

REPAIR OF IMPACT-DAMAGED PRESTRESSED BRIDGE GIRDERS WITH STRAND
SPLICES AND FABRIC REINFORCED CEMENTITIOUS MATRIX SYSTEMS

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ABSTRACT

This thesis investigates the repair of impact-damaged prestressed concrete bridge girders with strand splices and fabric-reinforced cementitious matrix systems, specifically for repair of structural damage to the underside of an overpass bridge girder due to an overheight vehicle collision. Collision damage to bridges can range from minor to catastrophic, potentially requiring repair or replacement of a bridge girder. This thesis investigates the performance of two different types of repair methods for flexural applications: strand splice repair, which is a traditional repair method that is often utilized, and fabric-reinforced cementitious matrix repair, which is a relatively new repair method. The overarching goal of this project was to provide guidance for assessment and potential repair of impact-damaged girders.

Prestressed concrete girders were tested to failure in flexure in this research. After a control test to establish a baseline for comparison, five tests were performed involving damaging a girder, repairing it using one of the repair methods, and testing it to failure. These tests showed that both strand splice repairs and fabric-reinforced cementitious matrix repairs can adequately restore the strength of an impact-damaged girder when up to 10% of the prestressing strands are severed. Combined repairs can also be a viable option if more than 10% of the prestressing strands are severed, though as the damage gets more severe, girder replacement becomes a more attractive option.

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GENERAL AUDIENCE ABSTRACT

This thesis investigates the structural repair of impact-damaged prestressed concrete bridge girders. Impact damage to these structural elements is most commonly an overheight vehicle colliding with the underside of an overpass bridge girder on a roadway. Damage to the structure can range from minor to catastrophic. Though more destructive impacts require replacement of the girder or entire bridge structure, repair of the damaged girder is possible in some cases.

This thesis studies two different methods of repair of these impact-damaged girders: strand splices and fabric-reinforced cementitious matrix. Full-scale prestressed concrete girders were tested to failure in this research. A total of five tests were performed involving damaging a girder, repairing it using one of the repair methods, and testing it to failure. These tests showed that both repair methods can adequately restore the strength of an impact-damaged girder for minor to moderate structural damage. The overarching goal of this project was to provide guidance for assessment and potential repair of impact-damaged girders.

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All photos by author, 2015.

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Abbreviations

AASHTO- American Association of State Highway and Transportation Officials

ACI- American Concrete Institute

AFRP- Aramid Fiber Reinforced Polymer

ASTM- American Society for Testing and Materials

CFRP- Carbon Fiber Reinforced Polymer

FRCM- Fabric Reinforced Cementitious Matrix

FRP- Fiber Reinforced Polymer

GFRP- Glass Fiber Reinforced Polymer

LVDT- Linear Variable Differential Transformer

MLVDT- Miniature Linear Variable Differential Transformer

NCHRP- National Cooperative Highway Research Program

NSM FRP- Near-Surface Mounted Fiber Reinforced Polymer

SRP- Steel Reinforced Polymer

PBO- Polyparaphenylene Benzobisoxazole

Tstrata IRC- Tstrata Infuseable Rapid Cure Restoration Material

VDOT- Virginia Department of Transportation

CHAPTER 1: INTRODUCTION

1.1: Background

Overheight vehicle collisions with overhead bridges occur frequently. Though data is unavailable on the exact number of overheight vehicle impacts that occur, it is estimated that 1100 of these collisions occur in the United States every year (Enchayan, 2010). Many of these collisions are simple scrapes, but significant damage to the overhead bridge can occur requiring repair or even replacement of an individual girder or entire bridge. Impact can cause property damage and injury or death to the driver or passengers of the overheight vehicle, adjacent vehicles, or people on an impacted overhead structure. For vulnerable structures such as through-truss bridges, overheight vehicle impact can cause collapse of the structure, as was the case with the Interstate 5 bridge collapse over the Skagit River in Mount Vernon, Washington in May 2013. In addition to the potential for injury and loss of life and immediate property damage, costs and damages result from repair or replacement of the damaged structure, diversion of traffic and potential delays, and economic damages to local industries. While prevention systems can reduce the number of overheight vehicle collisions, it is important to have practical, quick, and cost-effective repair schemes in place when collisions occur. Replacement of a girder is expensive and time consuming, as studies have shown that the average girder replacement costs \$8000 per ft of girder and takes one to two months to complete (Brice, 2013)

The use of prestressed concrete in bridges has significantly increased since the first use of prestressing in a bridge in 1949 (Aktan, et, al, 1994). In prestressed bridges, a girder is designed with a prestress force to offset the downward loads from the self-weight, superimposed dead loads, and live loads. Prestressing concrete reduces the tensile stress in the concrete and can prevent cracking. In prestressed concrete, high strength materials can be efficiently utilized, and

longer spans are possible. In addition, though the use of precast and segmental construction, projects can be completed on a quicker schedule. Figure 1 shows the increase in popularity of prestressed concrete bridges between 1950 and 1994. With the increase in the use of prestressed concrete in bridges, the need to develop effective repair methods for impact damaged prestressed concrete bridge girders is especially important. Current methods to repair prestressed concrete bridge girders include strand splicing, fiber reinforced polymer (FRP), and external post-tensioning. Other methods include steel jacketing, fabric reinforced cementitious matrix (FRCM), near-surface mounted (NSM) FRP, and steel reinforced polymer (SRP).

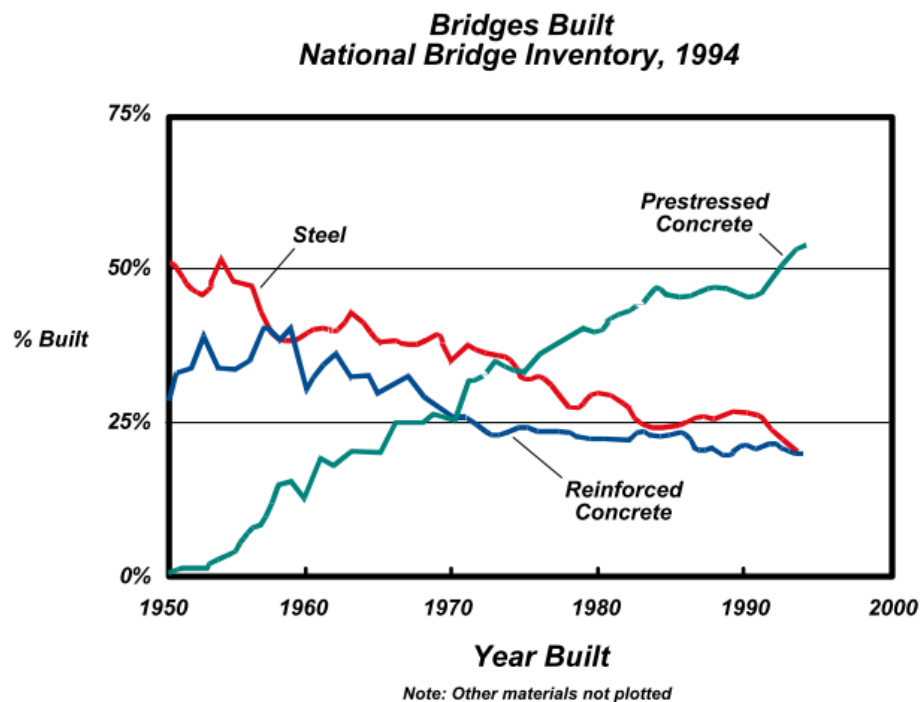


Figure 1- Percentage of Bridges Built Annually. Aktan, A. E., Culmo, M. P., Frangopol, D. M., French, C. W., Rabbat, B. G., Sanders, D. H., . . . Woods, S. W. (1994). *Concrete Bridges*. Washington D.C.: Transportation Research Board: Committee on Concrete Bridges. Retrieved May 5, 2014. Used under fair use, 2015.

This research project is intended to evaluate the performance of three repair methods for impact damaged bridges. Through an extensive review on previous applications and research of available repair techniques, the most promising repair methods were chosen for further research and testing. Four girders were provided by the Virginia Department of Transportation (VDOT) to test these repair techniques. The girders are from the Route 614 Bridge over Interstate 81 near Arcadia, Virginia. Bridge plans are provided in APPENDIX A: Route 614 Bridge Blueprints. The bridge, completed in 1960, has been replaced. The girders are AASHTO Type III prestressed girders, with a typical cross section shown in Figure 2.

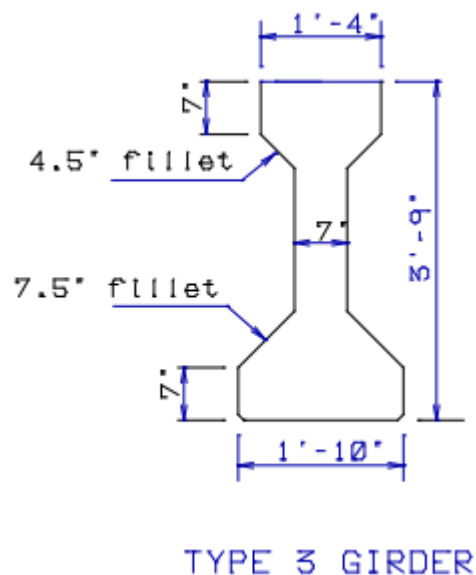


Figure 2- Typical AASHTO Type III Girder Cross Section

1.2: Purpose and Scope

This study is part of project sponsored by the Virginia Department of Transportation to focus on repair techniques for impact damaged prestressed concrete bridge girders. The investigation included repair installation and testing of the flexural strength of four AASHTO Type III girders that were intentionally damaged and repaired. Nonlinear finite element

modeling was used to aid in the evaluation of each repair method. This report discusses two of the three repair techniques that were evaluated. Three Master of Science students report on the project results: Mike Gangi, Mark Jones, and Justin Liesen. Liesen (2015) and Jones (2015) had responsibility for the installation and testing of the repaired girders and Gangi (2015) performed the finite element modeling of the girders. The investigation will include testing and analysis and will result in the development of design and installation recommendations.

1.3: Thesis Organization

CHAPTER II: Literature Review of this thesis reviews the prior literature on impact damage to bridges, evaluation of damage to prestressed concrete bridges, and repair techniques for damaged prestressed bridges. CHAPTER III: Methods discusses the methods, set-up, installation, and procedures followed for each test. CHAPTER IV: Results and Discussion discusses the results of each test and summarizes the analyses performed. CHAPTER V: Conclusions and Recommendations reviews and compares the results of the tests and proposes recommendations for evaluation, selection, and installation of repair techniques.

CHAPTER II: LITERATURE REVIEW

2.1: Impact Damage to Bridge Girders

According to the American Association of State Highway and Transportation Officials (AASHTO), the vertical clearance to structures passing over freeways should be at least 16 ft over the entire roadway width. However, structures that are more vulnerable to damage due to impacts, such as pedestrian overpasses, sign trusses, and cross bracing of through-truss structures should have a minimum clearance of 17 ft. In urban areas where meeting the 16 ft clearance for a particular structure is unreasonably costly and there is an alternate freeway facility that meets this clearance requirement, a minimum clearance of 14 ft may be used. In addition to these clearance requirements for structures passing over freeways, AASHTO recommends an extra 6 in of clearance to account for future resurfacing (American Association of State Highway and Transportation Officials, 2001).

In addition to investigating repair methods to use when vehicle collisions occur, engineers and planners have looked into preventative methods to solve this problem and reduce the number of overheight collisions. Preventative methods include permit requirements for overheight vehicles and fines for those without a permit, signs postings, overheight detection systems, and increasing clearance through grinding pavement or raising overpasses (Fu, Burhouse, & Chang, 2003). Many states have used some form of automated detection system to detect and warn operators of overheight vehicles. One such system, used in Virginia, used an infrared detection system to detect vehicles of a predetermined height. Upon detection, visual and audible alarms are activated to alert the driver, and blackout signs providing a warning message with directives for the driver are activated as well (Zarean & Register, 2008).

It is important to assess and classify the impact damage that occurs to determine how to proceed. Damage can be classified as minor, moderate, or severe (Harries, Kasan, Miller, & Brinkman, 2012). Minor damage includes shallow spalls, cracks, scrapes, and water stains, none of which affect the capacity of the girder (Harries, Kasan, & Aktas, 2009). Repairs of minor damage are not essential, though they are often done for aesthetic or preventative purposes. Moderate damage includes larger and deeper cracks and sufficient spalling to expose prestressing strands, though not severing them. Moderate damage does not affect the capacity of the girder, and repairs are done to prevent further deterioration (Harries, Kasan, & Aktas, 2009). Severe damage is further classified into three categories: Severe I, Severe II, and Severe III. Severe I damage requires structural repair to restore ultimate capacity, or the strength limit state. Serviceability is not addressed in Severe I damage and repair, so repair methods that restore some prestressing force, known as active repairs, are not necessary. Structural damage classified as Severe II damage requires structural repair to restore both the ultimate capacity and the service limit state, meaning that prestress force needs to be restored with the repair method. Severe III damage is too extensive for practical repair, and the member must be replaced (Harries, Kasan, & Aktas, 2009). A summary of damage classifications is shown in Table 1.

