Figure 2.8. The anchorage system details are shown in Figure 2.9. It was concluded that the anchors aided in the development of the full tensile capacity of the CFRP wraps for shear span-to-depth ratios greater than two. The anchors performed well when the following details were utilized, as listed in Quinn (2009).

- CFRP material used for the anchor should have an area that is greater than or equal to twice the area of the material within the primary strip that is being anchored to the concrete.
- Anchor holes should extend at least 4 in. into the core of the concrete member.

- Area of the anchor hole should be 40% greater than the area of the CFRP material used for the anchor.
- Anchor holes should be rounded to a minimum radius of 0.5 in.
- Anchor fans should be splayed at least 0.5 in. past the edge of the primary CFRP strip and at an angle not to exceed 60 degrees.
- Square CFRP patch sheets should be applied over the CFRP anchor. These sheets should be the width of the primary CFRP strip. The first patch sheet should be applied with its fibers oriented perpendicular to the CFRP strip, and the second patch sheet should be applied with its fibers oriented parallel to the CFRP strip.

A relatively new form of anchorage was studied by De Caso and Nanni (2016). This anchorage solution, shown in Figure 2.10, was developed for externally bonded FRP systems and consists of two main components: a flat staple composed of a pre-cured CFRP material and a saturated fiber sheet that wraps around the flat staple anchor. This entire assembly is placed over an externally bonded FRP sheet (in the case of this research, a longitudinal FRP laminate). The staple serves the purpose of transferring load into the concrete substrate, and the saturated fiber sheet increases the contact area to help transfer load from the FRP laminate to the staple. Double shear load tests, as shown in Figure 2.11, were performed on specimens that had bonded FRP laminates with no anchorage, bonded FRP laminates with only the pre-cured CFRP staple, and bonded FRP laminates with the two-component anchorage system. The results of the anchored specimens showed an improvement in load-carrying capacity as compared to the specimens with no anchorage, with the two-component anchorage system providing improved performance over the use of the CFRP staple alone (De Caso & Nanni, 2016).

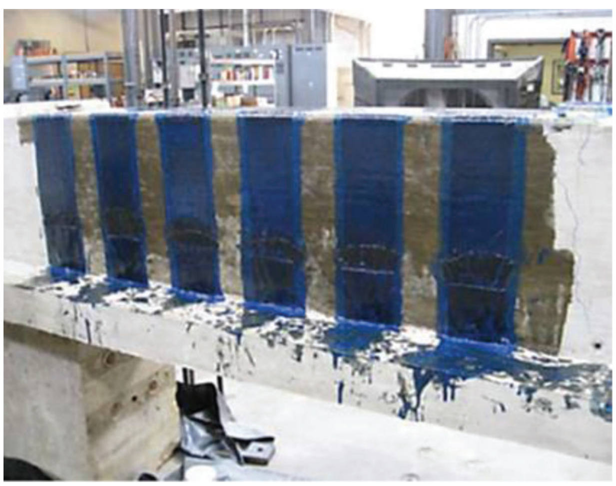


Figure 2.8 Anchored CFRP U-wraps on T-beams (Quinn, 2009).

One method of anchorage, especially for externally bonded longitudinal sheets, is the combination of FRP U-wrap and FRP anchors (Kim, 2006). The U-wrap can serve the purpose of shear strengthening a beam or helping to anchor a longitudinal FRP sheet to the tension face of the beam, while the FRP anchors are used to anchor the U-wrap to the sides of the beam. This effective form of anchorage was studied in a project completed by Kim (2006). In this study, multiple anchorage configurations were investigated to determine their effectiveness in anchoring flexural FRP sheets to reinforced concrete specimens. In the end, it was concluded that “it was necessary to use both CFRP anchors and CFRP U-wraps to achieve full strength of the CFRP sheet” (Kim, 2006).

2.2 Superstructure Repair and Strengthening

2.2.1 Introduction

The following subsections provide information related to FRP installation procedures and the structural behavior of concrete elements strengthened with FRP with specific focus on superstructure members. Additional information regarding the repair and strengthening of substructure elements is provided in Pevey (2018).

2.2.2 Installation Procedures

Fiber reinforced polymers exist in many commercially-available forms. For repair or strengthening of existing civil infrastructure, the commonly used forms include sheets, bars, strips/tapes, and meshes (ACI Committee 440, 2007). Depending on the damage level, repair accessibility, and other details of the structure,

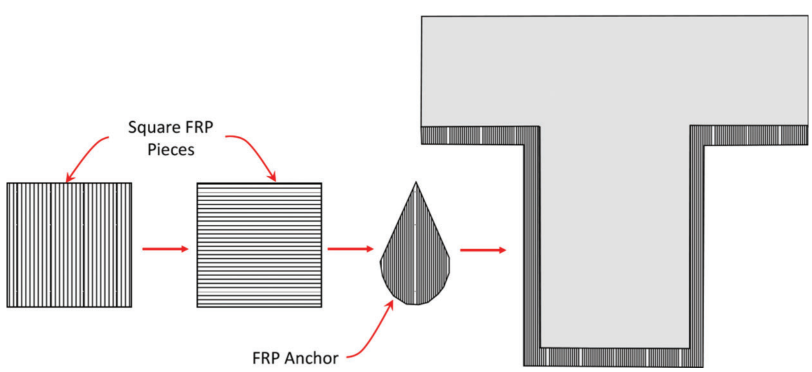


Figure 2.9 Anchorage details (adapted from Quinn, 2009).

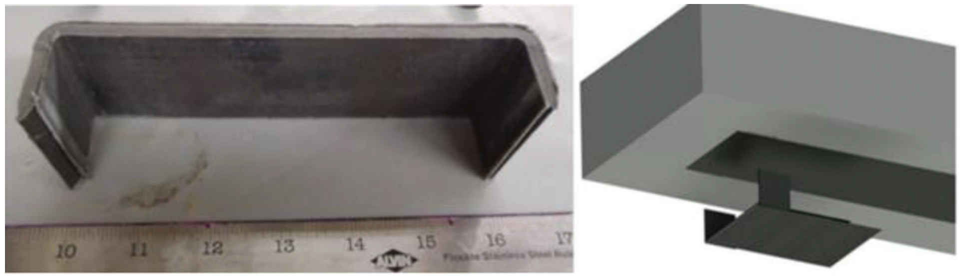


Figure 2.10 CFRP staple anchorage system (De Caso & Nanni, 2016).

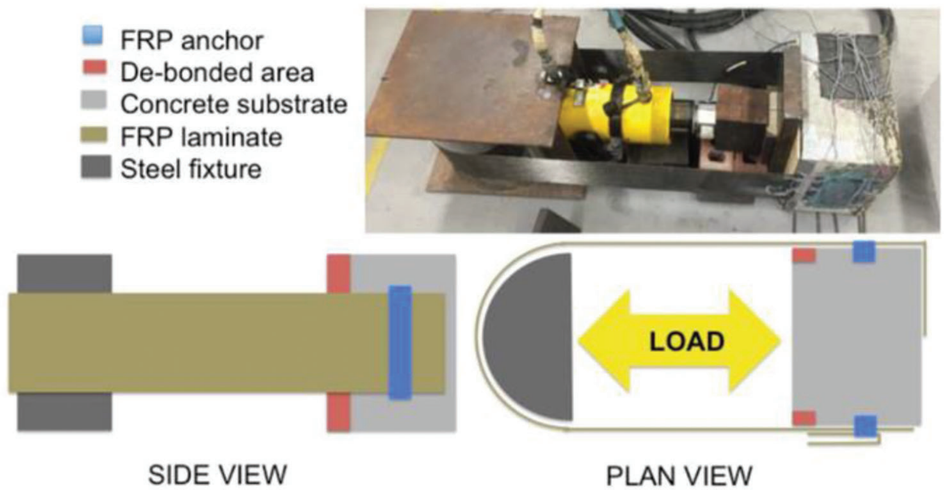
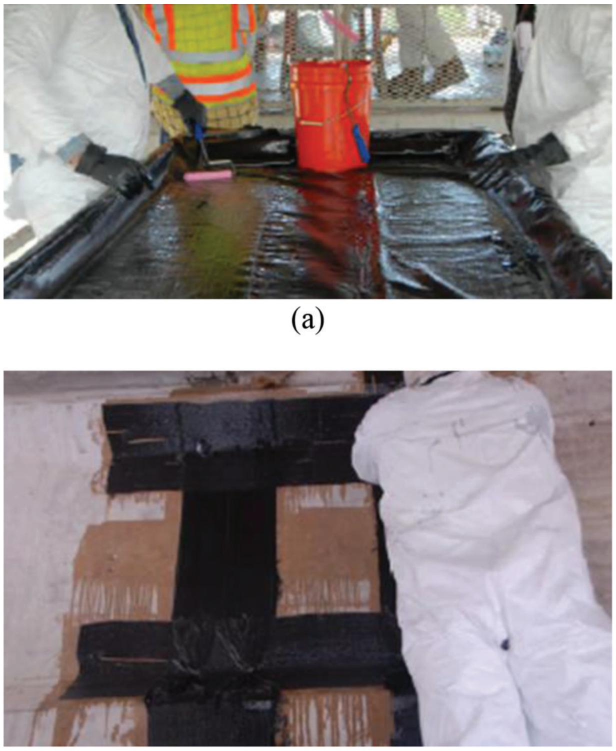


Figure 2.11 Double shear load test setup (De Caso & Nanni, 2016).

different repair and/or strengthening methods can be selected. Three common FRP repair and/or strengthening installation methods are externally bonded, near-surface-mounted, and externally-prestressed (ACI Committee 440, 2007). The externally bonded and externally-prestressed procedures consist of a surface preparation phase. This phase is imperative to the successful application and adequate performance of the FRP repair system, if the system is bond critical (ACI Committee 440, 2017). Surface preparation should ensure an adequate level of surface roughness. For application of externally bonded FRP, a concrete surface profile (CSP) of 3 is required (ACI Committee 440, 2017; ICRI, 2016). However, when roughening the surface, the integrity of the underlying layers of concrete should be preserved by not using high-impact devices that can undermine the strength of the concrete substrate or by following the use of impact devices with sandblasting or water-blasting to remove weakened surface concrete (Morgan et al., 1996).

Externally bonded FRP systems involve the application of FRP sheets to the concrete surface. The process begins with surface preparation, filling of damaged portions of concrete with the appropriate material, and ensuring a clean, workable surface for the FRP. Then, if specified by the manufacturer of the FRP system, a primer coating is applied to the concrete surface before the sheets are placed (ACI Committee 440, 2007, 2017). This primer coating provides a bondable surface for the application of the FRP, ensuring epoxy is not drawn into the concrete substrate from the composite (Karbhari et al., 2005). FRP application then takes place. The sheets are cut to size on site, and a dry- or wet-layup procedure is employed. For the dry-layup procedure, the concrete surface is saturated with epoxy resin and dry FRP sheets are then placed on top. A serrated roller is used to force epoxy through the FRP sheet and eliminate air voids before a final coat of epoxy is applied on top of the FRP to ensure that the fibers are fully impregnated. For the wet-layup procedure, FRP sheets are saturated with epoxy resin prior to being placed on the concrete surface. The saturated sheets are then placed where needed (Garcia et al., 2014; Kim et al., 2012; Quinn, 2009). Pre-saturated systems that are pre-impregnated by the manufacturer are also available (ACI Committee 440, 2017). The application of wet-layup FRP sheets to a bridge girder to increase shear capacity is shown in Figure 2.12. The saturation of sheets at ground level is depicted in Figure 2.12(a), and the application of the sheets to the concrete surface is shown in Figure 2.12(b). With either method, subsequent layers of sheets and adhesives may be placed depending on the desired level of strengthening (ACI Committee 440, 2017). Anchors, if specified, are



(a)

(b)

Figure 2.12 Wet-layup procedure for externally bonded FRP system (Garcia et al., 2014).

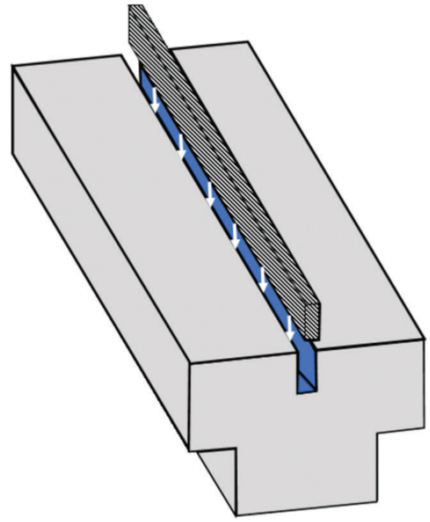


Figure 2.13 Near-surface-mounted system (adapted from Hughes Brothers, 2011).

then installed, followed by a final overcoating of epoxy. Paint can then be applied over the repaired region for aesthetics and, more importantly, UV protection.

Near-surface-mounted systems share some similarities with the externally bonded systems. However, these systems are placed inside the concrete substrate. Prior to applying the FRP material, a shallow groove is cut into the concrete substrate. Adhesive is then placed into the groove. The FRP, typically in the form of a bar or strip/tape, is installed into the groove (ACI Committee 440, 2007). These systems can be prestressed if needed for the specific repair or strengthening application (see Appendix B, Section B.1.1). Figure 2.13 depicts a typical near-surface-mounted system. In the figure, the groove has already been prepared, filled with epoxy, and the FRP strip/tape is being inserted into the groove. To complete the NSM application, additional adhesive is placed in the groove until it is filled (ACI Committee 440, 2007). A layer of paint can then be placed over the entire repaired surface.

The last method of FRP repair or strengthening is an active approach to rehabilitation using externally-prestressed FRP sheets. This method includes added benefits such as efficient material use by better utilization of the tensile capacity of the sheets, restoration of prestress in the member, improvement of serviceability performance, and the ability for the FRP to resist dead loads in addition to applied live loads (El-Hacha et al., 2004). Three methods for applying prestress to the FRP sheets include the cambered beam method, use of an external reaction frame, and direct prestressing (El-Hacha et al., 2004). The cambered beam method is performed by jacking the beam at midspan and applying the FRP sheets to the face of the member that experiences tension under service loads (Saadatmanesh & Ehsani, 1991). The jack is released after the epoxy cures, giving the sheets a moderate prestress. The cambered beam method is shown in Figure 2.14.

The external reaction frame method allows the sheets to be prestressed independent of the structural member. First, the sheets are prestressed prior to bonding to the concrete substrate. These prestressed sheets, along with a layer of adhesive, are then placed in contact with the structural member. After curing, the independent system is removed, and steel anchorage devices are typically put into place (Yue-lin et al., 2005). Lastly, sheets can be prestressed directly on the target member. Prior to prestressing procedures, the concrete surface is properly prepared for the externally bonded FRP system, and hardware is installed to the concrete substrate for both the jacking and dead ends. Epoxy is then applied to the sheets and concrete substrate according to project specifications (i.e., wet- or dry-layup). The sheets are placed into the prestressing hardware and stressed, ensuring bond along the length of the longitudinal FRP reinforcement (Wight et al., 2001). Following prestressing, the hardware assembly remains on the member for the life of the structure.

2.2.3 Flexural and Shear Behavior

In bridge superstructure members, common strengthening schemes to account for deficiencies include flexural and shear retrofits. In flexural strengthening schemes, the FRP systems are placed on the tension face of the deficient concrete member with the fibers oriented parallel to the longitudinal axis of the member (ACI Committee 440, 2017). Flexural strengthening systems usually extend along the length of the member

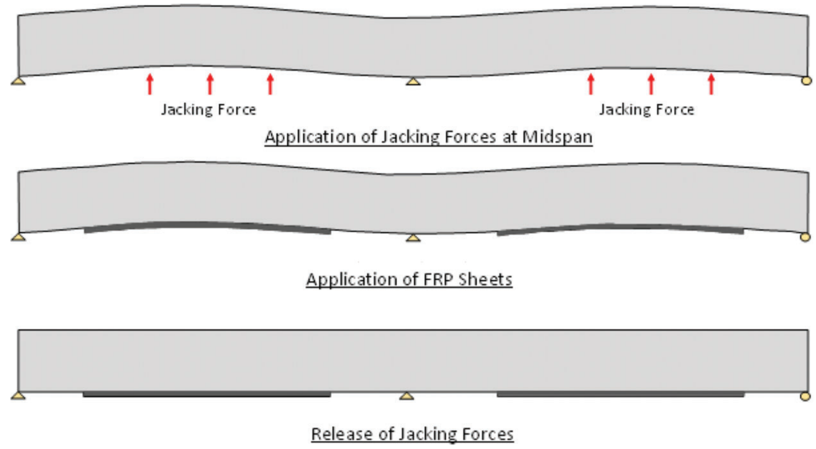


Figure 2.14 Cambered beam prestressing method (adapted from Saadatmanesh & Ehsani, 1991).

to be strengthened, while shear strengthening systems can be placed incrementally along the length. For shear strengthening schemes, systems are placed on the sides of the deficient member with fibers typically oriented perpendicular to the longitudinal axis of the member (ACI Committee 440, 2017). FRP strips (i.e., FRP sheets that are cut into individual strips) are typically unidirectional for both flexural and shear strengthening applications. That is, fibers in the strips are oriented in one direction.

For flexural strengthening of bridge superstructure elements, any of the installation methods discussed in Section 2.2.1 can apply, depending on the level of damage or deficiency present. For beams with severe damage or deficiencies, prestressed FRP systems may be necessary to return the beam to a satisfactory performance level, while nonprestressed sheets and near-surface-mounted systems may be applicable for most other cases.

In a study completed by Alagusundaramoorthy et al. (2002), the effectiveness of CFRP fabrics and "sheets" (better described as plates with a thickness of up to 4.78 mm) in strengthening reinforced concrete beams in flexure was investigated. Four-point bending tests were conducted on 14 beams that were comprised of two unstrengthened control specimens and 12 strengthened specimens. The strengthened specimens included five with CFRP sheets without anchors, three with anchored CFRP sheets, and four with CFRP fabrics. Anchorage was provided using bolts and nuts that were placed through the sheets and into pre-drilled holes in the concrete substrate. Loading was first applied cyclically, then statically until failure. The test results revealed that centerline deflections were reduced for all strengthened specimens at both service and failure loads compared to the control specimens. The failure load depended on the strengthening scheme being sheets, anchored sheets, or fabric as well as the number of layers of CFRP provided. The authors demonstrated that an analytical model assuming strain compatibility between the CFRP and concrete provides conservative strength estimates if used for design (Alagusundaramoorthy et al., 2002). It is important to note that anchorage should generally be provided for members strengthened with externally bonded FRP systems in order to achieve the full strength and effectiveness of the strengthening system.

When implementing near-surface-mounted systems for strengthening beams in flexure, different factors than those for externally bonded FRP systems must be considered, such as embedment length, groove geometry, and groove spacing (ACI Committee 440, 2017). In a study completed by Hassan and Rizkalla (2003), the bond characteristics of NSM CFRP strips installed in reinforced concrete beams were investigated. The experimental program consisted of nine small-scale reinforced concrete beams, of which eight were strengthened with NSM strips. All specimens were monotonically loaded to failure. The main testing parameter was the embedment length of the NSM strips. Embedment length here refers to the length of NSM strip extending out from each side of the applied point load at midspan that is embedded in the concrete substrate (i.e., length of strip measured from the point load toward both the left and right supports). The authors tested eight different embedment lengths to determine the required length to develop the ultimate force of the strip. If bond failure occurred in the strengthened specimen, the embedment length of the strips was increased, and the embedment length was decreased if flexural failure occurred. Using this methodology and differing embedment lengths, the minimum length required to develop the NSM strip was determined. Rupture of the NSM strips and a "full composite mechanism" between the CFRP and beam began at an embedment length of 850 mm. This translates to approximately 68% of the span length containing embedded NSM strip. The authors suggested that any embedment length beyond this would not provide extra strength to the beams of the test program. As for groove geometry, width and depth variances should be investigated as they can affect splitting of the epoxy cover. Furthermore, increasing the groove width can increase the debonding load and decrease the development length. Overall, the researchers concluded that

NSM strips are a viable method for the flexural strengthening of reinforced concrete members (Hassan & Rizkalla, 2003). This research is referenced in Section 14.3 of ACI 440.2R-17 regarding groove details and the calculation of the development lengths for NSM reinforcement.

To compare the effectiveness of NSM systems and externally bonded systems, El-Hacha and Rizkalla (2004) conducted a study that involved testing eight reinforced concrete T-beam specimens, of which four were strengthened using NSM bars or strips, and three were strengthened using externally bonded strips (i.e., sheets cut into strips). Both glass and carbon FRP products were used in this study. It was found that the NSM strips provided a greater load-carrying capacity compared to the NSM bars. The bars experienced early debonding failure that is possibly attributed to less bonded surface when compared to the strips. Failure modes for the NSM-strengthened specimens included rupture of the CFRP strips and debonding of the CFRP bars (from epoxy splitting). Failure modes in all externally-strengthened specimens were debonding of the strips. The authors concluded that NSM performance surpassed that of the externally bonded systems (El-Hacha & Rizkalla, 2004). It is important to note, however, that this study did not use anchors for the externally bonded strips.

Concrete bridge girders can be strengthened in shear using various techniques. As explained by Shekarchi (2016), the FRP system (typically referred to as a U-wrap for shear strengthening systems bonded on three sides of the member) can be continuous along the length of the member or split into discrete strips. One advantage of the discrete strips is that the member can be inspected for shear cracks throughout its service life (Shekarchi, 2016). Strips can be placed vertically on the member (i.e., at 90 degrees to the longitudinal axis of the member) or diagonally on the member (e.g., at 45 degrees to the longitudinal axis of the member) (Shekarchi, 2016). However, continuous U-wraps cannot be used in a diagonal orientation, but separate strips must be lapped on the bottom of the beam to achieve this arrangement (Kim, 2011; Shekarchi, 2016). In some applications, unidirectional strips (i.e., strips with fibers oriented in only one direction) are placed parallel and perpendicular to the longitudinal axis, creating a bidirectional strengthening scheme on the web of the member (Alotaibi, 2014; Kim et al., 2012; Shekarchi 2016). For side-bonded FRP materials that do not wrap around the member, experimental results have shown that members with laminate strips installed perpendicular to the assumed crack angle (45 degrees) result in improved behavior in terms of shear strength and shear cracking compared to vertical side-bonded strips (Quinn, 2009; Zhang & Zhu, 2005). However, a major disadvantage of these side-bonded diagonally-oriented systems is early debonding. Using the vertical U-wrap configuration has demonstrated improved behavior in terms of debonding (Bousselham & Chaallal, 2004, 2006; Quinn, 2009). Furthermore, as noted by Kim (2011), vertical U-wraps require less material than that of the diagonal U-wraps (consisting of lapping strips), which are longer due to geometry. According to Kim (2011), using U-wraps with wider vertical strips containing the same amount of CFRP as the diagonally-placed strip is more efficient.

It should be understood that for the shear resistance of an FRP laminate to be engaged, a crack must be formed and intersect the FRP laminate. Some localized debonding is necessary for this to occur. If anchors are used within the shear strengthening system, the FRP can continue to contribute to the shear resistance of the member after the occurrence of debonding (Bae & Belarbi, 2013; Quinn, 2009). Through experimental testing, Quinn (2009) concluded that "CFRP anchorage is required to develop the full strength of the CFRP laminates in shear" when wrapping the FRP around the entire member cross section is not possible.

Other important factors also affect the contribution of the FRP shear strengthening system. According to Quinn (2009), these include shear span-to-depth ratio, FRP layout and configuration, shear reinforcement in the member (e.g., stirrups), and number of layers of FRP reinforcement. For the beam tests conducted by Quinn (2009), when a shear span-to-depth ratio, defined as the distance from the support face to the point of loading, was less than two, failure was characterized by crushing of the strut extending from the load point to the support. This failure was determined to preclude the development of high tensile strains in the CFRP laminates. It was concluded by Quinn (2009) that there is a relationship between the shear span-to-depth ratio and the increase in shear strength provided by CFRP, but that further research into this matter is needed. Quinn (2009) also explains that the amount of internal steel reinforcement for shear resistance can affect the performance of the external FRP system. If a greater amount of internal shear reinforcement is present within the member, the less effect the FRP shear reinforcement will have (Deniaud & Cheng, 2001). It is important to note that, in an FRP-strengthened member, the traditional assumption of independence of shear strength contributions from steel stirrups, concrete, and FRP is not valid (Kim et al., 2017). According to Kim et al. (2017), the simultaneous attainment of the maximum shear contributions of concrete and FRP is not possible due to the brittle nature of the two materials. Kim et al. (2017) further explain that the point when the maximum contribution of shear strength provided by the FRP occurs may not correspond to the yielding of all transverse steel reinforcement. The authors present modification factors to the current shear design methodology in ACI 440.2R-17 to account for this altered interaction.

2.3 Strength and/or Stiffness Improvements

Multiple research projects across the United States and the world have proven the efficacy of fiber reinforced polymers for increasing the strength and/or stiffness of the elements to which they are applied.

Using the body of knowledge gathered through the literature review, a summary of research projects was generated and is presented in Table 2.2. This summary table represents the compilation of multiple types of research projects in which aspects such as fiber type, FRP strengthening system, loading configuration, and repaired element varied significantly. These projects were selected based on these variations in test parameters as well as the authors' explicit conclusions on strength increase and how the FRP systems affected stiffness characteristics.

