Field Test of a Bridge Deck with Glass Fiber Reinforced Polymer Bars as the Top Mat of Reinforcement

By

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(ABSTRACT)

The primary objective of this research project was to perform live load tests on a bridge deck with GFRP reinforcement in the field under service conditions. The strains and deflections in the span reinforced with GFRP in the top mat were recorded under a series of truck crossings, and these were compared to the span reinforced with all steel bars under identical loading conditions, as well as design values and other test results. Transverse strains in the GFRP bars, girder distribution factors, girder bottom flange strains, dynamic load allowances, and weigh-in-motion gauge results were examined. From the live load tests, it was concluded that the bridge was designed conservatively for service loads, with measured strains, stresses, distribution factors, and impact factors below allowables and design values.

The second objective was to monitor the construction of the bridge deck. To carry out this objective, researchers from Virginia Tech were on site during the bridge deck phase of the construction. The construction crews were observed while installing both the all-steel end span and the steel bottom/GFRP top end span. The installation of the GFRP bars went smoothly when compared to that of the steel bars. The workers were unfamiliar with the material at first, but by the end of the day were handling, installing, and tying the GFRP bars with skill. It was concluded that GFRP bars are an acceptable material in bridge deck applications with respect to constructibility issues.

The third objective was to set up the long term monitoring and data collection of the bridge deck. Electrical resistance strain gauges, vibrating wire strain gauges, and thermocouples were installed in the deck prior to concrete casting to provide strain and temperature readings throughout the service life of the bridge. It was concluded that the span reinforced with GFRP was instrumented sufficiently for long-term health monitoring.
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Chapter 1 – Introduction

1.1 Background

The deterioration of reinforced concrete structures in the United States has become increasingly evident over the recent years. More specific to the bridge industry, the vast majority of bridge replacements and rehabilitations is a result of the deterioration of the bridge deck. The concrete bridge decks become cracked and spalled and are dangerous to motorists. These functionally obsolete decks must then be replaced at high cost and usually with some type of traffic interruption involved. As long ago as 1992, the repair estimate for highway bridges in the United States alone was $50 billion, thus proving the need for more durable, longer lasting bridge decks (Bedard 1992).

Deterioration of bridge decks is caused by water and salts leaking into the deck through cracks on the surface, or by penetrating through uncracked concrete. Once the combination of water and salts penetrates to the top mat of steel reinforcement, the corrosion process ensues, resulting in an increase in the volume of the bar, which then causes the concrete to spall on the surface. Corrosion is the center of the bridge deck durability issue presented to engineers. If the corrosion can be eliminated or reduced, then the bridge deck service life can be increased without costly and time consuming replacement and rehabilitation procedures.

Many different methods have been employed to extend the service life of bridge decks. In Timothy Bradberry’s paper “Concrete Bridge Decks Reinforced with Fiber-Reinforced Polymer Bars,” he states that various departments of transportation have used techniques such as increased concrete cover, epoxy-coated steel bars, concrete sealants, deicing management, galvanized or stainless steel bars, cathodic protection systems, concrete admixtures, and high performance concretes (2001). Most of these methods are either very expensive and therefore not cost effective, or have not proven to solve the corrosion issue effectively. As a result, researchers have turned to composite materials for use in bridge decks as reinforcement to replace the top mat of steel reinforcement. Fiber-reinforced polymer (FRP) bars have excellent corrosion resistance and strength properties that allow them to be used in this type of structural application.
The development and use of FRP’s started in the 1940’s, when they were used primarily in the aerospace industry as a lightweight, high strength and stiffness material (TTI 2000). Since then, they have been used in other areas such as the manufacturing industry, automobiles, and sporting goods. FRP continues to be an expensive material to develop and implement. However, compared against the life-cycle costs of a bridge deck that must be replaced two to three times during its service life, FRP reinforcement is becoming a more viable option.

The question has been raised that if FRP reinforcements are suitable materials to replace traditional steel reinforcement in the top mat of bridge decks, then why not replace the bottom mat and use FRP as the positive moment reinforcement in the deck as well? Unfortunately, the stress-strain properties of FRP are not ideal for acting as the main reinforcement in the absence of steel. The stress-strain curve of FRP is linear-elastic to failure, exhibiting a brittle failure. This is unlike traditional reinforcing steel, which has a specific yield point in its stress-strain behavior and can undergo large strains well beyond yield without a reduction in load carrying capacity, making it a ductile material. Also, the modulus of elasticity of glass FRP (GFRP) is much less than that of steel, making steel the more desirable main reinforcement material for crack and deflection control. Therefore, researchers have focused on bridge deck applications with steel as the bottom mat of reinforcement and FRP as the top mat.

The Virginia Department of Transportation (VDOT) along with the Virginia Transportation Research Council (VTRC) partnered with the Virginia Polytechnic Institute and State University (VPISU) to implement this bridge deck durability technology into a new structure. The structurally deficient and functionally obsolete Route 668 bridge over Gills Creek in Franklin County, Virginia is pictured below in Figure 1.1. The bridge was replaced with a new, three-span, two-lane bridge, completed in July 2003. One end span of this bridge was constructed with GFRP reinforcement in the top mat replacing the traditional steel reinforcement. Also, the bridge was instrumented with strain gauges, thermocouples, and deflectometers in both end spans to monitor the bridge during both short-term live load tests and long-term evaluations. Prior to the construction of this new bridge, and through funding from VTRC, researchers at VPISU constructed and tested a prototype bridge deck in the laboratory, utilizing GFRP
in the top mat of reinforcement. This prototype deck was tested to verify the design of the Gills Creek bridge, evaluate the deck’s response under service load conditions, and determine the mode of failure and ultimate strength under various loading conditions. The tests showed the design met or exceeded all design criteria.

This project is sponsored by VDOT and the primary objectives are:

- live load testing on a bridge deck reinforced with GFRP bars in the field under service condition,
- monitoring the bridge deck construction,
- and setting up the long term monitoring and data collection of the bridge deck.
1.2 Objectives

1.2.1 Objective One

The primary objective of this research project is to perform live load tests on a bridge deck with GFRP reinforcement in the field under service conditions. More specifically, the strains and deflections in the span reinforced with GFRP in the top mat were recorded under a series of truck crossings, and these were compared to strains and deflections in the span reinforced with all steel bars under identical loading conditions. These observations are to determine whether or not the span reinforced with GFRP in the top mat is behaving as designers would predict. Also, this objective is very important in that the data from the live load test serves as pre-service life base line information for subsequent tests during the life of the structure.

In order to carry out this objective, several tasks were accomplished. The Gills Creek bridge was constructed and completed in July 2003 by A.R. Coffey and Sons, overseen by VDOT. During the construction, and prior to casting of the deck slab, researchers from VPISU instrumented various steel and GFRP bars in the deck with strain gages and placed other embedded strain gages and thermocouples in the deck slab. In addition, prior to the live load test, strain gages were adhered to the bottom and top flanges of various girders in both end spans, and deflectometers and weigh-in-motion gages were attached to the bottom flanges of various girders in both end spans. When the bridge, railings, approaches, and guardrails were all completed, researchers from VPISU and workers from VDOT performed multiple live load tests on the bridge while recording data from the embedded and external sensors with a mobile data acquisition system. These tests comprised a loaded VDOT dump truck with measured axle weights rolling over the bridge at various speeds and in different places on the deck. After the tests, the data were reduced and analyzed.

1.2.2 Objective Two

The second objective of this research project is to monitor the construction of the bridge deck. This is of high interest because GFRP is a relatively new material in bridge construction. Most construction workers are not familiar with this material and its
properties, which vary greatly from steel’s. For example, GFRP bars are quite a bit lighter to carry than steel bars. However, they are much less stiff and deflect quite a bit more than steel bars when placed in a bridge and walked on.

To carry out this objective, researchers from VT were on site during the bridge deck phase of the construction. The construction crews were observed while installing both the all-steel end span and the steel bottom/GFRP top end span. Observations were made such as the time of installation for both steel and GFRP bars, comments from the workers, spacing and workability of the bars, flexibility of the mats, and spacing of the bar chairs.

1.2.3 Objective Three

The third objective of this research project is to set up the long term monitoring and data collection of the bridge deck. This is to determine how the bridge deck responds to service loads over time. Concrete cracking, the full cycle of seasons, and deicing methods in the winter are but a few of the events during a bridge deck’s life that can cause long term deterioration. Knowing the actual conditions in the deck after periods of one, five, or ten years can give researchers and engineers a better understanding of the design guidelines and assumptions of bridge decks reinforced with GFRP. Also, the health of the bridge deck and the GFRP bars can be monitored constantly throughout their lives and problems can be remedied if there is a question of safety.

To achieve this objective, researchers installed different types of sensors in the bridge deck prior to casting. These sensors were connected to a long-term data acquisition system at the bridge. Future researchers can download important strain and temperature data to a laptop computer at the bridge site. This data can then be compared to data observed from the first live load test, other live load tests, or any other previous data acquisitions.

1.3 Thesis Organization

Chapter 2 presents a review of the literature and previous research on this topic and related topics. Chapter 3 discusses the methods and materials used to carry out the objectives listed above. Chapter 4 is a presentation and discussion of the results of the
field testing, as well as the observations made during the construction of the bridge deck and the implementation of the long term monitoring system. Chapter 5 presents conclusions to the research project, as well as recommendations resulting from the project and needs for further research.
Chapter 2 – Literature Review

2.1 Material Properties

Fiber-reinforced polymer (FRP) bars are a composite material comprising reinforcing fibers and a resin matrix (Yost 2001). The two constituents act together to form a material that is very desirable for many applications. The fibers are oriented nearly longitudinally, giving the bar high strength and stiffness properties in that direction. The resin matrix keeps the shape of the composite while protecting the fibers, and also distributes the stresses to the fibers, providing the composite action of the material (Bradberry 2001).

There are three types of fibers that are commonplace in FRP applications: glass, carbon, and aramid (Erki 1993). Glass fibers are common in structural FRP applications due to their comparatively low cost and high tensile strength. Disadvantages of glass fibers include low elastic modulus and resistance to moisture, sustained loads, and fatigue loads (TTI 2000). However, their properties are adequate for most common structural applications. The two most common types of glass fibers are E- and S-glass. E-glass fibers are used mostly in reinforced concrete applications due to their higher resistance to alkaline environments than S-glass fibers. E-glass is also less expensive than S-glass, but has less strength, stiffness, and ultimate strain characteristics (TTI 2000).

The resin matrix for FRP composites can either be a thermoset resin or a thermoplastic resin. Thermosets cannot be remolded once they have been cured with heat and catalyst. Common thermosets are polyester, vinyl ester, and epoxy. Thermoplastics, conversely, can be reshaped with heat after initial curing. Some examples of thermoplastics are PVC, polyethylene, and polypropylene (Bedard 1992). Thermosets are more widely used in FRP than thermoplastics. This is mainly because of their lower cost, but also due to their low melt viscosity, good fiber impregnation, and low processing temperatures (Hyer 1998). Within thermosets, vinyl ester resins are now the most common matrix material in FRP bars. This is due in part to their higher resistance to chemicals and temperature than polyester (TTI 2000).
FRP bars are manufactured using a process called pultrusion. The fibers are pulled through a bath of resin matrix, and then heated and cured in a die, producing bars made up of both the strong fibers and the binding resin matrix (Bedard 1992). Moreover, FRP bars are typically deformed in order to provide proper bond in applications with concrete. This deformation is usually achieved by wrapping the bar with fibers in a helical pattern (TTI 2000). Additionally, the bar’s surface can be impregnated with sand in order to improve the bar’s coefficient of friction.

2.2 Mechanical Properties

The mechanical properties of FRP, and more specifically GFRP, differ greatly from those of steel. Generally, the tensile properties of GFRP are as follows: high strength to weight ratio, low modulus of elasticity, and low failure strain. Other mechanical properties include: excellent corrosion resistance, very low specific gravity, and high transverse coefficient of thermal expansion (CTE).

One of the challenges presented to engineers in dealing with GFRP in structural applications is the variability of their mechanical properties between manufacturers (Bradberry 2001). There is no standardized stress-strain diagram for the different types of FRP, including GFRP (Brown et al. 1993). Therefore, strength properties listed by the manufacturer should be verified by the engineer. Unlike steel, a range of values defines strength properties for GFRP. According to the Texas Transportation Institute (2000), common ranges for the strength properties of GFRP bars are as follows:

- Tensile strength: 75-175 ksi.
- Modulus of elasticity: 6000-8000 ksi.
- Shear strength: 22 ksi.
- Ultimate strain: 0.035-0.05 in/in.

However, the ACI Committee 440’s Guide for the Design and Construction of Concrete Reinforced with FRP Bars (ACI 2003) recommends different ranges for these same properties:

- Tensile strength: 70-230 ksi.
- Modulus of elasticity: 5100-7400 ksi.
• Ultimate strain: 0.012-0.031 in/in.

As can be seen from above, the tensile strength of GFRP bars is greater than that of steel. However, the tensile modulus is approximately one fourth of that of steel. This causes deflections and crack widths to increase (Brown et al. 1993). Another consideration of GFRP’s low tensile modulus is the necessity for more bar chairs during the construction phase of the deck (Bradberry 2001).

The stress-strain curve of FRP is linear-elastic to failure, with no yield plateau. This results in a non-ductile failure of the bar, an undesirable characteristic in traditional structural design. Therefore, concrete sections reinforced with FRP must be designed as overreinforced sections so that concrete crushing occurs prior to FRP rupture. While this is still a brittle failure, it is considered more ductile than failure due to FRP rupture (Bradberry 2001).

Shear strength of FRP bars is governed primarily by the properties of the matrix material. On the average, shear strength of GFRP bars is one fourth the shear strength of steel bars (TTI 2000).

FRP’s high corrosion resistance is the primary reason this material has been introduced into structural applications. However, its transverse CTE is about four times higher than that of concrete, which could cause radial cracking extending from the bar to the surface under large increases in temperature. Therefore, concrete cover minimums should not be reduced because of the corrosion resistance of the material. FRP’s longitudinal CTE is similar to that of concrete (TTI 2000).

The specific gravity of FRP is very low, ranging from 1.25 to 2.0, making it light to carry and transport, and easier to install (TTI 2000). However, GFRP has a tendency to float when vibrated in some concrete mixtures. Therefore, bars must be tied down and secured to either the bottom mat of steel, the bar chairs, or the bottom of the deck forms (Bradberry 2001).
2.3 Experimental Results and Designs

2.3.1 Laboratory Testing

Cawrse (2002) constructed and tested in the laboratory a full-scale prototype of the Gills Creek bridge deck slab prior to its construction in the field. The bridge deck was tested in order to verify the design, as well as to determine the constructability of the bridge deck. The 24 ft 0 in. by 17 ft 4 in. deck was 7 ½ in. thick with an extra 1 in. of thickness in the overhangs, and was reinforced with steel bars as the bottom mat and GFRP bars as the top mat. Four separate tests were performed on the bridge deck – two overhang tests, one interior girder test, and one cantilever test. In each test, the deck was loaded at service levels, and then loaded to failure. Throughout the tests, strains, crack widths, and deflections were recorded.

Overall, the design was considered valid and it was determined that the bridge deck design was adequate to resist the design loads. More specifically, deflections in both overhang tests as well as the interior girder test were well under allowables at service load, while deflections in the cantilever test were above the recommended allowable. However, the large deflections resulting from the cantilever test were not an issue for the construction of the Gills Creek Bridge because its three spans were simple and not continuous. Similar to the deflection results, stresses in the reinforcement for both overhang tests and the interior girder test were under the recommended allowable, but the stresses during the cantilever test were greater than allowable. As for crack widths, measured values at service loads were well under allowable limits in both overhang tests as well as the interior girder tests. Again, crack widths during the cantilever test at service loads were equal to the recommended allowable, but much greater than predicted values.

Shortcomings in the design of the continuous region and prediction of deflections, stresses, and crack widths are cause for some concern. Design of GFRP reinforcement in negative moment regions for continuous structures must be further investigated. However, this was not a consideration in the researcher’s conclusions, because the bridge constructed in the field comprised three simple spans.
Brown and Bartholomew (1993) performed flexure tests on six beams reinforced with GFRP bars. The beams were 6 in. square in cross section and 30 in. long, and each was reinforced with one No. 3 GFRP bar in the bottom center. The beams failed in a ductile manner, with strengths very close to predicted values using the same ultimate strength design method as used for steel-reinforced beams. However, deflections were much greater – around four times that of steel-reinforced beams. The lower modulus of the GFRP bars resulted in increased cracking and higher deflections, and also greater crack widths. The researchers pointed out that another expression must be developed for the effective moment of inertia due to the very large crack widths that were observed. Overall, the researchers concluded that “FRP reinforcement can provide an attractive alternative to steel for structural applications in aggressive environments.”

Michaluk et al. (1998) tested eight one-way slabs in the laboratory under static conditions to determine their flexural and shear limit states as well as their behavior prior to cracking, after cracking, ultimate capacities, and modes of failure. All of the slabs were 11.5 ft long and 3.28 ft wide, with half of them having a thickness of 6 in. and half of them having a thickness of 8 in. Five of the slabs were reinforced with GFRP, one with CFRP, and two with steel. The slabs were tested in flexure using a spreader beam system. The authors concluded that the slabs with FRP reinforcement behaved in a bilinear elastic manner up to failure, with a significant reduction in stiffness after cracking for the GFRP slabs as compared to the CFRP and steel slabs. The slabs reinforced with GFRP exhibited adequate warning of failure with high deformations and crack widths. The under-reinforced slabs with GFRP failed in rupture of the reinforcement; however, the bars did not reach their ultimate tensile strength from tensile tests. This could be because of localized failures of the fibers at the crack due to sudden transfer of forces from the concrete to the bars at cracking. The over-reinforced slabs with GFRP failed in shear of the reinforcement over the very large shear crack, instead of the expected crushing of the concrete in the compression zone. Finally, the code equations for shear over-estimated the shear capacity of the slab with GFRP.

Two bridge deck slabs were tested in the laboratory by Hassan et al. (2000) under static loading conditions. The geometry and boundary conditions were the same for each. The slabs were 23.6 ft wide by 9.8 ft long, with a thickness of 8 in. One slab was
reinforced completely with CFRP, while the other one consisted of steel reinforcement on the bottom and GFRP on the top.

For both the CFRP and steel/GFRP decks, all the failure modes were punching shear, with the steel/GFRP deck exhibiting a higher punching shear strength due to the presence of the steel reinforcement. Deflections were very small at loads of up to more than double the service load.

An analytical model was developed for the slab using the finite element method. For the continuous bridge deck model, the predicted behavior of the slab agreed very well with the experimental data. Additionally, a parametric study was then implemented using the analytical model in order to determine optimum design boundary conditions. The researchers concluded that, from the results of the laboratory testing, as well as the analytical studies, the top reinforcement in continuous slabs is negligible in the slab’s punching shear capacity. Also, in order to satisfy serviceability and strength requirements for slabs with span to depth ratios between 9 and 15, 0.3% CFRP top and bottom reinforcement in both directions, or 1.2% GFRP bottom transverse reinforcement and 0.6% top transverse reinforcement with 0.6% top and bottom longitudinal reinforcement is safe. When using these recommended reinforcement ratios, deck slabs reinforced with CFRP and GFRP have ultimate strengths 1.8 and 1.6 times the required values by code.

Rahman et al. (2000) built a bridge deck prototype in the laboratory and tested it under static loading conditions. The deck slab was 20 ft square with a thickness of 7 in. The slab was reinforced with a CFRP grid as both mats of reinforcement, and was instrumented in order to obtain stresses and deflections during the test. The deck slab was loaded to cracking load at each wheel patch location, and then was loaded cyclically from zero stress to service conditions to simulate 50 years of truck traffic. Finally, the slab was loaded to failure. The researchers concluded that the behavior of the slab under service load conditions was satisfactory. Moreover, the deflections were within the L/800 limit and the stresses in the reinforcement were only 7% of its ultimate strength. The constructibility of the CFRP grid reinforced deck was deemed to be very satisfactory. The deterioration of the bridge deck under cyclically applied service load was negligible. Finally, the slab exhibited a very high ultimate strength of more than five times the
design wheel load. Overall, the CFRP grid reinforcement was judged suitable for bridge deck slabs.