Table 1: Damage Classifications. Harries, K. A., Kasan, J., Miller, R., & Brinkman, R. (2012). *Updated Research for Collision Damage and Repair of Prestressed Concrete Beams*. NCHRP Project 20-07, Task 307, University of Pittsburgh, Department of Civil Engineering, Pittsburgh. Retrieved May 10, 2015. Used under fair use, 2015.

Damage Classification	SEVERE I	SEVERE II	SEVERE III
Repair philosophy	ULS only	ULS and SLS	-
Action	non PT repair	PT repair	replace
Live load capacity replacement	up to 5%	up to 30%	100%
Ultimate load capacity replacement	up to 8%	up to 15%	100%
Replace lost strands	2-3 strands	up to 8 strands	>8 strands
Vertical Deflection	loss of camber	up to 0.5%	>0.5%
Lateral Deflection (Sweep) (Shanafelt and Horn 1985)	within construction tolerance		permanent lateral deflection exceeding construction tolerance

		strand loss	camber
MINOR	Concrete with shallow spalls, nicks and cracks, scrapes and some efflorescence, rust or water stains. Damage does not affect member capacity. Repairs are for aesthetic and preventative purposes only (<i>NCHRP 280</i>).	no exposed strands	no effect of girder camber
MODERATE	Larger cracks and sufficient spalling or loss of concrete to expose strands. Damage does not affect member capacity. Repairs are intended to prevent further deterioration (<i>NCHRP 280</i>).	exposed strands no severed strands	no effect of girder camber
SEVERE I	Damage affects member capacity but may not be critical – being sufficiently minor or not located at a critical section along the span [2.5]. Repairs to prevent further deterioration are warranted although structural repair is typically not required.	less than 5% strand loss	partial loss of camber
SEVERE II	Damage requires structural repair that can be affected using a non-prestressed/post-tensioned method. This may be considered as repair to affect the STRENGTH (or ultimate) limit state.	strand loss greater than 5%	complete loss of camber
SEVERE III	Decompression of the tensile soffit has resulted [2.6.1.2]. Damage requires structural repair involving replacement of prestressing force through new prestress or post-tensioning. This may be considered as repair to affect the SERVICE limit state in addition to the STRENGTH limit state.	strand loss exceeding 20%. In longer and heavily loaded sections, decompression may not occur until close to 30% strand loss.	vertical deflection less than 0.5%
SEVERE IV	Damage is too extensive. Repair is not practical and the element must be replaced.	strand loss greater than 35%	vertical deflection greater than 0.5%

2.2: Repair Techniques

Research of different repair techniques has been performed for many years. In 1980, Shanafelt and Horn began research for The National Cooperative Highway Research Program (NCHRP) for the NCHRP 12-21 project. The first phase of the project, a document published in 1980 titled Report 226, provided guidelines for assessment, inspection, and repair of damaged prestressed concrete girders (Harries, Kasan, Miller, & Brinkman, 2012). A damage classification system was proposed to categorize damage into three categories: minor damage, moderate damage, and severe damage. In addition, 11 different repair techniques were developed and discussed in detail to repair severe damage, including strand splicing, external post-tensioning, steel jacketing, combinations of the repair methods, and girder replacement (Harries, Kasan, & Aktas, 2009). The objective of the second phase of the NCHRP 12-21 project, Report 280, was to provide a manual for the evaluation and repair of damaged prestressed concrete girders. In addition, repair methods mentioned in Report 226 were tested, and further guidelines were provided (Harries, Kasan, & Aktas, 2009).

Additional repair methods have been developed since the completion of the NCHRP 12-21 project in 1985. Fiber reinforced polymer (FRP) has emerged as a viable repair technique, both as an externally bonded sheet and as near-surface mounted (NSM) reinforcement. Fabric reinforced cementitious matrix (FRCM) and steel reinforced polymer (SRP) have recently been developed as alternatives to externally bonded FRP. In the next six sections, this report looks in depth at strand splices, fiber reinforced polymer (FRP), fabric reinforced cementitious matrix (FRCM), steel reinforced polymer (SRP), near-surface mounted (NSM) FRP, and external post-tensioning.

2.2.1: Strand Splice

Strand splice repairs can be used to repair prestressed concrete beams that have one or more damaged or severed prestressing strands. Splices reconnect broken strands and restore prestressing force. Strand splicing has been shown to provide a quick and efficient repair which can easily be combined with an externally bonded repair method such as fiber reinforced polymer (FRP) or fabric reinforced cementitious matrix (FRCM).

Commercially available strand splice systems utilize a reverse thread coupler connected to threaded anchors on each end. A prestress force in the strand is introduced by turning the coupler, which moves the anchors toward each other (Harries, Kasan, Miller, & Brinkman, 2012). These splices are re-tensioned to 60-80% of the ultimate tensile strength of the undamaged strand. It is recommended to re-tension the strands to a value close to the long-term effective prestress of undamaged strands (Harries, Kasan, Miller, & Brinkman, 2012). Research has shown that splices restore 85-96% of the original tensile strength of the strand (Zobel, Jirsa, Fowler, & Carrasquillo, 1996).

Ensuring that the strands were re-tensioned to the proper value is important. Calibrating the amount of tension restored can be based on the torque wrench method or the turn of the nut method. The torque wrench method involves simply setting a torque wrench to a specified torque value and converting that to a prestress force. Table 2 shown an example of a conversion chart to achieve 70% of the ultimate tensile strength of the strand. Though this method is an easy way to measure prestress force applied, it can prove to be an inaccurate calibration of the re-tensioning force due to friction in the coupler and anchor system (Labia, Saiidi, & Douglas 1996). The turn of the nut method uses the displacement between the splice chucks on each end

of the coupler to measure stress. This measures the elongation of the strand, which can then be converted to strain and ultimately the applied stress.

Table 2- Torque Wrench Method Conversion to Prestress Force. Prestress Supply Incorporated. (2010).

Grabb-It Cable Splice. Retrieved April 22, 2015, from Prestress Supply:

<http://www.prestresssupply.com/Products/StrandRepair/GRABBITCableSplice.aspx>. Used under fair use,

2015.

GRABB-IT [®] CABLE SPLICE					
SUGGESTED INSTALLATION TORQUE VALUES					
STEM SIZE 3/4"-16 (UNT) NOM. DIA. 3/4" (.7500")					
STRESS AREA: .3750 IN ² (EQUIV. GRADE 7)					
GRADE	CABLE			GRABB-IT [®]	
	CABLE DIA.	70% ULTIMATE	ULTIMATE	TORQUE VALUES	
		(LBS)	(LBS)	LUBRICATED	
				IN-LBS	FT-LBS
250 K	3/8"	14000	20000	1575	131
	7/16"	18900	27000	2126	177
	1/2"	25200	36000	2835	236
270 K	3/8"	16100	23000	1811	151
	7/16"	21700	31000	2441	203
	1/2"	28900	41300	3252	271

Geometry of the girder is an important consideration for strand splice repairs. A splice chuck can have a diameter of up to 1.625 in, and strands may be too closely spaced to accommodate splices on both strands. This means that staggering the splices of adjacent strands is important to ensure that all of the strand splices in a repair fit together. Each strand splice repair requires clearing concrete cover around the strand for 24 to 30 in of length so that the coupler and splice chucks can fit in place. For strand splice repairs on the bottom row of strands in a girder, concrete cover may be issue, particularly if the repair concrete is not precompressed by applying a positive moment during installation. Releasing the positive moment (after the repair material reaches an acceptable compressive strength) induces a compression force in the

repair concrete and helps prevent cracking in the concrete at service loads and as a result helps prevent moisture from reaching the strand splices.

Strand splicing has been shown to be an effective way to restore original girder strength. Research on strand splices has yielded these recommendations for the use of strand splice repairs: a) when the ultimate flexural strength of the girder with the remaining undamaged strands is greater than the factored design moment; b) when fatigue is not a major concern; and c) when repairs consist of less than 10-15% of the total number of strands in the girder (Zobel & Jirsa, 1998).

2.2.2: Fiber Reinforced Polymer (FRP)

The use of externally bonded fiber reinforced polymer (FRP) is another method traditionally used to repair or strengthen reinforced or prestressed concrete beams. The repair method consists of FRP reinforcement sheets or fabrics, externally bonded to a concrete member with saturating resin. The FRP reinforcement material is commonly made from either glass (GRFP), aramid (AFRP), or carbon fiber (CFRP), though carbon fiber is most commonly used due to its stiffness (ACI Committee 440, 2008). Externally bonded FRP systems can be used to strengthen a member in flexure, shear, or axial compression. Since initial development of FRP systems in the 1980's, use of FRP systems for repair or strengthening has increased dramatically around the world. FRP systems have been implemented on virtually all types of structural elements and used to strengthen masonry, timber, and even steel in addition to reinforced and prestressed concrete (ACI Committee 440, 2008). ACI Committee 440 has created ACI 440.2R-08, a document titled "Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures". This document provides background

information on FRP systems and provides design and installation guidelines (ACI Committee 440, 2008).

Externally bonded FRP systems are often characterized based on how they are installed and delivered to the site. There are three main categories of FRP systems based on this classification: wet layup systems, prepreg systems, and precured systems. Wet layup systems consist of dry fiber sheets impregnated with a saturating resin on site and bonded to the concrete substrate using the same resin. Wet layup systems involve both the saturation and curing process occurring on site. However, the saturation process and/or curing can be done at the FRP manufacturer's facility off site as well. Prepreg systems are saturated off site and partially cured before they are delivered to the site for installation. Prepreg systems then use resin to bond to the concrete surface, and they often require additional heating on site for curing. Precured systems are saturated and cured off site, analogous to precast concrete. Adhesives are used to bond the precured FRP to the concrete surface. Regardless of the category of externally bonded FRP used, primer is often applied to the surface of the concrete to improve bond for the adhesive or saturating resin used. In addition, putty filler is used to fill small voids in the concrete surface to provide a smooth surface for the FRP to bond to. This also prevents bubbles from forming during curing (ACI Committee 440, 2008).

Installation of externally bonded FRP is relatively simple and quick. The resins used to bond FRP reinforcement to surface concrete can cause skin irritation and are harmful to breathe, so disposable suits, gloves, dust masks or respirators, and safety glasses or goggles are recommended during mixing and placing of resins. Similar to concrete application, FRP installation should not be done on cold surfaces without an auxiliary heat source, as the FRP system manufacturer should specify a minimum temperature for installation. FRP resins should

not be applied to wet or damp surfaces, as this can disturb the bond between the FRP and the surface. Surface preparation before installation includes rounding of corners and grinding or pressure washing the concrete surface (ACI Committee 440, 2008). Wet layup systems are often installed by hand after the dry sheets have been impregnated using a resin-impregnating machine. The fiber sheets are lightweight and relatively easy to install.

Unidirectional FRP systems are commonly used in repair of reinforced or prestressed concrete beams. In direct tension tests, these FRP systems exhibit a linear elastic stress-strain relationship until failure, which is an abrupt and brittle failure. Material properties of FRP systems, including tensile strength, can be either obtained from the manufacturer or tested in accordance with ASTM D3039. Failure modes in flexure for concrete beams strengthened by FRP include debonding of the FRP from the concrete substrate, shear/tension delamination of the concrete cover, and yielding of the steel in tension followed by rupture of the FRP laminate (ACI Committee 440, 2008). Research has shown that transverse clamping using FRP U-wraps along the length of the FRP sheets can improve bond behavior and increase the FRP debonding strain (Wipf, Klaiber, Rhodes, & Kempers, 2004b). Creep-rupture of the FRP reinforcement due to cyclic stresses and fatigue can occur, and must not exceed the limits in ACI 440.2R-08.

FRP systems, specifically carbon fiber reinforced polymer (CFRP) systems, have proven to be strong, lightweight, non-corrosive, durable, and quick repair systems (Alkhrdaji, Nanni, Chen, & Barker, 1999). In addition to lower labor costs, equipment costs for the use of FRP materials are lower than the equipment costs associated with concrete or steel (Nanni, 1999). Despite the effectiveness of FRP repairs, there are some drawbacks. Both the fiber reinforcement sheets (often carbon fiber) and saturating resins are relatively expensive (Nanni, 1999). A dry substrate surface is required for FRP installation, so inclement weather can delay

installation. The saturating resins used in FRP installation can be affected by exposure to ultraviolet light over time, and they also degrade when exposed to higher temperatures. Protective coatings can be applied to reduce the exposure to ultraviolet light and provide some fire protection for FRP reinforcement, somewhat negating these effects. Due to their effectiveness in performance, time, and cost, externally bonded FRP systems are commonly used as a technique for strengthening and repair of concrete structures.

2.2.3: Fabric Reinforced Cementitious Matrix (FRCM)

Fabric reinforced cementitious matrix (FRCM) is an alternative to traditional techniques such as strand splicing and fiber reinforced polymer (FRP). Fabric reinforced cementitious matrix (FRCM) consists of a cementitious matrix and structural reinforcement dry open mesh which is often made of either carbon fiber or polyparaphenylene benzobisoxazole (PBO) fiber (also known as zylon). FRCM is also commonly referred to as textile-reinforced concrete (TRC), textile-reinforced mortar (TRM), mineral-based composites (MBC), and fiber-reinforced cement (FRC) (ACI Committee 549, 2013). The structural reinforcement used in FRCM consists of individually coated fibers in an open mesh to allow for the cementitious matrix to saturate the fibers. Similar to externally bonded FRP, FRCM reinforcement can be used to strengthen a structure in flexure, shear, or axial compression, and has been utilized for a variety of different field applications, including tunnel linings, deck slabs, bridge piers, chimneys, and masonry walls (ACI Committee 549, 2013).