2.4 Summary and Conclusions

A literature review was conducted to identify past research relating to repair and/or strengthening of concrete bridges using fiber reinforced polymer technology. Research reports were gathered from state DOTs across the United States along with other documents, including journal papers, standards/specifications, design guides, and international research reports, gathered from various other sources. Research was presented to explore the state-of-the-art in FRP implementation for bridge rehabilitation with specific focus on bridge superstructures. A synthesis study was conducted to achieve certain objectives before proceeding with further studies related to the implementation of FRP in Indiana. The synthesis study achieved the following objectives.

- Identified proper materials and techniques that lead to successful applications of FRP.
- Established effective techniques for reliable anchorage of FRP sheets.
- Examined the state-of-knowledge of the flexural and shear behavior of reinforced concrete members strengthened with FRP.
- Determined the additional strength and/or stiffness provided by FRP.

The proper materials and techniques for successful application of FRP systems will depend significantly on the type and level of damage, deterioration, or deficiency present in the structure under consideration. Carbon, glass, and aramid are the most common fiber materials used for civil infrastructure (ACI Committee 440, 2007, 2017). For the repair of deteriorated concrete bridge members, FRP sheets applied using a wet- or dry-layup procedure are typically employed, depending on the geometry of the repaired-elements. Past research suggests that wet-layup systems are easier to apply, especially for irregular geometries such as at the intersection of a pier cap and columns (Garcia et al., 2014). This is primarily due to the flexibility and easy conformability of the saturated FRP sheets. Pre-saturated systems are also available (ACI Committee 440, 2017). For strengthening bridge elements, the level of deficiency will determine what type of strengthening system should be used. Strengthening systems include near-surface-mounted (NSM) strips or bars placed in the tension face of beams for flexural strengthening, externally bonded longitudinal sheets that are FRP-anchored to the tension face of beams for flexural strengthening, and FRP-anchored U-wraps for shear strengthening of beams and pier caps. If appropriate, prestressing NSM or prestressed externally bonded systems can be implemented to achieve improved serviceability of beams. Refer to Section 2.2.2 for more information on installation procedures.

Several methods of anchorage exist and have been tested for properly anchoring FRP to a concrete structure. These can be roughly grouped into metallic and non-metallic. Due to the inherent corrosion-resistant properties of FRP (non-metallic), the use of FRP-based anchors has gained popularity. One type of anchorage is the FRP fan anchor, or spike anchor. Its major benefits are compatibility with the FRP strengthening system, ability to anchor flexural or shear strengthening systems, corrosion resistance, and ease of constructability (Huaco Cárdenas, 2009; Orton, 2007; Pudleiner, 2016; Quinn, 2009; Wang, 2013). Another popular form of anchorage is the FRP U-wrap for anchoring longitudinal FRP flexural strengthening systems. U-wraps can function as additional shear strength for the beam or simply as a method for anchoring the flexural strengthening system. It is important to note that strengthening systems will either be bond critical or contact critical. Bond-critical applications rely heavily on the bond between the FRP system and the concrete substrate, while contact-critical applications do not (ACI Committee 440, 2017). Bond-critical applications include most beam flexural and shear strengthening systems. Therefore, anchorage should always be considered for these applications so that the strength of the FRP is fully utilized. Contact-critical applications include column confinement, which typically does not require the use of anchorage (ACI Committee 440, 2017). Refer to Section 2.1.7 for more information on anchorage techniques.

Through relatively recent research endeavors, the understanding of the shear and flexural behavior of concrete elements strengthened with FRP continues to expand. To provide an example of relevant experimental programs examining FRP systems, several studies were summarized in Table 2.2. Overall, research indicates that FRP is an effective strategy for increasing the strength and/or stiffness of multiple types of structural members, especially under normal loadings conditions.

TABLE 2.2
Strength and Stiffness Improvements Based on Experimental Programs Reported in the Literature

Year	Author(s)	Description of Work	Strengthening System	Strength Increase	Effects on Stiffness
1999	Naaman et al.	Shear strengthening of RC rectangular and T-beams subjected to ultimate loading conditions	CFRP plates and sheets	At least 30%	40% higher ultimate deflection in one case
1999	Pantelides et al.	Retrofit of a bridge bent subjected to in-situ quasi-static cyclic lateral loading conditions	CFRP sheets	16% (lateral load capacity increase)	Not explicitly stated
2002	Alagusundaramoorthy et al.	Flexural strengthening of RC beams subjected to service and ultimate loading conditions	CFRP fabric and anchored/ unanchored sheets	At least 40%	5%–25% reduction in deflection at failure
2002	Banthia et al.	Rehabilitation of deteriorated RC channel beams (laboratory testing and in-situ load tests)	GFRP wrap and GFRP spray-on	33% using wrap and 96% using spray (during laboratory testing)	30%–34% reduction in deflection (during load testing of bridge retrofitted with GFRP spray-on)
2003	Hassan & Rizkalla	Flexural strengthening of RC beams subjected to ultimate loading conditions	NSM CFRP	Up to 53%	“Substantial” stiffness increase
2003	Klaiber et al.	Flexural strengthening of damaged bridge girders (field tested before/after)	CFRP strips	Portion of strength restored	Deflections reduced by as much as 20%
2004	Green et al.	Flexural strengthening of prestressed concrete AASHTO girders subjected to ultimate loading conditions	CFRP laminates	At least 90.9% of original capacity regained	Deflections reduced by at least 23%
2004	Wipf et al.	Flexural strengthening of damaged prestressed concrete girders (laboratory testing and in-situ load tests)	CFRP sheets	31%–46% cracking load increase (for laboratory tested specimens)	Deflections reduced by as much as 20% (for in-situ testing)
2009	Quinn	Shear strengthening of RC T-beams subjected to loadings with different shear span-to-depth ratios	CFRP laminates with anchors	30%–40%	Not explicitly stated
2011	Kim	Shear strengthening of RC T-beams	CFRP sheets with anchors	30%–50%	More shear deformation capacity
2011	Kim et al.	Repair and strengthening of lap splices in large-scale RC columns subjected to monotonic and cyclic loading conditions	CFRP jackets with anchors	13%–56% (peak strength increase)	2.3%–10% peak drifts
2011	Satrom	Shear strengthening of RC T-beams subjected to fatigue and static loading	CFRP laminates with anchors	20%–30%	Smaller crack widths and lower steel strains
2016	Barros et al.	Shear strengthening of relatively deep RC T-beams	NSM-CFRP laminates	66%–81% (maximum load increase)	Increase in stiffness (after shear crack formation)
2016	Shekarchi	Shear strengthening of RC pile cap girders subjected to in-situ loading conditions	CFRP strips with anchors	22% (bidirectional FRP), 56% (unidirectional FRP)	Shear stiffness increased

TABLE 2.2
(Continued)

Year	Author(s)	Description of Work	Strengthening System	Strength Increase	Effects on Stiffness
2017	Chennareddy & Taha	Flexural strengthening of RC beams using two methods subjected to static loading to failure	U-wrap plus NSM CFRP	Up to 77%	Not explicitly stated
2017	Saljoughian & Mostofinejad	Strengthening of RC columns (using corner strip-batten method)	CFRP sheets	30.90%	67.9% (ductility increase)

3. CURRENT GUIDELINES

When implementing FRP systems in projects involving concrete bridge repair and strengthening, designers and contractors should consult the current guidelines applicable to their individual contributions to the project. The introduction of current guidelines in this chapter is primarily focused on the use of externally bonded and near-surface-mounted FRP systems used in repair and strengthening applications of in-service structures. Several design guidelines are available for the application of fiber reinforced polymer to existing concrete structures. The two most commonly used guides in the United States are from ACI and AASHTO. ACI developed the document *ACI 440.2R-17, Guide for the Design and Construction of Externally Bonded FRP Systems* (ACI Committee 440, 2017). The guide from AASHTO is titled *Guide Specification for Design of Externally Bonded FRP for Repair* (AASHTO, 2012). These guidelines from ACI and AASHTO contain repair procedure recommendations and design strategies for the use of FRP systems on various concrete structures for flexural and shear strengthening as well as column confinement. Other relevant documents from ACI that may be beneficial to understanding and implementing FRP repair and strengthening applications include the following.

- *ACI 440R-07, Report on Fiber-Reinforced Polymer (FRP) Reinforcement for Concrete Structures* (ACI Committee 440, 2007).
- *ACI 440.3R-12, Guide Test Methods for Fiber-Reinforced Polymer Composites for Reinforcing or Strengthening Concrete and Masonry Structures* (ACI Committee 440, 2012).
- *ACI 440.8-13, Specification for Carbon and Glass Fiber-Reinforced Polymer Materials Made by Wet Layup for External Strengthening of Concrete and Masonry Structures* (ACI Committee 440, 2013).

Guidelines on surface preparation of the concrete substrate are presented by the International Concrete Repair Institute (ICRI) in *Guideline No. 310.2R-2013, Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, Polymer Overlays, and Concrete Repair* (ICRI, 2013).

For the sake of conciseness, the most relevant guide documents, including the current guidelines from INDOT, are further discussed in Appendix A.

4. REVIEW OF BRIDGES WITH FRP REPAIRS

4.1 National and International Studies

Several examples of successful applications of FRP systems for repair and/or strengthening of deficient bridges in the United States and around the world have been reported in the literature. Four such examples include the Buhung Bridge in Gyeonggie, South Korea (Jung et al., 2017); Bridge A10062 on Interstate 44 in St. Louis County, Missouri (Tumialan et al., 2001);

the Wynantskill Creek Bridge in South Troy, New York (Hag-Elsafi et al., 2001, 2003); and the Uphapee Creek Bridge in Macon County, Alabama (Carmichael & Barnes, 2005). Case studies for each of these bridges are presented in Appendix B. These studies were chosen because they each describe a different application of FRP for various damage scenarios and installation procedures on in-service bridges. Repaired members were monitored to assess and verify the effectiveness of the repair. Collectively, the studies give a comprehensive glimpse of the many potential uses of FRP in civil infrastructure strengthening and rehabilitation.

4.2 Indiana Case Studies

4.2.1 Background

In the state of Indiana, there are over 19,000 bridges, of which more than 9,000 have either reinforced or prestressed concrete superstructures (National Bridge Inventory, 2019). The primary longitudinal members of these concrete bridge superstructures typically consist of box beams, I-beams, tee-beams, channel beams, and arches, among other types. Many of these bridges have experienced damage and deterioration over the years, and some have received a form of FRP repair or retrofit. With the help of INDOT, these bridges were identified, and a list was generated that consisted of 17 bridges within the state of Indiana. As the research team proceeded with the case studies, additional bridges in the state that have received FRP repairs were discovered. The research team visited and evaluated all the bridges in Indiana known to have been repaired or retrofitted with FRP. Other FRP-repaired bridges, however, are likely to exist in the state.

A total of 18 bridges were selected to represent the inventory of fiber reinforced polymer repairs in the state of Indiana. These 18 bridges are listed below. To accompany this list, the location of each bridge included in the case study is indicated in Figure 4.1.

1. Glendale Road at Aikman Creek
2. 116th Street at Keystone Parkway
3. I-65 Northbound at I-65 Southbound Ramp
4. Fruitridge Avenue at I-70 Eastbound/Westbound
5. County Road 275 East at I-74 Eastbound/Westbound
6. State Road 9 Northbound/Southbound at Salamonie River
7. State Road 9 at Catfish Lake
8. County 200 East at I-69 Northbound/Southbound
9. I-65 Northbound at I-70 Westbound Ramp
10. County Road 400 East at I-70 Northbound/Southbound
11. Waldron Road/County Road 600 East at I-74 Eastbound/Westbound
12. State Road 933 at Saint Joseph's River
13. I-94 Westbound at US 20/Willow Creek/CSX RR
14. State Road 101 at State Road 156 and Markland Dam
15. US 30 Eastbound/Westbound at Blue River/Park Drive
16. Old State Road 132 at I-69 Northbound/Southbound
17. County Road 28 at I-69 Northbound/Southbound
18. Interstate 70 near Madison Avenue Pier

4.2.2 Case Studies

Each case study involved the identification of the bridge that received an FRP repair, investigation of its history through archived inspection reports, evaluation of the methods of repair, and a site inspection to assess the repair. The goal of each case study was to document why the structure needed repair, what systems were implemented, how they were implemented, and if the FRP materials were properly utilized. The current condition of the FRP system was also evaluated to determine if it is still performing as intended. Inspections were primarily focused on observing the methods of FRP application. However, other aspects of the structure itself were identified as well (e.g., cracking, moisture intrusion, wearing surface and deck conditions, and deteriorated regions). Special consideration was given to characteristics of the FRP repair, such as debonding, voids, tears, FRP sheet layout, and the location of the repair on the structure. The details of each case study are provided in Appendix B, and the conclusions generated from the case studies are presented in Section 4.2.3.

4.2.3 Observations from Case Studies

After completing the series of case studies, five scenarios for which fiber reinforced polymers were being utilized in the state were identified as follows: concrete deterioration, fire damage, impact damage, crack growth mitigation, and general structural improvement. Concrete deterioration was the most common scenario for which FRP was used, with the primary scenarios being beam end deterioration and/or pier cap/column deterioration.

A recurring theme found from the case studies was the issue of leaking expansion joints. Due to the cold winters in Indiana, it is necessary to salt the roads to improve driving conditions. While this process is beneficial to the drivers on the roadway, it is detrimental to the underlying concrete structure. Salt-laden water slowly seeps through failed expansion joints and eventually makes contact with the concrete superstructure and substructure elements. Over time, this salt-laden water leads to the corrosion of the internal steel reinforcement. Once the steel corrosion begins, its volume expands, forcing the surrounding concrete to crack and spall. As seen from the case studies, beam end regions as well as pier caps and some columns located near these leaking expansion joints have experienced deterioration from this process. FRP has been extensively employed in Indiana in these regions of the bridge superstructure and substructure following repair of the concrete substrate. These FRP repairs were implemented to enhance the effectiveness of the concrete patchwork as well as extend the life of the entire repaired region.

FRP has also been used in Indiana after fire events. For certain fire-damaged bridges in the state, FRP was employed to improve regions of unsound, possibly delaminated concrete after experiencing high temperatures.

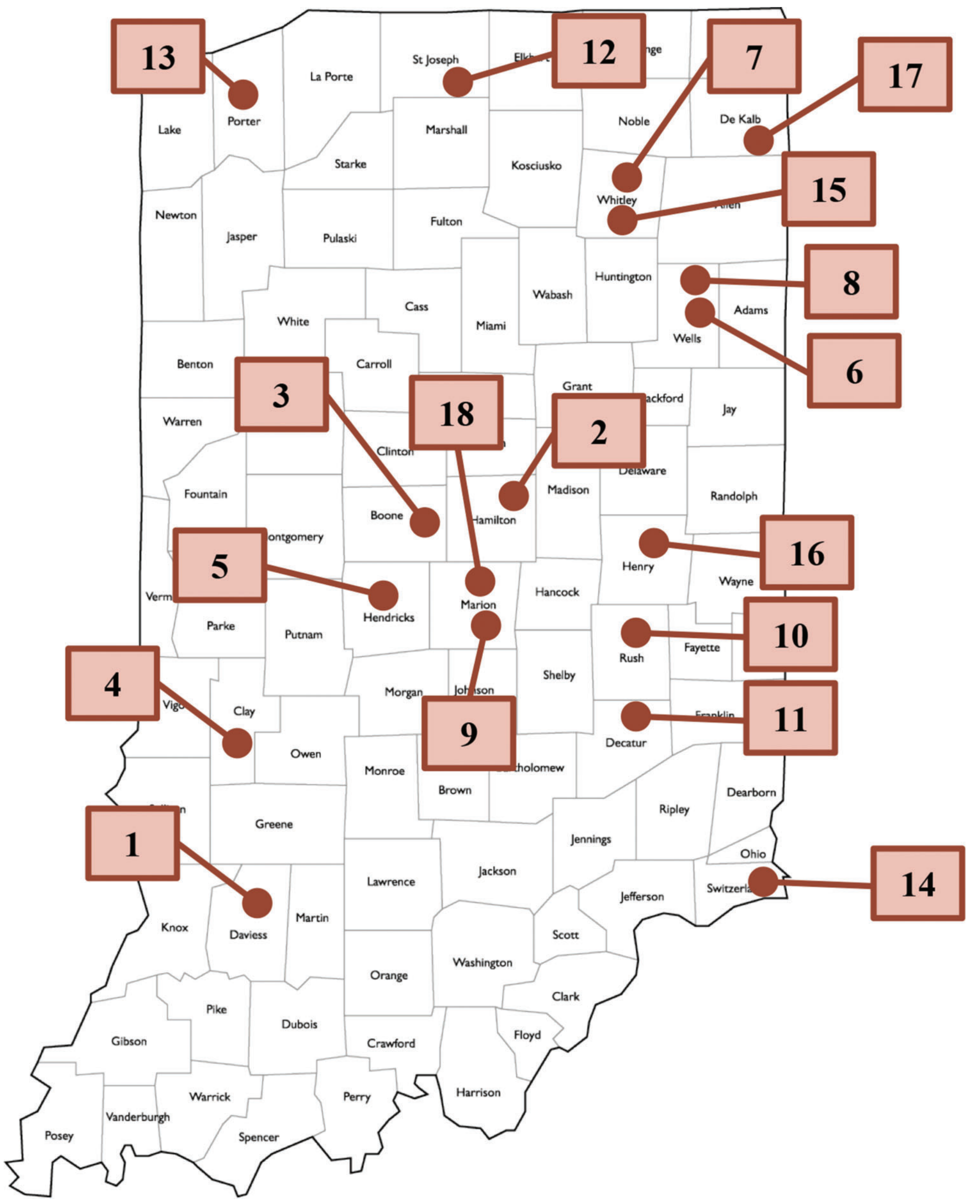


Figure 4.1 FRP-repaired bridges in Indiana (WorldAtlas, 2020).

One instance of impact damage to a bridge girder was identified through the case studies. The girder suffered no damage to the prestressing strands or apparent strength loss. The girder was wrapped with an FRP system at midspan where the impact occurred. It was assumed that since strengthening techniques are not to be utilized in Indiana at this time and that no longitudinal, flexural strips were placed under the wrap, that the repair to this impact-damaged girder was simply employed to ensure no residual concrete would spall onto traffic below. Other applicable reasons for this repair could be to protect the internal steel reinforcement and preserve the concrete and cross-section.

For what appears to have been crack growth mitigation, CFRP strips were utilized on the SR 933 bridge over St. Joseph's River. Once the cracks in the concrete arch rings were epoxy injected, single-ply, longitudinal CFRP strips were placed continuously along the entirety of the arch rings. These strips intersected the cracks at an approximate 90-degree angle.

Regarding general structural improvement, the US 30 bridges at Blue River/Park Drive was one case study in which FRP wraps were applied to the stem of solid stem piers that, according to past inspection reports, had not experienced any damage or deterioration. Following an extensive rehabilitation that involved the replacement of the deck, superstructure, and upper portion of the substructure, FRP wrap was placed around the bottom of the stems.

Common issues with the FRP repairs are mentioned in the case studies and summarized below. Refer to Chapter 6 for recommendations on improvements to circumvent these issues.

- Inconsistent layering of FRP sheets.
- Premature termination of FRP sheets near support locations or points of intersection of bridge elements.
- Improper epoxy quantities.
- Uneven distribution of epoxy.
- Inconsistent surface preparation.

Inconsistent layering of FRP sheets refers to the manner in which FRP sheets overlap each other on the repaired element. As long as the overlapping is sufficient to ensure chlorides are unable to penetrate under the FRP wrap, this issue mainly concerns aesthetics. If overlapping is more than what is sufficient or FRP manufacturer-specified, material waste becomes an issue. Premature termination of FRP sheets near supports or points of intersection of bridge elements can allow chloride entrance behind the FRP wrap. Furthermore, when FRP repair systems are applied to the concrete surface, adjacent areas of exposed concrete may receive chlorides at an accelerated rate due to the durability and resistance of the FRP. Therefore, prematurely terminating FRP sheets at intersection points and support locations can result in an exposed region, which may already contain some deterioration, to deteriorate more rapidly. Issues relating to epoxy use can result in major structural problems if not properly addressed. When an insufficient amount of epoxy is used to saturate sheets, proper bond between the elements within the FRP (i.e., between the fibers) is not achieved, thus resulting in insufficient stress transfer. Another important aspect of sufficient epoxy use is that the epoxy ensures adequate protection of the high-strength fibers within the matrix, as the epoxy is the primary source of protection for the fibers from environmental factors. Finally, inconsistent surface preparation is detrimental to an effective FRP repair or strengthening application. Various inspections showed that the concrete surface underlying the FRP repairs was not uniform. These regions of non-uniformity of concrete can cause voids or separation of the FRP system from the concrete substrate.

5. INDUSTRY SURVEYS

5.1 Overview

Following the compilation of current knowledge from the literature review, an industry survey was created that targeted engineers who have used FRP repair and/or strengthening systems in their designs. The surveys provided a direct approach for gathering information to aid in determining successful practices, the general level of familiarity with FRP systems, and what guidance is needed for the routine application of FRP in Indiana. Two versions of the survey were generated. One survey was developed for neighboring Midwestern states, while another survey was written for engineers in the state of Indiana. The two versions shared several similar questions. The FRP survey distributed to Midwestern states, however, contained questions related to practices and experiences in these states. The survey distributed to engineers in Indiana was comprised of questions related to the current use of FRP in Indiana as well as the perception engineers have with respect to FRP use. The results of the surveys were used by the research team to deliver relevant guidance for the use of FRP systems in Indiana. Copies of the survey responses can be found in Pevey (2018).

5.2 Midwestern FRP Survey

5.2.1 Introduction

The Midwestern FRP survey was distributed in November 2017 to engineers throughout the Midwest who participate in the Midwest Bridge Preservation Partnership (MWBPP). Responses were received from five state DOTs, referred to herein as States A, B, C, D, and E. The goal of the Midwestern FRP survey was to gauge the current state of knowledge of FRP use for concrete bridge repair in states that share similar climate and bridge deterioration issues as Indiana. The participants were asked questions about the following topics.

- Experience with FRP
- Criteria for assessing suitability of FRP for repair
- Guide and reference use
- Concrete substrate repair and surface preparation methods
- Detailing at bridge element intersection points
- End region repair
- Anchorage
- Durability
- Quality assurance/quality control and post-repair monitoring
- Constructability
- Contractor certification/training

The results of each question on the survey are provided in Appendix C, and the conclusions generated from the survey questions are presented next in Section 5.2.2.

5.2.2 Conclusions

Based on the results from the five Midwestern state DOTs (see Appendix C), FRP has been used on a relatively small number of bridges. The dominant applications of FRP systems include beam shear strengthening and column confinement. The application of FRP sheets through both wet- and dry-layup techniques have been used in Midwestern states. Repair durability appears to be sufficient based on the life of the repairs reported by the survey participants. Some criteria for determining the suitability of FRP for repair and/or strengthening concrete bridge elements include ensuring a sound concrete substrate, minimal existing crack widths, little to no exposure of internal steel reinforcement, and sufficient capacity of adjacent members. For design and construction recommendations, the guidelines from ACI 440.2R-17 and the AASHTO (2012) FRP guide specifications are typically used. Concrete surface preparation techniques are typically referenced from ICRI Guideline No. 310.2R-2013. Methods of concrete repair used by the Midwestern states include handheld breaking, scarifying, sandblasting after chipping and breaking, and saw-cutting. Methods of surface preparation include high-pressure water-jetting, abrasive blasting, grinding, needle scaling, and vapor blasting. Issues with beam end deterioration appear to prevalent in Midwestern states, typically caused by leaking expansion joints. No detailed methods of beam end deterioration repair methods were described. Further research into this topic, however, is being completed by State C. According to some survey participants, the determination of appropriate anchorage techniques is primarily conducted by the supplier of the FRP system. Anchorage techniques used by some Midwestern states include FRP spike anchors for anchoring FRP shear reinforcement and FRP U-wraps for anchoring FRP flexural reinforcement. In some cases, FRP systems have been used for the sole purpose of increasing the durability of concrete bridge elements. According to the survey participant from State D, no durability issues have been noted for pier cap and column protection systems in any inspection reports (where some applications have been in place for approximately 15 years). Visual and acoustic hammer sounding (tap) tests are used for non-destructive evaluation of FRP repairs by State D. These NDE methods are conducted during the normal bridge inspection programs (typically every 2 years). Common issues found during the FRP design and/or application process included lack of drawings from the consultant, lack of training of the field staff, and the formation of bubbles in the FRP systems.

5.3 Indiana FRP Survey

5.3.1 Introduction

The Indiana FRP survey was distributed in January 2018 to engineers at consulting firms and INDOT who have had experience with FRP or have used FRP in rehabilitation projects in the past. Responses to the Indiana survey were received from four participants. Two participants were from agencies within INDOT, one participant was from a consulting firm, and one participant did not specify an organization to which they were associated. The objective for the Indiana FRP survey was to determine the level of knowledge and interest that engineers in the state currently have regarding the use of FRP. The questions in this survey were aimed toward determining how designers can more effectively and efficiently implement fiber reinforced polymers into their projects. Participants were first asked about their experience, if any, with the design and/or application of FRP systems to concrete bridges. Next, participants were asked if they have encountered any specific issues with constructability or long-term performance of FRP systems. Finally, participants were asked to discuss their current problems when implementing FRP into their repairs, with the goal of generating feedback from Indiana engineers and their expectations for the development of the FRP guidebook.