Tannous and Saadatmanesh (1998) studied the change in mechanical properties of GFRP bars when exposed to the environmental effects of concrete and deicing salts. They examined the moisture absorption and the change in mechanical properties of the bars under accelerated exposure to environmental attack. Beams reinforced with GFRP were cast and placed in two deicing salt solutions for 1- and 2-year durations. Ten 8 in. by 16 in., 8 ft long beams were cast, with half of the beams reinforced with E-glass/polyester GFRP bars and half reinforced with E-glass/vinyl ester GFRP bars. The beams were tested in flexure after their aging periods, using one beam of each type as a control specimen.

The results showed that the bars exposed to concrete only lost less than 4% of their strength after one year. On the average, when exposed to the de-icing salts as well, the E-glass/vinyl ester bars lost less of a percentage of ultimate strength after one or two years than the E-glass/polyester bars. The maximum percentage of ultimate strength lost was 12.8% by the E-glass/polyester bars after 2 years. Therefore, the researchers stated that the “vinyl ester [bars] showed lower diffusivity and better resistance to chemical attack than polyester” (Tannous and Saadatmanesh 1998). The authors pointed out, however, that the beams remained uncracked while submerged in the salt solution and that the rate of penetration into the bars was very low. This would not be the case in the field, as most reinforced concrete structural elements would be cracked.

Bradberry (2001) provided a structural design for a GFRP-reinforced bridge deck. The design was to be incorporated in the aforementioned Sierrita de la Cruz Creek bridge in Texas. The author mentioned some challenges inherent in the design of GFRP-reinforced slabs. GFRP’s stress-strain curve, which is linear-elastic to failure with no yield point, is a concern. Therefore, the bridge deck must be designed as an overreinforced section, ensuring that the section fails in crushing of the concrete and thus preventing the more brittle tension failure of the GFRP bars. Furthermore, serviceability limit states, such as creep failure of GFRP bars and crack widths, were very important. Minimizing crack widths was the controlling case of the design.
The bridge deck for the Sierrita de la Cruz Creek bridge was to have an 8 in. thickness, with bridge geometry defined previously. The design forces for the one-way slab were determined using a 1 ft strip of slab over knife edge supports. The author concluded that strength and allowable stress limit states would not be an issue due to the slab’s close bar spacing necessary to control crack widths. The maximum crack width, recommended by the Canadian Standards Association, was 0.02 in. Ensuring this value meant that the bar reached only 15% of its guaranteed ultimate strength. Additionally, the long-term strength of GFRP was a design factor. Consequently, the author used the bar’s residual strength at the end of its design life in his analysis. Also, the estimated concrete strength at the end of its service life was used in the design. Combining the assumptions of lowest possible long-term strength of GFRP bars and highest estimated concrete strength over time ensures that the failure mode for the structure remains concrete compression for the life of the structure. Finally, the author concluded that bridge decks designed with GFRP reinforcement are a very good alternative to all steel-reinforced decks in harsh environments. However, more research must be conducted, especially concerning the long-term strength of GFRP as well as the ductility of GFRP reinforced sections.

2.3.2 Field Testing

Bice et al. (2002) instrumented the Sierrita de la Cruz Creek bridge in Potter County, Texas. The bridge is made up of seven equal spans of 79 ft each, with a total bridge length of 533 ft. The width of structure is 44 ft 9 in. and the deck thickness is 8 in. Spans 6 and 7 were constructed with GFRP bars as the top mat and epoxy-coated steel bars for the bottom. Spans 2 (all steel) and 6 (hybrid) were instrumented to record data from live load tests at certain time periods of the structure’s service life.

After about one year under service conditions, the bridge was tested. The load was applied using one or two Texas Department of Transportation dump trucks, each weighing approximately 54 kips. The front wheels each provided a load of about 5.4 kips directly on the deck, while the rear tandem wheels were placed on top of channel sections, which rested on top of wood blocks spaced 92 in. apart. This method was used
in order to load the overhang more than the barrier would allow. Each block provided a concentrated load of 21.6 kips directly on the deck.

No new cracks had formed during that initial year. Additionally, no old cracks had propagated further when load was applied. Therefore, the researchers assumed that all cracks on the bridge deck were shrinkage cracks. Furthermore, no strain gage data was reported by the researchers due to vandalism of the gages. One noteworthy result from the field research was the presence of a longitudinal crack close to the center stripe of the bridge in the span reinforced with GFRP. This crack was not present in the span reinforced with epoxy-coated steel, and would lead one to believe that the GFRP span was more flexible than the steel span in that part of the deck. Unfortunately, none of the tests carried out by the researchers would have caused this crack to widen or propagate.

Thippeswamy et al. (1998) instrumented a bridge for live load testing and observation. The bridge is a three-span, continuous structure over Buffalo Creek in McKinleyville, WV. The total bridge length is 177 ft, the deck thickness is 9 in., and the bridge is reinforced entirely with GFRP. The objective of this research was to “demonstrate the ability of FRP reinforced concrete technology to meet the demands of transportation infrastructure applications, including cost, construction productivity, and long-term durability” (Thippeswamy et al. 1998). Two different types of GFRP were used. Both bars were made up of E-glass fibers and a polyester resin, with a few modifications. The researchers commented on the constructibility of the bridge, noting that the bars were both lightweight and easy to carry around as well as that they were installed in relatively the same manner as epoxy-coated steel bars. Moreover, it was noted that gloves should be worn during installation due to the glass fibers of the bars, bar chairs should be spaced closer due to the increased flexibility of the GFRP mat, and that the mats should be tied down to the formwork in various locations in order to prevent floating of GFRP bars due to their lower specific gravity. Three load tests have been completed on the bridge so far, with more planned for the future. The load tests were performed in a static fashion, with trucks positioned to result in maximum positive and negative moments. The maximum bar strain resulting from the tests, when prorated for an AASHTO HS-25 loading, equated to a change in stress that was about 3% of the ultimate tensile strength. Additionally, the maximum live load deflection for the bridge,
when prorated for HS-25 loading as before, was equal to span/1500, which is well below the limit of span/1000 for urban areas. The authors concluded that the load test results showed that the bar stresses and deck deflections were well within allowable limits, and that the bridge would continue to be monitored through future tests in order to determine the long-term response of the bars.

2.4 Conclusions and Recommendations

From the research discussed in this review, it is clear that the use of GFRP as reinforcement in bridge decks is gaining momentum as an alternative to traditional black steel and epoxy-coated steel. The material’s high resistance to corrosion lends itself to applications in harsh environments, and its high strength enables it to be used as top mat reinforcement similar to steel. However, the material is relatively new and innovative, and questions remain regarding its properties, performance, and constructibility.

Due to the very nature of new materials, GFRP’s performance and durability in long-term applications must be addressed. The question of its long-term strength in the concrete environment must be explored. Initial research has shown, as discussed previously, that GFRP bars made up of E-glass fibers and vinyl ester resins seem to resist long-term degradation much better than those made up of polyester resins. However, real-life applications in the field must be investigated further.

Similarly, FRP-reinforced decks have been constructed and tested in both the laboratory and the field. However, the lexicon of test results, especially in field conditions, is thin at best, and more field tests validating bridge deck designs and therefore ensuring a safe and economical design for bridge decks reinforced with GFRP are necessary.

Again, due to the fact that GFRP is a new material for bridge deck applications, its constructibility must be investigated. Researchers have commented on constructibility in both the field and the lab. However, issues regarding installation of GFRP bars remain.

This thesis will attempt to answer questions regarding the short- and long-term strength of GFRP-reinforced bridge decks. Additionally, the issue of constructibility will be addressed. Through application and testing in the field under real life service
conditions, this research will broaden the industry’s knowledge and awareness of GFRP reinforcing as a new and innovative material in structural applications.
Chapter 3 – Methods and Materials

3.1 Introduction

This chapter presents the methods and materials used in this research project. First, section 3.2 discusses the laboratory testing of GFRP bars at Virginia Tech, included tensile testing and thermal testing. Next, section 3.3 discusses the Route 668 Gills Creek Bridge, including an overview of the bridge itself and its construction. Section 3.4 presents the bridge deck instrumentation, discussing electrical resistance strain gauges, vibrating-wire strain gauges, and thermocouples in Spans A and C. Section 3.5 describes the casting of the bridge deck and procedures involved in that. Section 3.6 discusses the preparations involved in the testing, including the girder instrumentation and the data acquisition system. Finally, section 3.7 explains the live load testing, with descriptions of the truck, its orientations, the type of tests, and the overall testing sequence.

3.2 Laboratory Testing

3.2.1 Tensile Testing

A sample of the glass fiber reinforced polymer (GFRP) bars to be used in the deck of the Route 668 Bridge over Gills Creek was tested in the Structures and Materials Laboratory at Virginia Tech. These tests were performed in order to determine representative tensile properties for the bars installed in the bridge. These tensile properties were ultimate strength and elastic modulus. The sample consisted of seven No. 6 bars, each 72 in. long. In a standard tensile test, the grips of the testing machine would locally crush the fibers of the FRP. Therefore, 20 in. long steel pipe sleeves were bonded to both ends of the bar, giving the specimen a 32 in. gauge length. The pipe used was 1¼ in. nominal diameter, standard weight pipe. The bars were bonded to the pipe with an epoxy mixture consisting of three parts sand, five parts epoxy resin, and one part epoxy hardener. The bars were placed and centered in the steel pipe with a PVC cap on the outside end, and then the epoxy mixture was poured in. When the end anchor was full, a PVC cap with a hole in the middle was slid over the bar and secured on the inside end of the pipe so that the epoxy mixture could cure without contamination from the
outside environment. The specimens cured one end at a time, allowing 24 hours to cure before working on the opposite end.

The specimens were gripped in a SATEC Universal Testing Machine and a clip-on extensometer was attached to record strain. The specimen was then pulled at a constant rate until approximately 70 percent of its predicted ultimate strength. At that point, the test was paused and the extensometer removed. The test was then resumed and continued until failure of the bar.

### 3.2.2 Thermal Testing

The question of transverse expansion of GFRP bars due to temperature increases, and the resulting stress on concrete was raised during construction of the Gills Creek Bridge. More specifically, VDOT wanted to ensure that the amount of clear cover concrete over the top mat of GFRP reinforcement was adequate to prevent cracking and spalling due to thermal expansion of the bars. A sample of GFRP bars representative of the bars to be installed in the bridge deck was tested at Virginia Tech to obtain reasonable values for longitudinal and transverse coefficients of thermal expansion (CTE). Two types of CTE tests were completed on bars provided by the manufacturer. The first type of test utilized specimens machined from No. 2, No. 3, and No. 6 bars. The tests were carried out using a linear dilatometer in the chemistry department at Virginia Tech. The specimens were tested in both the longitudinal and transverse directions. Longitudinal specimens for the No. 2 and No. 3 bars were obtained by machining cross-sections of the bars, about 3/8 in. long. Transverse specimens for the No. 2 and No. 3 bars were obtained by taking a specimen like that of the longitudinal tests, and machining off a flat portion on opposite sides of the bar, in order to give the push rod and the stop in the dilatometer flat spots to contact the specimen. For the No. 6 bars, 3/8 in. cubes were machined for both the longitudinal and transverse tests.

The specimens were then tested in the linear dilatometer. The temperature was varied from room temperature to 140 degrees Fahrenheit on some tests and 165 degrees Fahrenheit on others. Two specimens of each bar size were tested for both longitudinal and transverse CTEs. The change in length of the specimen was recorded, and the CTE was determined from the slope of the linear portions of the strain/temperature diagram.
An additional CTE test setup was devised for these bars in order to provide more data with which to make a recommendation on the appropriate amount of concrete cover for bridge. This test consisted of adhering an electrical resistance (ER) strain gauge circumferentially to the surface of the bar, attaching lead wires to the gauge, heating the bar to 140 degrees Fahrenheit in an oven, and recording the strain with a strain indicator. Two No. 6 bars were used, and each bar was tested twice. This is because negative strains were recorded in both bars when they returned to room temperature after being heated in the oven, and it was determined that the bars might have gone through a post-cure cycle at the high temperature. When the tests were performed the second time, the strains returned to essentially zero when the bars returned to room temperature. Additionally, the bars were cooled in a refrigerator to approximately 42 degrees Fahrenheit, and the strains were recorded at that temperature as well.

3.3 Bridge Construction

3.3.1 Route 668 Bridge Over Gills Creek

The Gills Creek Bridge consists of three simple spans. The total superstructure width is 30 ft 4 in. The spans are 45 ft, 80 ft, and 45 ft in length with Span A located on the downstation, south side, and Span C located on the upstation, north side. Figure 3.1 is a plan view of the bridge.
3.3.1.1 Bridge Deck

The bridge deck has a minimum thickness of 8 in. between the girders and a 9 in. thickness in the overhangs. Figure 3.2 presents a typical section of the bridge. In Spans B and C, the reinforcement is epoxy-coated steel for both the top and bottom mats. No. 4 steel bars are used in the longitudinal direction, while No. 6 steel bars are used in the transverse direction. In Span A, the reinforcement is epoxy-coated steel for the bottom mat and glass fiber reinforced polymer (GFRP) bars for the top mat. Again, the steel bars are No. 4 in the longitudinal direction and No. 6 in the transverse direction, while the GFRP bars are No. 6 in both directions. Concrete cover from the center of the bar to the top of the deck is a minimum of 2 ¾ in. for the epoxycoated steel bars and 2 in. for the GFRP bars. Figures 3.3 and 3.4 present a plan and section of Span A reinforcement. Figures 3.5 and 3.6 present a plan and section of Span C reinforcement.
Figure 3.2: Route 668 Bridge Typical Section

Figure 3.3: Span A Reinforcement Plan
Figure 3.4: Span A Reinforcement Section

Figure 3.5: Span C Reinforcement Plan
3.3.1.2 Girders

The girders for the Gills Creek bridge are W27x94 Grade 50 steel hot-rolled sections. They are spaced at 6 ft 6 in. from center to center. Shear studs are welded to the top flanges in order to provide full composite action with the bridge deck. The exterior girders are painted on the outside, while the interior girders remain unpainted.

3.3.2 Construction Observation

The construction of the Route 668 bridge over Gills Creek was monitored closely by the researcher, starting from the initial observation of the previous structure to the opening of the new bridge to traffic. The most important process to be observed was the bridge deck construction, or more specifically, the installation of both the epoxy-coated steel bars and the GFRP bars. The researchers noted the total man-hours required to install the reinforcement in both Spans A and C. Additionally, comments and complaints from the construction crew were logged. Handling of the bars, as well as familiarity with the material, or a lack thereof, were important considerations to be recorded. The researcher took note of the in-place appearance of both the steel and GFRP mats, as well as the flexibility of the mats during installation and when completed. Finally, the observations were compared against each other, in order to determine a recommendation on the constructibility of decks reinforced with GFRP.
3.4 Bridge Deck Instrumentation

3.4.1 Span A

3.4.1.1 Electrical Resistance Strain Gauges

Electrical resistance (ER) strain gauges were adhered to top mat reinforcement in Span A in four main areas. Figure 3.7 shows the instrumentation plan for Span A. Three of the areas involved gauges on transverse bars, while the fourth gauged only longitudinal bars. All of the gauges that survived the casting operation were monitored during initial live load tests and will be monitored during subsequent future live load tests. After testing, these gauges were connected to a data logger to be monitored for several years.

Figure 3.7: Span A Instrumentation Plan
The first gauging area investigates transverse bars over the first interior girder at the end of the span close to the abutment. This section is gauged to determine the stresses in the top mat transverse GFRP bars over an interior girder close to the abutment. Four bars are gauged, with each bar having two strain gauges attached over the top flange edges, for a total of eight strain gauges. Gauges in this section are labeled ATA1 (Span ‘A’-‘T’ransverse-‘A’butment) through ATA8.

A similar strain gauge layout makes up the second gauging area of transverse bars. However, instead of being at the end of the span, this section is at the midspan. Again, eight gauges make up this section. This section is gauged to determine the stresses in the top mat transverse GFRP bars over an interior girder at the midspan. Gauges in this section are labeled ATMI1 (Span ‘A’-‘T’ransverse-‘M’idspan-‘I’nterior) through ATMI8.

The third gauging area consists of gauges that are adhered to the same bars as the second area, however, the bars are gauged over the exterior girder. As before, eight gauges make up this section. It is gauged to determine the stresses in the top mat transverse GFRP bars in the overhang. Unfortunately, due to the width of the barrier rail, the closest that a wheel load can get to the overhang is directly over the exterior girder, so the overhang cannot be loaded to a great extent during the initial live-load test. Data will still be recorded, but the real importance of these gauges will be determining the stresses in these bars through long-term monitoring. Gauges in this section are labeled ATME1 (span ‘A’-‘T’ransverse-‘M’idspan-‘E’xterior) through ATME8. Figure 3.8 shows a wide view of span A with the three sections of transverse gauges noted. The dark color of the girders underneath the gauged sections can be seen.
The fourth and final area of ER strain gauges in Span A is at the midspan over the center girder and the two exterior girders. The longitudinal, top-mat GFRP bars running directly over each of these three girders are gauged at the midspan, with one gauge attached to each bar for a total of three gauges. The purpose of these gauges is to obtain the uppermost data point for a strain profile for each of these three girders. Gauges in this section are labeled AL1 (Span ‘A’-‘L’ongitudinal) through AL3.

All ER strain gauges were installed using procedures recommended by the manufacturer. Prior to weatherproofing, insulated lead wires were soldered to the gauges and run through the deck to a common access point located close to the abutment. The gauges were then weatherproofed using materials and procedures recommended by the manufacturer. Figure 3.9 is an illustration of gauged bars in Span A. The metallic covering over the gauges is the weatherproofing. Typically, lead wires were bound to bottom mat reinforcement in order to protect them from workers’ boots and also during placing of the concrete. An access hole was drilled in the stay-in-place metal deck forms.
and a small section of PVC pipe was installed in the hole with caulking between the PVC and the deck forms. The lead wires were then fed through the pipe, and the remaining open space in the pipe was filled with duct tape. Figure 3.10 is a picture of the access hole with all the wires running through it. Great care was taken to ensure that each wire was labeled properly and also that the wires were sufficiently long to reach the mobile data acquisition system.

![Figure 3.9: Span A Electrical Resistance Strain Gauges](image-url)
3.4.1.2 Vibrating-Wire Strain Gauges

Vibrating-wire (VW) strain gauges are a more accurate and longer-lasting tool to determine concrete strains. Six VW strain gauges were placed in the deck prior to casting. Figure 3.7, the instrumentation plan for Span A, indicates the VW gauge locations. At each section of transverse ER strain gauges, two VW strain gauges were installed in the transverse direction. One gauge was installed at the level of the bottom mat, and the other was installed at the top mat. These gauges are connected to the data logger in order to continuously monitor the concrete strains over time. Top mat strains will be compared against strains recovered from the ER strain gauges. Bottom mat strains will give an indication of where the neutral axis of the deck is located, helping future researchers to determine whether or not the cross-section can be considered to be cracked. The VW strain gauges are labeled VAT, VAB, VMIT, VMIB, VMET, and
VMEB. The designations are for top and bottom mats in the abutment, midspan-interior, and midspan-exterior locations, respectively.

All VW strain gauges were installed and connected to lead wires using procedures recommended by the manufacturer. Unlike the ER strain gauges, the VW strain gauges did not have to be weatherproofed. The lead wires were fed through the deck and the access hole similar to the ER strain gauges. Again, each lead wire was labeled carefully. Figure 3.11 shows two VW gauges in place. One gauge is connected to the top mat and the other is connected to the bottom mat.

![Vibrating Wire Strain Gauges](image)

**Figure 3.11: Vibrating Wire Strain Gauges**

3.4.1.3 Thermocouples

Three thermocouples were installed in each section of transverse ER strain gauges over an interior girder in order to determine the temperature gradient through the deck during both casting and throughout the deck’s life. Figure 3.7 indicates the thermocouple locations. These thermocouples are continuously monitored using the data logger. In
each group of three, one thermocouple is located at the bottom mat of reinforcement, one thermocouple is located at the top, and one is directly between the two. There are two groups of three, for a total of six thermocouples. Each thermocouple wire was fed through the deck and the access hole, and wire was labeled carefully. The thermocouples are labeled TAT, TAM, TAB, TMT, TMM, and TMB. The designations are for top, middle, and bottom orientations in the abutment and midspan locations, respectively. Figure 3.12 shows a set of thermocouples in place. Arrowheads lead to the twisted ends of the thermocouples at the three different depths in the deck.