Guidelines for the use of FRCM repairs, including design criteria, installation tips and techniques, and field application examples, have been addressed by ACI Committee 549, in a document titled “Guide to Design and Construction of Externally bonded Fabric-Reinforced

Cementitious Matrix (FRCM) Systems for Repair and Strengthening of Concrete and Masonry Structures” (ACI Committee 549, 2013). For design, FRCM is assumed to have bilinear behavior to failure in tension, though only the second linear portion of the stress-strain curve is used in design. An example of experimental tensile stress-strain test data is shown in Figure 3, with idealized stress-strain behavior shown in Figure 4. The initial linear portion of the idealized stress-strain curve corresponds to uncracked behavior of the FRCM specimen, with a bend-over point transitioning to the second linear portion representing cracked behavior.

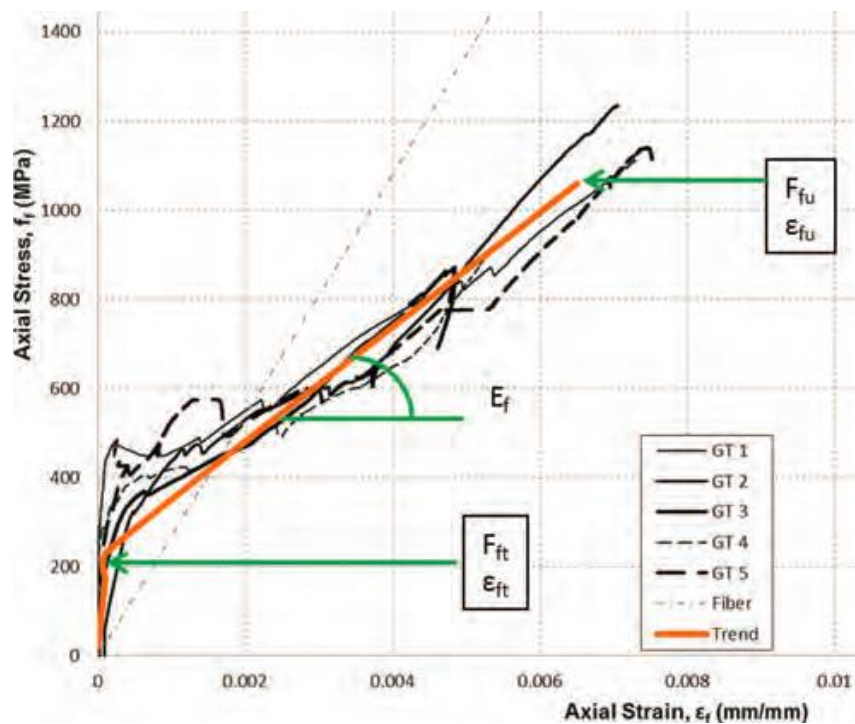


Figure 3- Experimental Tensile Stress-Strain Curve of FRCM Specimens. ACI Committee 549. (2013). *Guide to Design and Construction of Externally Bonded Fabric-Reinforced Cementitious Matrix (FRCM) Systems for Repair and Strengthening Concrete and Masonry Structures*. Farmington Hills: American Concrete Institute.

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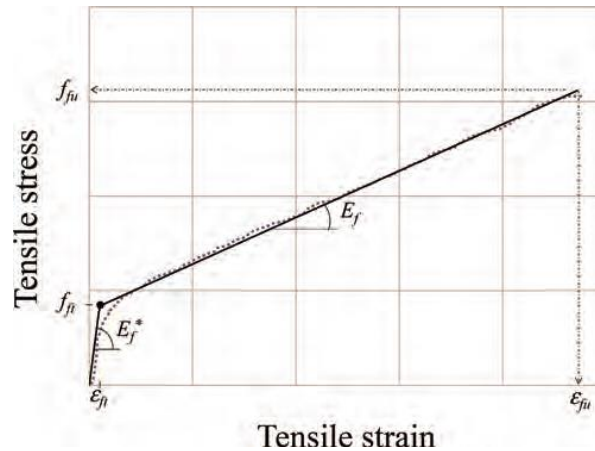


Figure 4- Idealized Tensile Stress-Strain curve of FRCM Specimens. ACI Committee 549. (2013). *Guide to Design and Construction of Externally Bonded Fabric-Reinforced Cementitious Matrix (FRCM) Systems for Repair and Strengthening Concrete and Masonry Structures*. Farmington Hills: American Concrete Institute. Used under fair use, 2015.

For design of FRCM repairs of concrete beams in flexure, failure modes in the FRCM must be considered in addition to crushing of the concrete in compression and yielding of the steel reinforcement in tension. Additional failure modes include shear/tension debonding of the concrete cover of the FRCM, debonding at the interface between the FRCM and the concrete substrate, interlaminar debonding, and slippage of the fiber mesh within cementitious matrix (ACI Committee 549, 2013). The effective tensile strain in the FRCM at failure is limited to 0.012 (or the design tensile strain listed by the manufacturer, if it is a lower value), in part to conservatively account for the bond-related failure modes. In addition to calculating the moment capacity in accordance with ACI 318, further limit states need to be satisfied, including serviceability constraints, design limitations, and creep-rupture and fatigue of the FRCM reinforcement (ACI Committee 549, 2013).

The use of FRCM repairs has some advantages over more traditional FRP repairs. Both repair types have similarly low labor and equipment costs, but the reinforcement and resin

materials used in externally bonded FRP systems are relatively costly. The use of a cementitious matrix in FRCM systems has many advantages versus using an epoxy resin, as used in FRP systems. Firstly, having similar mechanical, chemical, and physical properties as the concrete substrate can be advantageous for a repair material. Installation of the cementitious matrix involves simple troweling techniques, and no personal protective equipment (PPE) beyond what is worn for typical concrete application is needed. FRCM installation, unlike FRP, can be done on a wet surface and at a wide variety of temperatures (ACI Committee 549, 2013).

Cementitious materials perform well at higher temperatures and provide some level of fire resistance, whereas epoxy resins can fuel a fire and potentially release toxic fumes, and may require additional fire protection (De Caso y Basalo, Matta, & Nanni, 2009). The cementitious structure of FRCM also has a more porous structure than epoxy resin, increasing the vapor permeability, which can help decrease further degradation of the original structure. In addition, unlike FRP repairs, FRCM repairs can be undone without harming the original structure (ACI Committee 549, 2013). These numerous advantages of externally bonded FRCM repairs over traditional FRP repairs show why FRCM has emerged as a viable repair and strengthening method for reinforced or prestressed concrete structures.

2.2.4: Steel Reinforced Polymer

Steel reinforced polymer (SRP) systems are another alternative to FRP repairs. SRP systems contain interwoven ultra-high strength twisted steel wires assembled in a tape form. The tapes are made of high carbon steel cord with a micro-fine brass coating to optimize adhesion (Hardwire LLC, 2015). These tapes can be molded into either cementitious grout or polymeric resin, with the cementitious grout providing advantages such as fire resistance. Similar to FRP

fabrics in terms of ease of installation, SRP can be applied using by manual lay-up, and unlike FRP fabrics, rounding of corners is not required for SRP because the steel cords can be bent up to 90 degrees mechanically with no damage (Lopez, Galati, Alkhrdaji, & Nanni, 2007).

The high carbon steel cords used in SRP are made from the same process as reinforcement for automobile tires (Huang, Birman, Nanni, & Tunis, 2004). The steel cords are made by twisting five individual wires together, with three straight wires wrapped by two wires at a high twist angle to provide additional surface roughness and ultimately increase mechanical interlock. Cords can be laid out in the SRP tape at either low density (four cords per in), medium density (12 cords per in), or high density (18-20 cords per in) to optimize the material in specific instances (Hardwire LLC, 2015). Since cementitious mixtures have higher viscosity, low density tapes are ideal when grout is used as the resin, and high density tape should only be used for lower viscosity resins.

Research has shown that SRP tapes are an effective repair method to increase flexural capacity of reinforced or prestressed concrete beams. Debonding between the SRP and the concrete can occur, though U-wraps of SRP have been shown to prevent full detachment (Casadei, Nanni, Alkhrdaji, & Thomas). SRP tapes are been shown to be more effective than CFRP sheets at increasing both ultimate strength and ductility (El-Hacha 2006), though there are some differences and drawbacks when compared to CFRP. Though both SRP and CFRP are relatively easy to install, SRP rolls are heavier than CFRP rolls, and in overhead applications, the resin (whether cementitious or epoxy-based) has to hold the weight of the steel tape during application and while curing. This can control the selection of the resin, and the weight of the SRP can make it more difficult to install. Externally bonded SRP can also be susceptible to corrosion (El-Hacha 2006). Steel reinforced polymer is an effective material for repair and

strengthening of reinforced or prestressed concrete beams, but due to its difficult installation and vulnerability to corrosion, it may not be the most attractive repair method.

2.2.5: Near-Surface Mounted (NSM) FRP

Near-surface mounted (NSM) systems are a subset of FRP repairs. They consist of bars or plates installed and bonded into grooves cut into the tensile region of a concrete surface. NSM repair systems have been introduced to minimize some of the problems that have been reported with conventional externally bonded FRP systems, such as brittle debonding failures and susceptibility to damage from environmental conditions such as fire and temperature (El-Hacha & Rizkalla, 2004). NSM systems are a relatively recent development and a potentially promising technique for repair and strengthening of reinforced concrete.

FRP is preferred for NSM applications over traditional steel reinforcement due to its resistance to corrosion, an important consideration due to the close proximity to the concrete surface (El-Hacha & Rizkalla, 2004). The FRP reinforcement can either be reinforcing bars or strips. They are placed into the precut grooves in the concrete and can be bonded to the concrete using either an epoxy resin or cementitious grout. With NSM FRP bars, efficiency is controlled by bond characteristics at the interface between the bar and adhesive as well as the interface between the adhesive and the concrete, with tension rupture of the bars unlikely (Hassan & Rizkalla). While bars can be either sandblasted or deformed, deformed bars have been shown to be more effective in terms of bond performance (De Lorenzis & Nanni, 2002). Research has shown that strips are more effective due to the increased surface area between the FRP-adhesive interface, with strips failing in tension rupture and achieving full composite action with the concrete (El-Hacha & Rizkalla, 2004).

Near-surface mounted FRP is an effective method for both strengthening and repairing concrete. In cases of a repair due to an overheight vehicle impact, however, issues may arise with the use of NSM FRP. NSM repairs cannot easily be combined with Strand Splice repairs, since the thickness of the turnbuckle and splice chucks would conflict with cutting grooves into particular locations. The optimum groove depth varies with the FRP bar or sheet selection, but groove depths can be up to 1 in (De Lorenzis & Nanni, 2002). For repairs in the field, performing the required saw cuts overhead would be a difficult and time-consuming process for workers. NSM is an effective repair method and an efficient use of material, though increased labor costs may offset the decreased material costs.

2.2.6: External Post-Tensioning

External post-tensioning is a traditional repair method done by externally attaching steel strands or bars to the girder. These steel strands or bars are anchored by brackets, typically referred to as bolsters, which can be cast or mounted onto the side of the girder (Harries, Kasan, & Aktas, 2009). These steel strands or bars then are tensioned by jacking to restore some prestressing force to the girder. Research has shown that external post-tensioning repairs can restore ultimate strength back to the original capacity and restore some of the lost prestressing force as well (Preston et. al., 1987).

Though design of these repair systems is relatively simple, the anchorage point between the bolsters and the concrete requires extensive analysis. This interface can be subjected to significant shear as the force from the external steel is transferred to the concrete girder. To adequately resist this shear transfer, bolsters themselves often need to be post-tensioned (Harries,

Kasan, & Aktas, 2009). In addition to designing for this shear transfer at the interface, bolsters are subject to large concentrated compression forces and have induced moments due to the eccentric post-tensioning forces (Harries, Kasan, & Aktas, 2009).

Advantages to external post-tensioning include restoring prestress as well as ultimate strength. These external steel strands or bars can be straight along the girder length or harped to optimize restoring both ultimate strength and serviceability. External post-tensioning repairs have some drawbacks though. Significant analysis is required for design of the bolsters and the anchorage into the girder concrete. Bolsters may need to be post-tensioned to adequately resist the shear between the post-tensioned steel and the concrete substrate. Corrosion of the steel strands or bars and the bolsters is a concern that may need to be addressed during installation. Though steel bars or strands are traditionally used as the post-tensioned element, CFRP cables can also be used. Research on externally post-tensioned CFRP has shown comparable results to steel post-tensioned repair (El-Hacha & Elbadry, 2006). External post-tensioning has been shown to be an effective repair method for restoring both serviceability and ultimate strength of a concrete girder.