5.3.2 Survey Results

5.3.2.1 Experience with FRP. Two of the four survey participants indicated that they have had experience with FRP systems. The survey participant from the consulting firm indicated that the firm had specified the use of FRP systems for approximately 10 bridges. The consulting engineer indicated that FRP systems had been used for the purposes of beam end repair, pier cap repair, and column confinement. Over 20 bridges that have received FRP repairs were reported by one INDOT participant from the Greenfield District, and two bridges that received FRP repairs were reported by the other INDOT participant. One INDOT participant noted the use of FRP systems for column confinement and "wrapping the underside of an arch bridge." The INDOT participant from the Greenfield District stated that the FRP systems that have been implemented in their unit were for "covering and holding concrete patches."

5.3.2.2 Constructability issues and long-term performance. Two participants, one who indicated experience with FRP and one who did not, noted issues with long-term performance of FRP systems. The participant who indicated no experience with FRP described evidence of water seeping behind the FRP repairs on a pier, possibly causing "more rapid deterioration" to the underlying concrete. The participant having experience with FRP indicated that some areas of wrapping on a concrete arch bridge are separating from the substrate. No specific issues with constructability were noted in the survey responses.

5.3.2.3 Guidebook expectations. All four participants responded to this question. Responses included the following statements.

- "Detailed repair procedures and training in design of FRP systems. FRP has very different strength characteristics compared to concrete."
- "When is it appropriate to specify FRP for strengthening purposes since INDOT currently only specifies its use for repair purposes and not strengthening?"
- "Guidance on typical applications, reasonable strength gains, and what to avoid would be good information."
- "Suitable applications for the product."

6. SUMMARY, CONCLUSIONS, RECOMMENDATIONS, IMPLEMENTATION, AND BENEFITS

6.1 Summary

To identify the state-of-the-art in bridge repair and strengthening using FRP systems, a literature review was conducted. Using the information gathered during the literature review, a synthesis study was performed in order to achieve several important objectives that would help relate FRP repair and strengthening techniques to application in Indiana. Descriptions were provided that include details of the materials used in FRP systems for the repair and/or strengthening of bridges, FRP systems used for flexural and shear strengthening, anchorage techniques for externally bonded FRP, installation procedures, and the behaviors of members strengthened with FRP. Furthermore, an introduction to current guide documents for the design and implementation of FRP systems for concrete structures was provided.

In addition to generating a knowledge base of current practice used around the country, the research team needed to review the FRP-related repair work that has been completed in Indiana. Therefore, a series of case studies was conducted to evaluate existing FRP repairs in the state. This evaluation consisted of bridge inspections to assess repair integrity and to note any issues as well as a review of INDOT database documents (e.g., rehabilitation plans and archived inspection reports) to ascertain the causes of damage or deterioration that resulted in the need for the repair. Finally, industry surveys were distributed to neighboring Midwestern states and to Indiana engineers. The goal of each survey was quite different. The Midwestern survey was used to determine current practices of FRP use in states with similar climate and bridge deterioration issues, while the Indiana survey targeted Indiana engineers to gauge knowledge and interest related to FRP use.

6.2 Conclusions

Conclusions relating the use of FRP repair and/or strengthening methods to application in Indiana were generated from the literature review and synthesis study. The objectives of these tasks were to address concepts including (1) materials and application techniques, (2) anchorage techniques, (3) flexural and shear behavior of members strengthened with FRP, and (4) strength and stiffness increases provided by FRP. Additional information related to durability of concrete structures that have received FRP systems as well as quality assurance/quality control procedures is provided in Pevey (2018). The conclusions based on the literature review and synthesis study presented herein are as follows.

1. For the repair of deteriorated concrete bridge members, wet- or dry-layup sheets are typically employed, depending on the geometry of the repaired-elements. Some common strengthening systems include near-surface-mounted (NSM) strips or bars and externally bonded sheets. For more severe levels of damage or large imposed demands, prestressed NSM or prestressed externally bonded systems can be implemented to achieve improved serviceability of beams. Refer to Section 2.2.1 for more information on installation procedures.
2. A common type of anchorage is the FRP fan anchor, or spike anchor. Its major benefits include compatibility with the FRP strengthening system, ability to anchor flexural or shear strengthening systems, corrosion resistance, and ease of constructability. Another popular form of anchorage is the FRP U-wrap for anchoring longitudinal FRP sheets used for flexural strengthening. Refer to Section 2.1.7 for more information on anchorage techniques. Further consideration of FRP U-wraps as anchorage is provided in volume 2 of this report.
3. Past research has indicated that the application of FRP for the flexural strengthening of beam elements is a viable technique. When using externally bonded FRP or near-surface-mounted FRP for these applications, proper anchorage or embedment should be provided. Research into the use of externally bonded FRP for shear strengthening has also resulted in recommendations to ensure the strengthening measures are successful. The use of U-wraps with fibers oriented vertically on the sides of the beam member and anchored using FRP anchors have provided satisfactory results.
4. Using the body of knowledge gathered through the literature review, a summary table was generated and presented in Section 2.3. The research summarized in this table indicates that FRP is an effective strategy for increasing the strength and/or stiffness of multiple types of structural concrete members.

After completing a series of case studies evaluating FRP repair applications in Indiana, five reasons for which fiber reinforced polymers have been used in the state were identified as (1) concrete deterioration, (2) fire damage, (3) impact damage, (4) crack growth mitigation, and (5) general structural improvement. Reinforced concrete deterioration was the most common reason for which FRP was applied, with the primary scenarios being beam end deterioration and/or pier cap/column deterioration. Observed issues with the FRP repairs are mentioned in the case studies and included inconsistent layering of FRP sheets, premature termination of FRP sheets near support locations or points of intersection of bridge elements, improper epoxy quantities, uneven distribution of epoxy, and inconsistent surface preparation. Refer to Section 4.2.3 for more information on the observations gathered from the case studies.

Industry surveys were distributed to neighboring Midwestern states and Indiana engineers with the goal of determining the current state of practice when implementing FRP systems in nearby states as well as gauging knowledge and interest of Indiana engineers concerning FRP repair and strengthening methods. Based on the responses from the five Midwestern states, the dominant applications of FRP systems include beam shear strengthening and column confinement.

Some criteria for determining the suitability of FRP for repair and/or strengthening concrete bridge elements include ensuring a sound concrete substrate, minimal existing crack widths, little to no exposure of internal steel reinforcement, and sufficient capacity of adjacent members. For design and construction recommendations, the guidelines from ACI 440.2R-17 (ACI Committee 440, 2017) and the AASHTO (2012) FRP guide specifications are typically used. Concrete surface preparation techniques are typically referenced from ICRI 310.2R-2013 (ICRI, 2013). Methods of concrete repair used by the Midwestern states include handheld breaking, scarifying, sandblasting after chipping and breaking, and saw cutting. Methods of surface preparation include high-pressure water-jetting, abrasive blasting, grinding, needle scaling, and vapor blasting. According to some survey participants, the determination of appropriate anchorage techniques is primarily conducted by the supplier of the FRP system. Anchorage techniques used by some Midwestern states include FRP spike anchors for anchoring FRP shear reinforcement and FRP U-wraps for anchoring FRP flexural reinforcement. The use of visual and acoustic hammer sounding (tap) tests for non-destructive evaluation of FRP repairs was indicated for one state. These NDE methods are conducted during the normal bridge inspections (typically every 2 years).

6.3 Recommendations

Based on the findings from past research, successful implementation of FRP in other states (see Section 4.1 and Appendix B), and the conclusions from the experimental programs presented in volume 2 of this report, it is recommended that the *Indiana Design Manual* be updated to allow the use of FRP for strengthening purposes. Language in the *Indiana Design Manual* should allow designers to consider FRP systems for flexural strengthening of impact-damaged or otherwise structurally deficient beams, shear strengthening of beams with end-region deterioration or shear deficiencies, and flexural and/or shear strengthening of pier caps. The use of FRP for the confinement of columns and the strengthening of columns under axial compression and combined axial compression and flexure, although not explicitly addressed in this report, should also be considered. It should be noted in the *Indiana Design Manual* that the strength of the existing structure/structural elements must satisfy the limits proposed in the relevant design guidelines. The guidelines presented in ACI 440.2R-17 and the AASHTO (2012) FRP guide specifications should be referenced in the design manual as the standard that FRP designs should follow in Indiana. Reference to ICRI Guideline No. 310.2R-2013 should also be made regarding surface preparation. Scenarios when the use of FRP is not appropriate should also be noted in the design manual. For strengthening, if the existing strength of the structure does not satisfy the relevant requirements, FRP should not be used to strengthen the structure. Furthermore, designers should keep in mind that if continuity cannot be achieved within the FRP repairs (i.e., continuity of sheets near points of intersection of bridge elements), an alternative approach to repair should be considered.

To mitigate issues caused by the improper use and distribution of epoxy, the use of pre-saturated sheets can be considered when implementing FRP sheet systems. These pre-saturated systems may or may not require the application of additional resin on site (ACI Committee 440, 2017). In some circumstances, they may help to prevent issues such as uneven distribution of epoxy in the sheets. Finally, inconsistent surface preparation was seen in some repair applications in Indiana. Consistent application of the recommendations from ICRI Guideline No. 310.2R-2013 will help to eliminate this issue (ICRI, 2013).

Adopting practices identified from the results of the Midwestern survey, special long-term considerations for the inspection of FRP systems is recommended. Visual and tap tests of the FRP systems can be performed during the normal bridge inspection routine (i.e., every 2 years) to verify the integrity of the system.

Based on the case studies and consideration of the common deficiencies found in current FRP repairs, one of the more common issues with repairs was the premature termination of FRP sheets near support locations or points of intersection of bridge elements. This issue appears to stem from the common problem of irregular geometries or geometric discontinuities. The experimental phase of this research project presented in volume 2 includes procedures for implementing a wet-layup system to bridge elements, specifically prestressed I-beams, with geometries that present challenges. Another concept investigated in the experimental program is the application of FRP strengthening systems to box beams. Because the majority of box beam bridges in Indiana consist of adjacent beam systems, access to the sides of the beams is not possible. Therefore, U-wrap technology is not applicable and other forms of anchorage or strengthening systems should be considered for their applicability to deficient adjacent box beam bridges. Finally, for scenarios where beam end deterioration is present near a leaking expansion joint, options for repairing the girder ends are investigated in the experimental program in volume 2. Nevertheless, creation of a semi-integral abutment to eliminate the leaking joint altogether in these cases should be considered as an option.

6.4 Implementation

Based on the findings of the research, updates to the *Indiana Design Manual* to allow the use of FRP for strengthening purposes is recommended. Furthermore, in addition to the recommendation provided in the previous section, an FRP guidebook is being provided to INDOT. The guidebook includes an introduction to FRP systems and viable applications for Indiana bridges, comprehensive guidance for the implementation of FRP systems in Indiana, and recommendations for the *Indiana Design Manual*. The guidebook also directs engineers to the proper published guidelines for the successful design and installation of FRP systems.

6.5 Benefits

The benefits of this research are far reaching, as the project aligns with the economic competitiveness, asset sustainability, and innovation and technology strategic priorities of the *Indiana Department of Transportation 2019 Strategic Plan* (INDOT, 2019). The information contained in this report will help facilitate the implementation of FRP strengthening and repair systems, and the implementation of these systems is expected to result in significant time and cost savings. The establishment of proven repair procedures will reduce installation errors in the field, saving labor time and reducing material costs. Additionally, the development of effective repair techniques will provide INDOT with sustainable and cost-effective alternatives to replacing aging and deteriorated bridges. The application of FRP for the strengthening of Indiana bridges introduces an innovative technique to increase the service life of bridge infrastructure.

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APPENDICES

Appendix A. Current FRP Guidelines

Appendix B. Case Studies of Bridges with FRP Repairs

Appendix C. Midwestern FRP Survey Results

APPENDIX A. CURRENT FRP GUIDELINES

Within the following sections, the current guidelines from INDOT are first described. The guidelines from ACI will then be discussed followed by the AASHTO guidelines. To conclude, a brief overview of surface preparation strategies recommended by ICRI are provided. For the sake of conciseness, in-depth guidance for the methodology of each design guide will not be discussed. The goal for this chapter is to deliver assistance to engineers for navigating each of the guides.

A.1 Indiana Department of Transportation (INDOT)

The Indiana Department of Transportation currently does not have detailed design guidelines for the use of fiber reinforced polymers in structural strengthening applications. The use of FRP in Indiana has been limited to only preservation and seismic retrofit and has primarily been used to increase durability and for waterproofing. In Chapter 412 of the *Indiana Design Manual* (INDOT, 2013), the topics relating to bridge rehabilitation are presented. Fiber reinforced polymer composite materials are presented in Section 412-3.05(05). The manual states that "externally-bonded fiber reinforced polymer (FRP) composite materials or "fiber wrap" may be considered for bridge preservation and retrofitting. However, the Department currently does not allow the use of FRP systems to restore the structural capacity of bridge components." The manual explicitly allows fiber wrap to be used in conjunction with added confinement reinforcement as a seismic retrofit of columns, for concrete deterioration, or for vehicle impact to girders. Prestressed concrete girder preservation is described in Section 412-3.03(05). For girder repair, encasing the girder ends in concrete or fiber wrap or epoxy-injecting cracks and spalls followed by the application of fiber wrap are presented as some potential methods of preservation. It is explicitly stated in this section that the fiber wrap is to be used to encase deterioration and is not to be used for adding capacity.

The INDOT library contains a supplemental document for the application of FRP systems to bridges titled *Fiber Wrap Concrete Casing System* (INDOT, 2020). This document, referred to as Sample Unique Special Provision (USP) 702, provides an example provision to be added to the contract documents for bridge rehabilitation projects that will involve the use of fiber reinforced polymers. The casing system referenced in the document is specified as being an externally bonded fiber reinforced polymer composite system. The document contains sections related to materials and manufacturers, submittal requirements, construction requirements, field quality control procedures, method of measurement, and basis of payments.

In the Sample Unique Special Provision, it states that the materials to be used for patching or repair of the concrete member are to be as recommended by the manufacturer of the FRP system. Construction requirements in the Sample Unique Special Provision describe the required surface preparation, which includes removing any deleterious substances from the member and restoring the section to its original cross section or as specified in the plans. The field quality control section includes subsections related to installers, inspection, visual inspection and sounding, laboratory testing, substrate adhesion testing, repairs, and remedial measures.

A.2 American Concrete Institute (ACI)

Committee 440 of the American Concrete Institute developed a guide document to present design and construction guidance for the implementation of FRP systems to concrete structures. It is titled *ACI 440.2R-17, Guide for the Design and Construction of Externally Bonded FRP Systems* (ACI Committee 440, 2017). The first portion of the document describes the development of FRP systems and their current use within the concrete structural rehabilitation industry. Information on FRP constituent materials and mechanical properties is then presented. Construction recommendations for the application of FRP systems are given, including shipping/storing/handling, installation, inspection, and maintenance. The guide document contains design procedures that begin with general design recommendations, such as strengthening limits to check before designing FRP systems and an explanation of the material properties to use during the design process. The design methodologies in the guide document include those for flexural strengthening, shear strengthening, strengthening of members under axial force or combined axial force and flexure, and seismic strengthening. FRP reinforcement detailing recommendations, such as for lap splices and bond, are presented. The guide document concludes with design examples and an appendix that includes fiber material properties and a summary of standard test methods.

Before detailed design procedures for strengthening members for various types of loadings are presented in ACI 440.2R-17, general design considerations are described (see Chapter 9 of the guide) (ACI Committee 440, 2017). Basic principles are first introduced, such as the requirement for FRP systems to "be designed to resist tensile forces while maintaining strain compatibility between the FRP and concrete substrate." Designers should assume that FRP systems provide no resistance to compressive forces (i.e., any compressive strength of the material should be neglected). The design of FRP systems is based on limit-states-design principles, and the strength and serviceability requirements, including strength and load factors, of ACI 318 (ACI Committee 318, 2014) apply. Before designing a strengthening system, the existing strength of the structure should be evaluated. Primary information that needs to be gathered during the initial assessment of the structure should include existing load-carrying capacity, structural deficiencies and causes, and the condition of the concrete substrate. ACI 440.2R-17 imposes certain strengthening limits to circumvent undesired events such as collapse due to damage of the FRP system (ACI Committee 440, 2017). The sufficient level of existing strength of the structure is as follows (Eq. 9.2 of ACI 440.2R-17):

$$(\phi R_n)_{existing} \geq (1.1 S_{DL} + 0.75 S_{LL})_{new} \qquad \text{Equation A.1}$$

where:

ϕ = strength reduction factor from ACI 318
R_n = nominal strength of a member
S_{DL} = dead load effects
S_{LL} = live load effects

The designer should conduct additional analyses on the strengthened member to check overload conditions and ensure that a ductile flexural failure will occur rather than a brittle shear failure.

Design methodologies for flexural strengthening of both reinforced and prestressed concrete members are presented in Chapter 10 of ACI 440.2R-17 (ACI Committee 440, 2017). Here, both externally bonded FRP and NSM FRP are addressed. Design methodologies for shear strengthening are presented in Chapter 11. Next, the design of FRP systems for members under axial force or a combination of axial force and bending is given in Chapter 12. A detailed summary of design guidelines presented in ACI 440.2R-17 is provided in Pevey (2018).

A.3 American Association of State Highway and Transportation Officials (AASHTO)

The American Association of State Highway and Transportation Officials (AASHTO) developed a guide for the design and application of externally bonded FRP systems to reinforced concrete and prestressed concrete bridge elements titled *Guide Specification for Design of Externally Bonded FRP for Repair* (2012). The document contains information on surface preparation as well as inspection, evaluation, and acceptance of FRP systems following installation. The design methodologies include those for members under flexure, shear and torsion, and combined axial force and flexure. The document is to be used in conjunction with the most current *AASHTO LRFD Bridge Design Specifications* (2020). The guide incorporates design procedures from National Cooperative Highway Research Program (NCHRP) Reports 655 (Zureick et al., 2010) and 678 (Belarbi et al., 2011). NCHRP Report 655 delivers recommended design guidelines for externally bonded FRP systems, and Report 678 gives guidance on the design of FRP systems for the shear strengthening of girders. Loads and load combinations from *AASHTO LRFD* should be used for the design procedures presented in the guide.

To begin the process of designing FRP strengthening systems, the AASHTO (2012) guide gives general design considerations and limitations to the design procedures. A minimum capacity requirement is placed on bridge structural members that are to receive FRP strengthening systems. Externally bonded FRP strengthening systems can safely be applied to the member if the following equation is satisfied (Eq. 1.4.4-1 of AASHTO 2012):

$$R_r \geq \eta_i[(DC + DW) + (LL + IM)] \qquad \text{Equation A.2}$$

where:

R_r = factored resistance
η_i = load modifier taken as 1.0
DC = force effects due to component and attachments
DW = force effects due to wearing surfaces and utilities
LL = force effects due to live loads
IM = force effects due to dynamic load analysis

The design of FRP strengthening systems for members under flexure is presented in Section 3 of AASHTO (2012). The use of near-surface-mounted strengthening systems is not explicitly included in this section. The design of externally bonded FRP systems for strengthening members under shear and torsion is presented in Section 4. The design methodology given in this section was adopted from NCHRP Reports 655 (Zureick et al., 2010) and 678 (Kuchma et al., 2011) (see Section C4.1 of AASHTO, 2012). Section 5 then addresses design procedures for strengthening columns under axial compression, axial compression and bending, and axial tension. A detailed summary of design guidelines presented in AASHTO (2012) is provided in Pevey (2018).

A.4 International Concrete Repair Institute (ICRI)

The International Concrete Repair Institute developed *Guideline No. 310.2R-2013, Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, Polymer Overlays, and Concrete Repair* (ICRI, 2013) to present recommendations for the surface preparation of a concrete substrate prior to the installation of various overlays, including externally bonded FRP systems. Several types of protective systems (e.g., overlays and films) are presented in the document along with their typical thickness and appropriate concrete surface profile (CSP) to ensure effective bond of the protective system. The protective system corresponding to typical externally bonded FRP systems is the “thin film.” For thin films, a CSP 3 is to be used, which translates to a “light shotblast” according to ICRI (2013). Appropriate surface preparation methods that can achieve the desired CSP are presented in the document as well. According to the guideline, a CSP 3 can be achieved by acid etching, needle scaling, abrasive blasting, shotblasting, and high- and ultra-high-pressure water jetting. Molded replicas of the CSPs (i.e., CSP chips) can be ordered from ICRI if needed for reference in the field to assure the appropriate level of surface preparation has been achieved.

The risks to bond quality are also discussed in the ICRI (2013) guide. One risk to bond quality is that of microcracking (bruising), which creates a weakened layer of concrete caused by high-impact concrete preparation methods. When implementing any type of FRP system that is bond-critical, the phenomenon of microcracking should be avoided by using low-impact repair and surface preparation techniques. The surface preparation techniques with the lowest risk of inducing microcracking while still creating a CSP 3 include abrasive blasting, acid etching, water jetting, and shotblasting.

REFERENCES FOR APPENDIX A

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APPENDIX B. CASE STUDIES OF BRIDGES WITH FRP REPAIRS

B.1 National and International Case Studies

The following case studies have been summarized from the literature. The descriptions are based on the literature source(s) noted in each heading.

B.1.1 The Buhung Bridge (Jung et al., 2017)

The Buhung Bridge in Gyeonggi, South Korea, is a four-span bridge carrying two lanes of traffic. Its superstructure is supported by reinforced concrete T-beams. It is located near a military base and experiences frequent over-loadings from many large trucks and tanks. As a result, its original design became insufficient for its current imposed demands. An upgrade was implemented that involved a post-tensioned near-surface-mounted (NSM) system of carbon fiber reinforced polymer (CFRP) rods to improve the bridge capacity by about 30%.

The strengthening scheme involved an initial surface preparation that included cutting a groove in the concrete substrate for the placement of the NSM rods. Wider recesses were cut at the ends of these grooves to place post-tensioning anchorage devices. One end of the beam was designated as the dead, or anchored, end. The other end was the live end, where the jacking force was applied to impart the prestressing force to the CFRP rods. These anchorage devices were mounted into each recess using high-strength mechanical anchors. As illustrated in Figure B.1, the CFRP rod was installed along with the jacking apparatus, and the desired level of post-tensioning force was applied to the rod. Once jacking procedures were finished, the hardware required for the jacking operation was removed and the groove and recesses were grouted to complete the installation (Figure B.1).

Post-repair monitoring was performed to assess the performance of the structure with the retrofit. Dynamic load tests were conducted with a 30-ton truck to simulate the loading conditions. Maximum deflections of the strengthened beams decreased by 21% compared to the original unstrengthened beam. The researchers concluded that the strengthening scheme was effective from a serviceability perspective and that it provided the needed increase in capacity (Jung et al., 2017).

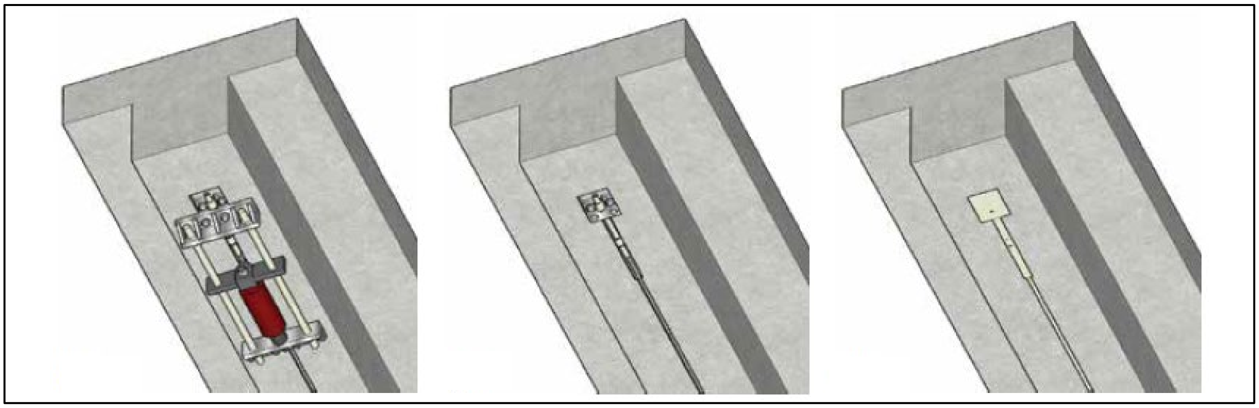


Figure B.1 Prestressed NSM strengthening system (from Jung et al., 2017).

B.1.2 Bridge A10062 (Tumialan et al., 2001)

At the interchange of Interstates 44 and 270 in St. Louis County, Missouri, an exterior prestressed concrete girder was impacted by an over-height truck. Upon removal of loose concrete, two prestressing strands were found to be exposed and fractured from the impact. Damage to the exterior girder is shown in Figure B.2. An externally bonded CFRP system was chosen for the retrofit of the damaged member, and the design of the system was carried out in accordance with ACI and AASHTO provisions.

Installation began with surface preparation that included restoration of damaged concrete with rapid setting, non-shrink, cementitious mortar. To improve the FRP wrap placement, the bottom edges of the girder flange were rounded. Sandblasting followed to expose aggregates and remove any loose concrete. An epoxy-based primer coat was applied and allowed to dry. To ensure the surface of the concrete substrate was completely uniform and free of small holes and imperfections, a thin layer of putty was then added. A dry-layup procedure was chosen to apply the CFRP sheets to the concrete surface along the tension (bottom) face. A layer of saturant was placed onto the concrete, followed by the CFRP sheet and a second layer of saturant. CFRP U-wraps were installed in a similar manner to the sides of the girder to anchor the flexural reinforcement. Finally, the completed installation was inspected for any defects. Design calculations indicated that the repair provided a factored nominal flexural strength similar to that of the undamaged girder (Tumialan et al., 2001).