Figure 3.12: Thermocouples

3.4.2 Span C

3.4.2.1 Electrical Resistance Strain Gauges

The layout of ER strain gauges in Span C is similar to that in Span A. However, three distinct sections of gauges are used instead of four. Figure 3.13 shows the instrumentation plan for Span C. All of these gauges are to be monitored during both
initial live load tests and subsequent future live load tests. However, these gauges are not connected to the permanent data logger.

Figure 3.13: Span C Instrumentation Plan

Similar to Span A, the first gauging area is transverse bars over the first interior girder at the end of the span close to the abutment. This section is gauged to determine the stresses in the top mat transverse epoxy-coated steel bars over an interior girder close to the abutment. Four bars are gauged, with each bar having two strain gauges attached over the top flange edges, for a total of eight strain gauges. Gauges in this section are labeled CTA1 (Span ‘C’-‘T’ransverse-‘A’butment) through CTA8.

The second section of gauges is, like Span A, similar to the first. Instead of being at the end of the span, this section is at the midspan. Again, eight gauges make up this section. This section is gauged to determine the stresses in the top mat transverse epoxy-
coated steel bars over an interior girder at the midspan. Gauges in this section are labeled CTM1 (Span ‘C’-‘T’ransverse-‘M’idspan) through CTM8. Figure 3.14 shows a transverse bar instrumented with two ER strain gauges, fully weatherproofed.

![Figure 3.14: Span C Electrical Resistance Strain Gauges](image)

Unlike Span A, there is no section of gauges over an exterior girder. However, the third section of gauges is at the midspan over the center girder and the two exterior girders. The longitudinal, top-mat epoxy-coated steel bars running directly over each of these three girders are gauged at the midspan, with one gauge attached to each bar for a total of three gauges. The purpose of these gauges is to obtain the uppermost data point for a strain profile for each of these three girders. Gauges in this section are labeled CL1 (Span ‘C’-‘L’ongitudinal) through CL3.

The ER strain gauges in Span C were wired and weatherproofed exactly like those in Span A. Another smaller access hole was installed in the stay-in-place metal deck forms close to the abutment. The lead wires were fed through the deck and the access
hole as in Span A, and again, care was taken to ensure proper labeling and length of each wire.

### 3.5 Bridge Deck Casting

After the reinforcing bars were installed and the embedded gauges were secured in the deck, concrete was placed into the forms. The casting was accomplished in one day, with all three simple spans cast in one operation. The contractor started at the abutment side of Span C and continued across the bridge, finishing at the abutment side of Span A. Figure 3.15 shows the casting operation at the beginning of the day in Span C. Test cylinders of concrete placed in Spans A and C were taken by the researcher during the casting operation. The cylinders were then match-cured with the deck for approximately two weeks and then taken back to Virginia Tech for storage. When tested for 28-day compressive strength, the Span A cylinders averaged 7001 psi and the Span C cylinders averaged 6882 psi.

![Figure 3.15: Span C Concrete Casting](image)
Great care was taken by the researcher as well as the construction workers to avoid contact with the installed gauges and wires during the casting operation. However, it should be noted that concrete was placed with a bucket about one to two feet above the bars, and in some cases this concrete was dropped directly on top of a section where gauges and thermocouples were present. Also, it should be noted that the workers tried to stay clear of the gauged sections while walking on the top mat of reinforcement. However, this was not always the case. Figure 3.16 shows the casting operation in the Span A end of the bridge.

Figure 3.16: Span A Concrete Casting

Since the construction crew was responsible for the casting of the deck, the researcher’s responsibilities for that day were limited to inspecting and keeping watch on the gauge and thermocouple installations, as well as monitoring the temperature of the concrete during casting and curing with the thermocouples and the data logger.
3.6 Test Preparation Procedures

3.6.1 Bridge Girder Instrumentation

3.6.1.1 Electrical Resistance Strain Gauges

The same type of ER strain gauges that were installed in the deck were also installed on selected girders prior to load testing. In both Spans A and C, the two exterior girders and the center girder were gauged. Figures 3.17 and 3.18 show girder instrumentation plans for Spans A and C. Each girder had two strain gauges installed, one on the inside top of the bottom flange, and the other on the inside bottom of the top flange. Figures 3.19 and 3.20 show ER strain gauges on the top and bottom flanges, respectively. The bottom flanges were gauged in order to determine common bridge design parameters such as girder distribution factors and dynamic load allowance, as well as to compare against theoretical predicted values. The inside top of the bottom flange was selected because the outside of the exterior girder, and the bottom of the bottom flange were to be painted, preventing proper gauge installations in those locations. The top flanges were gauged in order to obtain the strain profile of the composite girder. The gauges were to be used for the initial live load test only.

![Figure 3.17: Span A Girder Instrumentation Plan](image-url)
Figure 3.18: Span C Girder Instrumentation Plan

Figure 3.19: Electrical Resistance Strain Gauge on Girder Top Flange
Figure 3.20: Electrical Resistance Strain Gauge on Girder Bottom Flange

All ER strain gauges were installed using procedures recommended by the manufacturer. Prior to weatherproofing, insulated lead wires were soldered to the gauges and run along the girder flanges to the abutment. The gauges were then weatherproofed using materials and procedures from the manufacturer. Great care was taken to ensure that each wire was labeled properly and also that the wires were sufficiently long to reach the mobile data acquisition system.

3.6.1.2 Deflectometers

Deflectometers were also connected to the girders prior to load testing. Each girder in Spans A and C was instrumented at the midspan for deflection measurements. Figures 3.9 and 3.10 show deflectometers locations for Spans A and C. The deflectometers used in this project were the same as those used in Restrepo’s field testing of the Route 601 bridge over Dickey Creek in Sugar Grove, VA (2002). The deflectometers comprise a gauged aluminum plate, secured between two thicker
aluminum plates. This assembly is clamped to the underside of the bottom girder flange. The cantilevered aluminum plate was predeflected a nominal ½ in., producing a bending strain in the top of the plate, and secured to a wire with a heavy anchor at the end resting on the ground. Then, as the trucks ran over the bridge and the girders deflected, the bending strain in the cantilevered plate decreased, and a corresponding deflection was recorded. The deflectometers were calibrated in the Structures and Materials Laboratory at the Virginia Polytechnic Institute and State University using a trial and error method. The deflectometers were deflected by a known amount using a dial gauge, the display on the data acquisition system was checked, and if the measurement was off a gauge factor was adjusted in the system. Then, the process was repeated until the measurement on the dial gauge and the system were equal. Figure 3.21 shows a deflectometer in place, clamped on the girder bottom flange.

![Deflectometer](image.jpg)

**Figure 3.21: Deflectometer**

Each girder’s deflection was recorded to determine common bridge design parameters such as girder distribution factors and dynamic load allowance in both the
GFRP/epoxy-coated steel reinforced deck and the deck reinforced with epoxy-coated steel only. All deflectometers were installed using procedures garnered from past field testing projects. Cables were connected to the deflectometers using shop-installed plugs, and all were labeled carefully. After the testing was completed, the cables were unplugged and the deflectometers were unclamped from the flanges, saving them for future use.

3.6.1.3 Weigh-In-Motion Gauges

Clamp-on, weigh-in-motion (WIM) gauges were installed on three girders in Span A. The purpose of this instrumentation was to determine if strain measurements similar to those garnered from ER strain gauges could be made with the simpler WIM gauges. These gauges take much less time to install, and are a quick and easy alternative to the more intensive and time consuming process of adhering ER strain gauges to girder flanges. However, it is not yet known how accurate or reliable the WIM gauges are.

The WIM gauges were clamped to the underside of the bottom flanges of both exterior girders and the center girder in Span A (see Figure 3.9). Short cables were already attached to the gauges. Longer cables were plugged into the short cables and extended along the bottom flange to the mobile data acquisition system. The WIM gauges were unclamped and removed after the testing.

3.6.2 Data Acquisition

The data was collected during the live load test using a mobile data acquisition system. A Megadac 3108 AC data acquisition system from Optim Electronics was used for this project. It was placed in the rear of a large cargo van, which was parked in the bridge’s approach area.

For the Span A tests, the van was backed up to the bridge on the south side, in the northbound lane. 33 ER strain gauges, five deflectometers, and three WIM gauges were connected to the Megadac. The readings were zeroed by the operator and the tests commenced.

For the Span C tests, the van was backed up to the bridge on the north side, in the northbound lane. 25 ER strain gauges and five deflectometers were connected to the Megadac. Again, the operator zeroed the readings and the tests commenced.
3.7 Live Load Testing

3.7.1 Truck Description

A Virginia Department of Transportation dump truck was used to provide the moving loads for the testing. The truck was loaded with crusher run at a quarry and, when weighed at the quarry, the front and rear axles were 16.04 kips and 33.56 kips, respectively. The rear axle was actually a dual axle, with the total rear axle weight assumed to be split equally between the two. The distance from the front axle to the middle axle was approximately 15 ft, while the spacing of the middle and rear axles was approximately 4 ft. The transverse spacing of the wheels for each axle was approximately 7 ft. Figure 3.22 shows the dump truck used for the field testing.

Figure 3.22: VDOT Dump Truck
3.7.2 Truck Orientations

For tests in both Spans A and C, two different truck orientations were selected in order to provide the maximum effect in the instrumented reinforcement and girders. First, the truck was oriented directly above the first interior girder, in the southbound lane. Consequently, each wheel load was straddling the first interior girder on either side, producing the maximum effect in the deck over the girder, as well as in the girder itself at midspan.

Second, the truck was oriented 1 ft from the face of the parapet in the southbound lane. Consequently, the wheel load was essentially on top of the exterior girder. This orientation was selected in order to load the overhangs. However, due to the width of the parapet, the overhangs were not loaded to a great extent. However, this loading configuration did place a maximum effect on the exterior girder.

3.7.3 Quasi-static and Dynamic Tests

Two types of tests were conducted during the initial live load testing of the bridge. The first was the quasi-static test, in which the truck idled slowly across the bridge at a speed just fast enough to allow the truck to move in a slow, steady manner with no jerking or lurching ahead. This test could be categorized as a static test. However, since the truck was in fact moving across the bridge at a very slow rate, it will be termed hereafter as a quasi-static test.

The second type of test was a dynamic test, where the truck sped across the bridge at an approximate speed of 50 mph. Results from this type of test were then compared to results from the quasi-static test to obtain dynamic load allowances.

3.7.4 Test Sequence

The quasi-static tests were performed first, in order to accommodate painting crews that were finishing work on the bridge. The quasi-static tests proceeded in the following manner:

- Span A – interior truck orientation – five tests
- Span A – exterior truck orientation – five tests
- Span C – exterior truck orientation – five tests
• Span C – interior truck orientation – five tests

After the painting crews had given the researchers sufficient access, the dynamic tests were performed. For each test, the truck started approximately 300 ft away and build up speed so that, when traveling over the bridge, the truck was at a constant 50 mph speed. The process was then repeated, going the opposite direction, until all the tests were completed. The dynamic test sequence proceeded in the following manner:

• Span C – interior truck orientation – six tests
• Span A – interior truck orientation – six tests

Only the interior truck orientation was used for the dynamic tests for safety reasons. The speeds were deemed too fast for the truck to be traveling only 1 ft away from the parapet.

After all the tests were completed, the wires were disconnected from the Megadac, the data was saved to the computer’s hard drive as well as a separate removable zip disk, and the reusable external instrumentation was removed.
Chapter 4 – Results and Discussion

4.1 Laboratory Testing

4.1.1 Tensile Testing

A sample of glass fiber-reinforced polymer (GFRP) bars was provided by the manufacturer to be tested in the laboratory at Virginia Tech. Tensile tests were performed as discussed in Chapter 3 – Methods and Materials. The bars proved to be linear elastic to failure, which was expected, with an average ultimate strength of 109 ksi and an average modulus of elasticity of 5920 ksi. Table 4.1 shows a summary of the tensile test results. Graphs of stress versus strain can be seen for each test in Appendix B.

<table>
<thead>
<tr>
<th>Bar</th>
<th>Ultimate Load (k)</th>
<th>Ultimate Strength (ksi)</th>
<th>Elastic Modulus (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>45.5</td>
<td>103</td>
<td>6050</td>
</tr>
<tr>
<td>2</td>
<td>48.6</td>
<td>110</td>
<td>5840</td>
</tr>
<tr>
<td>3</td>
<td>49.5</td>
<td>112</td>
<td>5930</td>
</tr>
<tr>
<td>4</td>
<td>46.5</td>
<td>106</td>
<td>5940</td>
</tr>
<tr>
<td>5</td>
<td>48.5</td>
<td>110</td>
<td>5920</td>
</tr>
<tr>
<td>6</td>
<td>48.9</td>
<td>111</td>
<td>5940</td>
</tr>
<tr>
<td>7</td>
<td>48.4</td>
<td>110</td>
<td>5850</td>
</tr>
<tr>
<td>Avg.</td>
<td>48.0</td>
<td>109</td>
<td>5920</td>
</tr>
<tr>
<td>St.Dev.</td>
<td>1.42</td>
<td>3.23</td>
<td>69.5</td>
</tr>
</tbody>
</table>

These observed values are well within the range of tensile properties for GFRP bars as specified by the American Concrete Institute (ACI) in their Guide for the Design and Construction of Concrete Reinforced with FRP Bars (2003). For tensile strength, ACI gives a range from 70 ksi to 230 ksi for GFRP. Also, for GFRP they state a range for elastic modulus between 5100 ksi and 7400 ksi. So, the results from the tensile tests are well within these ranges. Some of the stress versus strain curves exhibited more than one linear portion. This is more than likely due to slip of the extensometer during the test.

Also included in ACI’s guide is an equation for the guaranteed tensile strength of an FRP bar:

\[ f^*_{fu} = f_{u,ave} - 3\sigma, \]  
(Eq. 4-1)
where $f_{u, ave}$ is the mean tensile strength of a sample of test specimens and $\sigma$ is the standard deviation. Using this equation, the guaranteed tensile strength of these bars is $109 - 3(3.23) = 99.3$ ksi. The specification for the FRP bars on the Gills Creek Bridge project states that the guaranteed ultimate tensile strength must be a minimum of 70 ksi. Therefore, a calculated strength of 99.3 ksi passes the specification by a wide margin.

4.1.2 Thermal Testing

Similar to the tensile tests, another sample of GFRP bars was provided from the manufacturer to be tested in the laboratory at Virginia tech. Thermal tests were performed as discussed in Chapter 3. For the first set of tests in the linear dilatometer, the average longitudinal coefficient of thermal expansion (CTE) was 4.97 microstrain per degree Fahrenheit ($\mu\varepsilon/°F$), and the average transverse CTE was 24.3 $\mu\varepsilon/°F$. Table 4.2 shows a summary of the results from the CTE tests using the linear dilatometer.

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Specimen 1</th>
<th>Specimen 2</th>
<th>Average</th>
<th>Overall Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>#2</td>
<td>6.22</td>
<td>5.62</td>
<td>5.92</td>
<td>4.97</td>
</tr>
<tr>
<td>#3</td>
<td>2.77</td>
<td>4.13</td>
<td>3.45</td>
<td></td>
</tr>
<tr>
<td>#6</td>
<td>4.75</td>
<td>6.32</td>
<td>5.54</td>
<td></td>
</tr>
</tbody>
</table>

Table 4.2: Thermal Test Results using the Linear Dilatometer

Typical values of longitudinal CTE for GFRP bars, as given by ACI, are from 3.3 $\mu\varepsilon/°F$ to 5.6 $\mu\varepsilon/°F$ (2003). The observed average of 4.97 $\mu\varepsilon/°F$ for the three bar sizes fits well into that range. However, the range given by ACI for transverse CTE for GFRP bars is 11.7 $\mu\varepsilon/°F$ to 12.8 $\mu\varepsilon/°F$. While the results for No. 2 and No. 3 bars are close to that range, the overall average for the three sizes is about twice that recommended by ACI. The transverse CTE’s for the No. 6 bars are very high, and as can be seen in Figures 4.1 and 4.2, the graphs of temperature versus strain exhibited more than one linear portion. It was unclear which slope to take as the transverse CTE. Therefore, the second group of tests were performed on the bars using the strain gauges and the oven, as described in Chapter 3.
Figure 4.1: Transverse CTE Test 1 on a #6 GFRP Bar

Figure 4.2: Transverse CTE Test 2 on a #6 GFRP Bar
As discussed in Chapter 3, strain gauges were adhered to two No. 6 bars and heated in an oven. Four CTE’s were recorded for each bar. The first was the initial heating from room temperature to 140 degrees, the second was the cooling from 140 degrees to room temperature, the third was the cooling from room temperature to 42 degrees, and the fourth was the second heating from room temperature to 140 degrees. Table 4.3 shows a summary of these results.

### Table 4.3: Thermal Test Results Using Strain Gauges and an Oven

<table>
<thead>
<tr>
<th></th>
<th>Bar A</th>
<th>Bar B</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st Heating Cycle</td>
<td>33.1</td>
<td>34.6</td>
</tr>
<tr>
<td>Cooling Cycle</td>
<td>40.4</td>
<td>42.4</td>
</tr>
<tr>
<td>Refrigerator Cycle</td>
<td>10.6</td>
<td>11.2</td>
</tr>
<tr>
<td>2nd Heating Cycle</td>
<td>41.8</td>
<td>43.2</td>
</tr>
</tbody>
</table>

Both bars exhibited very similar CTE’s during all four heating/cooling cycles. Within the first heating cycle, Bar A expanded at a rate of 33.1 με/°F and Bar B expanded at a rate of 34.6 με/°F. When cooled back to room temperature, Bars A and B contracted at rates of 40.4 με/°F and 42.4 με/°F, respectively. When placed in the refrigerator, Bars A and B contracted further at respective rates of 10.6 με/°F and 11.2 με/°F. Finally, when reheated in the oven, Bar A’s transverse CTE was found to be 41.8 με/°F and Bar B’s transverse CTE was 43.2 με/°F.