2.3: Repair vs. Replacement of Damaged Girder

In evaluating severe impact damage to a girder, repair of the girder may not be practical, with girder replacement being the more attractive option. Replacing a girder will cost much more and take longer to construct than repair of a damaged girder, but if the damaged girder cannot be restored to its original capacity and carry its share of the load, the girder should be replaced. The Washington State Department of Transportation (WSDOT) created a set of guidelines to describe damaged girder conditions which would require replacement, defining the

threshold between the SEVERE II and SEVERE III damage conditions (Brice, 2013). These damage cases are:

1. Over 25% of the strands have been severed
2. The bottom flange is displaced from the horizontal position more than $\frac{1}{2}$ in per 10 ft of girder length
3. Concrete damage at harping point resulting in permanent loss of prestress.
4. Abrupt lateral offsets may indicate that stirrups have yielded.
5. Severe concrete damage at girder ends resulting in permanent loss of prestress.

Other considerations are given by WSDOT that require further consideration and analysis. Significant concrete loss in the bottom flange may require verifying the level of stress remaining in exposed prestressing strands. Due to loss of stiffness, dead load from the damaged girder may be shed to adjacent girders. If this occurs, the adjacent girders must be able to accommodate this additional load. Damage to a girder that has been previously damaged and repaired is another situation that requires further consideration, as a previously repaired girder may not be able to be restored to sufficient capacity (Brice, 2013). Impact damage that meets the criteria given for girder replacement may still be repaired with extensive and costly methods such as section enlargement and external post-tensioning, but flexural capacity cannot easily be restored and girder replacement becomes a more appropriate solution (Harries, Kasan, & Aktas, 2009).

Cost of the repair versus replacement is an important factor as well. Even if impact damage to a girder does not meet any of the criteria for replacement, comparing the cost of a repair versus the cost of girder replacement is an important consideration. The Washington State DOT recommendations state that girder replacement may be warranted if the cost of a repair

reaches 70% of the replacement project cost (Brice, 2013). Deciding whether to repair or replace an impact damaged girder is an important consideration, and even with the guidelines and recommendations in place, there may be no definitive right or wrong answer.

2.4: Summary

Based on the literature review, three repair methods have been selected for evaluation: strand splicing, externally bonded fiber reinforced polymer (FRP), and fabric reinforced cementitious matrix (FRCM). Strand splices were chosen because they are traditionally used in repairs, can be combined with external repair methods, restore a portion of the prestressing force, and are simple and inexpensive to install. Externally bonded FRP was chosen because it is traditionally used in repairs, provides additional strength, and is relatively simple to install. FRCM was chosen because it is a relatively new repair method and has benefits over externally bonded FRP, such as fire protection and lower material cost. Table 3 summarizes the breakdown of responsibilities for investigating and reporting within this project.

Table 3: Project Scope Responsibilities and Summary

Girder Number	Tested Repair	Number of Severed Strands
1	Control Test	0
2	8-Strand Spliced	8
3	4-Strands Severed with FRP	4
	4-Strand Spliced	4
4	4-Strands Severed with FRCM	4
	4-Strand Spliced with FRCM	4

CHAPTER III: Experimental Testing Methods

This research project, intended to evaluate the performance of repair methods for impact damaged bridges, consisted of six flexural tests were conducted on four AASHTO Type III girders. These girders were from the Route 614 Bridge over Interstate 81 near Arcadia, Virginia. The bridge, which was completed in 1960, was replaced, and these four girders were transported to the Thomas M. Murray Structural Engineering Laboratory at Virginia Tech. A naming system was created for this report to identify each girder, with the four girders named Girder A, Girder B, Girder C, and Girder D. Each girder was 60 ft long while in service on the bridge. During transit from the demolished bridge to the laboratory, a section of Girder A fractured and separated from the rest of the girder, leaving a 44 ft section and a 16 ft section for Girder A.

Each girder cross section contains 50 prestressed strands, with two straight strands in the top flange, 40 straight strands in the bottom flange, and eight harped strands. Some strands are harped to decrease the eccentricity of the prestressing force at the end of the span, as this can cause cracking in the concrete. The harping points for the harped strands are located at 24 ft and 36 ft along the length of the girder. A cross section of the girder between the harping points is shown in Figure 5, and a cross section near the end of the girder is shown in Figure 6. Each prestressing strand has an area of 0.0799 in^2 , with a nominal diameter of 0.375 in. Seven-wire stress-relieved strand was used in the girder, with a minimum ultimate tensile strength of 250 ksi.

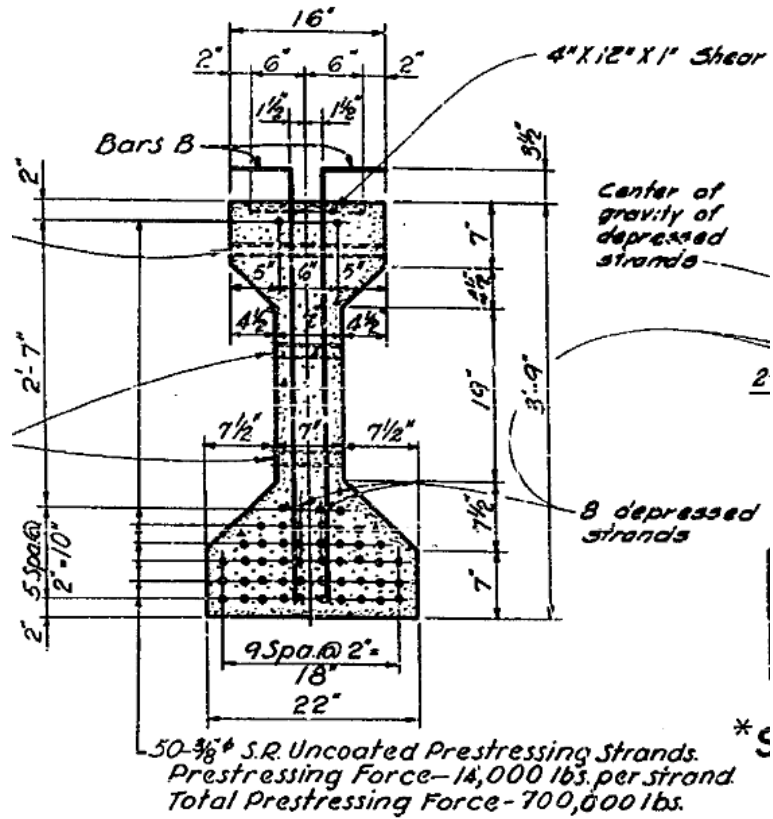


Figure 5- Cross Section of a Girder Between Harping Points

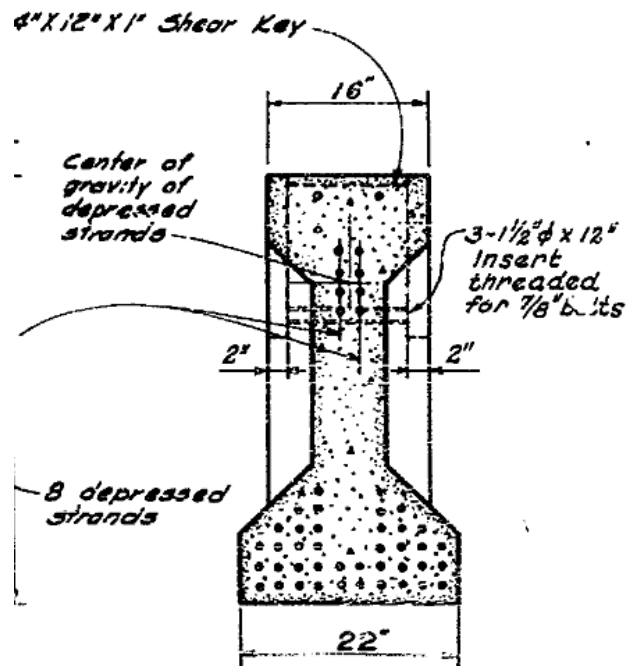


Figure 6- Cross Section Near the End of the Girder

In design and construction of the bridge that the girders were removed from, a composite concrete deck was placed on the girders. This composite deck spanned between the girders, which were placed 7 ft - 4 in apart from center to center. The girders were removed from the bridge by saw cutting through the deck, removing most of the width of the composite deck. Deck width varied for each girder, and though relatively constant, width varied along the length of each girder. A cross section of a girder with deck concrete is shown in Figure 7, and the deck dimensions and self weight of each girder are shown in Table 4.

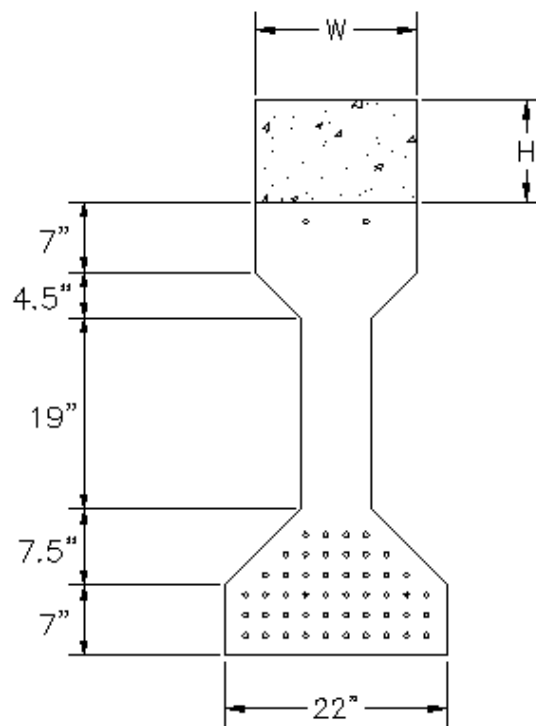


Figure 7- Cross Section with Existing Deck

Table 4- Deck Dimensions and Self Weight of Each Girder

Name	Deck Height, H (in)	Deck Width, W (in)	Self Weight (k/ft)
Girder A	10	24	1.00
Girder B	9.5	16	0.82
Girder C	9.5	16	0.78
Girder D	9.5	15	0.78

According to the blueprints shown in APPENDIX A: Route 614 Bridge Blueprints, the minimum 28 day compressive strength of the girder concrete was 5000 psi and the minimum 28 day compressive strength of the composite deck was 4000 psi. Shear reinforcement was placed along the length of each girder to provide additional resistance to vertical shear and to provide for horizontal shear transfer between the deck and the girder. This reinforcement consisted of two No. 5 single leg stirrups, with spacing along each girder shown in Figure 8, along with the center of gravity of the harped prestressing strands.

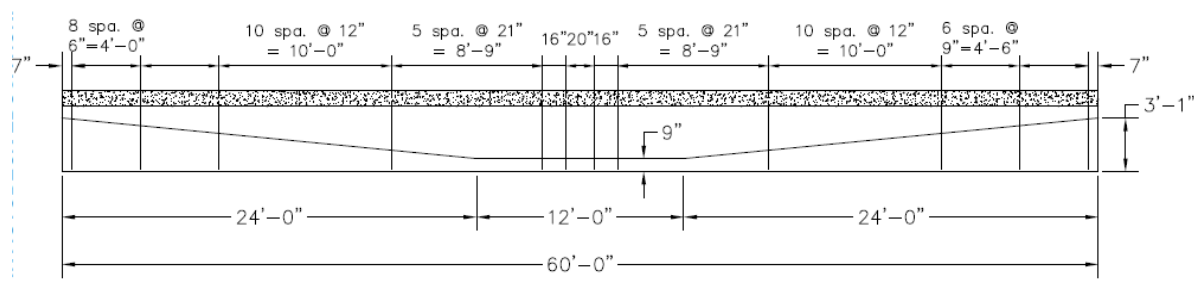


Figure 8- Stirrup Spacing and Harped Strand Layout

The goal of each test on the girders was to achieve a flexural failure. Since impact damage due to overheight vehicles typically occurs around the mid-span of the girder, where flexural strength dominates behavior and shear demand is typically low, flexural tests were conducted instead of shear tests. The first test was a flexural test of a 44 ft unrepaired section of Girder A to analyze the undamaged behavior and performance and establish a reference point for comparison with ensuing tests. The second test was a flexural test on a 60 ft girder, Girder B, which was damaged at the mid-span with eight severed strands and repaired using the strand splice method. The third and fourth tests were flexural tests on a 60 ft girder, Girder C, which was damaged in two locations, at approximately the one-third and two-thirds points along the length of the girder, each with four severed strands. The third test was repaired using the strand splice method and the fourth test was repaired using fiber reinforced polymer (FRP). The fifth

and sixth tests were flexural tests on a 60 ft girder, Girder D, which was damaged and repaired at the one-third and two-thirds points, similar to Girder C. The fifth test was repaired using fabric reinforced cementitious matrix (FRCM) and the sixth test was repaired using a combination of FRCM and the strand splice method. Table 5 outlines the testing schedule for the project, identifies the Master's Candidate responsible for investigating, analyzing, and reporting on each test, and summarizes the test set-up. Test 1, Test 2, Test 4, Test 5, and Test 6 will be discussed in this report. Test 4 is discussed by Justin Liesen (Liesen, 2015). Figure 9, Figure 10, Figure 11, Figure 12, Figure 13, and Figure 14 show the layout for each test.