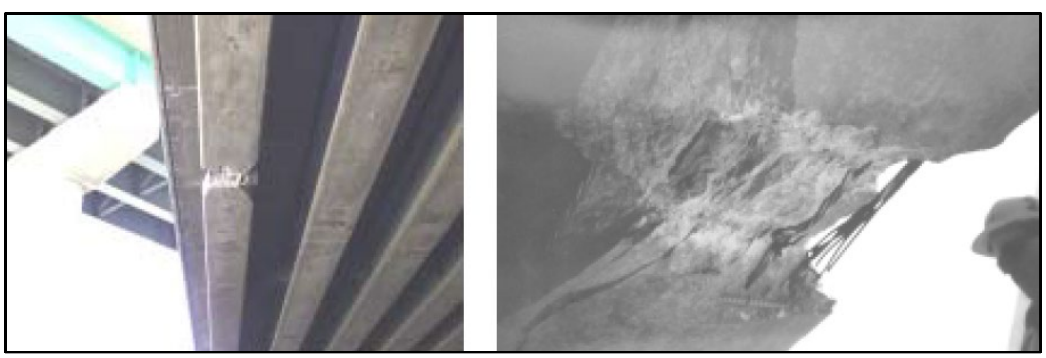


Figure B.2 Impact-damaged girder on Bridge A10062 (from Tumialan et al., 2001).

B.1.3 Wynantskill Creek Bridge (Hag-Elsafi et al., 2001 and 2003)

The Wynantskill Creek Bridge services State Route 378 in South Troy, New York. It is a simple span, reinforced concrete T-beam structure that carries five lanes of traffic. During a routine inspection of the bridge, the superstructure exhibited moisture and salt infiltration. Large areas containing freeze-thaw cracking and efflorescence were found on many of the beams, and some beams showed evidence of concrete delamination. The structure was rehabilitated using an FRP laminate system to confine the cracked concrete and increase the flexural and shear strength of the structure (Hag-Elsafi et al., 2001).

The strengthening system selected was an externally bonded CFRP system consisting of flexural and shear (U-wrap) reinforcement. Installation began with surface preparation by removing loose concrete and using a cement-based grout to fill cracks. A smooth, rounded finish was ensured at the sharp edges around beam corners, and the bridge underside was sandblasted and then pressure washed to remove laitance. A primer was then applied to the areas to be repaired once the beams were completely dry. Application of putty followed to ensure any small gaps or holes were filled and the surface was completely smooth. An epoxy resin was then added to the surface, the FRP laminates were placed, and a final layer of resin was applied over the laminates. To complete the installation, the repaired superstructure was painted for UV protection and to improve aesthetics (Hag-Elsafi et al., 2001). Figure B.3 shows one of the bridge girders that was strengthened.

Load tests were performed before the rehabilitation and approximately 10 days after completion of the FRP system installation. The testing program consisted of loading two lanes with four trucks, each with a weight of about 44 kips (196 kN). Several beams were instrumented to evaluate flexural and shear enhancements as well as laminate stresses to assess bond. Under the imposed service-like conditions, the retrofitted structure experienced reduced stresses in the primary longitudinal reinforcement and an improved transverse distribution of the load to the beams. The strengthening scheme was successful in regard to the intended repair and strengthening needs. It should also be noted that the total cost of the system was approximately $300,000 as opposed to the estimated structure replacement cost of $1.2 million (Hag-Elsafi et al., 2001). A supplemental investigation into this same bridge was conducted 2 years following the completion of the rehabilitation project (Hag-Elsafi et al., 2003). This separate investigation repeated the same load test previously conducted to examine the in-service performance of the retrofitted system. It was found that bond quality and system effectiveness did not diminish after 2 years of service. In addition to the load test, an infrared thermography camera inspection revealed that no significant delaminations were present within the system (Hag-Elsafi et al., 2003).


Figure B.3 Strengthening of a girder on the Wynantskill Bridge (Hag-Elsafi et al., 2003).

B.1.4 Uphapee Creek Bridge (Carmichael & Barnes, 2005)

The Uphapee Creek Bridge, also known as the War Memorial Bridge, is located in Macon County, Alabama. This structure is an 18-span, reinforced concrete bridge that was built in 1945. The original design was no longer adequate for the truck loads being imposed on it. The stresses and deflections experienced by the bridge were greater than expected during the original design. As a result, load limits were posted on the bridge. To increase the flexural capacity of the girders and eliminate the restrictions from the load posting, an FRP system was implemented. The section of the bridge chosen for rehabilitation was a three-span continuous portion, consisting of four variable-depth reinforced concrete T-beams.

The FRP system chosen for this strengthening project was a CFRP laminate strip system. The system design was completed with consideration of ultimate strength, anchorage, and serviceability. Strengthening configurations differed for exterior and interior girders as well as for end spans and the center span. The FRP strengthening systems were only designed for and installed in the positive moment regions of the reinforced concrete girders. These systems were comprised of longitudinal laminate strips bonded to the tension face of the girders. Standard concrete surface preparation techniques (i.e., sandblasting, grinding, and water blasting) were utilized to ensure proper bond of reinforcing materials. Cracks that could lead to premature failure of the FRP system (e.g., cracks that were 0.010 in. or wider) were injected with epoxy. Preparation of the FRP system was conducted on site each day, where epoxy batches were mixed daily and composites were cut to desired lengths from large rolls. The epoxy was applied to the concrete surface and the FRP strips, and the strips were then placed on the desired area of the girders. The application of a composite strip to a girder is shown in Figure B.4. During installation, the bridge was not closed to traffic. However, daily lane closures and traffic control measures were established to ensure all traffic was detoured as far from the target repair member as possible. Following the installation of the FRP system, inspections were conducted to assess bond quality. The tap test method was used for this procedure to locate voids or delaminations. No defects that required repair were found through the tap test evaluation.

Live load tests were performed on the Uphapee Creek Bridge before FRP installation, immediately after the FRP was applied, and six months after installation. The load tests were conducted by applying static and dynamic loads using two identical load test trucks. Following load tests after FRP installation, no significant evidence of delamination in the two instrumented girders was found. The authors describe further that the FRP installation was in good condition six months following installation; however, long-term testing should be conducted to ensure bond quality. Internal longitudinal steel reinforcement experienced an overall reduction in strain of about 5%. Finally, the integrity of the installation was found to be sound based on the testing completed six months after the installation of the FRP system (Carmichael & Barnes, 2005).

Figure B.4 FRP application to Uphapee Creek Bridge Girder (from Carmichael & Barnes, 2005).

B.2 Indiana Case Studies

The 18 case studies from bridges in Indiana mentioned in Chapter 4 are presented in this section. Each case study includes the bridge name, location, structure type, history and structural issues, construction information, inspection description, and repair description. The bridge name, location, and structure type were gathered from the Bridge Inspection Application System (BIAS), a bridge database system maintained by INDOT.

History and structural issues were obtained through an in-depth historical investigation of the inspection reports from the years prior to the FRP repair. The inspection reports were available through BIAS. The construction information includes details gathered from contract documents and bridge rehabilitation plans obtained through an online Electronic Records Management System (ERMS) used by INDOT. This information was investigated to gain a better understanding of the procedures taken from start to finish of each FRP repair. It was important to investigate the steps of concrete repair, surface preparation, and FRP system installation as well as the overall significance and purpose of the FRP installation to evaluate the integrity of the repaired surface and the effectiveness of the repair itself. Finally, the inspection description gives a brief description of the research team's site visit, and the repair description presents the findings and photographs from the visit.

The last case study described in Section B.2.18, pertaining to the repair of the bridge pier located at Interstate 70 near Madison Avenue in Indianapolis, is unique. The details presented include basic location and structural information, pre-repair evaluation including photographs and descriptions of deterioration, and a description of the processes involved in the application of the FRP repairs including photographs of the progression of work. The section concludes with observations of the repair three to four months after the work was completed.

B.2.1 State Road 933 over St. Joseph's River

Location: South Bend (La Porte District)

Structure Type: Reinforced Concrete Deck Arch (NBI 1-11)

Figure B.5 Panoramic view of the bridge facing east.

The bridge servicing SR 933 over St. Joseph's River is located in South Bend, Indiana, in the La Porte district. It is a three-span reinforced concrete deck arch bridge that carries four lanes of traffic.

History and Structural Issues: The following information was taken from various inspection reports found through the INDOT BIAS. These reports were generated by the following inspectors: FHWA (11/01/2007) (INDOT, 2007a), Wayne Skinner (06/21/2011) (Skinner, 2011), Bill Dittrich et al., (03/28/2013) (Dittrich et al., 2013), Bill Dittrich (12/04/2014) (Dittrich, 2014), and Brian Dilworth (08/17/2015) (Dilworth, 2015). The area of interest on this bridge is the superstructure elements, or more specifically, the arches supporting the roadway. Issues appeared to be cracking of the arch rings and some sag in the middle span. According to the November 2007 inspection report, the arch cracks were repointed and patched, and the arch condition rating was set at six. A major issue was also scour, as the bridge was considered "High Risk for Vulnerability for Scour." Permanent coffer dams were a portion of a 2006 rehab to help alleviate this issue. Underwater evaluation was also conducted for this structure. The condition rating of the superstructure fell to a two and the substructure was at a five in the June 2011 inspection report. The cracking of the arch rings was still prevalent, giving the superstructure such a low rating. It was explicitly stated in this report to immediately close the bridge, based on the superstructure remaining life. The inspection reports were not explicit, but FRP repairs were possibly made in 2012. These repairs also included epoxy injecting cracks in the underside of the arch rings and performing an impact echo survey to evaluate the presence of delaminations still left. The superstructure rating was increased to a four following this rehabilitation. Remaining life estimate was increased to 5 years.

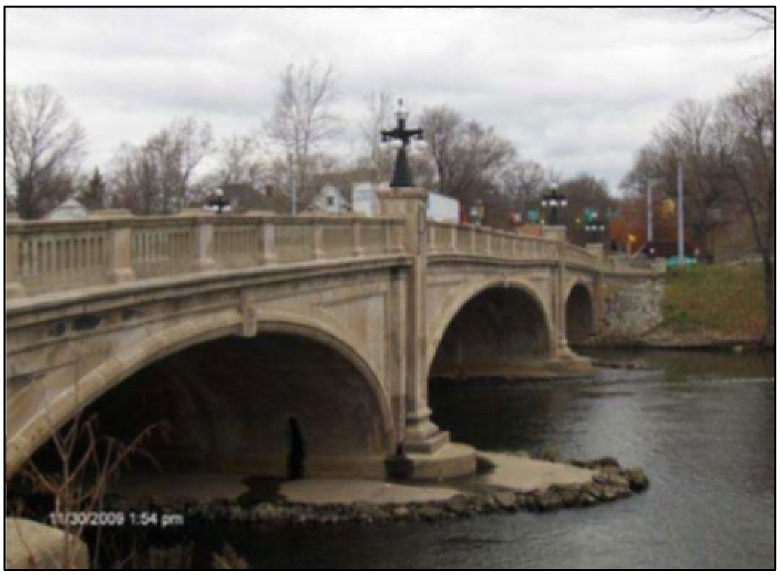


Figure B.6 East side of the bridge facing north (from Skinner (2011) inspection report).

Figure B.7 East Side railing facing north (from Skinner (2011) inspection report).

Figure B.8 Arch separation (from Dittrich et al. (2013) inspection report).

According to an August 2015 inspection report, the superstructure rating was raised to a five and no major changes were to be reported from the arch ring measuring. It was noted, however, that some CFRP strips were detached from the concrete arch in the middle span (totaling nine at the time).

Construction Information: According to the bridge rehabilitation plans (INDOT, 2012b), the work to be done to the underside of the arch rings included epoxy injecting all cracks, sandblasting and grinding all exposed surfaces, and applying wet layup CFRP strips, as described in the figure below.

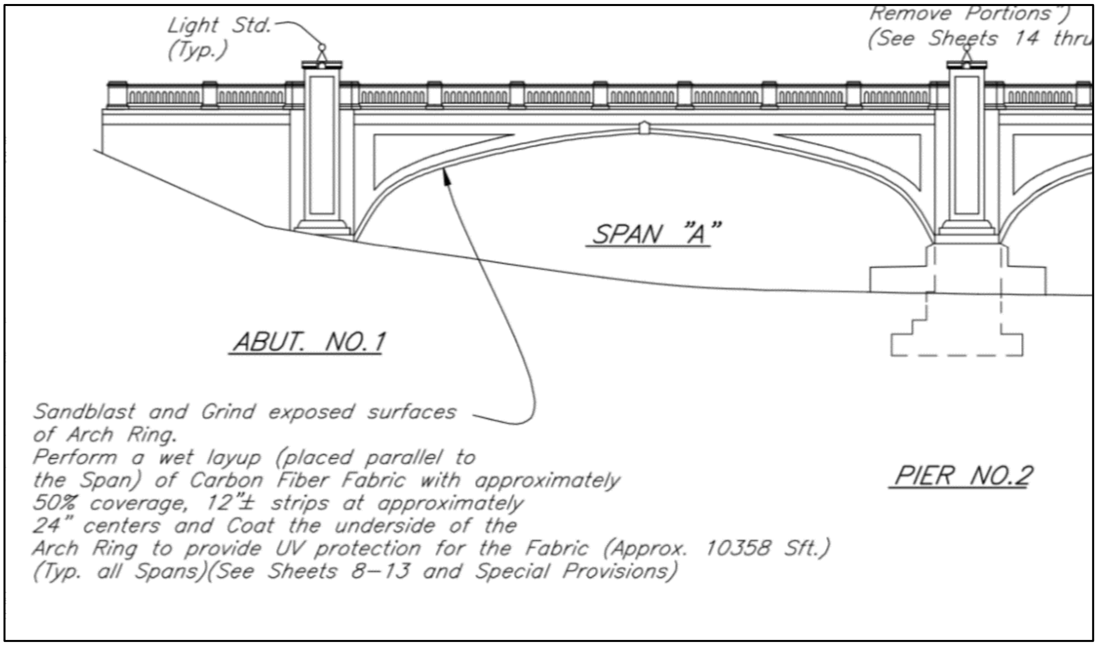


Figure B.9 FRP repair plans for arch rings (from INDO, 2012b).

These plans also included detailed crack mapping of the underside of all the arch rings alongside the strip plan for placement of the CFRP. The strips were to be continuous, single ply CFRP and intersect the cracks perpendicularly.

Inspection Details: Inspection was limited to only the southernmost span. Access was not possible at the northern span and the center span access was not possible without a marine vessel. Upon inspection, it was obvious that all the arch rings had experienced a repair using longitudinal, externally bonded CFRP strips.

Repair Description: All arch rings were covered with longitudinal CFRP strips where fibers were oriented parallel with the longitudinal axis of the superstructure. Strips were cut at an average width of 12 in. and spaced anywhere from 4- to 10-in. apart (sheets do not overlap). Sheets initiate and terminate approximately 7 ft from the base of the arch ring. No anchorage systems seem to be utilized, where the epoxy resin is the only bond between the FRP system and the concrete substrate. Several locations of inconsistent sheet width and spacing, inconsistent sheet lengths, uneven cuts made to the sheets, and some signs of debonding present at the ends of the sheets were observed. In addition to the issues noted from the field visit, the inspection reports noted that there was debonding present at the center span. However, this was not visible during the inspection made by the research team. Patches of CFRP were placed over old drainage pipe outlets that seem to have been filled with grout. All sheets were painted. This paint may not have been compatible with the CFRP sheet, as there were multiple locations of the paint peeling away from the sheet. There were some locations on the CFRP sheets that looked to be hairline cracks.

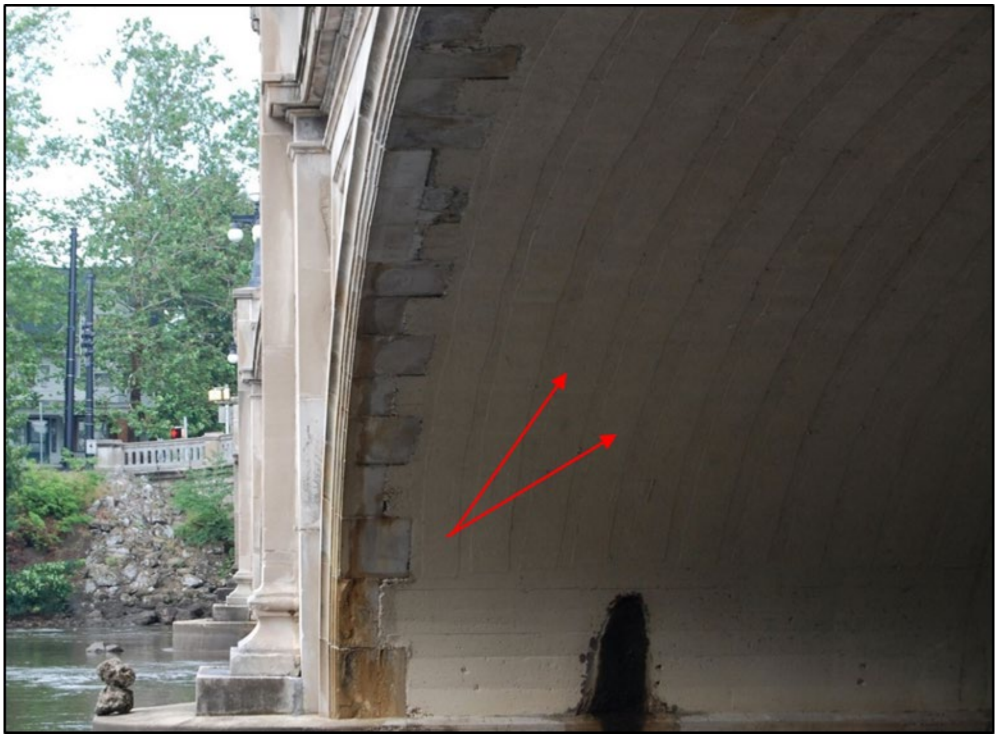


Figure B.10 CFRP strips on southern span.


Figure B.11 Close-up of cut made to CFRP sheet.

Figure B.12 Epoxy runoff.

Figure B.13 Debonding of CFRP on underside of arch ring at center span (from Dittrich (2014) inspection report).

Figure B.14 CFRP patching.

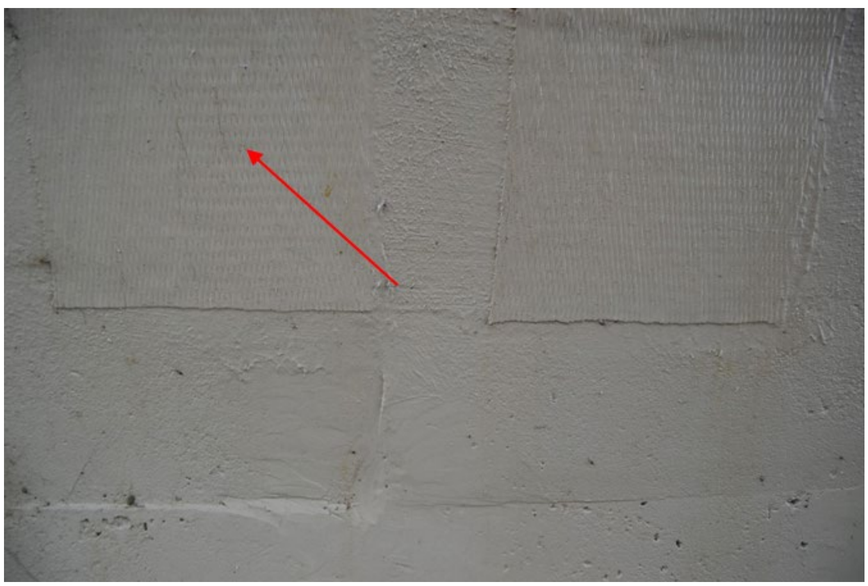


Figure B.15 Possible hairline cracks in CFRP sheet.

B.2.2 I-94 over US 20, Willow Creek, and CSX RR

Location: Portage (La Porte District)

Structure Type: Continuous Steel Stringer/Multi Beam or Girder (NBI 4-02)

Figure B.16 Panoramic view of bridge facing west (from Google, 2013).

Interstate 94 westbound and eastbound intersect US 20, Willow Creek, and CSX RR in Portage, Indiana, in the La Porte district. The structure is a continuous steel girder bridge, with the area of interest for this case study being the bents that support the bridge over US 20.

History and Structural Issues: The following information was taken from various inspection reports found through the INDOT BIAS. These reports were generated by the following inspectors: FHWA (11/28/2006) (INDOT, 2006c, 2006d), Wayne Skinner (01/30/2009) (Skinner, 2009a, 2009b), Wayne Skinner (01/22/2014) (Skinner, 2014a, 2014b), and Rich Fieberg (01/22/2015) (Fieberg, 2015a, 2015b). The main concern associated with this bridge is the substructure components, specifically the crash walls and columns of Bents 4 and 5, which support the I-94 bridge on either side of US 20. According to the inspection report from November 2006, the substructure condition rating was a five; however, the crash walls on Bents 4 and 5 were rated no lower than six. Again, the inspection report from January 2009 was similar, where the overall concern with the substructure was bridge seat cracking, backwall cracking and spalling, and cracking of slopewalls. The deck was in poor condition and replaced in 2014. The inspection report from January 2015 reflected the improvements to address the issues with backwalls and bridge seats. This report also stated that Bents 4 and 5 were patched and wrapped in fiber casing. No photos depicted Bents 4 and 5 to be in poor condition or in need of repair. Possible leaking from the cracked deck deteriorated the bents over time, but it is not clear from the reports what the actual cause was.

Construction Information: Construction plans (INDOT, 2012a) included replacement of guardrails, removal of the entire existing bridge deck, replacement of modular expansion joints, replacement of existing deck drains, removal of existing rocker bearing assemblies (construct RC pedestals and install elastomeric bearing assemblies), replacement of deteriorated end bent diaphragms, peening of the tapered and transverse edges of welded cover plate ends, repair of end of beams using welded beam repair, cleaning/painting of each beam end and all end bent diaphragms, replacement of RC approach slabs, installation of rip rap turnouts, installation of shear studs, and construction of terminal joint at the back of the RC approach slabs. According to the general plan drawings, an item was added to “remove the deteriorated concrete on Bent No. 4 and 5, install embedded galvanic anodes, patch with pneumatically placed mortar and welded steel wire fabric, and wrap with fiber wrap concrete casing system” on December 3, 2012. Elevation views of the bents and scheduled repairs are shown in the figure below. It is important to note that the sacrificial anodes were placed on the traffic face of the crashwall only.

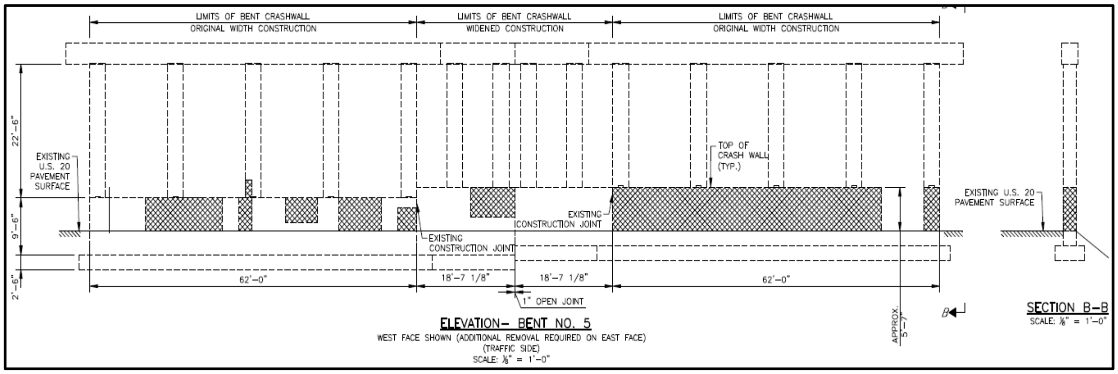


Figure B.17 Bridge rehabilitation plans: scheduled bent repairs (from INDOT, 2012a).

Inspection Details: Inspection was conducted on the substructure elements of the bridge. Two concrete bents were of concern, one on each side of US 20. The bents were reinforced concrete with a crashwall and rectangular (30″ × 36″) columns. Access was gained by a dirt road off of US 20.

Figure B.18 View of both bents facing west.

Repair Description: Bents 4 (north of US 20) and 5 (south of US 20) were wrapped using glass FRP sheets. Repair was completed sometime between 2013 and 2014. The crashwall and columns on each bent were wrapped. The crashwall on Bent 4 was wrapped with GFRP sheets to its base at the road level, with fibers running parallel to the longitudinal axis of the column. The crashwall at Bent 5 was wrapped in a similar fashion; however, the wrap did not extend to the ground on its backside opposite US 20. Columns were completely wrapped on all four sides with fibers oriented perpendicular to the longitudinal axis of the column. The columns were wrapped approximately 10 ft up measured from the top of the crashwall. Sheets on both columns and crashwalls overlapped and it appeared to be a single layer of GFRP. The GFRP wrap on end columns extended down to the base of the crashwall, so fiber orientation of the wrap on the crashwall remained perpendicular to the longitudinal axis of the end column. The FRP system is externally bonded with no evidence of anchorage use. Key concerns related to workmanship are surface preparation of the concrete and application errors when placing the GFRP sheets. There were multiple locations where bubbles/voids in the GFRP sheet were present as well as locations of depressions in the concrete surface. The lengths and overlapping of GFRP sheets were not consistent. There was debonding of the GFRP sheet from the concrete visible at the eastern most column of Bent 5. All sheets were painted.