These very high transverse coefficients of thermal expansion for No. 6 bars validated the results from the linear dilatometer tests. However, as discussed above, these results were not in agreement with numbers provided by the manufacturer, nor did they adhere to VDOT specifications. Therefore, the question of an acceptable transverse CTE was explored. The work of Aiello et al. (2001) was used to explore this question. In it, the authors present an equation to determine the temperature change at which the concrete around the bar will exhibit initial radial cracking due to pressure exerted on the concrete by the bar’s expansion resulting from that temperature change:

\[
(a_b - a_c)ΔT_{cr} = \frac{f_{cr}}{E_c} + \frac{\nu_c^2 - 1}{2} \left[ \frac{\nu_c f_{cr}}{E_c} + \frac{f_{cr}}{E_{Tb}} (1 - \nu_{Tb}) \right]
\]  

(Eq. 4-2)
Also, the authors present the same type of equation for which the radial cracking extends to the surface of the concrete and causes spalling:

\[
(\alpha_b - \alpha_c)\Delta T_{sp} = \frac{0.30f'_{ct} \gamma}{E_c} \left[ \ln(0.48\gamma) + 1.6 + \nu_c \right] + \frac{0.30f'_{ct} \gamma}{E_{Tb}} (1 - \nu_{TT}) \quad \text{(Eq. 4-3)}
\]

Variables and their assumed values are listed below:

- \(E_c\) = Young’s modulus of the concrete, calculated using ACI 318, with \(f'_{ct}\) equal to 4000 psi = 3605 ksi;
- \(E_{Tb}\) = Young’s modulus of the bar in the transverse direction, assumed to be 500 ksi;
- \(f_{ct}\) = tensile strength of the concrete, calculated using ACI 318, with \(f'_{ct}\) as above = 474 psi;
- \(\alpha_c\) = coefficient of thermal expansion of concrete, assumed to be 5.56 \(\mu\varepsilon/\degree\)F;
- \(\alpha_b\) = coefficient of thermal expansion of the bar in the transverse direction;
- \(\Delta T_{cr}\) = temperature increase producing the first concrete cracking;
- \(\Delta T_{sp}\) = temperature increase producing the spalling of the concrete cover;
- \(\gamma = r_2/r_0\);
- \(r_2\) = radius of the concrete cylinder, or the concrete cover to the center of the bar, taking into account a ½ in. sacrificial wearing surface, assumed to be 1.5 in.;
- \(r_0\) = radius of the bar, 0.375 in. for a No. 6 bar;
- \(\nu_c\) = Poisson’s ratio of concrete, assumed to be 0.18;
- \(\nu_{TT}\) = Poisson’s ratio of the bar in the transverse direction, assumed to be 0.30;

Both of these equations can be rearranged to solve for \(\alpha_b\) with a given \(\Delta T\). This will determine the maximum transverse CTE of the bar to prevent spalling or initial radial cracking of the concrete, given an expected temperature change under service conditions. This procedure was done for both equations with a conservatively assumed temperature change of 25°F. The resulting transverse CTEs were 35.0 \(\mu\varepsilon/\degree\)F and 52.7 \(\mu\varepsilon/\degree\)F for initial radial cracking and spalling of the concrete, respectively. The calculated maximum CTE for concrete spalling is greater than the values of CTE obtained from laboratory testing at Virginia Tech using the sample of bars provided by the supplier. The largest transverse CTE value obtained from laboratory testing was 47.4 \(\mu\varepsilon/\degree\)F. However, a close inspection
of the graphs of strain vs. temperature in the first set of CTE tests reveals that through the temperature range selected as a normal service life heating, 85°F to 110°F, the best fit lines exhibit an average slope of 16.8 \( \mu \varepsilon / ^\circ F \). These numbers suggest that the concrete is providing enough confinement around the bar to prevent radial cracking, and certainly enough to prevent spalling of any kind.

In order to determine a more accurate range of temperature in the bridge deck, the thermocouples in Span A were monitored during casting and curing of the concrete. Figure 4.3 shows a graph of temperature versus time for the six thermocouples in Span A during the casting operation. Concrete placement in Span A began at about 2:00 pm and the temperatures were monitored for over 24 hours, until about 6:00 pm the next day. The concrete was cast between 75°F and 80°F, and the heat generated by the hydration process caused the temperatures to reach a maximum of about 110°F at around 6:00 am. However, the time of zero stress in the bars and its associated temperature is unclear. Assuming that the time of zero stress in the bars was sometime between initial and final set of the concrete, it can be observed that the temperature was somewhere in the upper part of the range between 80°F and 110°F. Assuming that the temperature at the level of the bars never increases past 110°F, then the maximum increase in temperature after zero stress in the bar would be in the 15°F to 25°F range, which matches the assumed temperature change used in the above calculations.
4.2 Field Testing

4.2.1 Transverse Deck Strains

As discussed in Chapter 3, transverse bars in both Spans A and C were instrumented with electrical resistance strain gauges to determine the stresses in both the FRP and steel bars over the girders. In Span A, three locations were gauged. Over the first interior girder, or Girder 4, close to the abutment; over Girder 4 at the midspan; and over the exterior girder, or Girder 5, at the midspan gauges were placed to determine stresses in the top mat transverse FRP bars. In Span C, two locations were gauged. Gauges were located on the top mat transverse steel bars over Girder 4 close to the abutment and also at the midspan. In each location, four bars were gauged twice, for a total of eight gauges. The gauges were located directly above the edge of the top girder flange. However, many gauges were lost either prior to or during the concrete casting operation. In Span A, five gauges were lost in the section over Girder 4 close to the abutment. Four gauges were lost in the section over Girder 4 at the midspan. Finally, in
the section over Girder 5 at the midspan, one gauge was lost. Enough operable gauges remained in each section in Span A to investigate the transverse stresses in the deck. However, in Span C, the gauge survival rate was much lower. Six gauges were lost in the section close to the abutment, leaving only two operable. Additionally, in the section over Girder 4 at the midspan, seven gauges were lost during the installation and casting, and the eighth exhibited incoherent data. Therefore, it was determined that no data regarding transverse stresses in the deck could be garnered from the span C tests. Table 4.4 shows all of the embedded ER strain gauges and whether or not they were operable at the time of testing.

Table 4.4: Inventory of Operable/Non-Operable Embedded ER Strain Gauges

| Span A          | ATA1 operable | ATMI1 operable | ATME1 operable | AL1 operable | ATA2 non-operable | ATMI2 non-operable | ATME2 non-operable | AL2 operable | ATA3 non-operable | ATMI3 operable | ATME3 operable | AL3 operable | ATA4 operable | ATMI4 non-operable | ATME4 operable | AL4 operable | ATA5 non-operable | ATMI5 non-operable | ATME5 operable | AL5 operable | ATA6 operable | ATMI6 operable | ATME6 operable | AL6 operable | ATA7 non-operable | ATMI7 operable | ATME7 operable | AL7 operable | ATA8 non-operable | ATMI8 non-operable | ATME8 operable | AL8 operable |
|-----------------|---------------|----------------|----------------|--------------|-------------------|-------------------|-------------------|--------------|-------------------|----------------|----------------|--------------|----------------|-------------------|----------------|--------------|-------------------|-------------------|----------------|--------------|-------------------|-------------------|----------------|--------------|-------------------|-------------------|----------------|--------------|
| Span C          | CTA1 non-operable | CTM1 non-operable | CL1 non-operable | CTM1 non-operable | CTA2 non-operable | CTM2 non-operable | CL2 operable | CTM2 non-operable | CTA3 non-operable | CTM3 non-operable | CL3 operable | CTM3 non-operable | CTA4 non-operable | CTM4 non-operable | CTA5 non-operable | CTM5 non-operable | CTM5 non-operable | CTA6 operable | CTM6 non-operable | CTM6 non-operable | CTA7 non-operable | CTM7 non-operable | CTM7 non-operable | CTA8 operable | CTM8 non-operable | CTM8 non-operable |

The stress profile at each location of gauges in Span A under all of the tests was investigated. To do this, the time at which the truck passing over the deck, which produced the maximum stress in the group of gauges, had to be recorded. Therefore, the stress profile for each group of gauges was at a snapshot in time. To do this, for each test the maximum strains for each gauge were recorded along with their corresponding times. Then, the maximum strain within each group of gauges and its corresponding time was selected. This time was the time at which the snapshot of each gauge’s strain was taken for that group of gauges. The strains for all of the operable gauges in each group at the time of the maximum strain were then converted to stresses using the assumed elastic
modulus of the FRP bars, 6,300 ksi. For the gauges located on the same bar, these stresses were averaged to obtain a single stress for that bar. Then, for each group of gauges, the stresses for each gauged bar were plotted against their respective positions within the group. Table 4.5 is an example of this calculation for quasi-static test 1.

Table 4.5: Maximum and Minimum Strains and Corresponding Times

<table>
<thead>
<tr>
<th>ATA1 (µε)</th>
<th>ATA4</th>
<th>ATA6</th>
<th>ATMI1</th>
<th>ATM6</th>
<th>ATM7</th>
<th>ATME1</th>
<th>ATME3</th>
<th>ATME4</th>
<th>ATME5</th>
<th>ATME6</th>
<th>ATME7</th>
<th>ATME8</th>
</tr>
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<tbody>
<tr>
<td>Initial</td>
<td>1.4</td>
<td>6.9</td>
<td>-2.7</td>
<td>-1.9</td>
<td>1.6</td>
<td>-0.9</td>
<td>-0.2</td>
<td>0.9</td>
<td>-0.9</td>
<td>0.7</td>
<td>1.1</td>
<td>-0.2</td>
</tr>
<tr>
<td>Max µε</td>
<td>13.3</td>
<td>14.0</td>
<td>11.1</td>
<td>-1.1</td>
<td>3.3</td>
<td>0.7</td>
<td>1.5</td>
<td>2.4</td>
<td>5.2</td>
<td>3.6</td>
<td>4.8</td>
<td>5.5</td>
</tr>
<tr>
<td>2 - 7</td>
<td>11.0</td>
<td>7.1</td>
<td>8.4</td>
<td>0.8</td>
<td>1.7</td>
<td>1.6</td>
<td>1.7</td>
<td>1.5</td>
<td>4.3</td>
<td>4.5</td>
<td>3.9</td>
<td>4.4</td>
</tr>
<tr>
<td>Min µε</td>
<td>-2.0</td>
<td>4.1</td>
<td>-1.6</td>
<td>-16.7</td>
<td>-16.7</td>
<td>-16.6</td>
<td>-2.6</td>
<td>-0.9</td>
<td>-2.1</td>
<td>-1.8</td>
<td>-0.9</td>
<td>-3.7</td>
</tr>
<tr>
<td>4 - 7</td>
<td>-3.3</td>
<td>-2.8</td>
<td>-4.3</td>
<td>-14.8</td>
<td>-15.8</td>
<td>-16.4</td>
<td>-3.4</td>
<td>-1.8</td>
<td>-1.2</td>
<td>-2.5</td>
<td>-2.0</td>
<td>-3.5</td>
</tr>
<tr>
<td>Abs Max µε</td>
<td>11.9</td>
<td>7.1</td>
<td>8.4</td>
<td>0.8</td>
<td>1.7</td>
<td>1.6</td>
<td>1.7</td>
<td>1.5</td>
<td>4.3</td>
<td>4.5</td>
<td>3.9</td>
<td>4.4</td>
</tr>
<tr>
<td>Time</td>
<td>12.7</td>
<td>11.9</td>
<td>4.8</td>
<td>17.6</td>
<td>18.1</td>
<td>18.1</td>
<td>16.0</td>
<td>22.1</td>
<td>21.7</td>
<td>22.3</td>
<td>20.5</td>
<td>18.4</td>
</tr>
</tbody>
</table>

Row (1) is an average of baseline values leading up to the point at which the bridge deck was loaded. Rows (2) and (4) are the maximum and minimum strains for each gauge during the test, and rows (3) and (5) are the difference between these values and the baseline value from row (1). Row (6) is then the maximum of the absolute values of rows (3) and (5), and row (7) is the corresponding time for that value. The maximum strain within each group is row (8), and its corresponding time is row (9). Then, row (10) shows the strains for each gauge at the time indicated in row (9), and row (11) is that strain minus the baseline value in row (1). Therefore, row (11) presents the strains at each gauge at the time of the max strain for each group of gauges. Again, these strains were then converted to bar stresses and plotted against the bar’s position within each section of gauges. On the x-axis, 0.0 in. represents the middle of the gauged section. Figure 4.4 presents the stress profile for the ATA gauges, or the gauges in Span A over Girder 4 close to the abutment, during the quasi-static tests. Note that tests 1 through 5 utilized the interior truck configuration and tests 6 through 10 utilized the exterior configuration. It can be seen that the stresses in these bars ranged from 0.000 ksi to 0.080 ksi. In other words, when the truck straddled over the first interior girder directly above these gauges producing the maximum transverse stress over this location, the stresses in the bars were far smaller than the design limit for service stresses. In
Section 4.1.1 it was stated that the average ultimate strength of a sample of these bars tested in the laboratory was 109 ksi, and that the guaranteed tensile strength from ACI 440 was 99.3 ksi. The equation for the design tensile strength in ACI 440 is given as:

\[ f_{fu} = C_E f^*_{fu}, \]  

(Eq. 4-4)

where \( C_E \) is the environmental reduction factor, or 0.7 for glass fiber in concrete exposed to earth and water; and \( f^*_{fu} \) is the guaranteed tensile strength, or 99.3 ksi. Assuming that the allowable tensile strength is 20% of the design tensile strength, the allowable tensile strength under service conditions for these FRP bars can be calculated as 0.2(0.7)(99.3 ksi) = 13.9 ksi. The highest stress observed in the bars, or 0.080 ksi, is much less than the allowable limit of 13.9 ksi.

![Stress Profile - ATA Gauges - Span A Creep Tests](image)

**Figure 4.4: Stress Profile for ATA Gauges Under Quasi-static Loading**

Figures 4.5 and 4.6 show the stress profiles at the ATMI and ATME groups during the quasi-static tests. The ATMI gauges were located over Girder 4 at the midspan, and the ATME gauges were located over Girder 5 at the midspan. Again, the stresses in these bars are much smaller than design allowable values. Note that the stresses in the ATMI gauges are negative stresses. This is possible near the midspan if the bridge deck is acting as a plate element. The plate would be in positive bending in
both the longitudinal and transverse directions near the midspan, causing compressive stresses in the top mat bars. Locally, the bridge deck is bending transversely in negative moment over the interior girder. However, the global positive bending of the deck is superceded the local bending, and the top bars are in compression. Close inspection of Figure 4.5 shows that for Tests 1 through 5, the compressive bar stresses are higher than for Tests 6 through 10. This correlates with the testing in that Tests 1 through 5 utilized the interior truck orientation and Tests 6 through 10 utilized the exterior truck configuration. The maximum stress in the ATMI group is near the middle of the section, at about 0.110 ksi in compression. The stress profile for both interior and exterior configuration tests is fairly constant across, with the interior configuration tests giving the higher stresses. The stress profile for the ATME gauges is the more familiar sloping up to the center portion and then back down towards the end. The bar stresses towards the middle are in tension due to the local effect of the negative bending. However, the bar stresses on the outskirts exhibit the global effect of the plate in compression and positive bending. The maximum stress is 0.03 ksi. This is extremely low, but it does make sense because even for the exterior truck configuration tests, the truck’s wheels did not straddle girder 5, but simply rode on top. This produces a much smaller stress in the top transverse bars.
Figures 4.5, 4.8, and 4.9 show the stress profiles for the ATA, ATMI, and ATME gauges under dynamic loading. The ATA gauges exhibit no real pattern under dynamic
loads, and the stresses range from around 0.03 ksi to 0.10 ksi. Again, these stresses are very small. The ATMI gauges are negative values and very similar to the ATMI gauges during the quasi-static tests. They are in compression due to the global effect of the bridge deck as a plate in positive bending. The maximum stresses in this group are located towards the middle of the group and are around 0.13 ksi in compression. The dynamic load allowance phenomena is illustrated as the maximum stress in the ATMI gauges for the quasi-static tests was around 0.11 ksi. The ATME gauges under dynamic loads are similar to that under quasi-static loads. The shape of the stress profile is similar, as well as the maximum strain of around 0.022 ksi.

![Stress Profile - ATA Gauges - Span A Dynamic Tests](image)

Figure 4.7: Stress Profile for ATA Gauges Under Dynamic Loading
The results from the field testing can be compared to Cawrse’s (2002) data from his laboratory tests on the prototype deck. For the interior girder test, the bars reached a
maximum stress of 0.16 ksi under a static axle load of 30 kips, and 0.10 ksi under a static axle load of 20.5 kips. The results from the field tests show that the bars reached a maximum stress of 0.080 ksi under a static truck load at the ATA gauges and 0.10 ksi under a dynamic truck load at the ATA gauges. Therefore, the field test results were less than the lab results. Figures 4.4 and 4.7 show the Cawrse (2002) results from the 30 kips axle load along with the field testing results. The difference in bar stresses can be attributed to various factors. The flexibility of the supports, or girders, in the ATA region, while less than at the midspan, is still greater than in the laboratory. The girders in the lab were supported on a strong floor directly under the point of load application, ensuring no flexibility in the girders. Also, the deck constructed in the field utilizes stay-in-place metal deck forms and is stiffer than the deck in the lab, which was cast into wood forms that were removed prior to testing. Therefore, the bar stresses measured in the interior girder test in the laboratory were conservative for field conditions, and the bar stresses observed during the field testing were less than the lab results. The stresses from the ATMI gauges were in compression due to the apparent behavior of the bridge deck as a plate element in compression under the truck’s wheel loads, and difficult to compare to Cawrse’s laboratory results.

The results from the overhang tests in the laboratory showed maximum stresses of 0.65 ksi under a static wheel load of 20 kips and 0.45 ksi under a static wheel load of 16 kips for Overhang 1, and 0.11 ksi under a static wheel load of 8.1 kips and 0.075 ksi under a static wheel load of 6.2 kips for Overhang 2. The results from the field tests at the ATME gauges are considerably less due to the position of the truck wheel directly above the exterior girder. The ATME gauges show maximum stresses of 0.025 ksi under static load and 0.022 ksi under dynamic load.

The principle result to be garnered from the stress profiles is the very low amount of stress in each bar under live load. These low stresses should be compared with values from tests later in the bridge’s service life to determine the additional flexibility of the deck after cracking.
4.2.2 Girder Distribution Factors

4.2.2.1 Introduction and AASHTO Equations

Girder Distribution Factor (GDF) is a key concept in bridge design. If a moving load is placed on a bridge deck, the fraction of the total load which is carried by an individual girder is the GDF. GDF’s are influenced by the orientation of the applied load, as well as the spacing and number of girders. When a bridge is designed, the engineer must first calculate an acceptable GDF and then use that factor to apply the design loads to the bridge girders. The American Association of State Highway and Transportation Officials (AASHTO) specifies GDF’s in two different ways.

In the AASHTO Standard Specification (2002), Table 3.23.1 defines the fraction to apply to a wheel load when determining the live load bending moment for an interior girder as:

\[ g = \frac{S}{D}, \quad \text{(Eq. 4-5)} \]

where \( g \) is the girder distribution factor for a wheel line, \( S \) is the center-to-center spacing of the interior girders (ft), and \( D \) is a denominator dependant upon the type of bridge deck and girders. For a concrete bridge deck cast on steel I-Beam stringers the distribution factor is \( S/7.0 \) for bridges designed for one traffic lane and \( S/5.5 \) for bridges designed for two or more traffic lanes.

When determining the live load bending moment for an exterior girder, the AASHTO Standard Specification specifies the use of reaction at that girder with the assumption that the deck acts as simple spans between girders. This process is also known as the lever rule.

In the AASHTO LRFD Specification (1998), Table 4.6.2.2.2b-1 presents the equations to calculate distribution factors dependent on superstructure type. When designing for moment, for a concrete deck on steel beams with one design lane loaded:

\[ g = 0.06 + (\frac{S}{14})^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0L_t^3}\right)^{0.1}, \quad \text{(Eq. 4-6)} \]

where \( g \) is the distribution factor for interior beams, \( S \) is the center-to-center spacing of the interior girders (ft), \( L \) is the span length (ft), and \( t_s \) is the thickness of the deck slab (in.). The longitudinal stiffness parameter, \( K_g \), is given by the following equation:


\[ K_g = n(I + Ae) \frac{2}{2}, \quad (\text{Eq. 4-7}) \]

where \( n \) is the modular ratio defined as the modulus of elasticity of the beam material divided by the modulus of elasticity of the deck material, \( I \) is the moment of inertia of the beam, \( A \) is the cross-sectional area of the beam, and \( e_g \) is the distance between the centers of gravity of the basic beam and the deck.

Similar to above, when designing for moment with two or more design lanes loaded:

\[ g = 0.075 + \left( \frac{S}{9.5} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{K_g}{12L_t} \right)^{3} \frac{3}{0.1}, \quad (\text{Eq. 4-8}) \]

where the variables are as for one design lane loaded.

For both equations 4-6 and 4-8, the girder spacing must be between 3.5 ft and 16 ft, the deck slab thickness must be between 4.5 in. and 12 in., the span length must be between 20 ft and 240 ft, and the number of girders must be greater than 3.

For an exterior girder using the AASHTO LRFD Specification, with one design lane loaded the lever rule is to be used, similar to the Standard Specification. However, for two or more design lanes loaded, the distribution factor for an interior girder, \( g_{\text{interior}} \), is to be multiplied by the factor \( e \),

\[ g = e \cdot g_{\text{interior}}, \quad (\text{Eq. 4-9}) \]

where,

\[ e = 0.77 + d_e/9.1, \quad (\text{Eq. 4-10}) \]

where \( d_e \) is the width of the overhang, limited from –1.0 ft to 5.5 ft. A negative value of \( d_e \) would be for a web that is outboard of the curb or traffic barrier.