Table 5: Testing Schedule and Summary

Girder	Test	Description	Investigator	Number of Severed Strands	Span Length (ft)	Distance to Load Point 1 (ft)	Distance to Load Point 2 (ft)
A	1	Control Test (44 ft long)	Mark Jones, Justin Liesen	0	35	12.5	16.5
B	2	8-Strand Spliced	Mark Jones, Justin Liesen	8	58	25	33
C	3, Iteration 1	4-Strands Severed with FRP	Justin Liesen	4	48	17	21
	3, Iteration 2				53	17	21
	4, Iteration 1	4-Strand Spliced	Mark Jones, Justin Liesen	4	58	37	41
	4, Iteration 2				50	29	33
D	5	4-Strands Severed with FRCM	Mark Jones	4	52	31	35
	6	4-Strand Spliced with FRCM	Mark Jones	4	52	17	21

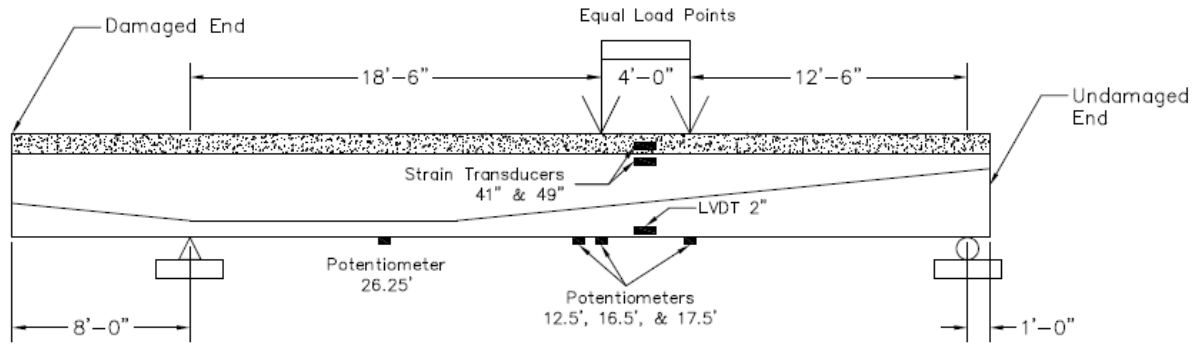


Figure 9- Test 1 Set-up

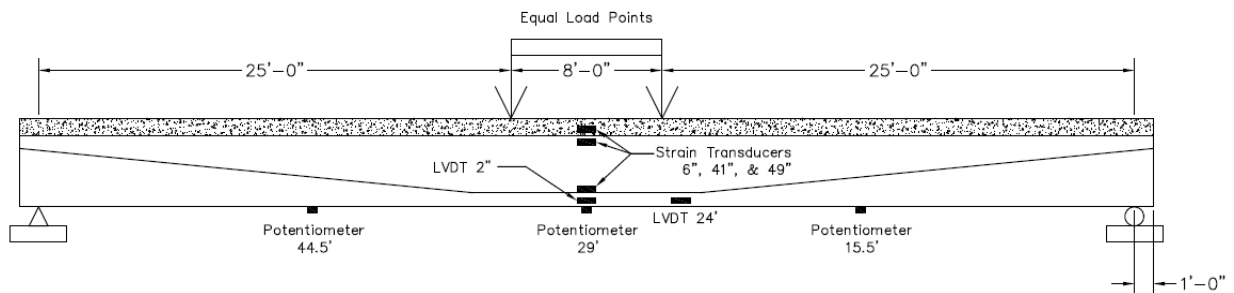


Figure 10- Test 2 Set-up

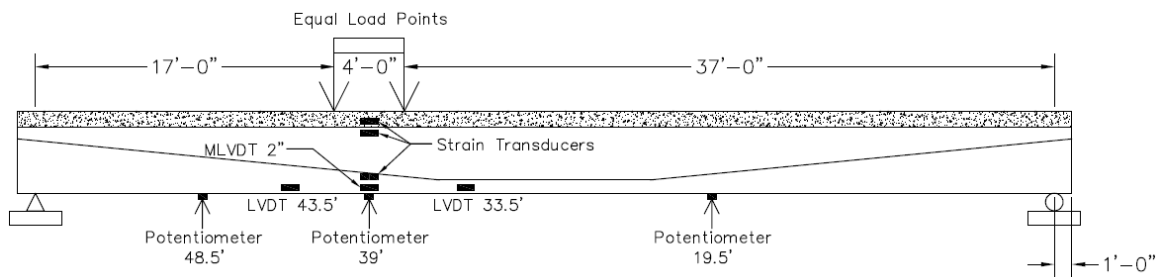


Figure 11- Test 4, Iteration 1 Set-up

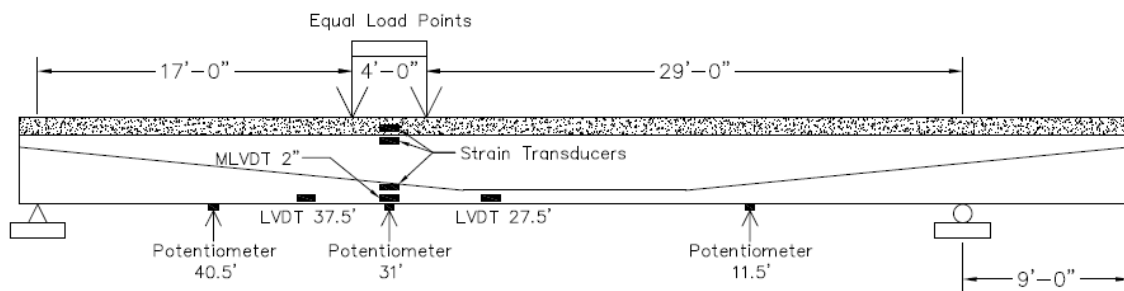


Figure 12- Test 4, Iteration 2 Set-up

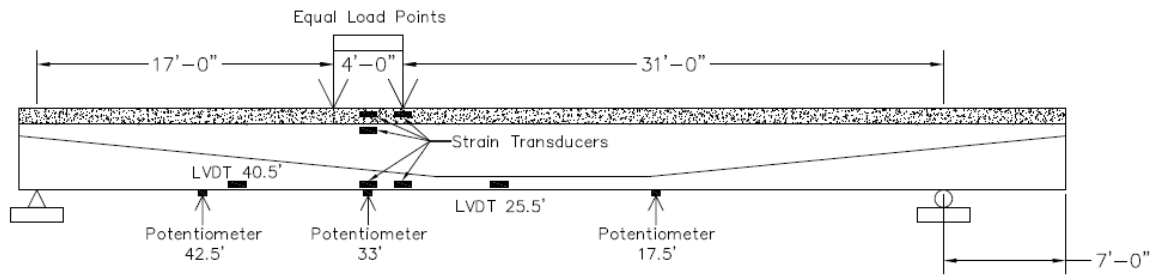


Figure 13- Test 5 Set-up

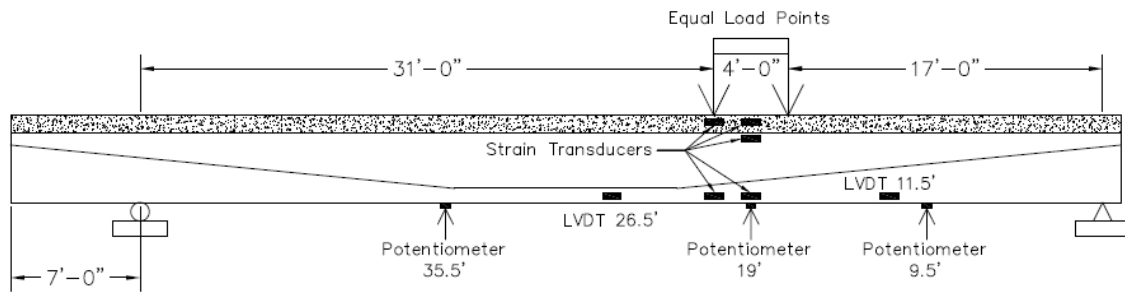


Figure 14- Test 6 Set-up

Girders B, C, and D were intentionally damaged to simulate overheight vehicle impact damage. The girders were damaged using a hydraulic hammer attached to a backhoe, known as a hoe ram. Repairs were designed and installed by Structural Technologies, a member of Structural Group specializing in repair and strengthening of structures (Structural Group Incorporated, 2015). Flexural tests on the girders were conducted after the repairs were completed, and observations and results were compared to the test of the undamaged girder, Girder A.

3.1: Materials Testing

To accurately model and assess behavior, materials tests were conducted on the girder concrete, the deck concrete, the prestressing strands, the shear reinforcing bars, and the repair

materials. Material tests were each performed in accordance with the proper standard set by the American Society for Testing and Materials (ASTM). Material samples obtained from the girders were excavated from the 16 ft section of Girder A that fractured from the rest of the girder during transit to the laboratory.

Core samples of the girder concrete and the deck concrete were taken from the 16 ft section of Girder A using a concrete core drill. Cylinder samples obtained were 2.75 in in diameter and approximately 6 in long. Two cylinders from the girder concrete and two cylinders from the deck concrete were obtained and tested for compressive strength in accordance with ASTM C42 (ASTM International, 2012). Two additional core sample cylinders were taken from the girder and tested for splitting tensile strength in accordance with ASTM C96. Both the compressive strength and splitting tensile strength tests were performed using a Forney compressive machine, with a precision to the nearest 500 lbs.

Two samples of reinforcing steel were taken from the 16 ft portion of Girder A and tested in tension to determine yield strength, ultimate strength, and develop a stress-strain curve. The samples were thought to be Grade 40 reinforcement. These samples were No. 4 reinforcing bars that were each approximately 3 ft long. The load rate for tensile testing was 0.05 in per minute for the first 4 minutes and increased to 0.5 in per minute until the bars each ruptured after approximately 8 minutes.

Prestressing strands, obtained from the 16 ft section of Girder A, were tested in tension to develop an accurate stress-strain curve and find the ultimate tensile strength. According to the bridge plans found in APPENDIX A, Route 614 Bridge Blueprints, the prestressing strands used were stress-relieved strands with a minimum ultimate tensile strength of 250 ksi. Strand samples were taken from the unused portion of Girder A and prepared for testing using the procedures

developed by Salomon (2014) and Loflin (2008). To reduce the potential for a premature failure in the grips of the machine, the ends of the strand were placed in copper tubing. The tubing was cut into sections approximately 10 in long, split in half along the length, and the inside face of the tubing was sandblasted to roughen the surface. To attach the strand sample to the copper tubing, an epoxy mix was made from West System 105A epoxy resin, West System 205 epoxy hardener, and 120 Aluminum Oxide Grit with a ratio of five fluid ounces of grit to five squirts each of the resin hardener from the pump of the bottle provided by the manufacturer. The strands were tested in the SATEC universal machine. Hydraulic V-grips were used to grip to the copper tubing, and an internal extensometer, an internal load cell, an external wire pot, and an external extensometer were used to measure the load and the displacement of the strand. The ends of the V-grips, the copper tubing, and the external extensometer are shown in Figure 15.



Figure 15- Tension Testing of Prestressing Strands

Tests were conducted to determine the effective prestress of the strands in Girder D. Six strands needed to be severed as part of the damage process, four in one damage location of

Girder D and two in the other damage location along the girder (two strands had been previously cut). The strands were cut with an electric grinder with a steel cutting blade, shown in Figure 16, and an extensometer was used to measure the change in length of the strand when cut, shown in Figure 17. Wire ties were utilized to reduce vibration and help to keep the strand in place as it was being cut, and duct tape was placed near the location of the saw cut to prevent unraveling of the strand.



Figure 16- Strand Cutting Using an Electric Grinder



Figure 17- Test Setup to Determine Effective Prestress During Strand Cutting

To patch the damaged concrete during repairs, Tstrata Infuseable Rapid Cure Restoration Material (Tstrata IRC) was used. Tstrata IRC is a premixed, dry packaged, rapid cure, self-consolidating concrete mixture used for repair (Structural Group Incorporated, 2015). During each repair of damaged sections on July 16, 2014 and March 17, 2015, six 4 in diameter by 8 in tall cylinder samples were taken from the Tstrata IRC repair concrete for compressive strength testing according to the ASTM 39 compression test. For each repair date, two cylinders were tested for their 24 hour compressive strength, two cylinders were tested for their 48 hour compressive strength, and two cylinders were tested for their 7 day compressive strength.

Three tension tests of the Grabb-it strand splices were performed (Bao, 2015). Grade 270 prestressing strands were anchored between two steel abutments with a Grabb-it strand splice between them. The abutments were placed 16 ft apart for the first test, 6 ft apart for the second test, and 4 ft apart for the third test. The end of the strand was attached to a load cell to measure the stress in the strand. The splice was tightened using a calibrated torque wrench and readings

were taken with the load cell at specific torque values to compare with the manufacturer provided values in Table 2. After completing the torque test, the strands were loaded to failure.

3.2: Test Procedures

3.2.1: Instrumentation

Different types of instruments were used to monitor and record behavior of each girder during testing. The following instrumentation types were used, potentiometers, linear variable differential transducers (LVDT), miniature linear variable differential transducers (MLVDT), strain transducers, and load cells. Potentiometers were used to measure the vertical deflection of the girder during loading as well as monitoring lateral movement of the girder. The LVDT's, MLVDT's, and strain transducers were placed on the side of the girder along its height to measure surface strain of the girder during applied load. A load cell was used during testing to monitor the magnitude of the applied load. All instruments were zeroed prior to recording data at the start of each test. During testing, data was continuously recorded using a high speed data acquisition system. In some instances during testing, certain instruments were removed when girders were nearing failure to prevent damage to the instruments. This was only done when it was considered safe to approach the girder.