Figure B.19 Repair procedures on Bent 5, September 2013 (Google, 2013).

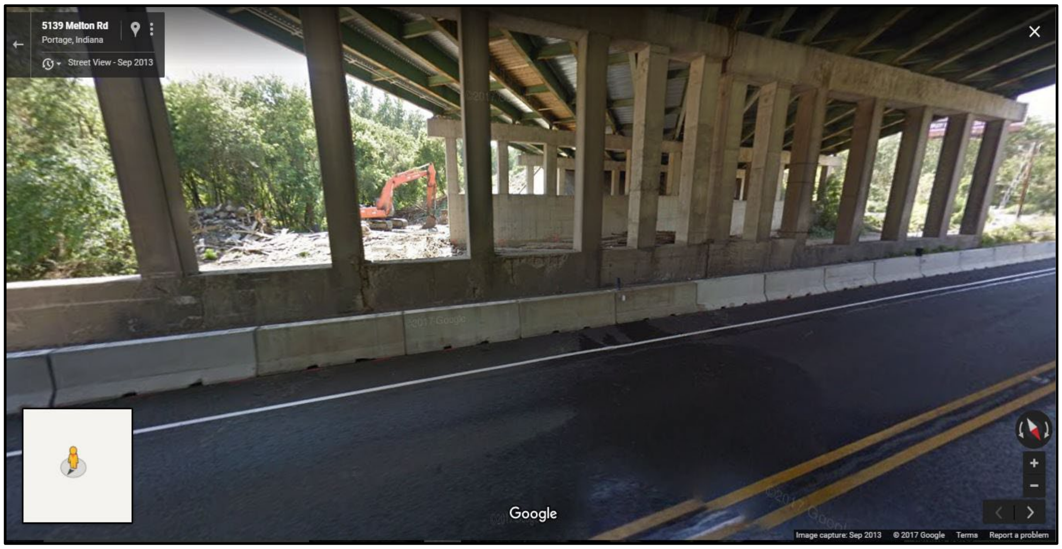


Figure B.20 Spalling and steel exposure of Bent 4 prior to FRP installation, September 2013 (Google, 2013).

Figure B.21 Variable length of GFRP sheets on Bent 5 crashwall.


Figure B.22 GFRP sheet overlapping and sheet detail at column to crashwall intersection.


Figure B.23 Debonding at easternmost column of Bent 5.

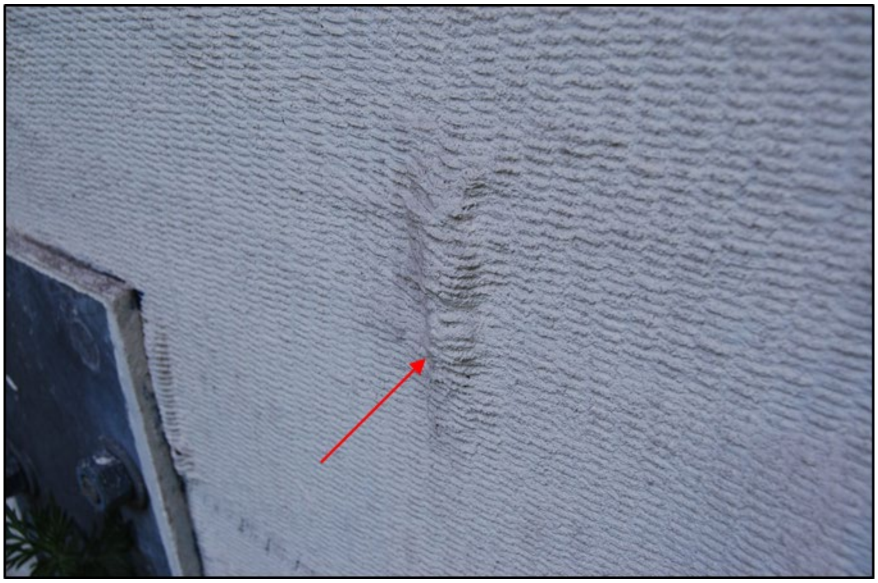


Figure B.24 Large bubble/void in GFRP sheet on crashwall of Bent 5.

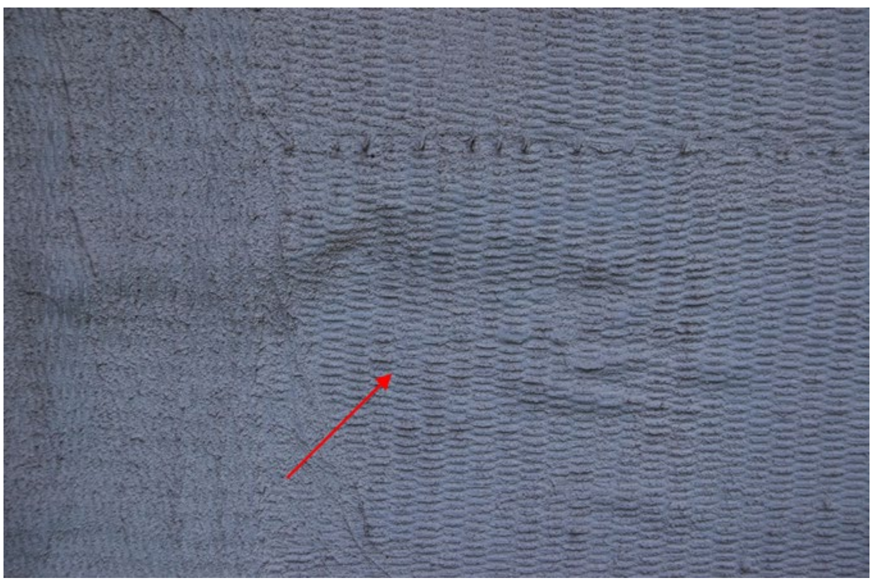


Figure B.25 Large depression in concrete surface at crashwall of Bent 5.

B.2.3 County Road 28 over I-69 NB/SB

Location: Auburn (Fort Wayne District)

Structure Type: Concrete Continuous Stringer/Multi Beam or Girder (NBI 2-02)

Figure B.27 Panoramic view of the bridge facing north (Google, 2016).

The CR 28 bridge is a four-span continuous concrete beam bridge located in Auburn, Indiana, in the Fort Wayne district. It services two lanes of traffic over Interstate 69 northbound and southbound. The areas of interest on this structure include several beam ends and Piers 2 and 4.

History and Structural Issues: The following information was taken from various inspection reports found through the INDOT BIAS. These reports were generated by the following inspectors: FHWA (05/03/2007) (INDOT, 2007f), Mark Hallien (04/28/2009) (Hallien, 2009c), Kirk Smith (04/26/2011) (Smith, 2011), Kirk Smith (04/26/2013) (Smith, 2013), Kirk Smith (04/21/2015) (Smith, 2015), and Joshua Biller (11/09/2015) (Biller, 2015b). The inspection report from May 2007 revealed the bridge substructure and superstructure were in good condition, evaluated as having condition ratings of six and seven, respectively. A comment was made regarding the concrete pillars at Bent 4 having some small spalls. Collision damage was noted on Bent 2 on the “west side of columns.” These same issues were represented in the April 2009 inspection report; however, with the additional comments of abutment cracking, slope wall cracking, and settlement at the top of the slope wall. In the April 2011 inspection report, it was noted that Girder 5 in Span C had surface spalls and delamination in five areas, and Girder 1 had delamination on the bottom side. The substructure issues were still present, with the additional comment that at Bent 4, Columns 1, 2, and 3 were delaminated. Column 3 had spalls with exposed rebar. At Bent 2 in Span A, there were spalls on three columns, and in Span B, there was a large spall with exposed rebar on one of the columns. There was one vertical spall with exposed rebar on the crashwall at Pier 2 in Span B. The following photos were taken from this 2011 report.

Figure B.28 Spalling and delamination with exposed rebar on girder (from Smith (2011) inspection report).

Figure B.29 Spalling and delamination at Pier 4, Span C (from Smith (2011) inspection report).

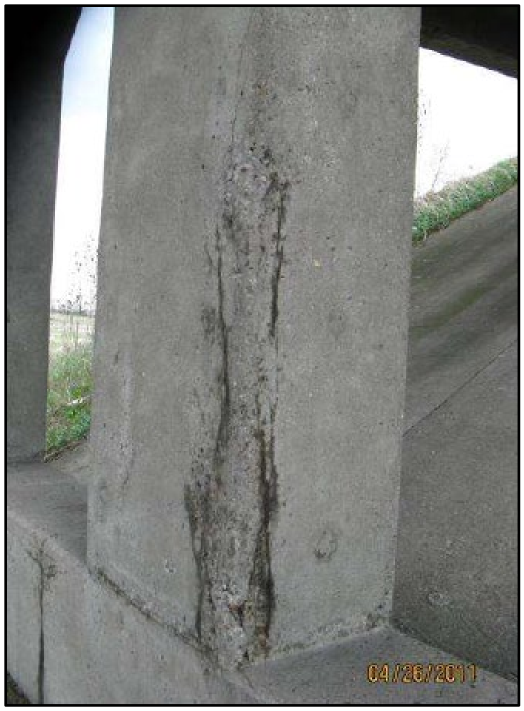


Figure B.30 Vertical delamination to the southwest corner of column at Pier 4, Span C (from Smith (2011) inspection report).

Figure B.31 Delamination at bottom of girder (from Smith (2011) inspection report).

Increased amounts of girder spalls and delamination was recorded in the April 2013 report. The concrete girder condition rating remained at a six, however. Concrete pillars showed more issues, in which Column 2 in Pier 4, Span C had vertical edge cracks, along with other elements showing spalling and delamination. The crashwall in Pier 4 had transverse and longitudinal cracks. The pillars and crashwall retained a condition rating of six. The inspection report from April 2015 revealed these issues were not yet remedied. The fiber wrapping and repairs were reflected in the November 2015 report, giving both the superstructure and substructure condition ratings of seven. The report stated that the girders had several new fiber wrap patches along their sides and bottoms at various locations. It was noted that few minor surface spalls and delaminations were present in the girders along with some hairline vertical cracks. Bent 3 was noted as having vertical cracks in the crashwall.

Construction Information: Bridge rehabilitation plans (INDOT, 2015) dated April 5, 2015, state to (1) patch spalled areas on Bents 2 and 4 and install fiber wrap around columns and crashwalls above ground line and below column fillets, (2) patch spalled areas on existing girders and install fiber wrap as directed by the Engineer, and (3) epoxy inject cracks in existing girders. Other bridge rehabilitation tasks included milling the deck surface, pouring a new latex modified concrete overlay, replacing reinforced concrete bridge approaches, and placing HMA pavement wedges and level at each end of bridge. Items 6 and 17 on the figure below correspond to the FRP repairs to the bents and beams.

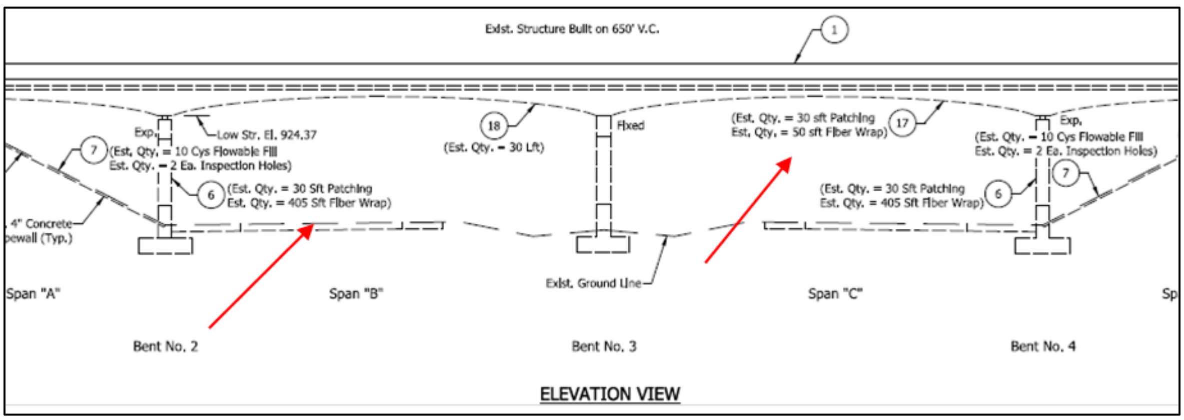


Figure B.32 Bridge rehabilitation plans: scheduled repairs (from INDOT, 2015).

It was stated in the contract information book that galvanic anodes be placed along the perimeter of the repair or interface with a typical spacing of 24 in. but not more than 28 in.

Inspection Details: Inspection was restricted to the underside of the bridge over I-69 southbound due to inclement weather. Photos were taken of the substructure and superstructure elements from this location only.

Repair Description: Various beam ends were repaired using FRP. Some of these repairs where three-sided wraps, while for a few locations at the beam ends, only a side was patched with FRP. One location on the deck appeared to be patched with an FRP sheet. The end piers were FRP-repaired as well. Here, the columns were completely wrapped. A longitudinal crack was seen on the bottom of the middle beam where it meets the eastern side of the middle pier.

Figure B.33 View of the FRP-repaired western end pier.

Figure B.34 Close-up of FRP-repaired beam end at western end pier.

Figure B.35 Possible FRP repair to underside of deck over I-69 SB.

Figure B.36 One-sided repair of middle beam on eastern side of middle pier.

Figure B.37 Cracking along middle beam.

B.2.4 County Road 200 S at I-69 NB/SB

Location: Huntington (Fort Wayne District)

Structure Type: Steel Continuous Stringer/Multi-Beam or Girder (NBI 4-02), Concrete approach spans

Figure B.38 View of bridge and FRP-repaired pier facing west.

The CR 200 South bridge is a four-span continuous steel beam bridge with concrete beam approach spans located in Huntington, Indiana, in the Fort Wayne district. It services two lanes of traffic over Interstate 69 northbound and southbound.

History and Structural Issues: The following information was taken from various inspection reports found through the INDOT BIAS. These reports were generated by the following inspectors: FHWA (02/22/2007) (INDOT, 2007e), Mike Hallien (02/19/2009) (Hallien, 2009b), Tim Atkinson and Kirk Smith (02/17/2011) (Atkinson & Smith, 2011), Joshua Biller (02/18/2015) (Biller, 2015a), and Corey Schamberger (08/18/2016) (Schamberger, 2016c). The area of interest on this structure is the pier on the I-69 southbound side. Issues began with the transverse joints missing joint material and leaking in 2007. In a report from February 2009, the joints were still in poor condition and the inside, west edge of the bottom of the hammerhead at Pier 2 was spalled. The report stated that the vertical rebar looked like it did not have enough concrete cover and the concrete had spalled. According to the inspection report from February 2011, random transverse cracks were present in the deck and both of the intermediate expansion joints were failed and leaking. The condition of Pier 2 had further declined and contained severe spalling with exposed rebar at the top at both ends. Issues with deterioration at Pier 2 persisted until the fiber wrapping was noted in an August 2016 inspection report.

Figure B.39 Pier 2 north hammerhead spalling and exposed steel (from Biller (2015a) inspection report).

Figure B.40 Pier 2 Deterioration (from Biller (2015a) inspection report).

Inspection Details: Inspection was focused mainly on the western end pier that was FRP repaired. No other members in the bridge appeared to have been repaired with FRP.

Repair Details: FRP was applied below the column cap of the western end pier of the bridge where the concrete approach span meets the continuous steel girder span over I-69 southbound. The repair initiates just below the column cap and terminates just below the point where the pier tapers to the column. Fibers in the sheets were oriented perpendicular to the longitudinal axis of the column and was a complete wrap. Several areas of the repaired surface had uneven sheet cuts, and poor details at the tapered section of the pier were noted. Some locations along the repaired surface showed signs of depressions and/or bubbling of the FRP sheet.

Figure B.41 Detailing of repair at tapered section of pier.

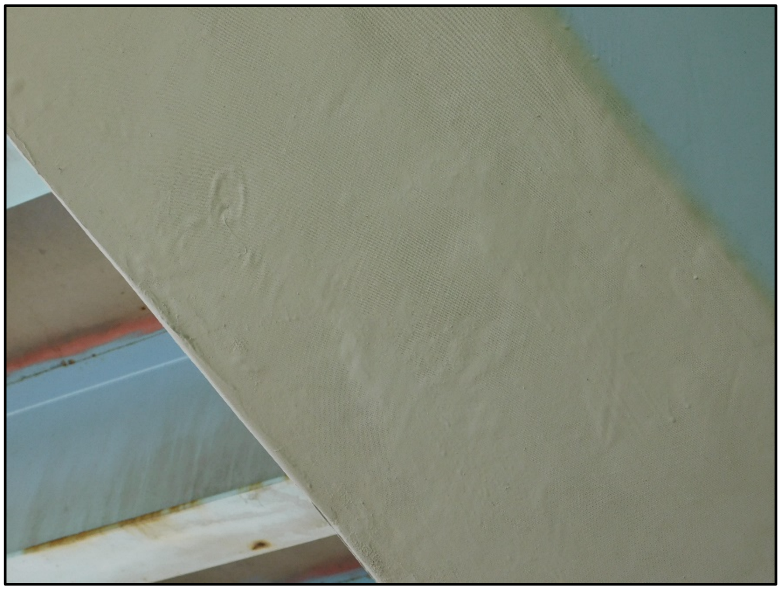


Figure B.42 Bubbling and depressions along tapered section of repaired pier.

Figure B.43 Paint/epoxy runoff on sheets and onto concrete.

Figure B.44 Joint and deck surface directly above repaired pier.

B.2.5 State Road 9 over Catfish Lake

Location: Columbia City (Fort Wayne District)

Structure Type: Prestressed Concrete Stringer/Multi-Beam or Girder (NBI 5-02)

Figure B.45 Panoramic view of the bridge facing west.

The SR 9 bridge over Catfish Lake is located in Columbia City, Indiana, in the Fort Wayne district. It is a ten-span, prestressed concrete beam bridge servicing two lanes of traffic.

History and Structural Issues: The following information was taken from various inspection reports found through the INDOT BIAS. These reports were generated by the following inspectors: FHWA (08/13/2007) (INDOT, 2007b), Mike Hallien (08/07/2009) (Hallien, 2009a), Tim Atkinson (08/05/2011) (Atkinson, 2011), Joshua Biller (07/30/2013) (Biller, 2013), and Joshua Biller and Cristin Gimbel (08/13/2014) (Biller & Gimbel, 2014). The area of interest on this structure is the beam ends at Piers 4 and 7 and the northern abutment. In the August 2007 inspection report, the bent caps at Piers 4 and 7 were noted as having cracks with rust staining and the piles were rusting just under the cap. The concrete pier caps at Piers 4 and 7 were still deteriorating and losing section. The piles at Piers 4 and 7 were noted as rusting as well in the inspection report from August 2009. In the report from August 2011, the first mention of superstructure issues was found. The fascia beam over Pier 7 contained a deep crack (spall) about a quarter of the way into the beam on the west end. The bottom of the diaphragm over Pier 7 was spalled on the east end between Beams 4 and 5, and 5 and 6. Issues with pier caps at Piers 4 and 7 were still present, with the addition of the end bent on the north end having spalling with exposed rebar and section loss between Beams 2 and 3, and 3 and 4. It was also noted that the east pier cap at Bent 4 was severely spalled with exposed rebar and section loss. A loss of the top 4 to 5 in. of the pier cap between Piers 5 and 6 as well as bad spalling of the cap over Piles 1 and 2 were noted in the report.

Figure B.46 Deterioration at Pier 4 (from Atkinson (2011) inspection report).

Figure B.47 Spalling of fascia beam and deep crack (from Atkinson (2011) inspection report).

Further superstructure issues were noted in the July 2013 inspection report. Several spalls with some exposed strands were found on Beams 1 and 5. All beams had spalling on their south ends with exposed strands on Span G. The deep crack with spalling on the fascia beam over Pier 7 was still present. Severe spalls with exposed rebar and section loss were noted for the diaphragms over Bents 4, 7, and 11. The interior diaphragm at Abutment 11 was deteriorated and the end bent on the north end had spalling with exposed rebar and section loss between Beams 2 and 3, and 3 and 4. Severe spalling and exposed rebar was noted at the pier caps of Piers 4 and 7. The piles at these piers had heavy rust with some section loss.

Figure B.48 Spalling and exposed strands at Beam 1 in Span A (from Biller (2013) inspection report).

Figure B.49 Deterioration at Beam 5 in Span A (from Biller (2013) inspection report).

Figure B.50 Deterioration at Pier 4 bent cap (from Biller (2013) inspection report).

Finally, the inspection report from August 2014 noted that the beam ends at Bents 4 and 7 (old expansion joint locations) and Abutments 1 and 11 were covered in fiber wrap. New diaphragms were noted at Bents 4 and 7. There were new semi-integral end bents, new caps for Bents 4 and 7, and all piles were cleaned and painted. The piles at Bents 4 and 7 were encased by concrete filled steel shells.

Figure B.51 Beam end region wrapping at Abutment 1 (from Biller & Gimbel (2014) inspection report).

Figure B.52 Repaired beam ends, new bent cap, and encased piles at Bent 4 (from Biller & Gimbel (2014) inspection report).

Figure B.53 Repaired beam ends, new bent cap, and encased piles at Bent 7 (from Biller & Gimbel (2014) inspection report).

Construction Information: The columns at Piers 4 and 7 of this bridge were encased with HSS 20 × 0.375 sections that were cut in half along the longitudinal axis for a two-piece construction. A flowable fill was placed inside the casing. The rehabilitation plans (INDOT, 2013) stipulate this procedure is for exterior columns only; however, during the site inspection, it seemed to be relevant in all of the columns of these piers. FRP repairs were proposed at the ends of all beams at the northern abutment and Piers 4 and 7 (wrapped into the span 10 linear feet for exterior beams and 3 linear feet for interior beams). Piers 4 and 7 were found to be the location of expansion joints following site inspection. Other construction procedures included construction of a new semi-integral end bent and pier cap reconstruction at Piers 4 and 7.

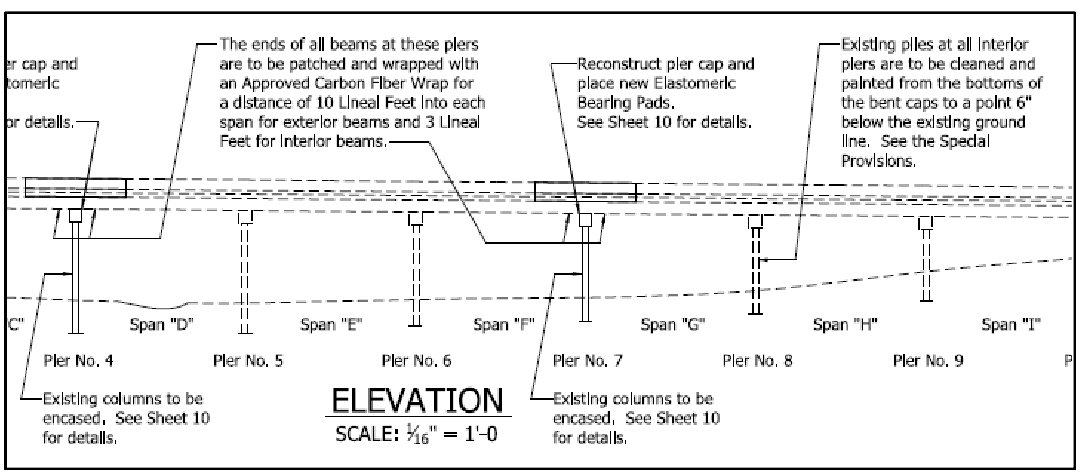


Figure B.54 Bridge rehabilitation plans (from INDOT, 2013).

Inspection Details: The inspection was restricted to the underside of the bridge due to inclement weather. It was found that the columns along the entire length of the bridge were confined with steel jackets. Some corrosion was visible at the base of the columns from the jackets. The columns at the expansion joints were wider than others, and it was assumed these columns were jacketed prior to jacketing of all the columns (possibly receiving two layers of steel jacketing in total). Two expansion joints are present in this structure. There were three spans between the western abutment and first expansion joint, three spans between the two expansion joints, and four spans between the second expansion joint and eastern abutment.

Repair Details: FRP repairs were completed on the beams ends at the location of possibly leaking expansion joints and at the southeastern abutment. No FRP repairs were seen on the northwestern end of the bridge. Local debonding of the sheets along the web and bottom flange of the interior beams at Pier 7 (from the southeast) were noted.

Figure B.55 Corrosion at base of steel jacket.

Figure B.56 Beam end repair at southern abutment.

Figure B.57 Different column width from steel jacketing.


Figure B.58 Beam end repair at Pier 7 at expansion joint (from the southeast).


Figure B.59 Local debonding at beam ends at Pier 7.

Figure B.60 Local debonding at beam ends at Pier 7.

B.2.6 State Road 9 over Salamonie River

Location: Huntington (Fort Wayne District)

Structure Type: Prestressed Concrete Continuous Stringer/Multi-Beam or Girder (NBI 6-02)

Figure B.61 Panoramic view of the southbound bridge facing northeast.

The SR 9 bridge over Salamonie River is located in Huntington, Indiana, in the Fort Wayne district. It is comprised of two, seven-span continuous prestressed concrete girder bridges servicing two lanes of traffic for both the northbound and southbound sides of SR 9. The areas of interest on this structure are several beam ends as well as the columns at Pier 4.