4.2.2.2 Determination of Girder Distribution Factors from Live Load Tests

Distribution factors were calculated from the raw data collected in each test. Due to the constant cross-sectional properties of the deck and the beams across the width of the bridge, girder deflections and bottom flange strains at each girder could be used to calculate distribution factors. However, only deflections were used to calculate GDF’s because girder deflections were measured at every girder. Unfortunately, strains could not be used to calculate distribution factors because only three of the five girders were instrumented with strain gauges. Using the procedure described below with only three out of five girders instrumented would result in GDF’s for a three-girder bridge instead of
a five-girder bridge. However, the distribution factors calculated from the deflections were used to verify the strains recorded at the exterior girders and the center girder. (See Section 4.2.4.)

The procedure for calculating the distribution factors from the deflections of each test was as follows:

1. Each girder’s maximum deflection was recorded, and then the maximum of these values along with the corresponding time was recorded.
2. Each girder’s deflection at that specific time was then recorded.
3. To obtain what fraction of the total deflection each girder experiences, the girder’s deflection was divided by the sum of the girder deflections at that specific time. This fraction is the girder distribution factor, or using AASHTO’s LRFD nomenclature, g.
4. Finally, the distribution factors were converted into the denominator, D, of equation 4-5. It should be noted that since the AASHTO Standard Specification defines the distribution factor, g, as the girder distribution factor for a wheel line, which is half of a lane load, the distribution factors from the live load tests must be multiplied by a factor of 2 to account for this. Therefore, the equation to calculate the denominator, D, is D = S/2g.

Table 4.6 shows a sample calculation of GDF’s. This data is from the first Span A quasi-static test, using the interior truck configuration.

### Table 4.6: Span A Quasi-static 1 (Interior Configuration) GDF Data

<table>
<thead>
<tr>
<th>Deflections</th>
<th>Δ_{i,max} (in.)</th>
<th>Time (sec.)</th>
<th>Δ_{corresp.} (in.)</th>
<th>GDF</th>
<th>&quot;D&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>G1</td>
<td>0.0131</td>
<td>19.6</td>
<td>0.0127</td>
<td>0.043</td>
<td>76.1</td>
</tr>
<tr>
<td>G2</td>
<td>0.0422</td>
<td>19.6</td>
<td>0.0419</td>
<td>0.141</td>
<td>23.1</td>
</tr>
<tr>
<td>G3</td>
<td>0.0708</td>
<td>19.9</td>
<td>0.0705</td>
<td>0.237</td>
<td>13.7</td>
</tr>
<tr>
<td>G4</td>
<td>0.0856</td>
<td>19.9</td>
<td>0.0854</td>
<td>0.287</td>
<td>11.3</td>
</tr>
<tr>
<td>G5</td>
<td>0.0873</td>
<td>19.6</td>
<td>0.0873</td>
<td>0.293</td>
<td>11.1</td>
</tr>
</tbody>
</table>

The maximum deflections were calculated along with their corresponding times, from 0.0131 in. at 19.6 seconds for Girder 1 to 0.0873 in. at 19.6 seconds for Girder 5. Girder 5’s deflection was the maximum, so all of the deflections were recorded at 19.6 seconds. Then, each corresponding deflection was divided by the sum of the deflections
to obtain the distribution factor. Finally, the 6.5 ft girder spacing was divided by twice the GDF to get the denominator, D.

The GDF results were then compiled within each type of test (Span A/Quasi-static/Interior, Span A/Dynamic/Interior, Span C/Quasi-static/Interior, Span C/Dynamic/Interior, Span A/Quasi-static/Exterior, and Span C/Quasi-static/Exterior). Tables 4.7 through 4.10 show the average maximum deflections, maximum and minimum GDF’s, and average GDF’s all within each type of test. The GDF in bold type represents the controlling GDF for each type of test. The average deflection of all five girders was computed and compared to a calculated theoretical deflection as well. This calculation is shown in Appendix A. The truck axle loads were placed on the bridge as discussed in Section 4.2.3, causing the maximum moment at the midspan. The deflection of the bridge at midspan was calculated for each concentrated load, and then superimposed to obtain the average deflection at midspan of all five girders. The moment of inertia used was the composite moment of inertia of the five bridge girders. This calculation is shown in Appendix A as well. The modulus of elasticity used was for the composite girders, or 29,000 ksi. Table 4.11 shows the comparison of deflections. The measured deflections are all slightly less than theoretical. The measured deflections range from 0.0591 in. for the Span A quasi-static tests with the exterior truck orientation to 0.0728 in. for the Span A dynamic tests. The theoretical deflection was calculated to be 0.0863 in.

Table 4.7: GDF Results for Span A Interior Configuration Tests

<table>
<thead>
<tr>
<th>Girder #</th>
<th>$\Delta_{\text{avg}}$ (in)</th>
<th>GDF$_{\text{max}}$</th>
<th>GDF$_{\text{min}}$</th>
<th>GDF$_{\text{avg}}$</th>
<th>GDF$_{\text{avg}}$ (in)</th>
<th>GDF$_{\text{max}}$</th>
<th>GDF$_{\text{min}}$</th>
<th>GDF$_{\text{avg}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>G1</td>
<td>0.013</td>
<td>0.044</td>
<td>0.042</td>
<td>0.043</td>
<td>0.012</td>
<td>0.044</td>
<td>0.024</td>
<td>0.033</td>
</tr>
<tr>
<td>G2</td>
<td>0.040</td>
<td>0.141</td>
<td>0.130</td>
<td>0.133</td>
<td>0.048</td>
<td>0.142</td>
<td>0.125</td>
<td>0.132</td>
</tr>
<tr>
<td>G3</td>
<td>0.068</td>
<td>0.237</td>
<td>0.223</td>
<td>0.226</td>
<td>0.079</td>
<td>0.224</td>
<td>0.212</td>
<td>0.218</td>
</tr>
<tr>
<td>G4</td>
<td>0.087</td>
<td>0.287</td>
<td>0.285</td>
<td>0.286</td>
<td>0.108</td>
<td>0.303</td>
<td>0.288</td>
<td>0.296</td>
</tr>
<tr>
<td>G5</td>
<td>0.095</td>
<td>0.320</td>
<td>0.293</td>
<td>0.313</td>
<td>0.116</td>
<td>0.332</td>
<td>0.305</td>
<td>0.320</td>
</tr>
</tbody>
</table>

$\Sigma = 1.000$          $\Sigma = 1.000$
Table 4.8: GDF Results for Span C Interior Configuration Tests

<table>
<thead>
<tr>
<th>Girder #</th>
<th>$\Delta_{avg}$ (in)</th>
<th>GDF$_{max}$</th>
<th>GDF$_{min}$</th>
<th>GDF$_{avg}$</th>
<th>$\Delta_{avg}$ (in)</th>
<th>GDF$_{max}$</th>
<th>GDF$_{min}$</th>
<th>GDF$_{avg}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>G1</td>
<td>0.015</td>
<td>0.052</td>
<td>0.047</td>
<td>0.049</td>
<td>0.015</td>
<td>0.053</td>
<td>0.033</td>
<td>0.042</td>
</tr>
<tr>
<td>G2</td>
<td>0.041</td>
<td>0.132</td>
<td>0.129</td>
<td>0.131</td>
<td>0.045</td>
<td>0.135</td>
<td>0.122</td>
<td>0.129</td>
</tr>
<tr>
<td>G3</td>
<td>0.068</td>
<td>0.217</td>
<td>0.216</td>
<td>0.217</td>
<td>0.076</td>
<td>0.220</td>
<td>0.214</td>
<td>0.217</td>
</tr>
<tr>
<td>G4</td>
<td>0.095</td>
<td>0.307</td>
<td>0.304</td>
<td>0.305</td>
<td>0.107</td>
<td>0.311</td>
<td>0.300</td>
<td>0.306</td>
</tr>
<tr>
<td>G5</td>
<td>0.093</td>
<td>0.302</td>
<td>0.294</td>
<td>0.298</td>
<td>0.107</td>
<td>0.321</td>
<td>0.296</td>
<td>0.306</td>
</tr>
</tbody>
</table>

Σ = 1.000

Table 4.9: GDF Results for Span A Exterior Configuration Tests

<table>
<thead>
<tr>
<th>Girder #</th>
<th>$\Delta_{avg}$ (in)</th>
<th>GDF$_{max}$</th>
<th>GDF$_{min}$</th>
<th>GDF$_{avg}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>G1</td>
<td>0.006</td>
<td>0.022</td>
<td>0.016</td>
<td>0.019</td>
</tr>
<tr>
<td>G2</td>
<td>0.028</td>
<td>0.097</td>
<td>0.090</td>
<td>0.094</td>
</tr>
<tr>
<td>G3</td>
<td>0.055</td>
<td>0.188</td>
<td>0.184</td>
<td>0.187</td>
</tr>
<tr>
<td>G4</td>
<td>0.087</td>
<td>0.295</td>
<td>0.293</td>
<td>0.294</td>
</tr>
<tr>
<td>G5</td>
<td>0.120</td>
<td>0.414</td>
<td>0.400</td>
<td>0.407</td>
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</tbody>
</table>

Σ = 1.000

Table 4.10: GDF Results for Span C Exterior Configuration Tests

<table>
<thead>
<tr>
<th>Girder #</th>
<th>$\Delta_{avg}$ (in)</th>
<th>GDF$_{max}$</th>
<th>GDF$_{min}$</th>
<th>GDF$_{avg}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>G1</td>
<td>0.006</td>
<td>0.020</td>
<td>0.018</td>
<td>0.019</td>
</tr>
<tr>
<td>G2</td>
<td>0.029</td>
<td>0.098</td>
<td>0.096</td>
<td>0.097</td>
</tr>
<tr>
<td>G3</td>
<td>0.056</td>
<td>0.190</td>
<td>0.187</td>
<td>0.189</td>
</tr>
<tr>
<td>G4</td>
<td>0.094</td>
<td>0.316</td>
<td>0.314</td>
<td>0.315</td>
</tr>
<tr>
<td>G5</td>
<td>0.114</td>
<td>0.384</td>
<td>0.377</td>
<td>0.380</td>
</tr>
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</table>

Σ = 1.000

Table 4.11: Comparison of Measured and Theoretical Deflections

<table>
<thead>
<tr>
<th>Span</th>
<th>Truck Orientation</th>
<th>Deflections, in.</th>
<th>Theoretical</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Measured (Average of 5 Girders)</td>
<td>Dynamic Tests</td>
</tr>
<tr>
<td>A</td>
<td>Interior</td>
<td>0.0606</td>
<td>0.0728</td>
</tr>
<tr>
<td>C</td>
<td>Interior</td>
<td>0.0625</td>
<td>0.0697</td>
</tr>
<tr>
<td>A</td>
<td>Exterior</td>
<td>0.0591</td>
<td>N/A</td>
</tr>
<tr>
<td>C</td>
<td>Exterior</td>
<td>0.0597</td>
<td>N/A</td>
</tr>
</tbody>
</table>
Tables 4.12 and 4.13 show the GDF’s for each span along with the corresponding D-values for each span. Note that D represents the denominator of equation 4-5, \( g = S/D \). Therefore, minimum and maximum GDF’s correspond to the largest and smallest values of D.

Table 4.12: GDF’s and D-values for Span A

<table>
<thead>
<tr>
<th>Truck Config.</th>
<th>Type</th>
<th># Passes</th>
<th>Min</th>
<th>Max</th>
<th>Avg</th>
<th>Min</th>
<th>Max</th>
<th>Avg</th>
<th>Comb. Avg.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interior</td>
<td>Creep</td>
<td>5</td>
<td>0.285</td>
<td>0.287</td>
<td>0.286</td>
<td>11.4</td>
<td>11.3</td>
<td>11.4</td>
<td>11.2</td>
</tr>
<tr>
<td></td>
<td>Dynamic</td>
<td>6</td>
<td>0.288</td>
<td>0.303</td>
<td>0.296</td>
<td>11.3</td>
<td>10.7</td>
<td>11.0</td>
<td></td>
</tr>
<tr>
<td>Exterior</td>
<td>Creep</td>
<td>5</td>
<td>0.400</td>
<td>0.414</td>
<td>0.407</td>
<td>8.1</td>
<td>7.9</td>
<td>8.0</td>
<td>8.0</td>
</tr>
</tbody>
</table>

Table 4.13: GDF’s and D-values for Span C

<table>
<thead>
<tr>
<th>Truck Config.</th>
<th>Type</th>
<th># Passes</th>
<th>Min</th>
<th>Max</th>
<th>Avg</th>
<th>Min</th>
<th>Max</th>
<th>Avg</th>
<th>Comb. Avg.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interior</td>
<td>Creep</td>
<td>5</td>
<td>0.304</td>
<td>0.307</td>
<td>0.305</td>
<td>10.7</td>
<td>10.6</td>
<td>10.7</td>
<td>10.6</td>
</tr>
<tr>
<td></td>
<td>Dynamic</td>
<td>6</td>
<td>0.300</td>
<td>0.311</td>
<td>0.306</td>
<td>10.8</td>
<td>10.5</td>
<td>10.6</td>
<td></td>
</tr>
<tr>
<td>Exterior</td>
<td>Creep</td>
<td>5</td>
<td>0.377</td>
<td>0.384</td>
<td>0.380</td>
<td>8.6</td>
<td>8.5</td>
<td>8.6</td>
<td>8.6</td>
</tr>
</tbody>
</table>

Figure 4.10 shows a graphical comparison of D-values. Again, note that D-values are actually denominators, so that higher D-values correspond to lower distribution factors.
It can be observed from Figure 4.10 that the design distribution factors are greater, and thus more conservative, than the factors obtained from the field testing. This is true for both interior and exterior girder design. For the design of interior girders, with one design lane loaded, the AASHTO Standard Specification distribution factor is the most conservative, S/7.0, while the AASHTO LRFD factor is slightly less, S/7.4. The values obtained from the tests for Spans A and C were S/11.2 and S/10.6, respectively. So the interior girders are taking less of a fraction of loads than designed for. For the design of the exterior girders, with one design lane loaded, the AASHTO Standard Specification GDF and the AASHTO LRFD GDF are the same value, both having been calculated using the lever rule. This distribution factor is S/6.5. The GDF’s obtained during testing were S/8.0 and S/8.6 for exterior girders in Spans A and C, respectively. Once again, the design distribution factor is conservative and the exterior girder is withstanding a smaller fraction of the total load.

Figure 4.11 compares the interior and exterior girder distribution factors for both spans under both quasi-static and dynamic loadings. Since distribution factors are not dependant on loading type, there should be no difference between GDF’s resulting from
either quasi-static or dynamic loads. It can be seen that for this bridge this is true. For span A quasi-static tests, the average distribution factor was S/11.4, while the factor for span A dynamic tests was S/11.0. Similarly for Span C, the GDF for quasi-static tests was S/10.7 and the GDF for dynamic tests was S/10.6.

**Figure 4.11: Distribution Factors Under Quasi-static and Dynamic Loads**

It can also be seen from Figure 4.11 that the GDF’s for both Spans A and C are essentially the same. The average GDF for Span A between quasi-static and dynamic loads is S/11.2, while the average GDF for Span C between the two types of loading is S/10.65. When using the girder spacing of 6.5 ft, this translates to factors of 0.580 and 0.610, or a difference of 0.030. In other words, the actual difference in observed distribution factors for Spans A and C is 3.0% of the total load per wheel line. Therefore, because the girder spacings are equal and the girder and deck section properties are constant across the width of the bridge, the distribution factors for the deck reinforced with GFRP and epoxy-coated steel and the deck reinforced with all epoxy-coated steel are the same.
4.2.3 Comparison of Recorded Bottom Flange Strains and Measured Distribution Factors from Deflections

In order to further verify the distribution factors resulting from the deflection data, bottom flange strains were calculated for the two exterior girders and the center girder from the measured weights of the truck, the composite section properties of the bridge girders, and the distribution factors calculated from the deflection results. These calculated bottom flange strains were then compared to the measured strains.

The following calculations are presented in Appendix A. First, the maximum moment at the midspan due to the truck weights was computed. The truck weight was measured at a nearby quarry to be 16.04 kips at the front axle and 33.56 kips at the rear tandem axles. The weight at the rear axles was assumed to be split equally between the two. The truck was then positioned on the span to create the maximum moment at the midspan. This orientation proved to be when the truck’s first rear axle was directly over the midspan. The moment at the midspan under this configuration was calculated to be 405 ft-kips. This moment was then distributed into each girder using the distribution factors calculated from the deflection data. For example, the distribution factor calculated at Girder 5 in Span A for the interior truck configuration was 0.317 per lane. The total moment at midspan was multiplied by this factor to obtain the total moment in Girder 5 at the midspan, or 128.4 ft-kips. The composite section modulus to the bottom of the girder was calculated to be 394 in$^3$, based on a nominal concrete compressive strength ($f'_c$) of 4,000 psi and ignoring the deck reinforcing. The moment in the girder was then divided by the section modulus to obtain the bottom flange stress, or 3.91 ksi for Span A girder 5 with the interior configuration. The elastic modulus of steel, 29,000 ksi, was then used to calculate the bottom flange strain, or 135 $\mu$e for Span A Girder 5 with the interior configuration. This calculated strain was compared with measured strains from both static and dynamic loading. Figure 4.12 shows a comparison of calculated and measured bottom flange strains using the interior configuration, and Figure 4.13 is the same for the exterior truck configuration.
Figure 4.12: Comparison of Midspan Bottom Flange Strains with the Interior Truck Configuration
Figure 4.13: Comparison of Midspan Bottom Flange Strains with the Exterior Truck Configuration

It can be seen that for both truck configurations, the calculated strains are greater than the measured strains. Using Girder 5 in Span A with the interior truck configuration as an example, the calculated strain of 134.8 µε is 25% greater than the measured strain of 107.7 µε. This would suggest that either the calculations of the loads acting on the bridge and in turn the moment in each girder are inaccurate, the composite section properties calculated for the girder are inaccurate, or the distribution factors calculated from the deflection data are too high. The correct answer is probably a combination of the three. While the determination of the moments in the girders was a simplified procedure and the composite section analysis is an assumption of the real properties of the cross-section and has a certain degree of error involved, the distribution factors are probably inaccurate as well. In Section 4.2.2, it was discussed that the distribution factors were conservative when compared against design values from AASHTO. If the distribution factors are high when compared with measured strains, than the bridge is designed even more conservatively.
Figure 4.12 also illustrates the error in the Span A strains from the dynamic tests. These extremely low strains are very probably erroneous. However, the Span C strains under dynamic loading are more reasonable. The dynamic load allowance effect is illustrated in this data, as the dynamic strains in Span C are greater than the static strains. However, the calculated strains from the deflection data are still greater than even the dynamic strains at Girders 5 and 3.

Tables 4.14 and 4.15 present the ratios of calculated bottom flange strain to measured bottom flange strain for both static and dynamic loading. For the static tests, Girders 3 and 5 in Span A are consistent between 1.25 and 1.42, and in Span C between 1.16 and 1.36. For both spans, the ratio at Girder 3 is higher than at Girder 5, and the ratio at Girder 1 is less than or equal to 1.00, ranging from 0.73 to 1.00. For the dynamic tests, the Span A ratios are inconsistent due to the low measured strains at Girders 3 and 5. The Span C ratios are between 1.0 and 1.25, with Girder 3 again the highest at 1.24, Girder 5 at 1.02, and Girder 1 at 0.96.