Potentiometers were used to measure vertical deflection of the girder and monitor lateral displacement. Potentiometers were calibrated before each test to a precision of 0.001 in. Calibration was done with a height gage. Potentiometers used to measure vertical deflection were placed below the bottom flange of the girder and tied with string to a magnet glued to the bottom face of the girder, shown in Figure 18. These instruments were placed at various points

along the length of the girder. Potentiometers used to monitor lateral displacement were attached to the side of the girder and tied to a magnet on a column of the load frame, shown in Figure 19.



Figure 18- Potentiometer Used to Measure Downward Deflection



Figure 19- Potentiometer to find Lateral Displacement; MLVDT and Strain Transducer to Obtain Strain Profile

Linear variable differential transformers (LVDT's) and miniature linear variable differential transformers (MLVDT's) were used to measure surface strain of the concrete. They were calibrated before each test to a precision of 0.001 in. For LVDT's and MLVDT's,

calibration was done using a height gage. These instruments were placed on the side of the girder along its height to measure strain at points along the depth and develop a strain profile at a certain point along the length of the girder. They were also used to measure slip at the interface between repair material and the concrete substrate. Figure 20 shows an LVDT in place on the side of a girder to measure slip at the interface between the repair material and the original concrete.



Figure 20- LVDT in place on the side of a girder

Strain transducers were used to measure surface strain of the concrete. Calibration data of each transducer was provided by the manufacturer and input into the system. Calibration data was provided to a precision of the nearest microstrain ($\mu\epsilon$). These instruments were placed on the side of the girder along its height to measure strain at points along the depth and, in conjunction with LVDT's and MLVDT's, develop a strain profile at a certain point along the length of the girder, shown in Figure 21.



Figure 21- LVDT and Three Strain Transducers to Obtain Strain Profile

A load cell was used to measure the applied load to the girder during testing. It was calibrated before each use using a Forney compression machine, with precision to the nearest 500 lbs. It was placed in the load frame, measuring compression applied by the actuator to the girder, shown in Figure 22. For ultimate tensile strength testing of the prestressing strand, an internal load cell in the SATEC Universal Machine measured applied load.



Figure 22- Test Set up with Actuator, Load Cell, Spreader Beam, and Rubber Pads

3.2.2: General Test Procedures

Each test was conducted using a simple span, supported by a pin bearing at one support and a roller bearing at the other support, each resting on W21x101 steel sections. Figure 23 and Figure 24 show the set-up of the pin and roller bearings. To prevent the girder from tipping over during testing, two bracing frames, each with a horizontal steel member placed with approximately 2 in of clear space from the girder, were put in place along the length of the girder, shown in Figure 25. For each test, a 400-kip actuator was used for the test to load the girder to failure. A spreader beam and rubber pads were used to simulate two equal point loads, creating a constant moment region where a flexural failure would be expected and the shear demand would be low.



Figure 23- Pin Bearing



Figure 24- Roller Bearing



Figure 25- Bracing Frame to Prevent Tipping

3.2.3: Test 1- Control Test

Girder A was tested for the undamaged control test to provide a baseline for comparison with subsequent tests. Girder A was approximately 44 ft in length. It was originally 60 ft, but suffered collision damage during the bridge's service life and during transit from the site to the laboratory, and a 16 ft section broke off. The bottom flange of the girder was spalled at the damaged end for about 7 ft of length from that damaged end. To ensure that the control test would represent an undamaged girder, the section of girder from the undamaged end to 37 ft from that end was considered to be undamaged. The composite deck was cut during demolition

of the bridge, but some existing deck still remained intact and attached to the girder. The existing deck on Girder A was roughly cut, with approximate dimensions 9.5 in deep and 24 in wide. A view of the existing deck can be seen in Figure 26. Despite some damage to the concrete deck, it was assumed to act as a fully composite deck.



Figure 26- 44 ft Section of Girder A

Girder A had been previously damaged by vehicle impact and repaired by VDOT. As the exterior girder closest to the direction of traffic, it was the only girder used in testing that had been previously damaged by impact. At 31 ft from the undamaged end, a spall was observed on the bottom flange with an exposed strand splice chuck, shown in Figure 27. A section of an intermediate diaphragm was still partially intact at about 20 ft from the undamaged end, as shown in Figure 28.



Figure 27- Spall with Exposed Strand Splice Chuck



Figure 28- Existing Section of Intermediate Diaphragm

A 35 ft span was used, with an overhang of one ft on the undamaged end and eight ft on the damaged end. The rubber bearing pads were spaced 4 ft apart, creating a constant moment region. To reduce the likelihood of a shear failure, Girder A was loaded off-center towards the undamaged end. Load points were applied at 12.5 ft and 16.5 ft from the support near the undamaged end, with the center of load 14.5 ft from that support. Despite the previous impact damage to the girder near the damaged end, it was determined that, since no damage was

observed in the section of girder that would have a high flexural demand, flexural testing on Girder A could be considered an appropriate control test.

Instruments were put in place to measure and record deflection, applied load, and strain at different points along the girder. A load cell was placed between the actuator and the cross-head of the load frame to measure load applied by the actuator. Potentiometers were placed at the two load points (12.5 ft and 16.5 ft from the support near the undamaged end), midspan (17.5 ft from either support), and the quarter point near the support near the damaged end (26.25 ft from the support near the undamaged end). LVDT's and strain transducers were used to attain a strain profile at the center of load. LVDT's were placed on the bottom flange on both sides, approximately 2 in up from the bottom of the girder, corresponding to the location of the bottom row of prestressed strands. Strain transducers were placed at 41 and 49 in from the bottom on one side of the girder. The full instrumentation diagram is shown in Figure 29, with a cross section at the center of load shown in Figure 30.

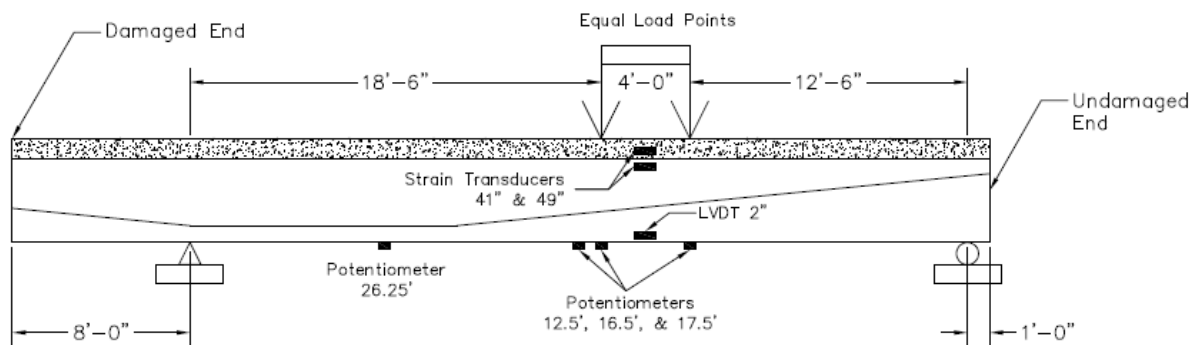


Figure 29- Control Test Instrumentation Diagram

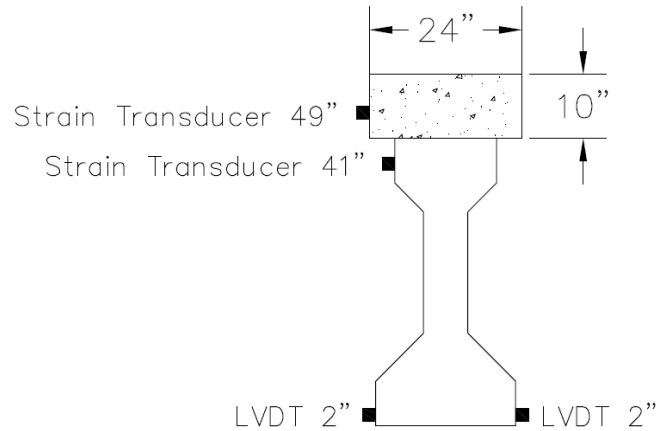


Figure 30- Control Test Instrumentation at Center of Load

The nominal moment and shear capacities of Girder A were determined using the provisions of AASHTO LRFD Bridge Design Specifications (AASHTO, 2012). The dimensions of the composite deck were approximately 10 in deep and 24 in wide, though for calculating the moment and shear capacities, the depth was assumed to be 9.5 in since the top of the deck had an uneven finish. The minimum compressive strength values specified in the plans, 5000 psi for the girder concrete and 4000 psi for the deck concrete, were used in this analysis instead of the compressive strength values obtained from taking core samples of the concrete. For the loading in this test, the critical point where failure would be expected was the load point located 16.5 ft from the support near the undamaged end. The nominal moment capacity at this point was 2952 k-ft, corresponding to an applied load of 366 kips after accounting for the distributed self weight of 1.0 kip/ft. The moment capacity versus the demand from an applied load of 366 kips is shown in Figure 31. Figure 32 shows the shear capacity versus demand for the 366-kip applied load. The shear capacity is close to the demand with that load, but does not exceed it. The jumps in the shear capacity along the girder are due to the variable spacing of shear stirrups, shown in Figure 8. Though other loading arrangements were considered, moving the load points closer to the midspan increased the shear demand in locations near the girder's intended midspan, where

shear stirrups were not closely spaced. In addition, moving the load points closer to the support increased the applied load required to reach flexural failure.

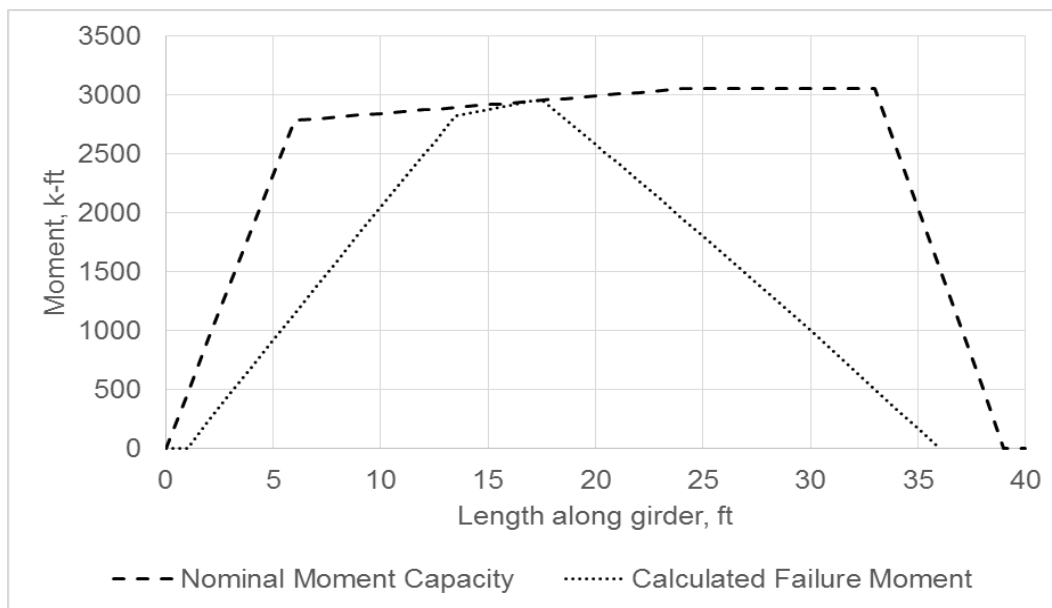


Figure 31- Nominal Moment Capacity versus Calculated Failure Moment for the Control Test

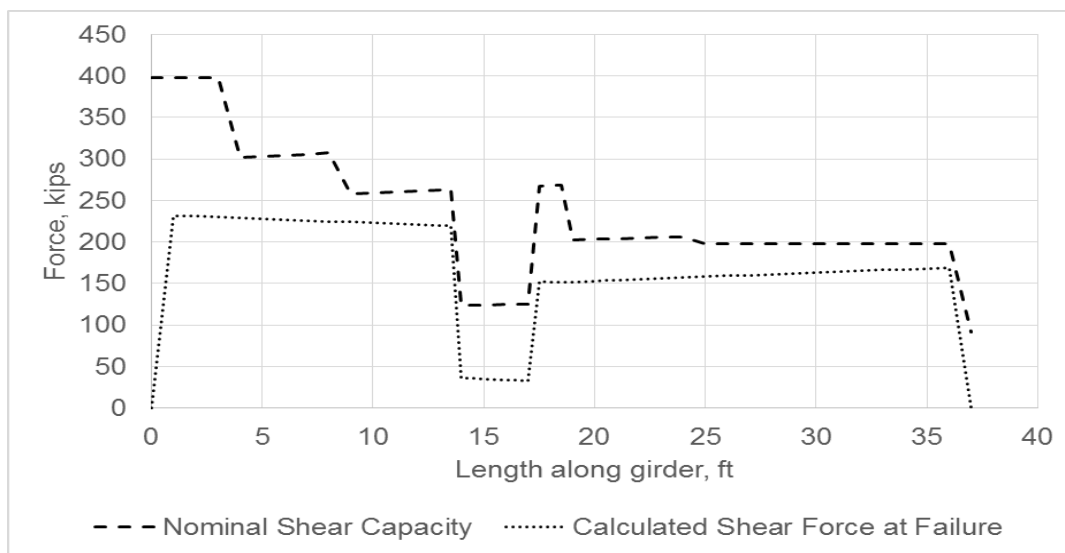


Figure 32- Nominal Shear Capacity versus Calculated Shear for the Control Test

The girder was tested on May 27, 2014 with an applied load reaching 353 kips and a deflection of 1.43 in. The load rate was managed to minimize effects of sudden loading. At 20 kip increments, the load was held steady and after pausing to ensure that the girder could be

safely approached, the girder was inspected and cracks were marked. At 280 kips of applied load, the strain transducers were removed to ensure they would not be damaged in case of failure and due to the potential for a failure, the girder was no longer closely inspected between load intervals. A hydraulic pump malfunctioned at 353 kips, and the test had to be stopped until the pump could be replaced. The pump was replaced, and the girder was re-tested on June 2, 2014. The load was applied in increments of 50 kips until reaching 250 kips, at which point 20 kip increments were used until reaching 401 kips. This exceeded the capacity of the test frame, so the test was concluded.