History and Structural Issues: The following information was taken from various inspection reports found through the INDOT BIAS. These reports were generated by the following inspectors: FHWA (12/06/2006) (INDOT, 2006a, 2006b), Mike Hallien et al. (12/01/2010) (Hallien et al., 2010a, 2010b), Tim Atkinson (11/28/2012) (Atkinson, 2012a, 2012b), Joshua Biller (11/21/2014) (Biller, 2014a, 2014b), and Corey Schamberger (11/15/2016) (Schamberger, 2016a, 2016b). The inspection report of the northbound bridge from December 2006 noted a spall on the downstream column of Pier 4 that was first found in December of 2002. No issues with the substructure or superstructure were noted in the southbound report at this date. The northbound inspection report from December 2010 noted that the pier cap at Pier 4 experienced extensive patching in 2010. The spall on the downstream column of Pier 4 was still present. Columns at Pier 4 were extensively patched and there was still a 3-ft by 3-ft delaminated area below the patching. It was also noted that the west fascia column had exposed steel. Loss of section at some of the beam ends at Abutment 1 was noted in the southbound report. Approximately 100 ft^2 of delamination with exposed steel, especially under Beam 4, was found at Pier 4. The inspection report of the northbound bridge from November 2012 noted spalling over the pier cap at Bent 5 and the fascia beam. The downstream column spall at Pier 4, the 3-ft by 3-ft delamination, and the exposed steel on the west fascia column were still present at this date. Significant deterioration of the beam ends at Pier 4 (the area below the expansion joint), minor deterioration at the abutments, and major deterioration of the pier cap at Pier 4 due to the failure of the above joint were all noted in the November 2014 report of the southbound bridge. Further deterioration of the cap at Pier 4 and spalling with exposed spiral steel on the west column at Pier 4 were noted in the northbound report.

Figure B.62 Column spalling and cap deterioration at northbound Pier 4 (from Biller (2014a) inspection report).

In the November 2016 inspection report, the patching and fiber wrapping of the beams at Pier 4 were noted from the 2016 rehabilitation. Spalls and deterioration of the cap at Pier 4 were also found to be patched from the rehab. The top 12 ft of the columns at Pier 4 were patched and fiber wrapped. Other beam ends at the abutment were fiber wrapped as well.

Figure B.63 Fiber Wrapped End of Beam 1 at Abutment 1 (from Schamberger (2016b) inspection report)

Figure B.64 Fiber wrapped beam ends at Pier 4 (from Schamberger (2016b) inspection report).

Construction Information: According to the bridge rehabilitation plans (INDO, 2015) that were certified in September of 2015, various concrete repair and carbon fiber patching procedures were to be implemented at the abutments and Pier 4. The existing SS expansion joint at Pier 4 was to be removed and replaced on both bridges. The deteriorated beam ends at Pier 4 were scheduled to be cleaned and wrapped with carbon fiber for added confinement protection. The deteriorated concrete on the pier cap at Pier 4 was to be removed on both bridges and galvanic anodes were to be installed before patching with concrete and welded wire steel fabric. The elastomeric bearing pads at Pier 4 were to be removed and replaced. At the abutments, the beam ends were to be patched and then wrapped with carbon fiber. In Span F of the northbound bridge, the bottom flange of the west fascia beam had a spall that was to be patched with concrete and then wrapped with carbon fiber for added confinement protection.

Inspection Details: Inspection of the south end of both the northbound and southbound SR 9 bridges was conducted. These end bents had been replaced, with evidence showing that the entire abutment had been removed and the approach spans rebuilt.

Figure B.65 Top of deck above the south end abutment of southbound bridge.

Figure B.66 Sand Layer on top of deck above the abutment diaphragm.

Repair Details: Beam ends at Pier 4 and both abutments were fiber wrapped. Sheet fibers were oriented perpendicular to the longitudinal axis of the beam and were painted. Beam end repairs at Pier 4 are assumed to be completed due to a leaking expansion joint. FRP repairs to the beam ends appear to be three-sided. On interior beams, the end repairs initiate at the end of the beam and terminate just before the diaphragm in some locations. Some of the interior beam end repairs were not complete three-sided wraps. In some locations, the wrap covers one complete side of the beam and bottom flange, and then terminates just a few inches up on the other side of the beam. It was common to see debonding of the sheet from the concrete surface in these cases. The top of the columns at the southbound Pier 4 were fiber wrapped and the top and bottom of the columns at the northbound Pier 4 were fiber wrapped. These columns are circular, and the wraps completely encased the top and/or bottom of the column. Fibers in the column wraps were oriented perpendicular to the longitudinal axis of the column. Breaks and/or poor cutting of the FRP sheets at the columns was noted as well as areas of depressions/bubbles. The column wraps overlapped.

Figure B.67 Beam end repair at south end abutment of southbound bridge.

Figure B.68 Debonding of FRP at beam end.

Figure B.69 Local debonding at interior beam end.

Figure B.70 Sheet termination at bottom of deck.

Figure B.71 Pier 4 Repairs on northbound bridge.

Figure B.72 Beam end repairs at southbound bridge Pier 4.

Figure B.73 Top of column repaired on southbound Pier 4.

Figure B.74 Epoxy runoff and sheet bubbling.

Figure B.75 Interior beam end repair (sheet termination at diaphragm).

B.2.7 US 30 over Blue River and Park Drive

Location: Columbia City (Fort Wayne District)

Structure Type: Concrete Continuous Slab (NBI 2-01)

Figure B.76 View of the US 30 eastbound bridge facing southwest.

The US 30 eastbound and westbound bridges over Blue River and Park Drive are located in Columbia City, Indiana, in the Fort Wayne district. These bridges are continuous concrete slab bridges servicing two lanes of traffic for both the eastbound and westbound sides of US 30.

History and Structural Issues: The following information was taken from various inspection reports found through the INDOT BIAS. These reports were generated by the following inspectors: FHWA (02/18/2008) (INDOT, 2008d, 2008e), Kirk Smith (02/18/2010) (Smith, 2010a, 2010b), Tim Atkinson (02/13/2012) (Atkinson, 2012c, 2012d), Kirk Smith and Linda Holzinger (06/23/2014) (Smith & Holzinger, 2014a, 2014b), and Joshua Biller (11/24/2014) (Biller, 2014c, 2014d). The areas of interest are the columns supporting both bridges. The major issue with this structure was failing and leaking joints, eventually causing a replacement of the entire superstructure of both the westbound and eastbound bridges. The original superstructure was composed of reinforced concrete girders and precast concrete box beams. According to the February 2008 inspection report for the eastbound bridge, the transverse joints received an overall condition rating of five. The joints were leaking and causing delamination and spalling on both sides of Piers 2, 3, and 4. Beam deterioration and rebar exposure was also reported.

Figure B.77 Spalled beam end and exposed rebar at eastbound bridge (from Smith (2010a) inspection report).

Figure B.78 Beam end deterioration at eastbound bridge (from Smith (2010a) inspection report).

Figure B.79 Wet pier at leaking joint in eastbound bridge (from Smith (2010a) inspection report).

Figure B.80 Patching of deck underside of eastbound bridge (from Smith (2010a) inspection report).

Issues with failing joints and beam deterioration was prevalent in the February 2012 report for the eastbound bridge, with the addition of pier deterioration. Spalled areas and exposed rebar were found in the intermediate piers. Similar issues were present in the westbound bridge. According to the June 2014 inspection report, both eastbound and westbound bridges would receive extensive rehabilitation, both receiving a new deck, superstructure, and upper portions of the substructure. The new structure will be a four-span continuous slab with haunches at the piers, and the substructure upper portion would be widened to eliminate the hammerheads to create rectangular walls. This rehabilitation was noted in the November 2014 inspection report. In this report, the bottom stems of the substructure were noted as being fiber wrapped. It was not explicitly stated in the inspection reports that the bottom portion of the stems of the substructure were deteriorated or needed rehabilitation.

Inspection Details: Access to inspect the underside of the US 30 westbound bridge was gained by way of Park Drive. Inclement weather limited inspection of all aspects of both bridges. Details and photos were taken of the US 30 westbound piers on either side of Park Drive.

Repair Details: The base of all piers on both bridges were fiber wrapped. Wrapping encased the pier completely and sheets overlapped by approximately 4 to 6 in. The wrap initiates at the size reduction in the pier (pier cap) and terminates at the ground level. Several locations of voids, depressions, bubbles, and what resembled alligator cracking were found on the repaired surfaces. All wraps were painted, and some locations the paint was chipping and peeling off.

Figure B.81 Sheet overlapping of pier adjacent to Blue River.

Figure B.82 Continuous wrap at curves of pier.

Figure B.83 Local debonding of wrap.

Figure B.84 Cracking and tearing of paint.

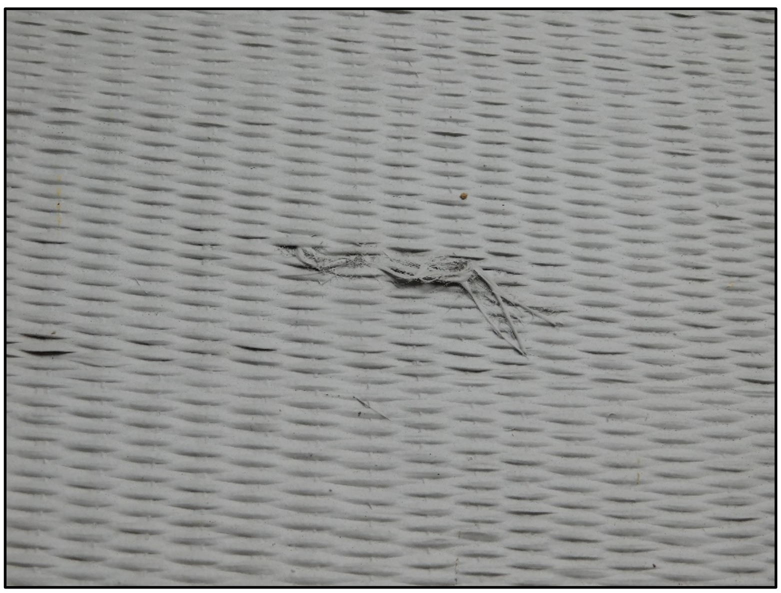


Figure B.85 Damaged and hanging fibers**.**

Figure B.86 Alligator-cracking.

Figure B.87 Termination of wrap at pier cap.

B.2.8 I-65 Northbound at I-65 Southbound Exit Ramp

Location: Lebanon (Crawfordsville District)

Structure Type: Concrete Continuous Stringer/Multi-beam or Girder (NBI 2-02)

Figure B.88 Panoramic view of bridge facing west.

History and Structural Issues: The following information was taken from various inspection reports found through the INDOT BIAS. These reports were generated by the following inspectors: FHWA (04/16/2007) (INDOT, 2007c), Matt Ference (04/29/2009) (Ference, 2009), and Dan Bewley (04/20/2011) (Bewley, 2011a). The areas of interest in this bridge include the beam ends at both abutments and the northernmost pier as well as some areas of all three piers. In an inspection report from April 2007, Pier 2 was noted as having vertical cracking in the pier cap and that two columns had cracking. It was also stated within this report that the southwest corner bridge seat was spalled, delaminated, and wet. The interior transverse intermediate joint was found to have some missing joint material. The condition of Pier 2 worsened, as the inspection report from April 2009 describes heavy cracking and light spalling within the pier. Heavy cracking and spalling were noted for some of the columns within Pier 2. The report goes on to explain that there is a wide range of deterioration present in the substructure of the bridge, with light cracking in some areas and heavy cracking with moderate spalling and exposed rebar in other areas (mainly Pier 2). In a report from April 2011, the superstructure rating was dropped to a four due to all spans containing girders with spalled areas and exposed steel at the piers. No changes in the deterioration of the substructure were noted.

Figure B.89 Spalled east girder with exposed steel above Pier 2 (from Bewley (2011a) inspection report).

Figure B.90 Crumbling west girder at southwest corner of southern abutment (from Bewley (2011a) inspection report).

Figure B.91 Spalled west girder and deteriorated Pier 2 (from Bewley (2011a) inspection report).

Figure B.92 Spalled column with exposed steel at Pier 2 (from Bewley (2011a) inspection report).

Inspection Details: It was possible to inspect the entirety of the underside of this bridge. All beams and substructure components were inspected to evaluate FRP repairs that were present.

Repair Description: Repairs to this bridge included wrapped beam ends, pier caps, and columns. Beam end repairs were completed at both the north and south abutment as well as on either side of Pier 4 on the north side of the exit ramp. The beam end repairs initiated approximately 1 to 1.5 ft from the support and extended 2 to 3 ft into the beam. Pier caps at the piers were wrapped on three sides, initiating and terminating at the diaphragm. The wrapping at the pier caps did not overlap to create a continuous wrapping. There was some spacing of the U-wraps. The columns at Pier 4 were the only columns wrapped on the structure. These columns were wrapped from their intersection with the pier cap to the ground. Generally, the repair quality was good, with the exception of a large void (about 2.5 ft by 3 ft) on Pier 4, corrosion and large areas of discoloration on Pier 4, and transverse cracking and peeling of the pier cap wrapping at the beam-to-pier cap intersection.

Figure B.93 Beam end repair of fascia girder at southern abutment.

Figure B.94 Beam end wrapping up to deck.

Figure B.95 Wrapped pier cap at Pier 2.

Figure B.96 Termination of pier cap wrapping at diaphragm.

Figure B.97 Termination of pier cap wrapping at edge of cap.

Figure B.98 Pier 4 wrapped beam cap and columns.

Figure B.99 Pier cap-column joint.

Figure B.100 Large void and discoloration on wrapping at Pier 4.

Figure B.101 Cracking and peeling of beam cap wrapping at beam end support.

B.2.9 116th Street East at Keystone Parkway

Location: Carmel (Greenfield District)

Structure Type: Prestressed Concrete Continuous Stringer/Multi-beam or Girder (NBI 6-02)

Figure B.102 Panoramic view of the bridge facing southeast.

History and Structural Issues: The following information was taken from an inspection report found through the INDOT BIAS. The report was generated by the following inspector: Adam Post (11/25/2013) (Post, 2013). The main concern for this structure was a vehicle impact to the north fascia girder of the west span of the 116th street bridge. Prior to this incident, no damages or deterioration were noted. The report from November 2013 first noted the vehicle impact. The structural appraisal rating was very good at eight. Only cracks and spalls were noted from the impact. There were no exposed or severed strands, and the bridge sufficiency rating was okay.

Figure B.103 Impact-damaged girder prior to FRP Repair (from Post (2013) inspection report).

Inspection Details: The research team was able to access the western span of the 116th Street Bridge over southbound Keystone Parkway. This span contained the impacted girder that was repaired. Once under the bridge, it was evident only the fascia girder was damaged. All other girders within the span were in good condition and did not show any signs of damage. The eastern span over northbound Keystone Parkway did not appear to have any impact-damaged girders.

Repair Description: An FRP wrapping scheme was employed on an impacted fascia girder. The wrapping was restricted to just a small area at midspan of the girder and the bottom flange of this area. Wrapping at the bottom flange of the impacted region is approximately 8-ft wide along the girder, and the edges of the repair are 15 ft from the supports. No wrapping was present at the web of this girder. Judging from visual inspection, it appeared no final protective coating (i.e., paint) was used on this wrapping. As seen on many other case studies, some final layer of paint or UV protective coating is placed over the FRP wrapping. The wrapping scheme on this impacted fascia girder did not appear to have this coating; however, there could have been a transparent coating placed on the wrap that is not visible from the roadway level. No significant issues or defects were prevalent with this repair.

Figure B.104 Wrapping at impacted region.

Figure B.105 Minor frayed edges at termination along web.

Figure B.106 Impacted fascia girder and undamaged adjacent girder.

Figure B.107 Repair from underside of 116th Street Bridge.

B.2.10 Old State Road 132 at I-69 Northbound/Southbound

Location: Pendleton (Greenfield District)

Structure Type: Steel Continuous Stringer/Multi-beam or Girder (NBI 4-02), Concrete approach spans (NBI 1-02)

Figure B.108 Panoramic view of the bridge facing southeast.

History and Structural Issues: The following information was taken from various inspection reports found through the INDOT BIAS. These reports were generated by the following inspectors: FHWA (11/30/2007) (INDOT, 2007d) and Jim Mickler (03/09/2009) (Mickler, 2009). The areas of concern for this bridge were Piers 2 and 4. According to an inspection report from November 2007, minor longitudinal cracks were present at the joints, and the deck underside had transverse cracks and efflorescence. Heavy spalling was noted at the transverse joint at Pier 2. Pier 3 had minor chipping and spalling. The interior pier caps had delaminations and heavy spalls on their underside at Piers 2 and 4 with exposed steel and heavy section loss. The concrete columns were noted as having cracks, delaminations, and spalls with section loss with steel exposure at Piers 2 and 4. The crashwalls had vertical cracks, minor spalls, and some honeycombing. Following the partial rehabilitation from 2008, the columns and pier caps of Piers 2 and 4 were wrapped and the condition of the substructure was improved, according to an inspection report from March 2009.

Inspection Details: Access was possible to the underside of the northwest side of the bridge near the concrete approach span. The research team could not access the southeast pier due to fencing restricting access from the top of the bridge along with an active construction site in the median dividing the interstate.

Repair Description: FRP wrapping was used on the columns and pier cap at Piers 2 and 4. The crashwalls and the entirety of the middle pier were left unwrapped. No superstructure elements on this bridge received FRP repairs. The pier caps that received repairs were transversely U-wrapped up to the diaphragm on both sides of the pier, with the location at the pier cap-to-column intersection cut out. The columns were completely wrapped transversely along their entire length. Large areas of voids and discoloration were found on the northwestern pier cap. Inconsistent layering of sheets and epoxy quantities were present throughout various regions of the repaired surfaces. Large areas of rust staining on the repaired surface of the southeastern pier were visible from across the interstate. This rust staining was due to minor corrosion of steel superstructure elements above the pier.

Figure B.109 Wrapped columns and pier cap at northwestern pier.

Figure B.110 Large area of void and discoloration at northwestern pier cap.

Figure B.111 Inconsistent use of epoxy at pier cap-to-column intersection.

Figure B.112 Discoloration at northwestern pier cap.

Figure B.113 Rust staining and other discoloration at southeastern pier.

B.2.11 County Road 400 East at I-70

Location: Greenfield (Greenfield District)

Structure Type: Steel Continuous Stringer/Multi-beam or Girder (NBI 4-02)

Figure B.114 Panoramic view of the bridge facing Southeast.

History and Structural Issues: The following information was taken from various inspection reports found through the INDOT BIAS. These reports were generated by the following inspectors: FHWA (07/01/2008) (INDOT, 2008c), John Routh (05/21/2010) (Routh, 2010), Jim Mickler (05/10/2012) (Mickler, 2012), Brian Harvey and John Routh (05/19/2014) (Harvey &

Routh, 2014), and Jim Mickler (07/14/2015) (Mickler, 2015b). The areas of interest within this bridge are Piers 2 and 4, specifically the regions underneath the hammerhead caps of each pier. In the inspection report from July 2008, the north pier cap was noted from 2004 as having spalls with rebar exposure and the south face of Pier 2 had minor cracks and delaminations from a collision damage from 1998. The poured sealer for the transverse joints at the end bents had minor chipping and debonding. The polymer modified at Piers 2 and 4 were debonding, with a large crack over Pier 4. The spalling and rebar exposure were still present in the north pier according to the May 2010 inspection report. This report also describes issues in the approach slabs, which had several large epoxy sealed cracks, random longitudinal cracks, and continued issues at the transverse joints. The interior diaphragms were noted as having minor cracks and efflorescence. Issues with the approach spans worsened according to the May 2012 inspection report. No specific changes were noted regarding the superstructure or substructure elements. In the May 2014 inspection report, corrosion was found over Piers 2 and 4, and the concrete girders contained minor vertical cracks, delamination, and spalling with exposed rebar over Pier 2. Finally, the July 2015 inspection report describes that the superstructure in the end spans were replaced in 2014, now eliminating the concrete girder approach spans. The fiber wrapping was noted on the stems below the hammerheads of Piers 2 and 4 with the reconstructed pier caps.

Inspection Details: Piers 2 and 4 were accessed for inspection. These two piers were the only elements in this bridge to have received FRP repairs.

Repair Description: The single column Piers 2 and 4 had received FRP wraps at their bases. The transverse wraps initiate at ground level and extend up to the tapered hammerhead portion of the pier. Inconsistent layering of sheets was noted. A large sag in the transverse sheets at the northern pier was seen. There were no significant voids or areas of discoloration visible. Some issues that were seen included cracking similar to alligator cracking. These small, hairline cracks in the FRP sheets may be due to UV exposure.

Figure B.115 Wrapped column at northern pier.

Figure B.116 Wrap termination at hammerhead.

Figure B.117 Sag in transverse column wrap at northern pier.

Figure B.118 Proper smooth surface of repair.

Figure B.119 Possible UV damage.

Figure B.120 Areas of voids on the repaired surface of the southern pier.

Figure B.121 Inconsistent layers and sagging of sheets.

B.2.12 I-65 Northbound at I-70 Westbound Ramp

Location: Indianapolis (Greenfield District)

Structure Type: Steel Continuous Stringer/Multi-beam or Girder (NBI 4-02)

Figure B.122 Panoramic view of the bridge facing south.

History and Structural Issues: The following information was taken from various inspection reports found through the INDOT BIAS. These reports were generated by the following inspectors: FHWA (04/01/2008) (INDOT, 2008b) and James Mickler (06/26/2015) (Mickler, 2015a). The structural issues of this bridge, as they pertain to the FRP repair, are localized to the middle concrete pier. Major issues did not occur until a collision on June 26, 2015, that caused the bridge to catch fire, according to the inspection report from that date. Before this incident, minor collision scrapes were found on the concrete columns during an April 2008 inspection. These minor collision damages were prevalent in the reports until the fiber wrapping repairs scheduled for letting in March 2016. As for the major collision in 2015, the inspection reports explain that the fire burned for about 30 minutes directly below the bridge after a semi-truck struck the north concrete barrier near Bent 2. Fire damages were sustained to the pier cap and columns at the north end of Bent 2. The north end of the pier cap and the two northern columns had extensive but shallow scaling from the fire; however, there was no exposed rebar. The following photographs were taken from the July 26, 2015, inspection report and depict the fire damages to Bent 2 prior to repair.

Figure B.123 Fire damage to Bent 2 (from Mickler (2015a) inspection report).

Figure B.124 Scaling of the north end of bent cap (from Mickler (2015a) inspection report).

Inspection Details: FRP repairs were restricted to the middle pier. This bridge carries I-65 over four lanes of interstate traffic. At the time of inspection, traffic was extremely busy; therefore, the research team was not able to access all points around the middle pier. All photographs were taken from the eastern shoulder.

Repair Description: FRP was used to wrap all three columns and the north end of the beam cap. Some inconsistent layering of the sheets was noted. There was a large region on the northernmost column with tears in the FRP sheets exposing the underlying concrete. The research team believes this is due to vehicle or vehicle component impact.

Figure B.125 Wrapped northernmost column and beam cap edge.

Figure B.126 Damage to northernmost column with exposed concrete.

Figure B.127 Inconsistent layering of FRP sheets with signs of vehicle damages.

B.2.13 County Road 275 East at I-74 Eastbound/Westbound

Location: Pittsboro (Crawfordsville District)

Structure Type: Concrete Continuous Stringer/Multi-beam or Girder (NBI 2-02)

Figure B.128 Panoramic view of the bridge facing southeast.

History and Structural Issues: The following information was taken from various inspection reports found through the INDOT BIAS. These reports were generated by the following inspectors: FHWA (06/19/2007) (INDOT, 2007h), Dan Bewley (06/08/2009) (Bewley, 2009), Dan Bewley (06/02/2011) (Bewley, 2011b), and Dan Bewley (11/03/2016) (Bewley, 2016). Issues with deterioration at Piers 2, 3, and 4 were first noted in an inspection report from June 2007. The report describes the piers having columns with exposed rebar, spalling, and other delaminations, with the crashwall containing vertical cracks. Efflorescence and minor transverse cracks were still present on the deck underside and were first noted in a report from 1999. The transverse joints at both the north and south ends received a structural condition rating of three.

Figure B.129 Spalling and exposed rebar at Pier 2 (from Bewley (2009) inspection report).

Figure B.130 Spalling and exposed rebar at Pier 3 (from Bewley (2009) inspection report).

Figure B.131 Spalling and exposed rebar at Pier 4 (from Bewley (2009) inspection report).

According to the June 2011 inspection report, both of the transverse joints had failed, and the joint material was torn and protruding from the joint. Hairline cracks in all girders were noted with no leaching present. Issues with spalling on the piers was still prevalent, and the condition rating of the concrete in the substructure was lowered to a five. This deterioration of the piers was present until the November 2016 report that noted the fiber repairs to all three piers.

Inspection Details: The entirety of the underside of the structure was accessed during inspection. FRP repairs were located at all piers. Traffic at the time of inspection was light, therefore the research team was able to access all piers for evaluation.

Repair Description: Piers 2, 3, and 4 received FRP repairs. The columns and pier caps at Piers 2 and 4 are wrapped, along with the columns at Pier 3 (middle pier). Areas along the repaired surface contained multiple voids, debonding, and inconsistent overlapping and lengths of wrap. Another issue with the repair was poor detailing at the cap-column intersection. Large gaps were present at this location. Gaps between the fiber tows within the FRP sheets were seen as well.