Table 4.14: Calculated/Measured Bottom Flange Strains (Static Loading)

<table>
<thead>
<tr>
<th>Calculated/Measured (Static Loading)</th>
<th>Interior Configuration</th>
<th>Exterior Configuration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span A, G5 =</td>
<td>1.25</td>
<td>Span A, G5 =</td>
</tr>
<tr>
<td>Span A, G3 =</td>
<td>1.35</td>
<td>Span A, G3 =</td>
</tr>
<tr>
<td>Span A, G1 =</td>
<td>0.86</td>
<td>Span A, G1 =</td>
</tr>
<tr>
<td>Span C, G5 =</td>
<td>1.16</td>
<td>Span C, G5 =</td>
</tr>
<tr>
<td>Span C, G3 =</td>
<td>1.28</td>
<td>Span C, G3 =</td>
</tr>
<tr>
<td>Span C, G1 =</td>
<td>0.96</td>
<td>Span C, G1 =</td>
</tr>
</tbody>
</table>

Table 4.15: Calculated/Measured Bottom Flange Strains (Dynamic Loading)

<table>
<thead>
<tr>
<th>Calculated/Measured (Dynamic Loading)</th>
<th>Interior Configuration</th>
<th>Exterior Configuration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span A, G5 =</td>
<td>9.01</td>
<td>Span A, G5 =</td>
</tr>
<tr>
<td>Span A, G3 =</td>
<td>28.47</td>
<td>Span A, G3 =</td>
</tr>
<tr>
<td>Span A, G1 =</td>
<td>-2.41</td>
<td>Span A, G1 =</td>
</tr>
<tr>
<td>Span C, G5 =</td>
<td>1.02</td>
<td>Span C, G5 =</td>
</tr>
<tr>
<td>Span C, G3 =</td>
<td>1.24</td>
<td>Span C, G3 =</td>
</tr>
<tr>
<td>Span C, G1 =</td>
<td>0.96</td>
<td>Span C, G1 =</td>
</tr>
</tbody>
</table>
4.2.4 Dynamic Load Allowances

4.2.4.1 Introduction and AASHTO Definitions

Another important factor in bridge design is the dynamic load allowance, or impact factor (IM). It was stated above that the distribution factors for interior or exterior girders should be the same under both static, or quasi-static, and dynamic loads because transverse load distribution is not dependent on loading type or magnitude. However, the total load imposed on the bridge structure from the moving vehicle increases as the vehicle increases in speed. This is a phenomenon known as the dynamic load allowance. Various bridge deck characteristics such as imperfections in the deck surface, rough deck joints, vertical curvature, or uneven approach slabs all contribute to the dynamic load allowance.

AASHTO defines IM two different ways. In the AASHTO Standard Specification (2002), equation 3-1 defines the impact formula as,

\[ I = \frac{50}{(L+125)}, \]

(Eq 4-11)

where I is the impact fraction (30% maximum) and L is the length of the span loaded to produce the maximum effect in the member (ft). For the Rte. 668 bridge, with a span length of 45 ft, the impact fraction would be \( I = \frac{50}{(45+125)} = 29\% \). In the AASHTO LFRD Specification (1998), Table 3.6.2.1-1 gives values for IM instead of an equation. For all bridge components other than deck joints, and for all limit states other than fatigue and fracture, \( IM = 33\% \).

4.2.4.2 Determination of Dynamic Load Allowances from Live Load Tests

Similar to girder distribution factors, dynamic load allowances were calculated from the raw data of each test. Midspan deflections were used to calculate IM at all five girders in both spans, and midspan, bottom flange strains were used to calculate IM at the two exterior girders and the center girder. No dynamic tests were made over the exterior girder due to the safety concerns of the truck traveling so close to the barrier at high speeds, so only the interior truck configuration was used to calculate IM. The procedure to calculate IM was as follows:
1. For each quasi-static and dynamic live load test, the base line girder response (deflection or strain) was calculated by averaging the data points leading up to the moment that the truck first moves on to the bridge deck.

2. For each quasi-static and dynamic live load test, the maximum girder response was recorded, and the base line response was subtracted from this. This result was the calculated response for each test.

3. For each girder, the calculated responses under quasi-static loads were averaged to obtain an average “static” response. These average static responses are shown in row 1 of Tables 4.16 and 4.17.

4. For each girder, the calculated responses under dynamic loads were divided by the average static response and then subtracted by 1 to obtain the dynamic load allowance, or IM. These can be seen in rows 8 through 13.

5. Because there were six dynamic tests per span, there were six IM per girder, per span for both deflections and strains. A maximum, minimum, and average IM was calculated for each girder, with the maximum average IM being the controlling IM for each span.

Tables 4.16 and 4.17 illustrate the calculation of IM for both spans. The shaded values are maximum responses and the values in bold are the four controlling dynamic load allowances.
### Table 4.16: Calculation of IM for Span A

<table>
<thead>
<tr>
<th></th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
<th>G4</th>
<th>G5</th>
<th>G1</th>
<th>G3</th>
<th>G5</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Static</strong></td>
<td>0.010</td>
<td>0.039</td>
<td>0.068</td>
<td>0.086</td>
<td>0.091</td>
<td>20.7</td>
<td>72.9</td>
<td>109.5</td>
</tr>
<tr>
<td><strong>Dynamic 1</strong></td>
<td>0.111</td>
<td>0.042</td>
<td>0.071</td>
<td>0.101</td>
<td>0.111</td>
<td>19.0</td>
<td>6.0</td>
<td>14.6</td>
</tr>
<tr>
<td><strong>Dynamic 2</strong></td>
<td>0.130</td>
<td>0.049</td>
<td>0.082</td>
<td>0.111</td>
<td>0.125</td>
<td>28.8</td>
<td>3.8</td>
<td>17.2</td>
</tr>
<tr>
<td><strong>Dynamic 3</strong></td>
<td>0.120</td>
<td>0.044</td>
<td>0.073</td>
<td>0.099</td>
<td>0.104</td>
<td>18.2</td>
<td>3.6</td>
<td>13.6</td>
</tr>
<tr>
<td><strong>Dynamic 4</strong></td>
<td>0.150</td>
<td>0.053</td>
<td>0.085</td>
<td>0.114</td>
<td>0.125</td>
<td>26.2</td>
<td>4.5</td>
<td>17.4</td>
</tr>
<tr>
<td><strong>Dynamic 5</strong></td>
<td>0.132</td>
<td>0.046</td>
<td>0.076</td>
<td>0.103</td>
<td>0.108</td>
<td>19.9</td>
<td>4.0</td>
<td>15.9</td>
</tr>
<tr>
<td><strong>Dynamic 6</strong></td>
<td>0.150</td>
<td>0.056</td>
<td>0.088</td>
<td>0.115</td>
<td>0.122</td>
<td>31.4</td>
<td>4.7</td>
<td>18.5</td>
</tr>
<tr>
<td><strong>IM 1</strong></td>
<td>0.113</td>
<td>0.070</td>
<td>0.042</td>
<td>0.174</td>
<td>0.220</td>
<td>-0.081</td>
<td>-0.918</td>
<td>-0.867</td>
</tr>
<tr>
<td><strong>IM 2</strong></td>
<td>0.360</td>
<td>0.277</td>
<td>0.183</td>
<td>0.298</td>
<td>0.374</td>
<td>0.394</td>
<td>-0.948</td>
<td>-0.843</td>
</tr>
<tr>
<td><strong>IM 3</strong></td>
<td>0.225</td>
<td>0.127</td>
<td>0.071</td>
<td>0.155</td>
<td>0.143</td>
<td>-0.122</td>
<td>-0.951</td>
<td>-0.875</td>
</tr>
<tr>
<td><strong>IM 4</strong></td>
<td>0.543</td>
<td>0.376</td>
<td>0.246</td>
<td>0.331</td>
<td>0.370</td>
<td>0.265</td>
<td>-0.939</td>
<td>-0.841</td>
</tr>
<tr>
<td><strong>IM 5</strong></td>
<td>0.352</td>
<td>0.185</td>
<td>0.123</td>
<td>0.207</td>
<td>0.189</td>
<td>-0.040</td>
<td>-0.944</td>
<td>-0.855</td>
</tr>
<tr>
<td><strong>IM 6</strong></td>
<td>0.832</td>
<td>0.445</td>
<td>0.287</td>
<td>0.341</td>
<td>0.337</td>
<td>0.650</td>
<td>-0.935</td>
<td>-0.831</td>
</tr>
<tr>
<td><strong>IM_{max}</strong></td>
<td>0.832</td>
<td>0.445</td>
<td>0.287</td>
<td>0.341</td>
<td>0.374</td>
<td>0.650</td>
<td>-0.918</td>
<td>-0.831</td>
</tr>
<tr>
<td><strong>IM_{min}</strong></td>
<td>0.113</td>
<td>0.070</td>
<td>0.042</td>
<td>0.155</td>
<td>0.143</td>
<td>-0.122</td>
<td>-0.951</td>
<td>-0.875</td>
</tr>
<tr>
<td><strong>IM_{avg}</strong></td>
<td>0.403</td>
<td>0.246</td>
<td>0.159</td>
<td>0.251</td>
<td>0.272</td>
<td>0.178</td>
<td>-0.939</td>
<td>-0.841</td>
</tr>
<tr>
<td><strong>IM_{avg,NB}</strong></td>
<td>0.578</td>
<td>0.359</td>
<td>0.239</td>
<td>0.323</td>
<td>0.360</td>
<td>0.436</td>
<td>-0.941</td>
<td>-0.838</td>
</tr>
<tr>
<td><strong>IM_{avg,SB}</strong></td>
<td>0.230</td>
<td>0.128</td>
<td>0.079</td>
<td>0.179</td>
<td>0.184</td>
<td>-0.081</td>
<td>-0.938</td>
<td>-0.866</td>
</tr>
</tbody>
</table>

### Table 4.17: Calculation of IM for Span C

<table>
<thead>
<tr>
<th></th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
<th>G4</th>
<th>G5</th>
<th>G1</th>
<th>G3</th>
<th>G5</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Static</strong></td>
<td>0.016</td>
<td>0.041</td>
<td>0.066</td>
<td>0.091</td>
<td>0.090</td>
<td>20.9</td>
<td>70.6</td>
<td>109.0</td>
</tr>
<tr>
<td><strong>Dynamic 1</strong></td>
<td>-0.001</td>
<td>0.042</td>
<td>0.071</td>
<td>0.103</td>
<td>0.108</td>
<td>19.5</td>
<td>68.7</td>
<td>128.1</td>
</tr>
<tr>
<td><strong>Dynamic 2</strong></td>
<td>0.000</td>
<td>0.038</td>
<td>0.065</td>
<td>0.093</td>
<td>0.096</td>
<td>18.7</td>
<td>66.3</td>
<td>115.4</td>
</tr>
<tr>
<td><strong>Dynamic 3</strong></td>
<td>-0.001</td>
<td>0.048</td>
<td>0.077</td>
<td>0.108</td>
<td>0.112</td>
<td>21.7</td>
<td>72.2</td>
<td>127.8</td>
</tr>
<tr>
<td><strong>Dynamic 4</strong></td>
<td>0.000</td>
<td>0.036</td>
<td>0.065</td>
<td>0.097</td>
<td>0.105</td>
<td>16.9</td>
<td>66.7</td>
<td>125.7</td>
</tr>
<tr>
<td><strong>Dynamic 5</strong></td>
<td>0.000</td>
<td>0.046</td>
<td>0.075</td>
<td>0.106</td>
<td>0.113</td>
<td>19.2</td>
<td>69.0</td>
<td>128.1</td>
</tr>
<tr>
<td><strong>Dynamic 6</strong></td>
<td>0.000</td>
<td>0.041</td>
<td>0.070</td>
<td>0.099</td>
<td>0.102</td>
<td>20.1</td>
<td>73.4</td>
<td>126.1</td>
</tr>
<tr>
<td><strong>IM 1</strong></td>
<td>-1.034</td>
<td>0.033</td>
<td>0.081</td>
<td>0.127</td>
<td>0.202</td>
<td>-0.068</td>
<td>-0.027</td>
<td>0.176</td>
</tr>
<tr>
<td><strong>IM 2</strong></td>
<td>-0.983</td>
<td>-0.057</td>
<td>-0.006</td>
<td>0.016</td>
<td>0.065</td>
<td>-0.108</td>
<td>-0.061</td>
<td>0.059</td>
</tr>
<tr>
<td><strong>IM 3</strong></td>
<td>-1.056</td>
<td>0.189</td>
<td>0.174</td>
<td>0.187</td>
<td>0.243</td>
<td>0.034</td>
<td>0.023</td>
<td>0.173</td>
</tr>
<tr>
<td><strong>IM 4</strong></td>
<td>-0.979</td>
<td>-0.103</td>
<td>-0.010</td>
<td>0.060</td>
<td>0.164</td>
<td>-0.195</td>
<td>-0.055</td>
<td>0.153</td>
</tr>
<tr>
<td><strong>IM 5</strong></td>
<td>-1.012</td>
<td>0.124</td>
<td>0.134</td>
<td>0.167</td>
<td>0.252</td>
<td>-0.083</td>
<td>-0.022</td>
<td>0.176</td>
</tr>
<tr>
<td><strong>IM 6</strong></td>
<td>-0.990</td>
<td>-0.003</td>
<td>0.055</td>
<td>0.092</td>
<td>0.129</td>
<td>-0.038</td>
<td>0.040</td>
<td>0.157</td>
</tr>
<tr>
<td><strong>IM_{max}</strong></td>
<td>-0.979</td>
<td>0.189</td>
<td>0.174</td>
<td>0.187</td>
<td>0.252</td>
<td>0.034</td>
<td>0.040</td>
<td>0.176</td>
</tr>
<tr>
<td><strong>IM_{min}</strong></td>
<td>-1.056</td>
<td>-0.103</td>
<td>-0.010</td>
<td>0.016</td>
<td>0.065</td>
<td>-0.195</td>
<td>-0.061</td>
<td>0.059</td>
</tr>
<tr>
<td><strong>IM_{avg}</strong></td>
<td>-1.009</td>
<td>0.031</td>
<td>0.071</td>
<td>0.108</td>
<td>0.176</td>
<td>-0.076</td>
<td>-0.017</td>
<td>0.149</td>
</tr>
<tr>
<td><strong>IM_{avg,NB}</strong></td>
<td>-0.984</td>
<td>-0.054</td>
<td>0.013</td>
<td>0.056</td>
<td>0.119</td>
<td>-0.114</td>
<td>-0.025</td>
<td>0.123</td>
</tr>
<tr>
<td><strong>IM_{avg,SB}</strong></td>
<td>-1.034</td>
<td>0.115</td>
<td>0.130</td>
<td>0.160</td>
<td>0.232</td>
<td>-0.039</td>
<td>-0.009</td>
<td>0.175</td>
</tr>
</tbody>
</table>
4.2.4.3 Dynamic Load Allowance Results from Deflection Data

It can be seen from Table 4.16 that the maximum average impact factor, or IM\textsubscript{avg}, from deflection data was at Girder 5, or the exterior girder closest to the truck’s path. Therefore, the deflections under quasi-static and dynamic loads at Girder 5 will be investigated. The average of the maximum deflections under quasi-static loads, or Static\textsubscript{Avg}, for Span A at Girder 5 was 0.091 in. The maximum deflections under dynamic loads for Span A at Girder 5 ranged from 0.104 in. to 0.125 in. Figure 4.14 shows a graph of Girder 5 deflection vs. time for both quasi-static and dynamic loads in Span A. The data shown in Figure 4.14 is from Span A quasi-static test 1 and Span A dynamic test 1, and it illustrates the concept of dynamic load allowance.

![Span A Girder 5 Deflection vs. Time](image)

**Figure 4.14: Span A Deflections Under Quasi-static and Dynamic Loading**

Referring back to Table 4.16, IM\textsubscript{avg} was the average maximum deflection divided by Static\textsubscript{Avg} minus 1, or 0.272. The design impact factors from AASHTO Standard and AASHTO LRFD were 0.29 and 0.33, respectively. Therefore, the calculated impact factors for Span A using deflection data were less than the design values, and thus, the design was adequate. Figure 4.15 shows a comparison of impact factors from deflection data.
Figure 4.15: Dynamic Load Allowance Summary from Deflection Data

It can be seen from Table 4.17 that the maximum IM$_{avg}$ from deflection data was at Girder 5. Therefore, the deflections under quasi-static and dynamic loads at Girder 5 will be investigated. Static$_{Avg}$ for Span C at Girder 5 was 0.090 in. Note that the maximum Static$_{Avg}$ for Span C was at Girder 4. However, the dynamic deflections at Girder 5 were greater than that at Girder 4, making IM$_{avg}$ greater at Girder 5. The maximum deflections under dynamic loads for Span C at Girder 5 ranged from 0.096 in. to 0.113 in. Figure 4.16 shows a graph of Girder 5 deflection vs. time for both quasi-static and dynamic loads in Span C. The data shown in Figure 4.16 is from Span C quasi-static test 4 and Span C dynamic test 4.
For Span C, IM_{avg} was calculated to be 0.176. Similar to Span A, the design impact factors from AASHTO Standard and AASHTO LRFD were 0.29 and 0.33, respectively. The calculated impact factors for Span C using deflection data were less than the design values, making the design adequate. Again, see Figure 4.13.

It can be seen from Figure 4.15 that the impact factors differ from Span A to Span C. While both are less than the design values, the IM from Span A is 0.27, greater than the IM from Span C, 0.18. This could be due to many different factors related to the bridge deck surface and roadway profile, as well as the presence of GFRP reinforcement in the top mat in Span A. In Appendix A, the transformed moment of inertia for the composite girder (I_c) is calculated for both Spans A and C. Typically, the reinforcement in the bridge deck is such a small amount of material in the composite girder that it is not taken into account. However, because the top mat reinforcement is the only difference in the two spans, it is included in these calculations. I_c for Span A was calculated as 10480 in^4 and I_c for Span C was calculated as 10599 in^4, resulting in a decrease of 119 in^4 or 1.12% when GFRP is used as the top mat reinforcing. This very small difference in
transformed moment of inertia of the composite girder for the two spans is not enough to cause the difference in impact factors between the two spans that are seen here.

Taking a closer look at the two spans, Static$_{Avg}$ at Girder 5 for both spans was essentially equal, or 0.091 in. for Span A and 0.090 in. for Span C. Therefore, the difference in impact factor for the two spans must lie in the dynamic deflections. The deflections from dynamic loads in Span A were greater than that in Span C, and that probably has to do with imperfections in the profiles of the deck and the approaches. Both spans were tested with the truck driving approximately 50 miles per hour traveling both north and south. However, if the profiles for the approaches and center span were not perfect, the results could vary. For instance, a northbound test for Span A means that the truck traveled first over the south approach, then Span A, then Span B, and so on. A northbound test for Span C means that the truck traveled first over Span B, then Span C, then the north approach. The exact opposite is true for the southbound tests. Therefore, if the joints at both ends of Span B were not exactly the same, or if the joints at both ends of the bridge were not exactly the same, or if the approaches did not ride exactly the same, then the dynamic deflections for the two spans would be different. The average impact factors for both the northbound and southbound directions for each span are given in the last two rows of Tables 4.16 and 4.17. It is clear that when the truck crosses over the approach first (i.e., a northbound test for Span A or a southbound test for Span C) the impact factors are higher than when the truck crosses over Span B first. For instance, the average impact factor for Span A in the northbound direction is 0.36, compared to 0.18 in the southbound direction. Not only is the northbound value twice that of the southbound value, it is higher than the design values from AASHTO of 0.33 and 0.29.