3.2.4: Test 2- Girder with Eight Severed/Re-Tensioned Strands

Girder B was a 60 ft long girder that was damaged at the midspan and repaired using the strand splice method. The applied damage consisted of removing 8 ft of concrete around the midspan, approximately 4 ft on either side of midspan, and severing eight prestressing strands. The damage location is shown in Figure 33, and a cross section view showing the damaged strands is shown in Figure 34. A length of 8 ft was chosen as the length the damage area, based on the average width of a semi-trailer (YRC Worldwide, 2015). Three strands were severed in the bottom row of strands, three strands were severed in the second row from the bottom, and two strands were severed in the third row from the bottom. To damage the girder, it was set on its side and struck with a hydraulic hammer attached to a backhoe, as shown in Figure 35.

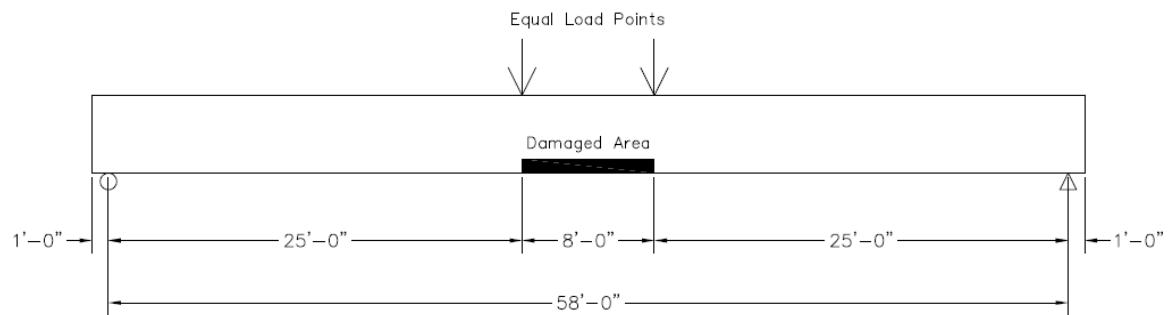


Figure 33- Girder B Damage Location

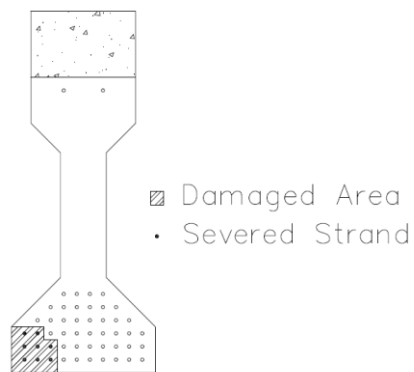


Figure 34- Girder B Damage Cross Section



Figure 35- Damaging Girder B with Hydraulic Hammer

Eight strands were severed and re-tensioned to represent significant enough damage that girder replacement may be considered. The amount of prestressing force lost in a girder to

require replacement varies by state. Some state departments of transportation will replace a girder if any strands are damaged. Other departments of transportation will replace a girder if 10% of the strands are damaged, and some state departments of transportation will consider repair methods if less than 25% of strands are damaged (Wipf, Klaiber, Rhodes, and Kempers 2004a). Ignoring the two top strands, severing eight strands, or 17% of the strands, was chosen for this test. Ensuing tests were inflicted with less damage (four strands severed) to represent a repair with more moderate damage.

This repair was performed by Structural Technologies. Strands were spliced with Grabb-it strand splices and tightened with a torque wrench to 80% of minimum ultimate tensile strength of the strand, based on Table 2 provided by the Grabb-it manufacturer (Prestress Supply Incorporated, 2010). After the splices were installed and tightened, the damaged concrete was repaired using Tstrata IRC high strength, quick cure concrete. APPENDIX F: Common Repair Procedures provides the repair procedures followed for installation of the repair concrete. Appendix G: Strand Splice Repair Procedures provides the repair procedures followed for the installation of strand splices. The repair was performed in two days by three Structural Technologies workers, though the second day solely consisted of removing formwork following the concrete placement the day before. Figure 36, Figure 37, Figure 38, Figure 39, and Figure 40 show the progress of the repair, including installation of the strand splice, formwork placement for the concrete placement, and removal of formwork.



Figure 36- Damage of Girder B Before Repair



Figure 37- Splice Installation for Girder B Repair



Figure 38- Completed Installation of Strand Splices



Figure 39- Formwork for Concrete Placement



Figure 40- Completed Repair after Concrete Placement and Form Removal

A 58 ft span was used for the test set-up, with an overhang of 1 ft on each end. A spreader beam and rubber pads were used to simulate two equal point loads 8 ft apart, creating a constant moment region. Steel braces were attached from the spreader beam to each column of the load frame to prevent the spreader beam from tipping. Figure 41 shows the loading setup, including the actuator, load cell, spreader beam, spreader beam braces, and the bearing pads.



Figure 41- Test 2 Loading Set-up

Instruments were put in place to measure and record deflection, applied load, and strain at different points along the girder. A load cell was placed above the actuator system to measure

load applied by the actuator. Potentiometers were placed at the center of load as well as the one-quarter point and three-quarters point of the span. In addition, two potentiometers were placed on the side of the girder at the center of load to measure lateral movement. LVDT's and strain transducers were used to obtain a strain profile at the center of load. An LVDT was placed on the bottom flange on each side at 2 in from the bottom and a strain transducer was placed on the bottom flange on each side at 6 in from the bottom. Strain transducers were also placed on the undamaged side of the girder at 41 in and 49 in from the bottom. Additionally, an LVDT was placed at approximately 25 ft from one end to measure slip between the repair zone and the unrepaired zone. The full test set-up and instrumentation plan is shown in a profile view in Figure 42, with a cross section at the center of load shown in Figure 43.

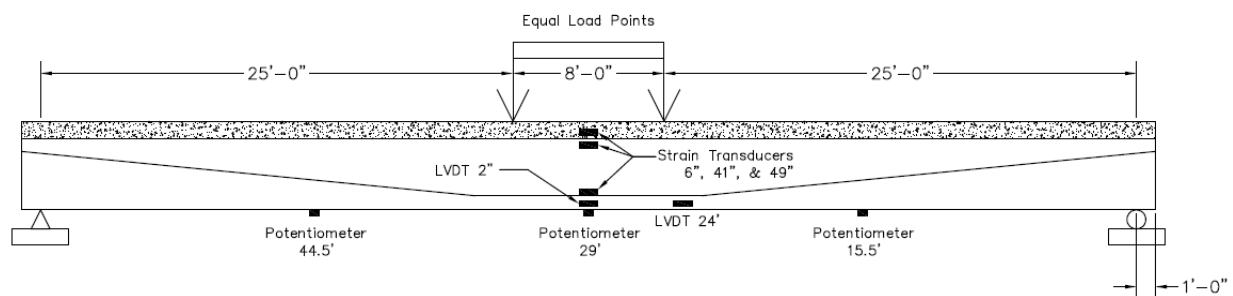


Figure 42- Test 2 Set-up and Instrumentation Plan

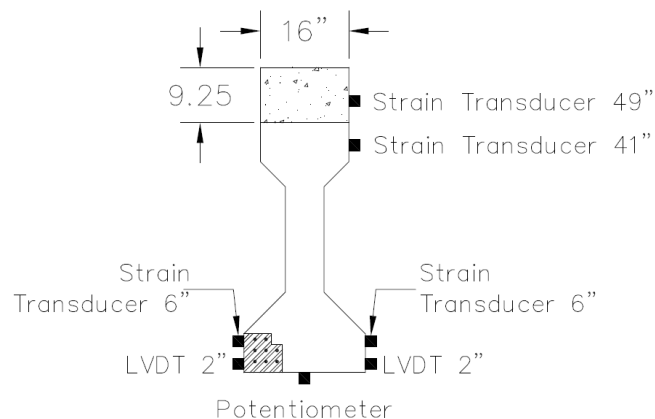


Figure 43- Test 2 Cross Section at Center of Load

The nominal moment and shear capacities of Girder B were determined using the provisions of AASHTO LRFD Bridge Design Specifications (American Association of State Highway and Transportation Officials, 2012). Strands that were spliced were assumed to be able to reach 85% of the ultimate tensile strength of the undamaged strand. The dimensions of the composite deck were approximately 9.25 in deep and 16 in wide. The minimum compressive strength values specified in the plans, 5000 psi for the girder concrete and 4000 psi for the deck concrete, were used in this analysis instead of the compressive strength values obtained from taking core samples of the concrete. For loading in this test, failure was anticipated at midspan, 29 ft from each support. The nominal moment capacity at this point was 2773 k-ft, corresponding to an applied load of 191 kips after accounting for the distributed self weight of 0.82 kips/ft. The moment capacity versus the demand from an applied load of 191 kips is shown in Figure 44. The shear capacity versus demand for this applied load is shown in Figure 45. The jumps in the shear capacity along the girder are due to the variable spacing of shear stirrups, shown in Figure 8.

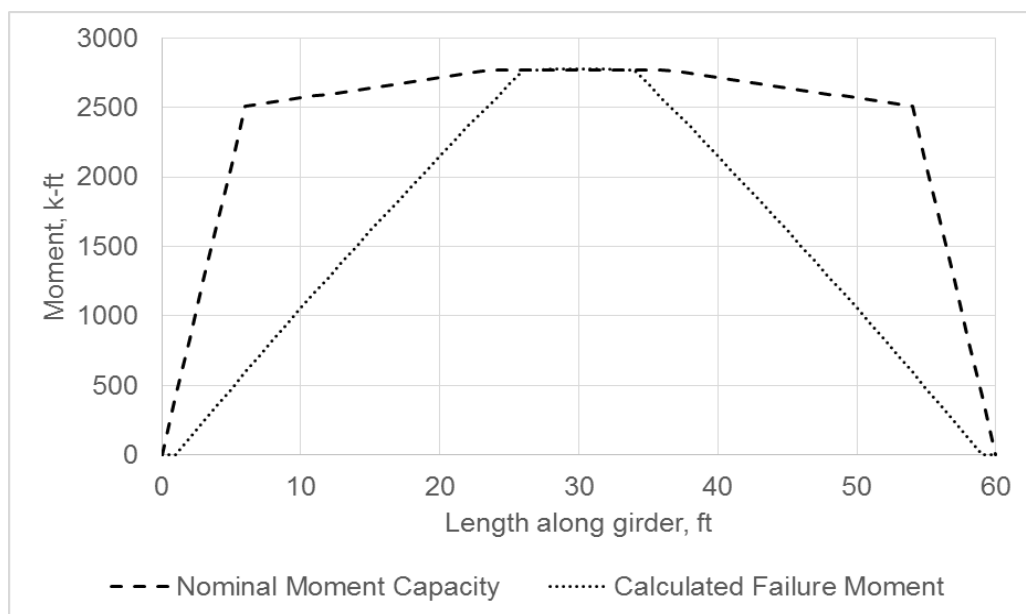


Figure 44- Nominal Moment Capacity versus Calculated Failure Moment for Test 2

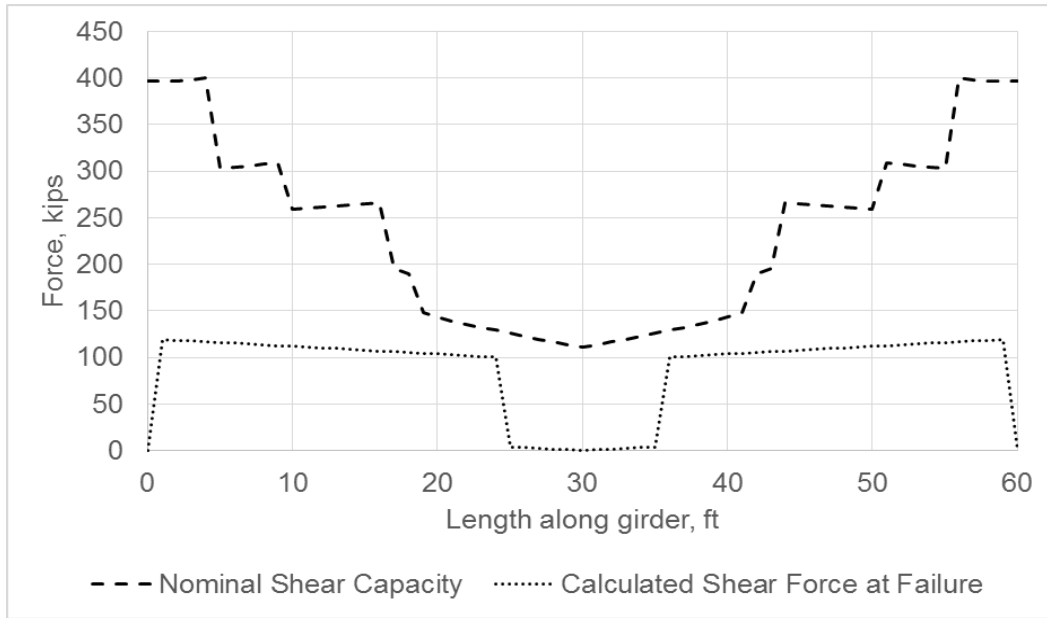


Figure 45- Nominal Shear Capacity versus Applied Shear for Test 2

Girder B was tested on August 21, 2014. The load rate was managed to minimize the effects of sudden loading. At 40 kip increments to 80 kips, the load was held steady and after pausing to ensure that the girder could be safely approached, the girder was inspected and cracks were marked. From 80 kips until 160 kips, the girder was inspected at 20 kip increments, at which point the load increment was decreased to 10 kips.