Figure B.132 Multiple voids in column wrap.

Figure B.133 Inconsistent sheet lengths and overlapping.

Figure B.134 Local debonding of column wrap.

Figure B.135 Gaps between fiber tows within sheets.

Figure B.136 Poor detailing at pier cap-to-column intersection.

Figure B.137 Poor detailing at pier cap-to-column intersection.

B.2.14 Fruitridge Avenue at I-70

Location: Terra Haute (Crawfordsville District)

Structure Type: Continuous Concrete Stringer/Multi Beam or Girder (NBI 2-02)

Figure B.138 Panoramic view of bridge.

Interstate 70 eastbound and westbound intersects South Fruitridge Avenue in Terra Haute, Indiana, in the Crawfordsville District. The structure is a four-span continuous concrete girder bridge.

History and Structural Issues: The following information was taken from various inspection reports found through the INDOT BIAS. These reports were generated by the following inspectors: FHWA (12/11/2007) (INDOT, 2007g), Melvin Hughes (04/17/2009) (Hughes, 2009), Melvin Hughes (04/07/2011) (Hughes, 2011), Dan Bewley et al. (03/12/2013) (Bewley et al., 2013), and Melvin Hughes (03/17/2015) (Hughes, 2015). The areas of interest for this structure are the fire-damaged regions of the northern spans over I-70 westbound towards the northern pier. According to an inspection report from December 2007, the fire damage occurred in 2002. The damage was caused from an accident on the roadway in which portions of the deck underside, girders, and substructure were affected. The report explains that the concrete was "white hot" causing concrete to spall and delaminate in a few areas. A field check was conducted that involved sounding the fire-damaged areas of the bridge. Some areas of delamination were found in the east girder in the northern span, pier cap, columns, and crashwall of the northern pier. The FRP repairs were noted as early as 2006, which improved the condition rating of the fire-damaged portions of the structure.

Inspection Details: Inspection of the entire structure was possible. The substructure elements and beam ends at the northern pier were of main concern. Evidence of fire damage was present on the underside of the bridge spanning over I-70 westbound and the adjacent span over the slope walls towards the northern abutment. No deterioration or fire damage was present on the southern spans over I-70 eastbound towards the southern abutment.

Repair Description: FRP repairs were located on portions of the northern pier, two girders in the northernmost span, and portions of the deck in the northernmost span. Three of the columns of the northern pier were wrapped. Two columns contained a full-length wrap and one was wrapped about three-quarters of its length. Portions of the pier cap and crashwall of the northern pier were wrapped between the columns. The column wraps did not all extend to the bottom of the crashwall, where some terminated at the top of the crashwall. In the northernmost span on the eastern edge of the bridge, two girders and the deck portion between them was wrapped with FRP. These wraps initiated just over the pier and terminated approximately 3 to 4 ft into the span towards the northern abutment. These wraps extended up to the deck on both sides of the two girders. The portion of the diaphragm between these two girders was wrapped with FRP as well. In general, the FRP repairs seemed to be completed adequately. The FRP repairs were in good condition, except for a few areas of chipped paint on the surfaces of the column wraps. The only major damage present in the repairs was a large tear in the westernmost column wrap of the northern pier. The tear extended to the underlying concrete and appears to have been damaged by a passing vehicle.

Figure B.139 Fire-damaged northern spans.

Figure B.140 Wrapped girders, diaphragm, and deck portion in northern span.

Figure B.141 Detailing of FRP repair at girder-diaphragm intersection.

Figure B.142 Column wrap terminated at top of crashwall.

Figure B.143 FRP Repairs to column and portions of pier cap and superstructure.

Figure B.144 Large tear in westernmost column wrap.

Figure B.145 Areas of chipped paint on column wraps.

B.2.15 Glendale Road at Aikman Creek

Location: Washington (Vincennes District)

Structure Type: Concrete Channel Beam (NBI 1-22)

Figure B.146 Panoramic view of bridge.

Glendale Road (East 500 South) intersects Aikman Creek in Washington, Indiana, in the Vincennes District. The structure is a three-span reinforced concrete channel beam bridge supported on steel encased concrete piles.

History and Structural Issues: The following information was taken from various inspection reports found through the INDOT BIAS. These reports were generated by the following inspectors: InspectTech Administrator and Daviess County (03/23/2009) (InspectTech Administrator, & Daviess County, 2009), Patrick Conner (06/21/2011) (Connor, 2011), Alfred Wessling (06/04/2013) (Wessling, 2013), and Jonathan Olson (06/09/2015) (Olson, 2015). The area of interest for this bridge case study was the piles located within Aikman Creek (eastern pier). In a March 2009 inspection report, special remarks were given that described heavy leakage at the piers from degraded or missing joint material, and that all eight piles in the eastern pier contained corrosion holes. The overall substructure received a condition rating of four, mainly due to the poor condition of the piles at the eastern pier. No photographs or in-depth information was found prior to the installation of the steel encasements. The following photographs were taken from a November 2011 report and depict the deterioration of the steel-encased piles prior to the FRP repairs.

Figure B.147 Pile deterioration at eastern pier (from Conner (2011) inspection report).

Figure B.148 Pile deterioration at eastern pier (from Conner (2011) inspection report).

The reports from June 2013 and June 2015 still note heavy leaking at the piers; however, the piles were noted as having the fiber wrap repairs. The overall condition rating of the substructure was raised to a five. In the report from 2015, damage to the column wrap was noted at the eastern pier as shown in the photograph below.

Figure B.149 Column wrap damage at eastern pier (from Olson (2015) inspection report).

Inspection Details: Inspection was limited due to the water level in the creek. Therefore, close-up inspection of the wrapped piles located within Aikman Creek was not possible.

Repair Description: FRP repairs were located on the steel-encased piles on the eastern pier. Column wraps extended into the water of Aikman Creek and terminated at varying heights (approximately 3 ft above the waterline) up each of the eight columns at the eastern pier. These repairs appeared to wrap around each of the columns at least two full turns; however, this wrapping scheme was not consistent for every column. The repairs seemed to be properly applied to the steel-encased columns, with only one area of debonding visible on one of the columns. All wraps were painted to match the steel-encased columns. Some of the paint had chipped, exposing the FRP wrap. It was not immediately clear why these steel-encased piles were wrapped, as the western pier columns were not wrapped.

Figure B.150 FRP wraps on steel-encased columns of eastern pier.

Figure B.151 Paint chipping on column wrap.

Figure B.152 Debonded area at top of column wrap.

Figure B.153 Unwrapped columns at western pier.

Figure B.154 Seepage at fascia beam in easternmost span.

B.2.16 State Road 101 at Markland Dam

Location: Florence (Seymour District)

Structure Type: Prestressed Concrete Continuous Stringer/Multi Beam or Girder (NBI 6-02)

Figure B.155 Panoramic view of bridge spanning IN-156.

Figure B.156 Panoramic view of bridge near Markland Dam.

State Road 101 intersects IN-156 and Markland Dam in Florence, Indiana, in the Vincennes District. The structure is a continuous prestressed concrete girder bridge.

History and Structural Issues: The following information was taken from various inspection reports found through the INDOT BIAS. These reports were generated by the following inspectors: FHWA (03/06/2008) (INDOT, 2008a), Chris Everman (02/23/2010) (Everman, 2010), Chris Everman (01/27/2012) (Everman, 2012), Chris Everman (01/09/2014) (Everman, 2014), and Chris Everman (11/12/2015) (Everman, 2015). The areas of interest for this structure are the beam ends near the joints at the two locations shown in the above photos. In an inspection report from March 2008, it was noted that the south joint was leaking and causing damage to an underlying beam. The north joint was described as loose and vibrating when traffic traveled over it. The transverse joints received an overall condition rating of five. The underlying beam previously mentioned contained cracking, spalling, and deterioration. The inspection report from February 2010 explicitly stated that the leaking joints were causing deterioration to the beams below it. The condition of the beams appears to have declined further, with spalling and exposed steel in the beams over Piers 22 and 28 (at the two joint locations previously discussed). It was also noted there was a spalled region at midspan in Span X with exposed steel. The substructure condition remained unchanged.

Figure B.157 Beam end deterioration at Pier 28 (from Everman (2010) inspection report).

Figure B.158 Beam end deterioration at Pier 22 (from Everman (2010) inspection report).

Figure B.159 Midspan spalling and exposed steel (from Everman (2010) inspection report).

Issues with deck/joint degradation and beam end deterioration persisted through the reports from January 2012 and January 2014. Fiber wrap repairs were noted in the November 2015 inspection report, and the superstructure was listed in good condition.

Construction Information: According to the bridge rehabilitation plans (INDOT, 2014), there were multiple other areas of this bridge scheduled for FRP repair that were not seen during inspection. These repairs included two beams within Span W and one beam within Span Y. The figures below were taken from the rehabilitation plans for the beam end repairs at the two joint locations.

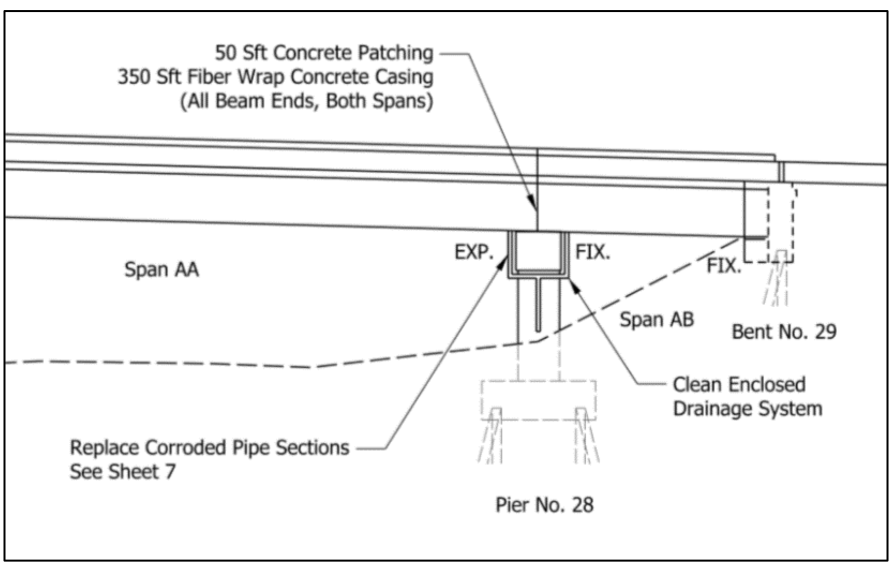


Figure B.160 Fiber wrap plans for beam ends at joint near IN-156 (from INDOT, 2014).

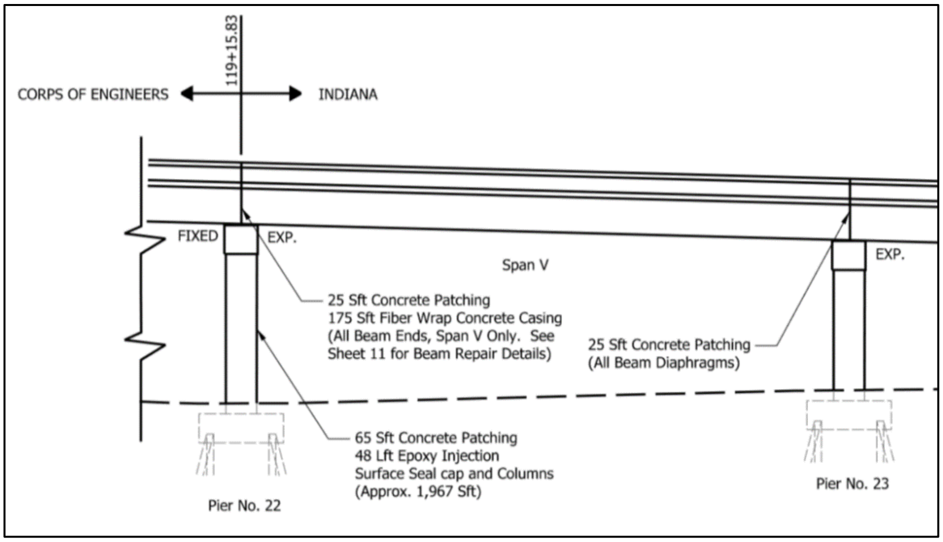


Figure B.161 Fiber wrap plans for beam ends at joint near Markland Dam (from INDOT, 2014).

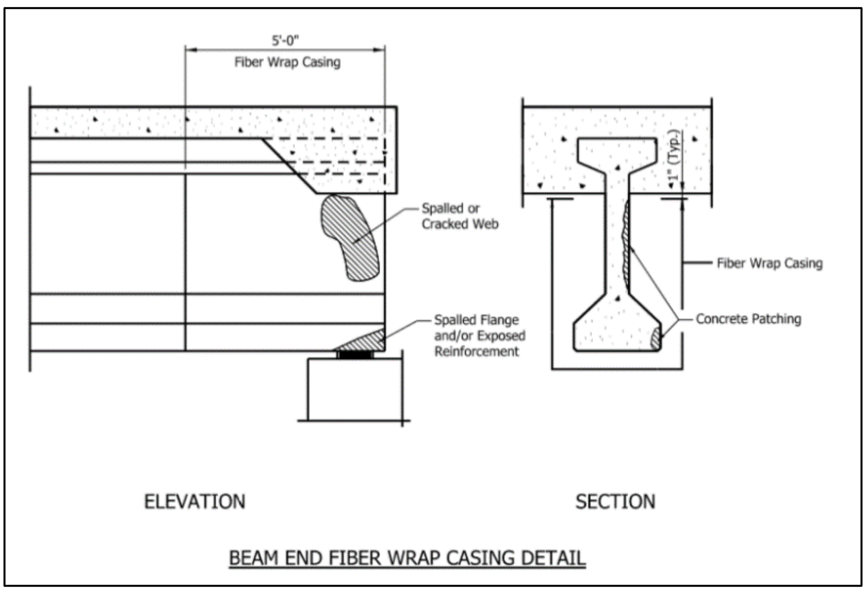


Figure B.162 Beam end repair details (from INDOT, 2014).

Inspection Details: Inspection of the entirety of this structure was possible. The first phase of the inspection consisted of the beam ends at the northernmost pier near IN-156. The second phase involved inspecting the beam ends at Pier 8, located within the facility adjacent to Markland Dam. The piers at both joint locations appeared to be in good condition with no immediate signs of deterioration. The steel girders at the joint near Markland Dam looked to be in condition as well and appeared to be recently painted.

Repair Description: FRP repairs are located at the beam ends at the expansion joints near IN-156 and Markland Dam. At the joint near IN-156, beam ends were repaired on both sides of the joint; however, the joint near Markland Dam was a transition point from prestressed concrete girders to steel girders. Therefore, the prestressed concrete girder ends were the only ends to receive FRP repairs at the Markland Dam joint. Beam end repairs were U-wraps initiating and terminating at the bridge deck and diaphragm. Wraps did not extend onto the deck or diaphragm at any point. The repairs at the IN-156 joint appeared to be cracking/splitting where the repaired surface met the pier cap support locations. There was evidence of some debonding in the repaired regions. Some of the repaired beam ends contained inconsistent layering of FRP sheets. It appeared that regions of the repaired surfaces had been patched, and it was unclear if the member was completely wrapped on three sides.

Figure B.163 Beam end repairs at joint near IN-156.

Figure B.164 Premature termination of FRP wrap at support location.

Figure B.165 Cracking/splitting of sheet at support location.

Figure B.166 Patching of FRP material.

Figure B.167 Beam end repairs at Markland Dam joint.

Figure B.168 Corrosion at beam end near support location.

B.2.17 Waldron Road at I-74

Location: Waldron (Greenfield District)

Structure Type: Concrete Continuous Stringer/Multi Beam or Girder (NBI 2-02)

Figure B.169 Panoramic view of bridge.

Waldron Road (County Road 600 East) intersects Interstate 74 in Waldron, Indiana, in the Greenfield District. The structure is a four-span continuous reinforced concrete girder bridge.

History and Structural Issues: The following information was taken from various inspection reports found through the INDOT BIAS. These reports were generated by the following inspectors: FHWA (11/20/2007) (INDOT, 2007i), John Routh (10/19/2009) (Routh, 2009), Brian Harvey (08/20/2013) (Harvey, 2013), James Yapp (08/31/2015) (Yapp, 2015), and Jim Mickler (08/23/2016) (Mickler, 2016). The area of interest for this case study is Piers 2 and 4. In an inspection report from November 2007, it was noted that the transverse joints had many areas that were debonded and torn, and that were leaking and failing. The concrete columns had cracking, delaminations, spalling, and rebar exposure at the bases since 2005. The crashwall was found to have vertical cracks as early as 1999. The inspection report from October 2009 lowered the substructure condition rating from a seven to a six. Issues with the leaking joints and minor substructure deterioration persisted through the August 2015 inspection report until an August 2016 report, which noted the fiber repairs to the various regions of the piers.

Inspection Details: Inspection was conducted on the substructure elements of the bridge. Piers 2 and 4 supporting Waldron Road over I-74 received some fiber patching.

Repair Description: FRP patching repairs were located on various regions of Piers 2 and 4. No full wraps were found on these piers. Patches were located on the corners of the columns on both piers, specifically on the side of the pier facing the direction of the flow of traffic of I-74. Some areas of the crashwall were patched on the northernmost pier. There were a few regions within the patches that appeared to contain voids or possible excess epoxy. Pier 3 did not contain any FRP repairs; however, there was spalling and cracking present throughout the pier.

Figure B.170 Southern pier FRP patching.

Figure B.171 Possible void or excess epoxy in repair region.

Figure B.172 Corner patching of columns at northern pier.

Figure B.173 Larger column patch at northern pier.

Figure B.174 View of entire pier and beam ends.

Figure B.175 Spalling of Pier 3.

Figure B.176 Spalling of Pier 3.

B.2.18 Interstate 70 near Madison Avenue

B.2.18.1 Introduction

This case study investigates the repair of two reinforced concrete piers of a bridge on Interstate 70 in downtown Indianapolis, as denoted by the marker in Figure B.177. The FRP repair work was conducted in October and November of 2017.

The bridge is a multi-span steel girder bridge supported by reinforced concrete piers. The piers are comprised of four circular reinforced concrete columns supporting the pier cap. The piers scheduled for repair are denoted as #4 in the bridge rehabilitation plans (INDOT, 2017) gathered from INDOT and are located in a fenced yard for the Indianapolis Sub-District Maintenance Facility. The equipment shown in the figures are for general maintenance and are not related to the bridge repair. Figure B.178(a) shows the bridge span over Madison Avenue and Figure B.178(b) depicts the piers scheduled for repair, Pier #4.

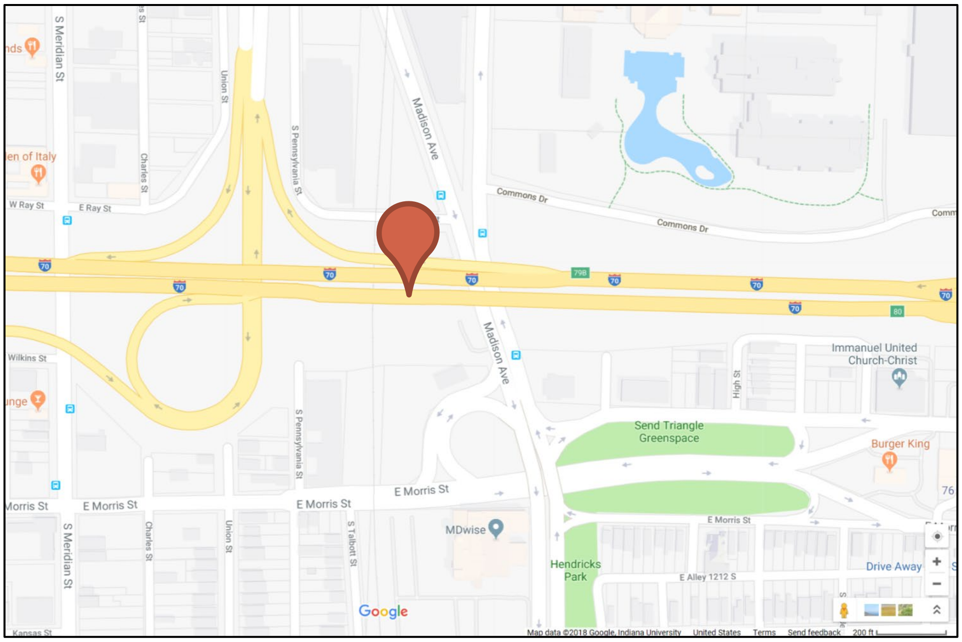


Figure B.177 I-70 bridge location (Google, 2018).

(a)

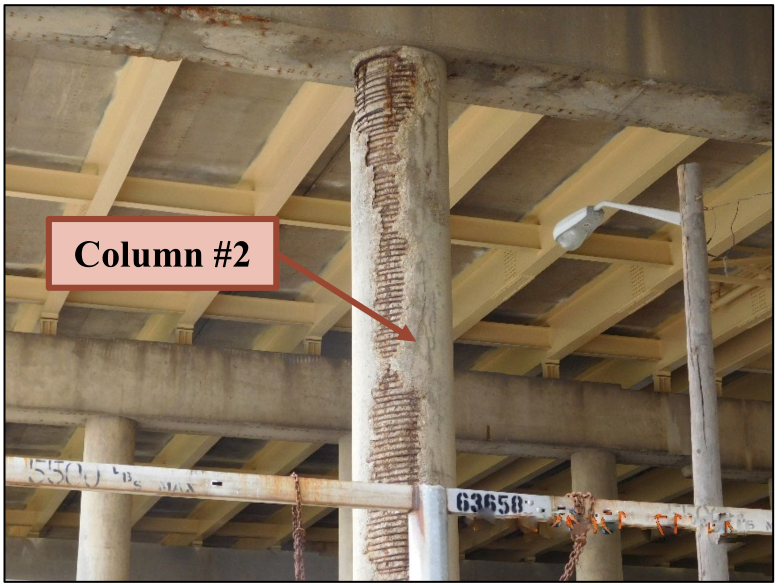
(b)

Figure B.178 I-70 Bridge near Madison Avenue–(a) bridge section spanning Madison Avenue; (b) pier prior to repair and adjacent bridge elements.

B.2.18.2 Before the Repair

The bridge piers were first inspected to assess the deterioration and other deficiencies prior to any repair work. The piers that were scheduled for repairs are located at an expansion joint in the bridge. The pier caps on both piers were heavily deteriorated with some columns exhibiting similar levels of damage as the caps. Large regions of spalled concrete and exposed steel reinforcement were found in multiple locations throughout the piers. Column #2, shown in

Figure B.179, contained the most severe deterioration of all columns. Concrete had spalled, and the steel reinforcing cage was exposed in areas that extended over a majority of the height of the column. In some areas, the concrete loss included all of the cover as well as some of the internal section (see Figure B.179(b)).



(a)

(b)

Figure B.179 Column 2–(a) overall deterioration and (b) deterioration at top of column.

Another column that experienced deterioration was located at the other end of the structure from Column #2. The level of deterioration for this column did not match the severity of the spalling exhibited by Column #2; however, the column had an area of spalled concrete and exposed steel cage as shown in Figure B.180.

Figure B.180 Deterioration of fascia column at I-70 westbound pier.

The pier caps also experienced severe deterioration in three regions. These regions resembled the damage in the columns with large areas of spalled concrete and exposed reinforcing steel. Details of the three deteriorated regions are shown in Figure B.181.

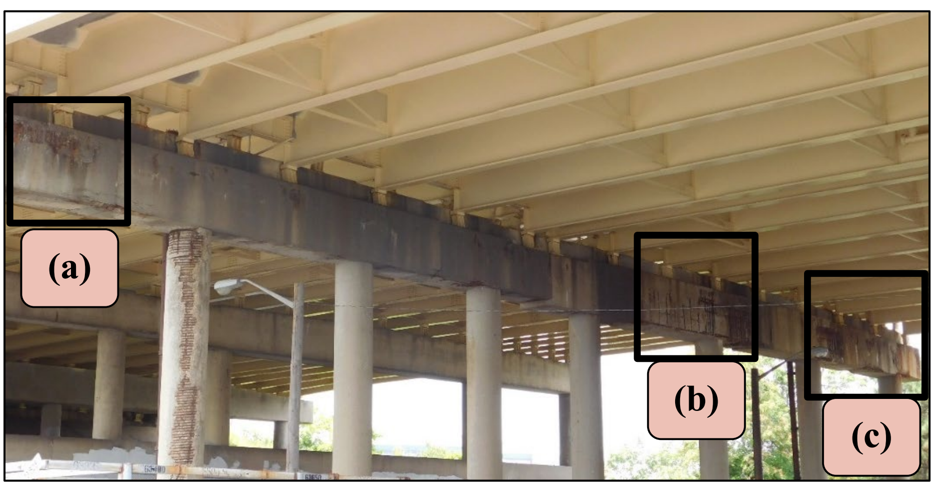


Figure B.181 Deterioration of pier caps.

(a)

(b)

(c)

Figure B.181 (cont.) Deterioration of pier caps.