Another, more simple explanation for the difference in impact factors is that the deck surface of Span A may simply be more rough than that of Span C. However, the important result from these tests is that the average impact factors from deflection data for both the span reinforced with FRP and steel, and the span reinforced with steel alone, were conservative. However, the average impact factor in the northbound direction for Span A was higher than design values, and must be investigated further.
4.2.4.4 Dynamic Load Allowance Results from Strain Data

Impact factors resulting from strain measurements were investigated as well. It can be seen from Table 4.16 that StaticAvg for Span A from strain data was 109.5 microstrain (µε) at Girder 5. However, the maximum girder strains under the six dynamic tests for Span A ranged from 13.6 µε to 18.5 µε, resulting in an average impact factor of –0.852. This obviously is due to an experimental error during the Span A dynamic tests. Because the Span A bottom flange strain gauges were recording reasonable values during the quasi-static tests, it is this researcher’s assumption that the gauges had either ceased to function properly after the quasi-static tests, or that an error was made in the process of connecting the wires to the data acquisition system and recording and processing the dynamic test data from those gauges. As discussed in Chapter 3, the quasi-static tests for Span A were performed first. Then, the Span A instrumentation was disconnected, the data acquisition system was relocated, the Span C instrumentation was connected, and the Span C quasi-static tests were performed. The Span C dynamic tests were next to be completed, and then the Span C instrumentation was disconnected. When the Span A instrumentation was reconnected to perform the Span A dynamic tests, an error must have occurred with the bottom flange gauges. Therefore, the Span A bottom flange strains resulting from the dynamic tests cannot be used. Figure 4.17 shows a graph of Girder 5 bottom flange strain vs. time for both quasi-static and dynamic loads in Span A. The data shown in Figure 4.17 is from Span A quasi-static test 5 and Span C dynamic test 5.
Table 4.17 shows that the maximum IM_{avg} from strain data in Span C was at Girder 5. Therefore, the strains under quasi-static and dynamic loads at Girder 5 will be investigated. Static_{Avg} for Span C at Girder 5 was 109 \, \mu \varepsilon. The maximum strains under dynamic loads for Span C at Girder 5 ranged from 115.4 \, \mu \varepsilon to 128.1 \, \mu \varepsilon. Figure 4.18 shows a graph of Girder 5 bottom flange strain vs. time for both quasi-static and dynamic loads in Span C. The data shown in Figure 4.18 is from Span C quasi-static test 4 and Span C dynamic test 4.
Figure 4.18: Span C Bottom Flange Strains Under Quasi-static and Dynamic Loading

IM_{avg} for Span C was calculated to be 0.149. Again, the design impact factors from AASHTO Standard and AASHTO LRFD were 0.29 and 0.33, respectively. The calculated impact factors for Span C using strain data were less than the design values, making the design adequate. See Figure 4.19 for a comparison of impact factors from strain data. Additionally, the Span C impact factors resulting from deflections and strains were very similar. IM_{avg} from deflections was 0.176 and IM_{avg} from strains was 0.149. This agreement of like data provides further assurance that the extremely low strains under dynamic loads for Span A were erroneous.
4.2.5 Comparison of Weigh-In-Motion Gauge Results

As discussed in Chapter 3, weigh-in-motion (WIM) strain gauges were attached to the bottom flanges of Girders 1, 3, and 5 to compare against electrical resistance (ER) strain gauges. This was to determine whether or not the simpler, easier-to-install WIM gauges give results in line with the ER strain gauges. Strains from the WIM gauges were recorded in Span A for the quasi-static tests using both of the truck configurations, and for the dynamic tests as well. Two data sets from both the interior and exterior configuration quasi-static tests are presented here, as well as four data sets from the dynamic tests. These data sets were deemed to be representative of all the tests. Table 4.18 compares the maximum bottom flange strains from the ER strain gauges and the WIM strain gauges for the eight selected tests.

Figure 4.19: Dynamic Load Allowance Summary from Strain Data
Table 4.18: Comparison of Maximum Strains

<table>
<thead>
<tr>
<th>Girder 1</th>
<th>Girder 3</th>
<th>Girder 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strain Gauge</td>
<td>WIM Gauge</td>
<td>Strain Gauge</td>
</tr>
<tr>
<td>Creep 1</td>
<td>23</td>
<td>21</td>
</tr>
<tr>
<td>Creep 2</td>
<td>20</td>
<td>19</td>
</tr>
<tr>
<td>Creep 6</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Creep 7</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Dynamic 1</td>
<td>30</td>
<td>1</td>
</tr>
<tr>
<td>Dynamic 2</td>
<td>29</td>
<td>1</td>
</tr>
<tr>
<td>Dynamic 4</td>
<td>26</td>
<td>1</td>
</tr>
<tr>
<td>Dynamic 5</td>
<td>20</td>
<td>1</td>
</tr>
</tbody>
</table>

Note: Creep Tests 1 and 2 and Dynamic Tests 1, 2, 4, and 5 used the interior truck configuration. Creep Tests 6 and 7 used the exterior truck configuration. All strains are expressed in µε.

Figures 4.20, 4.21, and 4.22 show the comparison of ER strain gauges and WIM strain gauges for Span A quasi-static test 1 for Girders 1, 3, and 5, respectively. It can be seen that during the quasi-static tests, the WIM gauges at Girders 1 and 3 provide strains very similar to the ER gauges. However, at Girder 5, or the exterior girder closest to the truck load, the comparison is poor. Because the agreement of strains under quasi-static loads is very good for Girders 1 and 3, but poor for Girder 5, it is reasonable that the WIM gauge at Girder 5 may have been installed or connected to the data acquisition system improperly. Also, the shape of the WIM gauges at Girder 5 mimics that of the ER gauges, but the WIM gauges seem to be off by a common factor. Table 4.19 compares the ratio of maximum strains from the ER gauges and the WIM gauges at Girders 1, 3, and 5. Again, the comparison is very good for Girders 1 and 3, with ratios ranging from 1.01 to 1.10. All of the Girder 5 ratios are from 1.70 to 1.71, leading to the assumption that a scale factor of some sort was erroneous for that WIM gauge. Regardless, more work with these gauges in the field is required to determine if this is in fact true.
Comparison of WIM and ER Strain Gauges -- Girder 1 (Span A Creep 1)

Figure 4.20: Comparison of Strain Gauges at Girder 1 for Span A Quasi-static Test

Comparison of WIM and ER Strain Gauges -- Girder 3 (Span A Creep 1)

Figure 4.21: Comparison of Strain Gauges at Girder 3 for Span A Quasi-static Test
Figure 4.22: Comparison of Strain Gauges at Girder 5 for Span A Quasi-static Test

Table 4.19: Comparison of Ratios of Maximum Strains of ER Gauges and WIM Gauges

<table>
<thead>
<tr>
<th></th>
<th>Girder 1</th>
<th>Girder 3</th>
<th>Girder 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Creep 1</td>
<td>1.10</td>
<td>1.07</td>
<td>1.71</td>
</tr>
<tr>
<td>Creep 2</td>
<td>1.09</td>
<td>1.06</td>
<td>1.70</td>
</tr>
<tr>
<td>Creep 6</td>
<td>1.04</td>
<td>1.03</td>
<td>1.71</td>
</tr>
<tr>
<td>Creep 7</td>
<td>1.01</td>
<td>1.03</td>
<td>1.70</td>
</tr>
</tbody>
</table>

During the dynamic tests, none of the WIM gauges recorded any reasonable values, leading the researcher to believe that either these WIM gauges are not to be used to dynamic loads or that the gauges were either installed or connected to the data acquisition system incorrectly. Figures 4.23, 4.24, and 4.25 illustrate the comparison of the ER gauges and the WIM gauges for the Span A dynamic test 1 at Girders 1, 3, and 5, respectively. Again, more work with these gauges in the field is required to determine the acceptability of WIM gauges in this application.
Figure 4.23: Comparison of Strain Gauges at Girder 1 for Span A Dynamic Test 1

Figure 4.24: Comparison of Strain Gauges at Girder 3 for Span A Dynamic Test 1
Figure 4.25: Comparison of Strain Gauges at Girder 5 for Span A Dynamic Test 1
Chapter 5 – Conclusions and Recommendations

5.1 Introduction

The purpose of this research project was to investigate glass fiber reinforced polymer (GFRP) as top mat reinforcing in concrete bridge decks in the field. This investigation focused on three main objectives, using the new Route 668 bridge over Gills Creek in Franklin County, Virginia. The primary objective was to test a new bridge deck reinforced with GFRP bars in the top mat in the field. The second objective was to monitor the construction of the bridge and to comment on the constructibility of bridge decks reinforced with GFRP bars. The third and final objective was to instrument and prepare the bridge deck for long-term monitoring. The following sections present the conclusions pertaining to the three objectives of the research, as well as provide recommendations for researchers and professionals alike.

5.2 Conclusions From Live Load Testing

Results from the live load tests were compiled in five major areas: investigation of the strains and stresses in the top mat transverse GFRP bars, comparison of girder distribution factors from Span A and Span C, comparison of impact factors from Span A and Span C, comparison of calculated and measured bottom flange strains, and comparison of the weigh-in-motion gauges and the electrical resistance strain gauges. Conclusions and recommendations regarding the live load testing of the bridge can be garnered from all five areas.

5.2.1 Strains and Stresses in the GFRP Reinforcement

In each of the three gauged sections in Span A, during both the quasi-static and dynamic tests, the stresses in the top mat transverse reinforcement were well under their limit. The highest tensile bar stresses occurred during the dynamic tests in the section of gauged bars directly over the first interior girder close to the abutment. The truck’s path was directly over that girder, with the two wheel lines straddling it. The maximum bar stress was approximately 0.10 ksi. Compared to the maximum allowable stress from ACI 440 discussed in Chapter 4 of 13.9 ksi, a maximum service stress of 0.10 ksi is not a
concern. Additionally, the stresses measured in the field were compared to Cawrse’s (2002) data from the laboratory. The field testing results were less than the lab results. Cawrse’s maximum bar stresses under a service axle load of 30 kips was 0.16 ksi, compared to 0.10 ksi under a service truck load in the field. It can be concluded that the top mat transverse GFRP reinforcement was designed very conservatively with respect to stresses, and that tensile failure of the GFRP bars under service conditions is not a concern. It is recommended that the design of deck slabs reinforced with GFRP in the top mat be further investigated to reduce the extreme level of conservatism and to improve the cost effectiveness. However, the issue of cracking, which was not investigated in this research, must be considered in the design as well.

No conclusions can be made regarding the comparison of stresses and strains in the Span A GFRP bars and the Span C steel bars. This is because too many of the electrical resistance strain gauges attached to the steel bars in Span C were no longer functioning after the bars were instrumented and the concrete was cast.

5.2.2 Girder Distribution Factors

The girder distribution factor results show that the Route 668 bridge was designed conservatively when compared with design values from the AASHTO Standard Specification and the AASHTO LRFD Specification. For both the interior and exterior girders, and for both Span A and Span C, the distribution factors measured during the live load tests were less than the AASHTO design values. For the interior girder distribution factor, the values of S/11.2 and S/10.6 measured from Span A and Span C, respectively, were much less than the distribution factors from AASHTO Standard and LRFD, or S/7.0 and S/7.4, respectively. For the exterior girder distribution factor, the values of S/8.0 and S/8.6 measured from Span A and Span C, respectively, were less than the design distribution factor calculated using the lever rule, S/6.5. It can then be concluded that the bridge was designed conservatively with respect to transverse load distribution, and it does not appear that any girder would ever see a greater fraction of lane load than what it was designed for under service conditions.

Furthermore, it can be seen that there is not a significant difference in girder distribution factors between Span A and Span C. Since the difference between Span A
and Span C interior girder distribution factors, or S/11.2 and S/10.6, as well as the
difference between the two spans’ exterior girder distribution factors, or S/8.0 and S/8.6,
is small, it can be concluded that the span reinforced with steel on the bottom and GFRP
on the top is no different with respect to transverse load distribution than the span
reinforced with all steel. However, this field testing was performed when the deck was
brand new and uncracked. The deck’s response in terms of transverse load distribution
may be different after cracking.

5.2.3 Comparison of Calculated and Measured Bottom Flange Strains

For both the exterior and interior truck orientations, the bottom flange strains
calculated using the distribution factors from the deflections were slightly higher than the
actual bottom flange strains recorded during the tests. As discussed in Chapter 4, this
could be a result of inaccuracies involved in the field testing itself. However, the
important conclusion to draw from this comparison is that because the bottom flange
strains calculated from the measured girder distribution factors are higher than the actual
bottom flange strains measured during the tests, the composite girder is acting as an
uncracked section and the strains are below expected values.

5.2.4 Dynamic Load Allowance

The dynamic load allowance results show that the Route 668 bridge was designed
conservatively when compared with design values from the AASHTO Standard
Specification and the AASHTO LRFD Specification. For both Span A and Span C, the
impact factors measured during the live load tests were less than the AASHTO design
values. For the impact factors measured from deflection data, the values of 0.27 and 0.18
measured from Span A and Span C, respectively, were less than the impact factors from
AASHTO Standard and LRFD, or 0.29 and 0.33, respectively. However, the value of
0.36 from the northbound tests at Span A was greater than design values, causing it to be
unconservative in the direction from the approach onto the bridge. This must be
investigated further in future live load tests. For the impact factors measured from strain
data, the value of 0.15 measured from Span C was less than the AASHTO design values
as well. The impact factor measured from strain data in Span A, -0.85, was based on
erroneous data reported by malfunctioning gauges and therefore cannot be included in
this discussion. The smaller impact factors recorded from the live load tests means that the bridge girders are seeing less of a dynamic effect than designed for. It can then be concluded that the bridge was designed conservatively with respect to dynamic loads.

When comparing Spans A and C, it can be seen that the impact factors from deflection data differ slightly. Data from Span A reported an impact factor slightly higher than that of Span C. However, it is difficult to ascertain whether or not that has anything to do with the cross-sectional properties of the two spans, or more specifically, the presence of GFRP bars as the top mat of reinforcing in Span A. The transformed moment of inertia of the composite sections at Spans A and C were found to be 10480 in$^4$ and 10599 in$^4$, respectively. Therefore, the presence of GFRP in the top mat of reinforcement instead of steel decreased the moment of inertia of the composite girder by 1.12%. Therefore, the difference in impact factors of the two spans has more to do with differences in the deck surface roughness of each span, and also profile differences in the two approaches. Future live load tests on the Route 668 bridge will investigate this question further.

5.2.5 Comparison of Weigh-In-Motion Strain Gauges and Electrical Resistance Strain Gauges

The results from the weigh-in-motion (WIM) strain gauges show that for the quasi-static tests, the strains recorded from the WIM gauges and the electronic resistance (ER) strain gauges were very similar at Girders 1 and 3, but at Girder 5, closest to the load, the strains from the WIM gauges were much less than these from the ER gauges. However, with only one group of data, it is difficult to get an idea of the accuracy of these two types of gauges. From this data, it can be concluded that the WIM gauges are not adequate to record accurate strains in place of ER gauges. Future tests utilizing WIM gauges and ER gauges in tandem should be performed to further evaluate the WIM gauges.

For the dynamic tests, the strains recorded from the WIM gauges are much smaller than the strains recorded from the ER gauges. However, as before, it is difficult to make any conclusions on the acceptability of WIM gauges for dynamic tests when using only one group of data. In the absence of any future research on WIM gauges, it
can be concluded from these results that WIM gauges are not an acceptable means to measure strains under dynamic loads in place of ER gauges.

5.3 Conclusions from Construction Monitoring

As discussed in Chapter 3, the construction of the Route 668 bridge over Gills Creek, and more specifically the bridge deck, was monitored to determine the constructibility of bridge decks reinforced with GFRP. The same crew installed the bars in both Spans A and C, and the installation of the Span A GFRP bars took about a half day longer than the Span C steel bars. Installation of Span A reinforcement lasted about a full day, while the installation of Span C reinforcement lasted about a half day. The time it took for the crew to install the top mat GFRP bars in Span A was about the same as it took for them to install both mats of steel bars in Span C. It was clear that the crew was very experienced in installing epoxy-coated steel bars. In Span C, both mats were installed with little or no conversation amongst the crew. The bars were handled, installed, and tied with a minimal amount of delays or questions. Conversely, the crew was clearly inexperienced in handling the GFRP bars. However, after an hour or two its production was just as swift as with the steel bars. The major factor in the increased amount of installation time for the GFRP bars was the fact that there was almost two times the number of bars as in Span C. Moreover, the bars were spaced very closely, so more ties were necessary and due to the tight spaces, it was more difficult to install the ties. Some comments on the installation of the GFRP bars from the crew were as follows:

- the rough, sand-impregnated surface of the bars required gloves to be worn at all times,
- the lightweight nature of the bars made it much easier to carry a bundle of bars and lay them in their approximate locations,
- the increased flexibility of the bars was a concern, but chairs were spaced at closer intervals than in Span C to alleviate that, requiring more time to install the bars,
• the increased number of ties was the biggest complaint, due to the increased number of bars and the necessity of tying the top mat to the bottom mat to prevent floatation of the bars during concrete casting.

Overall, the installation of the GFRP bars went smoothly when compared to that of the steel bars. The increased installation time for the GFRP bars was mainly due to the increased number of bars in Span A. The workers were unfamiliar with the material at first, but by the end of the day were handling, installing, and tying the GFRP bars with skill. From the monitoring of the construction of the Route 668 bridge over Gills Creek, it can be concluded that GFRP bars are an acceptable material in bridge deck applications with respect to constructibility issues.

5.4 Long-Term Monitoring Conclusions

Sensors were installed in Span A to monitor the GFRP-reinforced deck for the long term. ER strain gauges, which were also used during the initial live load tests, vibrating wire (VW) strain gauges, and thermocouples were installed in the deck prior to concrete casting to provide strain and temperature readings throughout the service life of the bridge. All of the VW gauges and thermocouples were confirmed to be working properly after the concrete was placed, and will be used in conjunction with a remote data logger to record data at specific time intervals. Additionally, the ER gauges that were deemed to be in working condition after the concrete was placed will be connected to the data logger. These sensors will be able to report information regarding the long-term health of the GFRP bars at the future researchers’ discretion. It can be concluded that Span A reinforced with GFRP bars on the top mat is instrumented sufficiently for long-term monitoring. At this time, no other conclusions can be made regarding the long-term health of the GFRP bars and the bridge deck.

5.5 Recommendations for Future Research

Bridge decks reinforced with GFRP bars are a new and innovative idea in bridge engineering. Very few bridges in service today utilize this technology, and more research is necessary for GFRP reinforcement to be accepted as a safe and cost effective material
in bridge decks. Some recommendations for future research involving GFRP reinforcement are presented in this section.

The Route 668 bridge is instrumented for long-term health monitoring, and this is the first recommendation. Strain and temperature readings from inside the deck should be taken at regular intervals. Long-term strength degradation of GFRP bars in an alkaline environment like that of concrete is a major question among engineers and researchers today, and these questions and concerns will be addressed using real-life data from this bridge.

Additionally, future live load tests using both quasi-static or static, and dynamic loads should be completed at specific intervals in the bridge’s life. First, a test after one year in service conditions would be helpful to answer questions regarding the cracking of the deck and the ensuing increased flexibility in the GFRP-reinforced span that is expected. The strains in the top mat GFRP bars can be measured to determine whether or not the stresses in the bars have increased.

A comparison of dynamic load allowance data for Spans A and C can be accomplished with future live load tests as well. The difference in impact factors between Spans A and C during this project can be investigated with future tests, as well as the high impact factors observed during the Span A northbound tests.

Also, girder distribution factors can be calculated from test results during future live load tests, and these distribution factors can be compared against those calculated from this project.

It is recommended that for future live load tests of the bridge, all five girders be gauged with ER strain gauges. This is so that girder distribution factors can be calculated from both deflection and strain data. These distribution factors can then be verified by comparing them against each other.

The acceptability of WIM strain gauges is still a question, and should be investigated further in future live load tests. It is recommended that the bottom flanges in both Spans A and C be instrumented with WIM gauges as well as ER gauges, and these strains be compared against each other.

Finally, the effect of multiple trucks on the bridge was not investigated in this project. It is recommended that future live load tests use two trucks to load the bridge so
that girder distribution factors for “2 or more lanes loaded” be calculated and compared to design values in the AASHTO Standard Specification and the AASHTO LRFD Specification.
References


*ACI Guide for the Design and Construction of Concrete Reinforced with FRP Bars*, ACI 440.1R-03 (2003), American Concrete Institute Committee 440, Farmington Hills, MI.


*TIT FRP Reinforcing Bars in Bridge Decks: State of the Art Review*, TTI Report 1520-2, Texas Transportation Institute, College Station, TX, 2000.

Appendix A – Calculations

A.1 Composite Section Analysis

Cast-in-place slab:

\[ t_s = 8.0 \text{ in.} \]

Structural thickness = 7.5 in.

\[ f'_c = 4000 \text{ psi} \]

Haunch = 1.5 in.