B.2.18.3 During the Repair

I-70 was kept open to traffic for most of the pier repair work. There was one night that two of the four bridge lanes were closed in order to perform jacking and repairs near an exterior girder (C. Donlan, personal communication, March 21, 2018). The repair procedure began with the restoration of the deteriorated concrete substrate. First, all deteriorated concrete was removed, and the entire surface of the affected steel was exposed. A saw-cutting method was used to remove a majority of the deteriorated concrete. Remaining portions of unsound concrete were then chipped off. Portions of concrete surrounding the reinforcing bars were removed as well. The pier cap and columns, which contained a surface area of about 3,530 ft^2, experienced a removal of approximately 1,350 ft^2 of unsound concrete with an average depth of about 4 in. (C. Donlan, personal communication, March 21, 2018). Once the deteriorated concrete was removed, the steel reinforcement was cleaned, and an epoxy primer was applied to the bars, as shown in Figure B.182. Additional rebar was placed in areas that exhibited rebar section loss greater than 50% or where rebar displayed breaks or ruptures (C. Donlan, personal communication, March 21, 2018). After this process, the areas to be repaired were ready for concrete placement. Shotcrete, or sprayed concrete, was used to restore the members to their original cross-section. Shotcrete was placed over the top of the primer-coated steel bars and missing areas of concrete until the entire section was restored, as shown in Figure B.183 and Figure B.184. Then, the surface was finished smooth. Due to the cold temperatures at the time of these repairs, insulated curing blankets were used to cover the repaired regions after the concrete surfaces were finished, as shown in Figure B.185.

Figure B.182 Cleaned steel and removed concrete substrate prior to shotcrete application.

Figure B.183 Shotcrete material preparation station.

(a)

(b)

Figure B.184 Application of shotcrete to pier cap.

Figure B.185 Use of insulated curing blankets for temperature control after shotcrete application.

The second phase of repair was the surface preparation of the concrete. Grinding was used to roughen the surface for FRP application. This process involves the use of a hand grinder along with a vacuum that is needed to collect debris and dust (Vartiak, 2017). An air compressor with a blower attachment was used after grinding to further clean the surface and remove laitance. The last phase of repair involved the installation of the FRP sheets. The FRP system consisted of a carbon fiber sheet installed using a wet-layup procedure. Due to the relatively small size of the job, the sheets were saturated by hand using a roller. For larger jobs, rolling drums and vats are employed to saturate the FRP sheets prior to applying them to the concrete surface (Vartiak, 2017). Figure B.186 and Figure B.187 show the preparation of the FRP sheets as they were cut and saturated at ground level. The epoxy was a two-part mixture prepared at ground level. In addition to being applied to the FRP sheets, the epoxy was also applied to the concrete surface, as shown in Figure B.188. The pot life of the mixture was approximately 2 to 4 hours once mixed. Temperature and humidity are important aspects to consider when applying FRP systems. At the time of the FRP installation, the temperature was approximately 46°F on November 14, 2017, and the FRP application proceeded without problem. Since it is important to keep all material in a climate-controlled environment, the materials were stored in a heated trailer on-site when not being used.

Figure B.186 Cutting the FRP sheets at ground level.

Figure B.187 Hand-saturating FRP sheets using a roller at ground level.

Figure B.188 Epoxy application to pier prior to FRP sheet application.

Once the sheets were saturated and epoxy was applied to the surface of the concrete, the FRP sheets were placed. Access was gained to the pier cap and portions of the columns using aerial lifts. The application of the FRP sheets is shown in Figure B.189. Applying FRP sheets near the intersection of the columns with the pier caps required precise trimming of the sheets. This process is shown in Figure B.190.

(a) (b)

Figure B.189 Applying saturated FRP sheets to pier cap.

Figure B.190 Detailing of FRP sheets at column to pier cap intersection.

It is important to note that during these processes, the surface of the concrete must be dry. Therefore, the joint above the pier needed to be completely sealed. This joint, at the time of the pier repair, was still leaking and had not yet been repaired. Figure B.191 shows the plastic sheets attached to the deck to protect the underlying pier from the leaking expansion joint.

Figure B.191 Sealing of deck joint to prevent leaking onto pier.

B.2.18.3 After the Repair

The completed repairs were inspected on March 20, 2018, to assess the repair integrity and note any potential issues after being in service. The completed repairs, shown in Figure B.192, appeared to be in adequate condition. Close inspection for the presence of debonding, delaminations, or voids was not possible. Some of the painted areas of the repaired surface were noticeably chipping, as shown in Figure B.192(b). It was evident that the expansion joint was still leaking on the piers, and the repaired surfaces were significantly stained.

(a)

(b)

Figure B.192 Completed Repairs

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APPENDIX C. MIDWESTERN FRP SURVEY RESULTS

C.1 Experience with FRP

The first section of the Midwestern survey was used to identify each state DOT's experience with the use of FRP for the repair and/or strengthening of concrete bridges. Four of the five participants stated that they have had experience with the design and application of FRP systems. The one participant who responded negatively stated that they are currently considering the use of FRP. All four participants with experience exclusively use nonprestressed FRP sheets consisting of either carbon or glass fibers. No use of prestressing or near-surface-mounted technologies was indicated in the responses. Two of the survey participants noted that the reason for choosing carbon fiber is the added strength benefit, and one of those participants added that glass is typically used for confinement applications. Figure C.1 illustrates the number of bridges by state that have received FRP repairs. Figure C.2 depicts the types of FRP systems and the percentage use of each system by state.

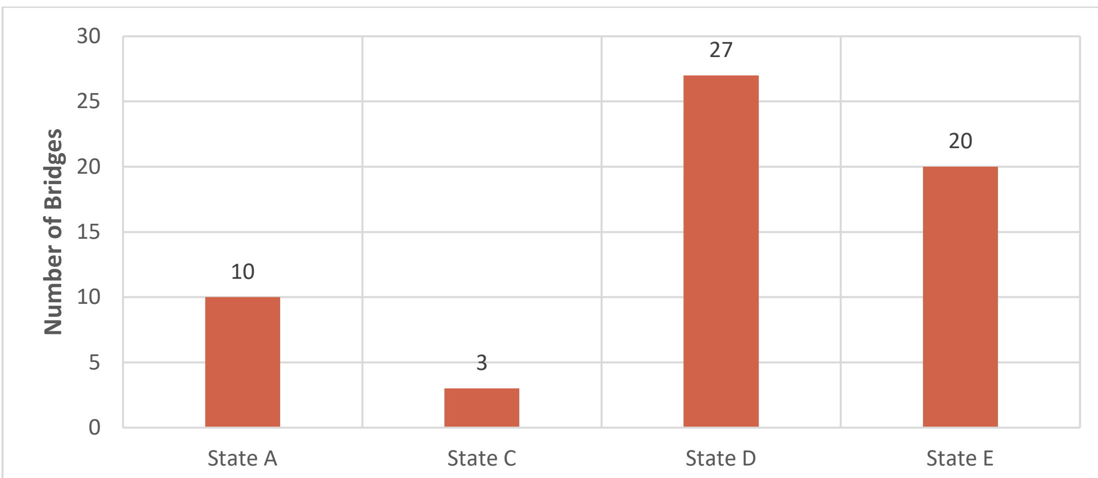

Figure C.1 Bridges to receive FRP systems by state according to survey responses.

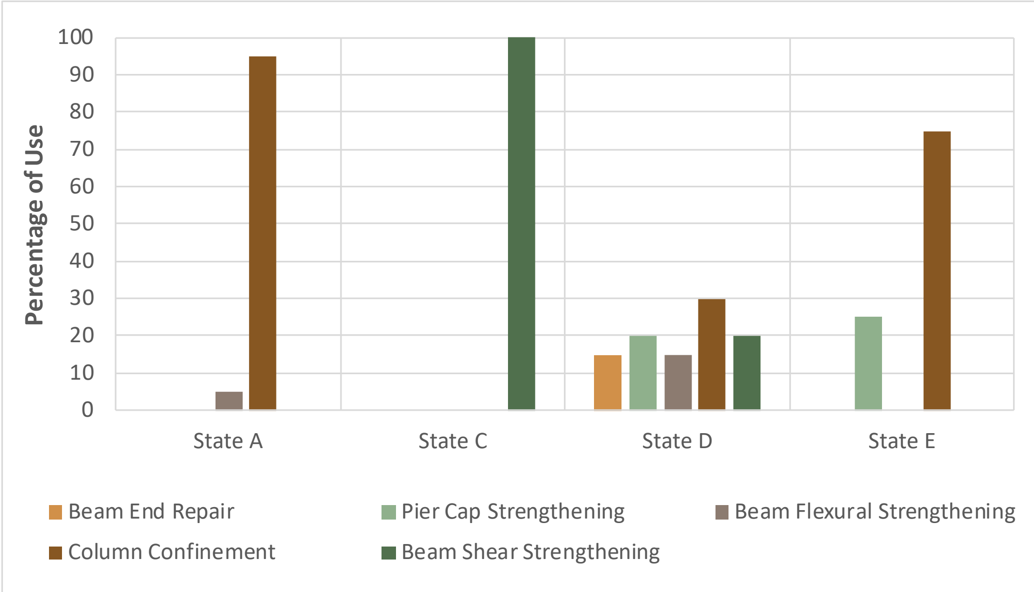

Figure C.2 FRP system types by state according to survey responses.

Participants were next asked to fill out a table to describe their FRP repairs in more detail. This table required information regarding the repaired-element type, FRP system type, installation method, number of related projects, success of the project, and the life of the repair/retrofit currently or at the time of its removal. Repair of three concrete T-girders using externally bonded FRP sheets was reported by State C. The project was successful, and the life of the repairs were 10 to 15 years. Repair of an impact-damaged AASHTO girder with wet-layup sheets on two related projects was noted by State A. These repairs were listed as having "mixed" success with a repair life of 15 years. The State A DOT participant also listed two column repair projects using wet-layup sheets that have been successful after being in service for 1 to 2 years. The use of dry-layup methods to shear strengthen pier caps on eight projects and to confine circular columns on 15 different projects was reported by State E. These projects were noted as being successful, and the repairs were either "very new" or "fairly new." The State D DOT participant provided an extensive list of the bridge elements that have been repaired in the state. The earliest repairs noted were implemented in 1993. Repairs in the state included column wraps, pier cap U-wraps, shear strengthening of pier caps, flexural strengthening of girders, and an emergency girder repair. The State D DOT participant did not provide information on the installation methods, repair success, repair life, or FRP systems used on these projects.

C.2 Criteria for Assessing Suitability of FRP for Repair

The design and application process of FRP systems includes several steps that do not occur simultaneously but are very much dependent upon each other. The predominant section of the Midwestern survey focused on these topics and began by asking participants to describe the criteria they follow when determining if a bridge is a suitable candidate for FRP repair and/or strengthening. The State C DOT participant reported that FRP can be used if cracks in the concrete were "tight" and the concrete surface was in good condition for the attachment of FRP sheets. FRP shear strengthening is considered as a repair strategy "any time a rehab project is being scoped" and if the shear ratings of adjacent elements in the bridge are sufficient, according to the response from the State D DOT participant. The State A DOT participant noted that the criteria for using FRP on concrete bridges are still being developed. However, some insight was given into the benefits of the repair methods they were implementing at the time of the survey (e.g., speed of repair work completion, low dead load addition from the FRP systems, and limitation of moisture ingress to repair patchwork). The State E DOT participant reported criteria for pier cap shear strengthening applications. When analysis reveals insufficient capacity when a significant change in shear loading is expected to occur and the beam is in "fair condition" (e.g. no exposed or deteriorated rebar), strengthening using FRP is a viable option.

C.3 Guide and Reference Use

Various guides and references are used for the design and application of FRP systems for concrete bridges. Survey participants were asked which of these guides and references they use when implementing FRP systems in their states. The popular guide documents along with the number of states that use them are shown in Table C.1. These results were gathered from the four participants who indicated they have experience with FRP systems.

Table C.1 Guide/Reference Use by State

Guide/Reference	Number of States
ACI 440R.2-17, Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures (ACI Committee 440, 2017)	4
AASHTO, Guide Specifications for Externally Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements (AASHTO, 2012)	3
ICRI Guideline No. 330.2, Guide Specifications for Externally Bonded FRP Fabric Systems for Strengthening Concrete Structures (ICRI, 2016)	1
NCHRP Report 678, Design of FRP Systems for Strengthening Concrete Girders in Shear (Belarbi et al., 2011)	1

Table C.1 (cont.) Guide/Reference Use by State

Guide/Reference	Number of States
ICRI Guideline No. 310.2R-2013, Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, Polymer Overlays, and Concrete Repair (ICRI, 2013)	2
Internal DOT Documents (for surface preparation)	1

From the results shown in Table C.1, it is evident that the guideline from ACI 440.2R-17 is the most widely used among the survey participants for design and application of FRP systems (ACI Committee 440, 2017). Half of the survey participants that indicated they have experience with FRP reported they reference ICRI Guideline No. 310.2R (ICRI, 2013) for concrete surface preparation.

C.4 Concrete Substrate Repair and Surface preparation Methods

Proper concrete substrate repair and surface preparation is integral to the success of an FRP system. Various methods of achieving a healthy concrete substrate exist; therefore, survey participants were asked what methods they typically employ for concrete repair and surface preparation. Participants were given a list of methods found from the literature and were allowed to list other methods they may employ. For the participants that indicated they have experience with FRP, the results from their survey submissions are shown in Table C.2.

Table C.2 Concrete Repair and Surface Preparation by State

Substrate Repair Methods	Number of States
Saw Cutting	1
Handheld Breaking	3
Scarifying	1
Scabbling	0
Hydro-Demolition	0
Other (Sandblasting)	1
Surface Preparation Methods	**Number of States**
Grinding	2
Acid Etching	0
Needle Scaling	1

Table C.2 (cont.) Concrete Repair and Surface Preparation by State

Surface Preparation Methods	Number of States
Abrasive Blasting	3
High-Pressure Water-Jetting	1

The most popular methods for concrete substrate repair and surface preparation determined through the survey results are handheld breaking and abrasive blasting, respectively. The combination of saw-cutting and handheld breaking was specified by State A. State A also noted that they "would be open to contractor proposals with a mockup" for other methods of concrete surface preparation.

C.5 Detailing at Bridge Element Intersection Points

A common issue determined through the case studies in Indiana was discontinuity of FRP systems at points of intersecting elements within bridge substructures. If FRP repairs (i.e., sheets or wraps) are terminated prematurely at these locations, chlorides can penetrate and become trapped under the FRP. This may result in accelerated deterioration of the underlying concrete. Participants were asked to discuss any special detailing requirements they specify in regions such as the column-to-crashwall and column-to-pier cap intersection points. Figure C.3 illustrates the figure that was shown for this question in the survey. The State A DOT does not specify any detailing requirements. However, the State A DOT participant did mention the consideration to "avoid terminations in tensile zones or areas that are traditionally occupied by hooked reinforcement because of concern about termination exposure to environment (e.g., pier cap overhang)." The State D DOT participant reported no involvement with projects in which FRP was applied at intersection points between columns and adjoining elements. The other two participants that indicated experience with FRP did not comment on this topic.

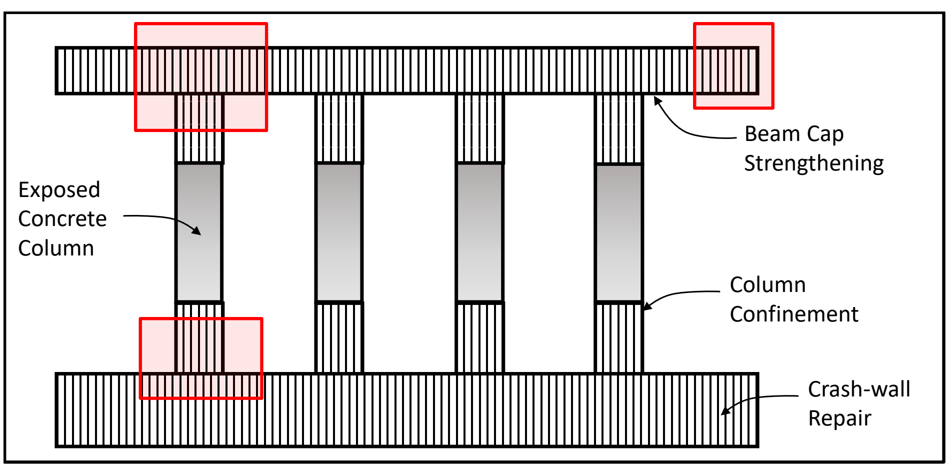

Figure C.3 Bridge element intersection points.

C.6 End Region Repair

The deterioration of beam end regions is a common occurrence in Midwestern states that treat roadways with salt during the cold winter months. Salt-laden water migrates through failed expansion joints in bridges, eventually causing the ends of the underlying beams to deteriorate, as shown in Figure C.4. FRP has been used in Indiana to repair beam ends that have experienced this form of deterioration. To gauge how other Midwestern states handle beam end region repair, participants were asked to describe what methods they employ to ensure the region is properly repaired, especially when the region is obstructed by the bearing pad and/or diaphragm. The State A DOT participant could not specifically comment on this subject; however, the participant did mention the FRP applied to the beam ends may have issues if the joints begin to leak. The State C DOT is currently working on a research project related to the repair of deteriorated I-beam ends. The State C DOT participant noted that this is a common deterioration scenario at locations where the expansion joints have failed. The State D DOT participant stated that contractors are required to remove unsound concrete and subsequently apply FRP to the beam ends. Removal is done manually and with mechanical equipment. The State D DOT participant noted that in most of the cases of beam end repair to date, the girder ends have been integral with the abutment/pier diaphragms, limiting the need to work around the bearings or diaphragms.

(a) (b)

Figure C.4 Beam end region deterioration.

C.7 Anchorage

In order to fully utilize the strength of FRP systems (i.e., flexural or shear strengthening systems), anchorage is commonly used. Anchorage techniques vary according to the literature. Therefore, participants were asked about the anchorage techniques they specify in their designs and what factors are considered when determining which technique is appropriate. Figure C.5 was displayed in the survey and depicts some of the common forms of anchorage found from the literature. The State E DOT does not specify anchorage and the participant stated that any anchorage details are currently left to the discretion of the manufacturer. The State A DOT sometimes specifies anchorage, specifically FRP U-wraps for the anchorage of flexural reinforcement. The criteria that are considered for anchorage are the responsibility of the supplier of the system. The State C DOT currently does not specify anchorage in their projects. The State D DOT specifies the use of FRP anchors (i.e., spike anchors) to anchor shear reinforcement.

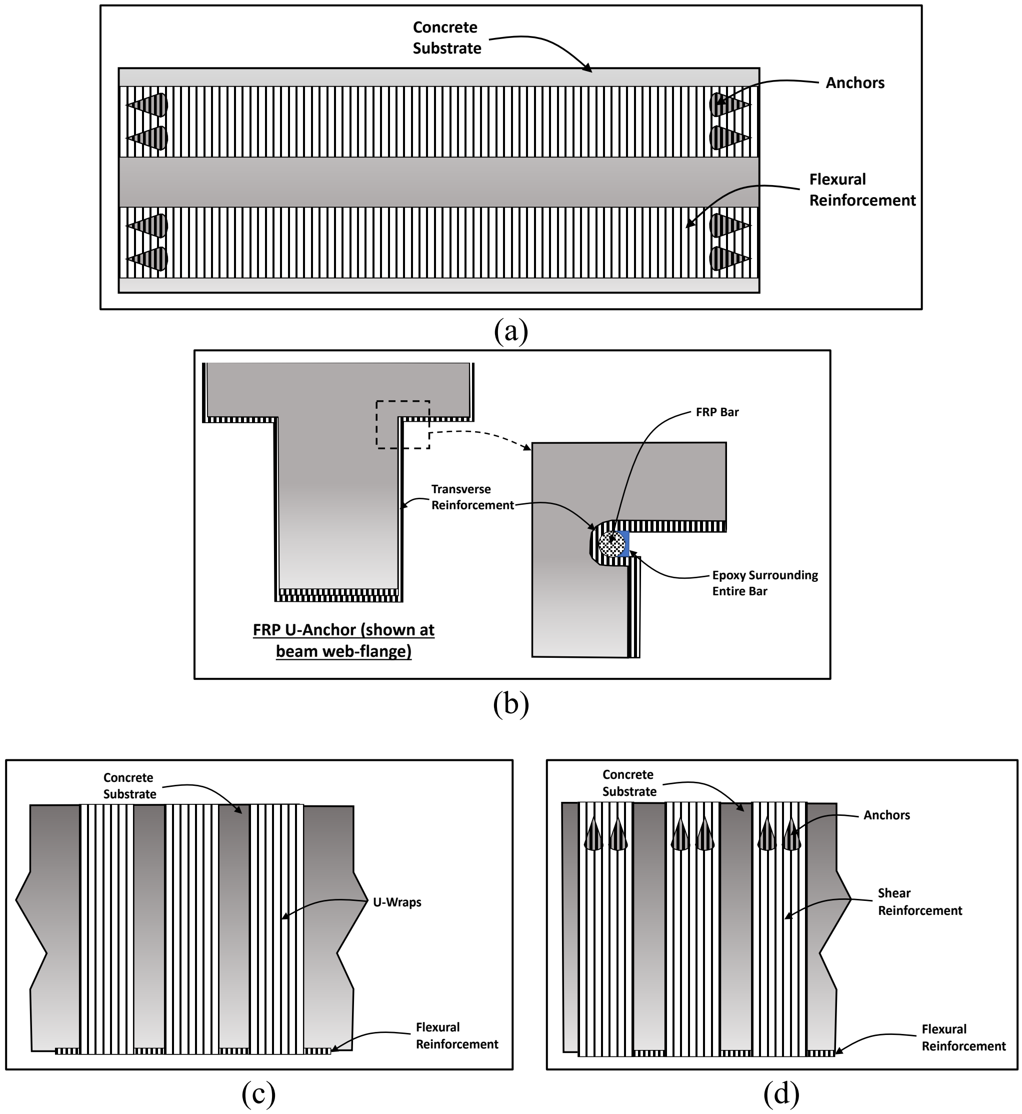

Figure C.5 Common anchorage techniques–(a) FRP spike anchors for flexural reinforcement; (b) FRP U-anchor; and (c) FRP U-wraps for flexural rein.

C.8 Durability

Participants were then asked if they have used FRP specifically to increase the durability of a concrete element or structure. The State A DOT has used FRP for this purpose on about 10 to 12 pier caps and more than 10 columns. The repairs were fairly new at the time of the survey, and their long-term performance could not yet be assessed. For one of the impact-damaged beam repairs from State A implemented in 2002, a loss of bond in some portions was noted during a recent inspection. The State C DOT participant could not extensively discuss this topic. However, the participant did note that the FRP repairs they have implemented seem to be performing well. The State D DOT has utilized FRP for the protection of pier caps and columns in approximately 15 bridges since 2003. No durability issues have been noted in any inspection reports for these repairs.

C.9 Quality Assurance/Quality Control and Post-Repair Monitoring

An important step in the process of FRP implementation is to ensure the integrity of the repair or strengthening system. Through the use of visual inspection and non-destructive evaluation (NDE) techniques, the effectiveness of the FRP system can be ascertained. Participants were asked to choose which methods of quality assurance/quality control they employ from a list of NDE techniques found through the literature. The State E DOT participant stated that the methods of QA/QC for FRP repairs are simply visual and follow the normal routine inspections every 2 years. The State A DOT uses visual and hammer sounding for post-installation evaluation and has not implemented any long-term maintenance program. The State D DOT employs visual and acoustic emission techniques and their long-term FRP maintenance program follows their regular NBIS inspection program.

C.10 Constructability Issues

Since the implementation of FRP systems in Indiana is not yet common practice, one question in the Midwestern survey was directed at constructability issues with FRP systems. By hearing common issues encountered during the application of FRP systems from different state DOTs, the research team could help Indiana avoid these same problems. The State A DOT participant commented that bubbles and trapped air were common issues with constructability in their FRP systems. The participant noted that as the resin hardens, bubbles are formed as the FRP pulls away from the concrete substrate in some areas. To avoid these bubbles, supervision is required to re-apply resin as needed during the curing process. The State D DOT participant reported three major issues: lack of drawings, lack of training, and possible improper fiber placement. The lack of drawings from the consultant results in extended correspondence between the installation crew and the manufacturer to determine how the FRP should be properly installed. Lack of training appears to be common for the field inspection staff. It was noted that improper fiber placement or placing the fabric with the fibers oriented in the incorrect direction has not been observed.

C.11 Contractor Certification/Training

The last question of the survey asked participants to discuss any requirements for contractors installing FRP systems. The State C DOT does not require any certification or training. Three state DOTs reported that certification from the manufacturer is required for the contractor completing the FRP installation. The State E DOT requires a minimum of 5 years of experience installing FRP systems. The State D DOT requires that installers have a minimum of 3 years of experience installing similar systems. The contractor must submit a list of all completed FRP-related projects from the past 3 years using materials from the manufacturer of the system specified for the project being proposed. This list must include at least 10 projects with the proposed system.

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On March 11, 1937, the Indiana Legislature passed an act which authorized the Indiana State Highway Commission to cooperate with and assist Purdue University in developing the best methods of improving and maintaining the highways of the state and the respective counties thereof. That collaborative effort was called the Joint Highway Research Project (JHRP). In 1997 the collaborative venture was renamed as the Joint Transportation Research Program (JTRP) to reflect the state and national efforts to integrate the management and operation of various transportation modes.

The first studies of JHRP were concerned with Test Road No. 1—evaluation of the weathering characteristics of stabilized materials. After World War II, the JHRP program grew substantially and was regularly producing technical reports. Over 1,600 technical reports are now available, published as part of the JHRP and subsequently JTRP collaborative venture between Purdue University and what is now the Indiana Department of Transportation.

Free online access to all reports is provided through a unique collaboration between JTRP and Purdue Libraries. These are available at http://docs.lib.purdue.edu/jtrp.

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