Steel I-beams:

\[ W27x94 \]

\[ A = 27.7 \text{ in}^2 \]

\[ d = 26.9 \text{ in.} \]

\[ t_w = 0.49 \text{ in.} \]

\[ b_t = 9.99 \text{ in.} \]

\[ t_f = 0.745 \text{ in.} \]

\[ I_x = 3270 \text{ in}^4 \]

\[ S_x = 243 \text{ in}^3 \]

Composite Section:

Effective Flange Width:

\[ b_e = \min \left( \frac{1}{4} L, S, 12 t_s \right) \]

\[ b_e = \min \left( \frac{1}{4} \times 45 \times 12, 6.5 \times 12, 12 \times 7.5 \right) = 78 \text{ in.} \]

Modular Ratio Between Slab and Beam Material:

\[ n = \frac{E_c}{E_s} = \frac{57 \sqrt{4000}}{29000} \]

\[ n = 0.1243 \]

Transformed Section Properties:

\[ b_{c,t} = 0.1243(78) = 9.695 \text{ in.} \]

\[ A_{flg,t} = 9.695(7.5) = 72.72 \text{ in}^2 \]

Transformed haunch width = 9.99(0.1243) = 1.242 in.
Transformed haunch area = 1.242(1.5) = 1.863 in²

Properties of Composite Section:

<table>
<thead>
<tr>
<th></th>
<th>Area (in²)</th>
<th>y_b (in)</th>
<th>A_y (in³)</th>
<th>y_bc (in)</th>
<th>A(y_bc-y_b)² (in⁴)</th>
<th>I (in⁴)</th>
<th>I + A(y_bc-y_b)² (in⁴)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>27.70</td>
<td>13.45</td>
<td>372.57</td>
<td>27.00</td>
<td>5088.61</td>
<td>3270.00</td>
<td>8358.61</td>
</tr>
<tr>
<td>Haunch</td>
<td>1.86</td>
<td>27.65</td>
<td>51.51</td>
<td>0.78</td>
<td>0.78</td>
<td>0.35</td>
<td>1.13</td>
</tr>
<tr>
<td>Deck</td>
<td>72.72</td>
<td>32.15</td>
<td>2337.95</td>
<td>1925.91</td>
<td>1925.91</td>
<td>340.84</td>
<td>2266.75</td>
</tr>
<tr>
<td>Sum</td>
<td>102.28</td>
<td>--</td>
<td>2762.02</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>10626.48</td>
</tr>
</tbody>
</table>

\[ A_c = 102.28 \text{ in}^2 \]
\[ h_c = 26.9 + 1.5 + 7.5 = 35.9 \text{ in.} \]
\[ I_c = 10626 \text{ in}^4 \]
\[ y_{bc} = 27.0 \text{ in.} \]
\[ y_{tg} = 35.9 - 27.0 - 7.5 - 1.5 = -0.10 \text{ in.} \]
\[ y_{tc} = 35.9 - 27.0 = 8.9 \text{ in.} \]

\[ S_{bc} = \frac{I_c}{y_{bc}} \]

\[ S_{bc} = \frac{10626}{27} = 394 \text{ in}^3 \]

\[ S_k = \left( \frac{1}{n} \right) \left( \frac{I_c}{y_{tc}} \right) \]

\[ S_k = (1/0.1243)(10626/8.9) = 9605 \text{ in}^3 \]

**A.2 Detailed Composite Section Analysis for Dynamic Load Allowance Investigation**

Slab and beam properties equal for Spans A and C except for top reinforcement type.

Span A Composite Section:

Cast-in-place slab:

Same as above

Bottom mat reinforcement:

No. 4 epoxy-coated steel bars

\[ A = 0.20 \text{ in}^2 \]
\[ E = 29000 \text{ ksi} \]

Top mat reinforcement:

No. 6 GFRP bars
\[ A = 0.44 \text{ in}^2 \]
\[ E = 5920 \text{ ksi (from laboratory tensile tests)} \]

Steel I-beams:

Same as above

Composite Section:

Effective Flange Width:

\[ b_e = 78 \text{ in. (same as above)} \]

14 bottom mat steel bars within flange

13 top mat GFRP bars within flange

Modular Ratio Between Slab and Beam Material:

\[ n = 0.1243 \text{ (same as above)} \]

Modular Ratio Between Bottom Mat Steel Bars and Beam Material

\[ n = 1.0 \]

Modular Ratio Between Top Mat GFRP Bars and Beam Material

\[ n = \frac{5920}{29000} = 0.2041 \]

Transformed Section Properties:

\[ b_{e,tr} = 0.1243 \times 78 = 9.695 \text{ in}. \]

\[ A_{flg,tr} = 9.695 \times 7.5 - 14 \times 0.20 - 13 \times 0.44 = 64.2 \text{ in}^2 \]

Transformed haunch width = 9.99 \times 0.1243 = 1.242 in.

Transformed haunch area = 1.242 \times 1.5 = 1.863 \text{ in}^2

Transformed steel rebar area = 14 \times 0.20 \times 1.0 = 2.8 \text{ in}^2

Transformed GFRP rebar area = 13 \times 0.44 \times 0.2041 = 1.17 \text{ in}^2

Properties of Composite Section

<table>
<thead>
<tr>
<th></th>
<th>Area (in²)</th>
<th>(y_b) (in)</th>
<th>(Ay_b) (in²)</th>
<th>(y_{bc}) (in)</th>
<th>(A(y_{bc}-y_b)^2) (in⁴)</th>
<th>(I) (in⁴)</th>
<th>(I + A(y_{bc}-y_b)^2) (in⁴)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>27.70</td>
<td>13.45</td>
<td>372.57</td>
<td>26.73</td>
<td>5088.61</td>
<td>3270.00</td>
<td>8358.61</td>
</tr>
<tr>
<td>Haunch</td>
<td>1.86</td>
<td>27.65</td>
<td>51.51</td>
<td>0.78</td>
<td>0.78</td>
<td>0.35</td>
<td>1.13</td>
</tr>
<tr>
<td>Bot Rebar</td>
<td>2.80</td>
<td>30.53</td>
<td>85.47</td>
<td>34.72</td>
<td>34.72</td>
<td>0.00</td>
<td>34.72</td>
</tr>
<tr>
<td>Top Rebar</td>
<td>1.17</td>
<td>33.15</td>
<td>38.79</td>
<td>44.20</td>
<td>44.20</td>
<td>0.02</td>
<td>44.21</td>
</tr>
<tr>
<td>Deck</td>
<td>64.20</td>
<td>32.15</td>
<td>2064.03</td>
<td>1700.26</td>
<td>1700.26</td>
<td>340.84</td>
<td>2041.10</td>
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<tr>
<td>Sum</td>
<td>97.73</td>
<td>--</td>
<td>2612.36</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>10479.77</td>
</tr>
</tbody>
</table>

\[ A_c = 97.73 \text{ in}^2 \]

\[ h_c = 35.9 \text{ in.} \]

\[ I_c = 10480 \text{ in}^4 \]

\[ y_{bc} = 26.73 \text{ in.} \]
\[
y_{tg} = 35.9 - 7.5 - 1.5 - 26.73 = 0.17 \text{ in.}
\]
\[
y_{tc} = 35.9 - 26.73 = 9.17 \text{ in.}
\]
\[
S_{bc} = \frac{10480}{26.73} = 392 \text{ in}^3
\]
\[
S_{tc} = \frac{1}{0.1243}(10480/9.17) = 9194 \text{ in}^3
\]

Span C Composite Section:

Cast-in-place slab:

Same as above

Top and bottom mat reinforcement:

No. 4 epoxy-coated steel bars
\[
A = 0.20 \text{ in}^2
\]
\[
E = 29000 \text{ ksi}
\]

Steel I-beams:

Same as above

Composite Section:

Effective Flange Width:

\[b_e = 78 \text{ in. (same as above)}\]

Assume 14 bottom mat steel bars within flange
Assume 15 top mat steel bars within flange

Modular Ratio Between Slab and Beam Material:
\[n = 0.1243 \text{ (same as above)}\]

Modular Ratio Between Steel Bars and Beam Material
\[n = 1.0\]

Transformed Section Properties:

\[b_{e, tr} = 0.1243 \times 78 = 9.695 \text{ in.}\]
\[A_{flg, tr} = 9.695 \times 7.5 - 14 \times 0.20 - 15 \times 0.20 = 66.9 \text{ in}^2\]

Transformed haunch width = 9.99 \times 0.1243 = 1.242 \text{ in.}
Transformed haunch area = 1.242 \times 1.5 = 1.863 \text{ in}^2
Transformed bottom mat steel area = 14 \times 0.20 \times 1.0 = 2.8 \text{ in}^2
Transformed top mat steel area = 15 \times 0.20 \times 1.0 = 3.0 \text{ in}^2

Properties of Composite Section
### A.3 Calculation of Girder Distribution Factors

**AASHTO Standard Specification:**

For concrete deck on steel I-beam stringers:

\[ g = \frac{S}{7.0}, \quad \text{for bridges designed for one traffic lane} \]

\[ g = \frac{S}{5.5}, \quad \text{for bridges designed for two or more traffic lanes} \]

**AASHTO LFRD:**

Distribution of live load per lane for moment in interior beams:

\[
DFM_1 = 0.06 + \left( \frac{S}{14} \right)^{0.4} \left( \frac{S}{L} \right)^{0.3} \left( \frac{K_g}{12.0L^3t_s^4} \right)^{0.1}
\]

- \(3.5 \leq S \leq 16.0\)
- \(4.5 \leq t_s \leq 12.0\)
- \(20 \leq L \leq 240\)
- \(N_b \geq 4\)
- \(10,000 \leq K_g \leq 7,000,000\)

\[ K_g = n \left( I + A e^2 \right) \]

\[ S = 6.5 \text{ ft} \]

\[ t_s = 7.5 \text{ in.} \]

\[ L = 45 \text{ ft} \]
\[ N_b = 5 \]
\[ n = 8.05 \]
\[ I = 3270 \text{ in}^4 \]
\[ A = 27.7 \text{ in}^2 \]
\[ e_g = 32.15 - 13.45 = 18.7 \text{ in} \]
\[ K_g = 8.05(3270 + 27.7(18.7^2)) = 104,299 \text{ in}^4 \]

\[ DFM_1 = 0.06 + (6.5/14)^{0.4}(6.5/45)^{0.3}(104299/(12*45*7.5^3))^{0.1} \]

\[ DFM_1 = 0.441 \text{ lanes per beam, for one lane loaded} \]

\[ DFM_2 = 0.075 + \left( \frac{S}{9.5} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{K_g}{12.0Lt_s^3} \right)^{0.1} \]

\[ DFM_2 = 0.075 + (6.5/9.5)^{0.6}(6.5/45)^{0.2}(104299/(12*45*7.5^3))^{0.1} \]

\[ DFM_2 = 0.575 \text{ lanes per beam, for two lanes loaded} \]

For one lane loaded:

\[ DFM = 0.441 \text{ lanes per beam} \]

\[ g = S/D = 6.5/D = 0.441 \]

\[ D = 6.5/0.441 = 14.7 \]

\[ g = S/14.7 \text{ per lane load, or} \]

\[ g = S/7.35 \text{ per wheel line} \]

Distribution of live load per lane for moment in exterior beams:

Use lever rule for one lane loaded:

\[ \Sigma M_{IG} : -R(6.5) + P/2(5.4167) = 0 \]

\[ R = 0.417P \]

\[ DFM_1 = 0.417m = 0.417(1.2) = 0.500 \text{ lanes per beam} \]

\[ D = (S/DFM_1)(1/2) = (6.5/0.50)(1/2) = 6.5 \]
\[
g = \frac{S}{6.5} \text{ per wheel line}
\]

**A.4 Calculation of Maximum Moment at Midspan due to Truck Weights/Distribution of Moment into Girders/Calculation of Bottom Flange Strains**

Calculate maximum moment at midspan due to truck weights:

- Front axle: \( 16.04 \) kips
- Rear axles: \( 33.56 \) kips

Assume axle orientation like that of Restrepo (2002), similar VDOT dump trucks used for both tests:

Find resultant:

\[
F_R = 16.04 + 16.78 + 16.78 = 49.6 \text{ kips}
\]

\[
x_R = \frac{(16.04 \times 0 + 16.78 \times 15 + 16.78 \times 19)}{49.6} = 11.5 \text{ ft}
\]

Resultant located 3.5 ft in front of first rear axle.

Maximum moment at midspan is when truck’s first rear axle is at midspan:

\[
M_{\text{mid}} = 28.7 \times 22.5 - 16.04 \times 15 = 405 \text{ ft-kips}
\]

Distribute moment into girders and calculate bottom flange strain:

Span A Interior (Sample Calculation):

\[
M = 405 \times 0.317 = 128 \text{ ft-kips}
\]

\[
\sigma_{\text{bot}} = 128 \times 12 / 394 = 3.90 \text{ ksi}
\]

\[
\varepsilon_{\text{bot}} = 3.90 / 29000 = 134 \mu\varepsilon
\]
Max Moment @ Midspan: 405 ft-kips
Composite Section Modulus, Bottom: 394 in³
Steel Modulus: 29000 ksi

<table>
<thead>
<tr>
<th>Interior Configuration</th>
<th>Exterior Configuration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder Distribution Factors:</td>
<td>Girder Distribution Factors:</td>
</tr>
<tr>
<td>Span A, G5 = 0.317</td>
<td>Span A, G5 = 0.407</td>
</tr>
<tr>
<td>Span A, G3 = 0.222</td>
<td>Span A, G3 = 0.187</td>
</tr>
<tr>
<td>Span A, G1 = 0.038</td>
<td>Span A, G1 = 0.019</td>
</tr>
<tr>
<td>Span C, G5 = 0.302</td>
<td>Span C, G5 = 0.380</td>
</tr>
<tr>
<td>Span C, G3 = 0.217</td>
<td>Span C, G3 = 0.189</td>
</tr>
<tr>
<td>Span C, G1 = 0.046</td>
<td>Span C, G1 = 0.019</td>
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</tbody>
</table>

Calculated Moments (ft-k):
<table>
<thead>
<tr>
<th>Interior Configuration</th>
<th>Exterior Configuration</th>
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</thead>
<tbody>
<tr>
<td>Span A, G5 = 128.4</td>
<td>Span A, G5 = 164.8</td>
</tr>
<tr>
<td>Span A, G3 = 89.9</td>
<td>Span A, G3 = 75.7</td>
</tr>
<tr>
<td>Span A, G1 = 15.4</td>
<td>Span A, G1 = 7.7</td>
</tr>
<tr>
<td>Span C, G5 = 122.3</td>
<td>Span C, G5 = 153.9</td>
</tr>
<tr>
<td>Span C, G3 = 87.9</td>
<td>Span C, G3 = 76.5</td>
</tr>
<tr>
<td>Span C, G1 = 18.6</td>
<td>Span C, G1 = 7.7</td>
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Calculated Bottom Flange Stresses (ksi):
<table>
<thead>
<tr>
<th>Interior Configuration</th>
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<tbody>
<tr>
<td>Span A, G5 = 3.91</td>
<td>Span A, G5 = 5.02</td>
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<td>Span A, G3 = 2.74</td>
<td>Span A, G3 = 2.31</td>
</tr>
<tr>
<td>Span A, G1 = 0.47</td>
<td>Span A, G1 = 0.23</td>
</tr>
<tr>
<td>Span C, G5 = 3.73</td>
<td>Span C, G5 = 4.69</td>
</tr>
<tr>
<td>Span C, G3 = 2.68</td>
<td>Span C, G3 = 2.33</td>
</tr>
<tr>
<td>Span C, G1 = 0.57</td>
<td>Span C, G1 = 0.23</td>
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</tbody>
</table>

Calculated Bottom Flange Strains (µε):
<table>
<thead>
<tr>
<th>Interior Configuration</th>
<th>Exterior Configuration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span A, G5 = 135</td>
<td>Span A, G5 = 173</td>
</tr>
<tr>
<td>Span A, G3 = 94</td>
<td>Span A, G3 = 80</td>
</tr>
<tr>
<td>Span A, G1 = 16</td>
<td>Span A, G1 = 8</td>
</tr>
<tr>
<td>Span C, G5 = 128</td>
<td>Span C, G5 = 162</td>
</tr>
<tr>
<td>Span C, G3 = 92</td>
<td>Span C, G3 = 80</td>
</tr>
<tr>
<td>Span C, G1 = 20</td>
<td>Span C, G1 = 8</td>
</tr>
</tbody>
</table>

Measured Bottom Flange Strains (µε), Under Static Loading:
<table>
<thead>
<tr>
<th>Interior Configuration</th>
<th>Exterior Configuration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span A, G5 = 108</td>
<td>Span A, G5 = 136</td>
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<tr>
<td>Span A, G3 = 70</td>
<td>Span A, G3 = 56</td>
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<tr>
<td>Span A, G1 = 19</td>
<td>Span A, G1 = 8</td>
</tr>
<tr>
<td>Span C, G5 = 111</td>
<td>Span C, G5 = 136</td>
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<tr>
<td>Span C, G3 = 72</td>
<td>Span C, G3 = 59</td>
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<tr>
<td>Span C, G1 = 20</td>
<td>Span C, G1 = 11</td>
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Measured Bottom Flange Strains (µε), Under Dynamic Loading:
<table>
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<th>Interior Configuration</th>
<th>Exterior Configuration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span A, G5 = 15</td>
<td>Span A, G5 = N/A</td>
</tr>
<tr>
<td>Span A, G3 = 3</td>
<td>Span A, G3 = N/A</td>
</tr>
<tr>
<td>Span A, G1 = 7</td>
<td>Span A, G1 = N/A</td>
</tr>
<tr>
<td>Span C, G5 = 126</td>
<td>Span C, G5 = N/A</td>
</tr>
<tr>
<td>Span C, G3 = 74</td>
<td>Span C, G3 = N/A</td>
</tr>
<tr>
<td>Span C, G1 = 20</td>
<td>Span C, G1 = N/A</td>
</tr>
</tbody>
</table>

Notes: 1. Calculated strains were developed using measured truck weights, composite section properties, and measured girder distribution factors.
A.5 Calculation of Theoretical Deflection due to Truck Weights

\[ I_{c,1\text{girder}} = 10626 \text{ in}^4 \]
\[ I_{c,5\text{girders}} = 53130 \text{ in}^4 \]

Maximum moment at midspan = 405 ft-kips

\[ \Delta_{\text{mid}} = \Delta_1 + \Delta_2 + \Delta_3 \]
\[ \Delta_1 = \left(\frac{16.04 \times 7.5 \times 22.5}{6EI \times 45}\right) \left(45^2 - 7.5^2 - 22.5^2\right) = \frac{14662}{EI} \]
\[ \Delta_2 = \left(\frac{16.78 \times 45^3}{48EI}\right) = \frac{31856}{EI} \]
\[ \Delta_3 = \left(\frac{16.78 \times 18.5 \times 22.5}{6EI \times 45}\right) \left(45^2 - 18.5^2 - 22.5^2\right) = \frac{30435}{EI} \]

\[ \Delta_{\text{mid}} = \frac{(14662 + 31856 + 30435)}{EI} = \frac{76953}{EI} \]

\[ \Delta_{\text{mid}} = \frac{76953}{(29000 \times 144 \times 53130/12^4)} = 0.00719 \text{ ft} = \textbf{0.0863 in.} \]
Appendix B – GFRP Tensile Test Graphs

Stress vs. Strain

$y = 6047802x - 438$

Figure B.1: GFRP Tensile Test 1
Figure B.2: GFRP Tensile Test 2

Figure B.3: GFRP Tensile Test 3
Figure B.4: GFRP Tensile Test 4

Figure B.5: GFRP Tensile Test 5
Stress vs. Strain

Figure B.6: GFRP Tensile Test 6

Figure B.7: GFRP Tensile Test 7


Vita

Matthew Harlan was born to John and Patricia Harlan on November 9, 1978 in Lewistown, PA, and was raised in Lewistown and Charleston, WV. He graduated with honors from Capital High School in May, 1997. He then attended Virginia Polytechnic Institute and State University from 1997 to 2001, where he received his Bachelor of Science degree in Civil Engineering, graduating summa cum laude. While at Virginia Tech, Matt participated in the engineering co-op program with the Eastern Federal Lands Highway Division of the Federal Highway Administration in Sterling, VA and Sevierville, TN. He continued his studies at Virginia Tech from 2002 to 2004, where he received his Master of Science degree in Civil Engineering with an emphasis in Structural Engineering. Master Matthew Harlan accepted a position in August 2003 as a structural engineer with the URS Corporation, and he currently works as a part of the bridge group in their Tampa, FL office. He is licensed as an Engineer In Training in the Commonwealth of Virginia.