3.2.5: Test 4- Girder with Four Severed/Re-Tensioned Strands

For Test 4, Girder C, a 60 ft girder, was damaged for approximately 4 ft of length at the one-third and two-thirds points along the length of the girder. Four of the 48 prestressed strands in the bottom flange of the girder were severed at each of the two damage locations on Girder C. Having two damage locations on one girder allowed two different repair techniques to be tested on the same girder. Damage to each repair location was limited to 4 ft of damage and four strands severed to ensure that one repair would not influence the results of the other repair.

Figure 46 shows the damage locations along the length of the girder, and Figure 47 shows a cross

section at the damage locations. At each location, two strands were severed in the bottom row of strands, one strand was severed in the second row from the bottom, and one strand was severed in the third row from the bottom. To damage the girder, it was set on its side and struck with a hydraulic hammer attached to a backhoe. Figure 48 shows the girder after this damage.

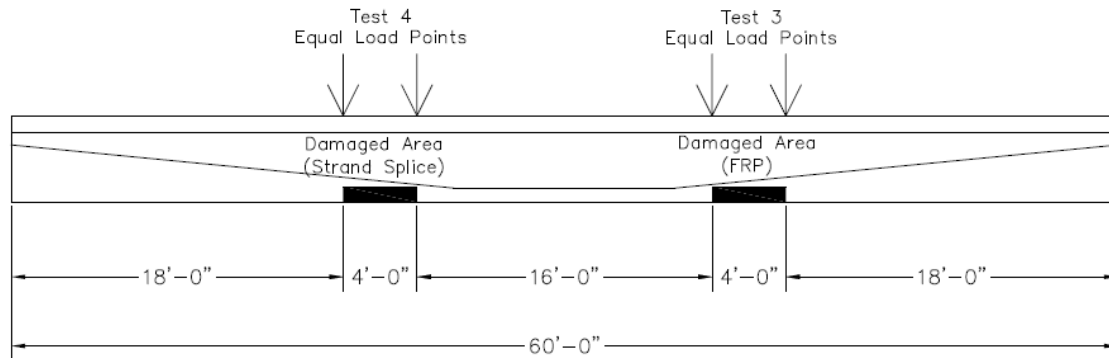


Figure 46- Girder C Damage Locations

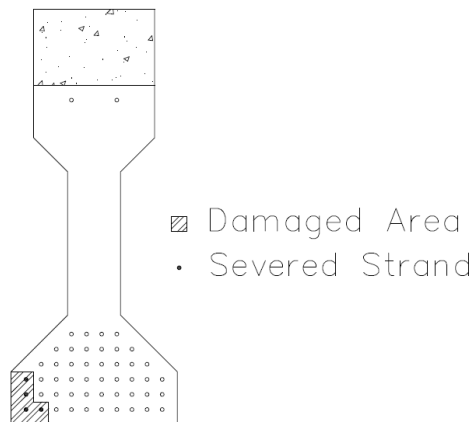


Figure 47- Girder C Cross Section Through Damaged Area



Figure 48- Girder C after Damage from Hydraulic Hammer

This repair was performed by Structural Technologies. Strands were spliced with Grabb-it cable splices and tightened with a torque wrench to 80% of the minimum ultimate tensile strength of the strand, based on Table 2 provided by the Grabb-it manufacturer (Prestress Supply Incorporated, 2010). After the splices were installed and tightened, the damaged concrete was repaired using Tstrata IRC high strength, quick cure concrete. APPENDIX F: Common Repair Procedures provides the repair procedures followed for installation of the repair concrete. APPENDIX G: Strand Splice Repair Procedures provides the repair procedures followed for the installation of the strand splices. The repair, in addition to the strand splice repair in Test 2, was performed in two days by three Structural Technologies workers, though the second day solely consisted of removing formwork following the concrete placement the day before. Figure 49, Figure 50, Figure 51, Figure 52, Figure 53, and Figure 54 show the progress of the repair, including sandblasting of the concrete, installation of the strand splices, formwork placement and placing concrete, and removal of formwork.



Figure 49- Damage Location for Test 4 Before Repair



Figure 50- Sandblasting of the Girder Concrete



Figure 51- Installation of the Strand Splices



Figure 52- Completed Installation of the Strand Splices



Figure 53- Placing the Repair Concrete



Figure 54- Completed Repair after Concrete Placement and Form Removal

Test 4 was conducted in two iterations with a 58 ft span length for the first iteration and a 50 ft span length for the second iteration. A 58 ft span was used, with an overhang of 1 ft on each end. A spreader beam and rubber pads were used to simulate two equal point loads 4 ft apart, creating a constant moment region. This constant moment region was located from 17 ft to 21 ft from one support. A 58 ft span was chosen for the first iteration to decrease the shear

demand near the original midspan of the girder, where shear stirrups are further apart. Steel braces were attached from the spreader beam to each column of the load frame to prevent the spreader beam from tipping. Figure 55 shows the loading setup, including the actuator, load cell, spreader beam, spreader beam braces, and the bearing pads.



Figure 55- Test 4 Loading Set-up

Instruments were installed to measure and record deflection, applied load, and strain at different points along the girder. A load cell was placed above the actuator system to measure load applied by the actuator. Potentiometers were placed at the center of load as well as the halfway point from the center of load to the support on each side. In addition, a potentiometer was placed on the side of the girder near the center of load to measure lateral movement. MLVDT's and strain transducers were used to obtain a strain profile at the center of load. An MLVDT was placed on the bottom flange on each side at 2 in from the bottom and a strain transducer was placed on the bottom flange at 6 in from the bottom on the damaged side and 4 in from the bottom on the undamaged side. Strain transducers were placed at 41 and 49 in from the

bottom on both sides. Additionally, an LVDT was placed at the interface between the repair zone and the unrepaired zone, approximately 14.5 ft and 24.5 ft from the closest support. LVDT's were placed at these two locations on the undamaged side as well. The full test set-up and instrumentation plan for the first iteration of Test 4 is shown in a profile view in Figure 56, with a cross section at the center of load shown in Figure 57.

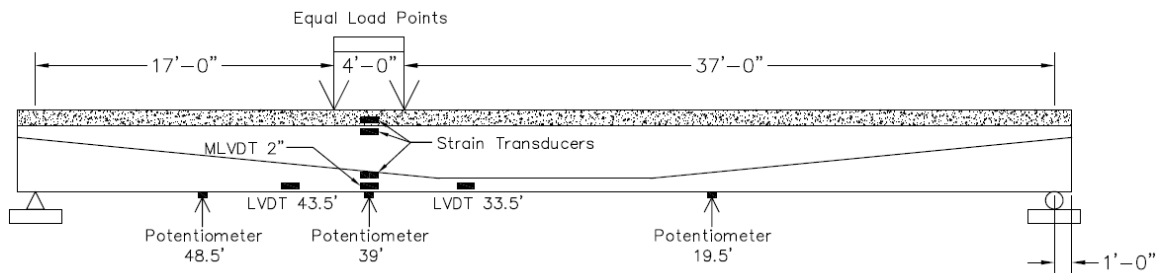


Figure 56- Test 4, Iteration 1 Set-up and Instrumentation Plan

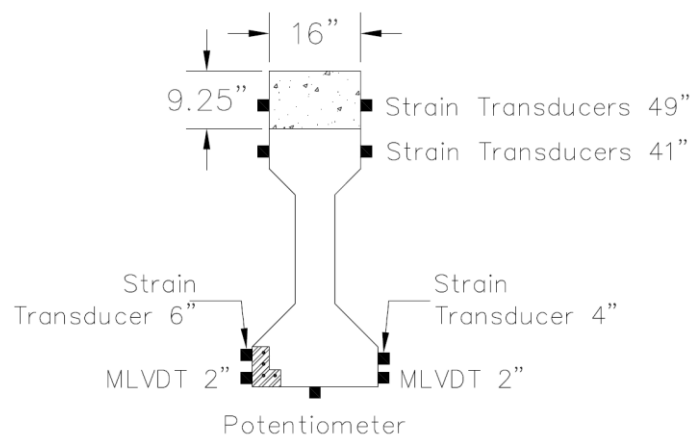


Figure 57- Test 4, Iteration 1 Instrumentation Cross Section

The nominal moment and shear capacities of Girder C were determined using the provisions of AASHTO LRFD Bridge Design Specifications (American Association of State Highway and Transportation Officials, 2012). Strands that were spliced were assumed to be able to reach full minimum ultimate tensile strength of the undamaged strand in this analysis. The dimensions of the composite deck were approximately 9.25 in deep and 16 in wide. The minimum compressive strength values specified in the plans, 5000 psi for the girder concrete and

4000 psi for the deck concrete were used in this analysis instead of the compressive strength values obtained from taking core samples of concrete. For loading in this test, failure was anticipated at the second load point, 21 ft from the closest support. The nominal moment capacity at this point was 2744 k-ft, corresponding to an applied load of 202 kips after accounting for the distributed self weight of 0.78 kips/ft. The moment capacity versus the demand from the expected failure load, 202 kips, is shown in Figure 58. The shear capacity versus demand for this applied load is shown in Figure 59. The jumps in the shear capacity along the girder are due to the variable spacing of shear stirrups, shown in Figure 8.

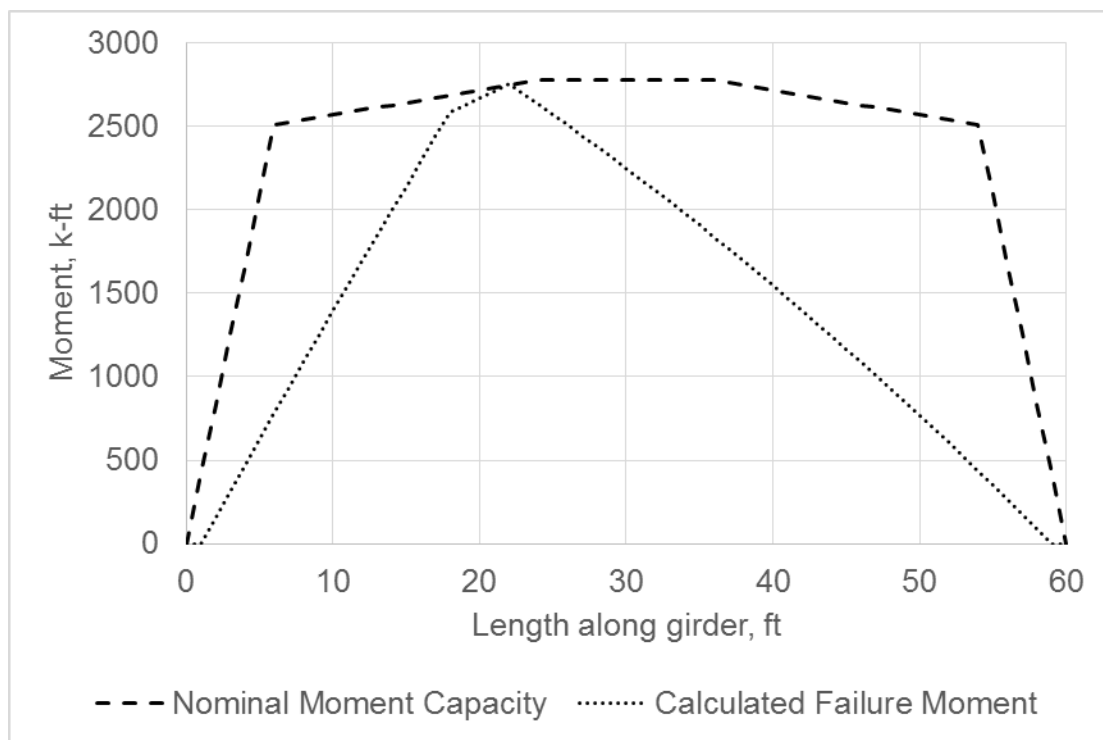


Figure 58- Nominal Moment Capacity versus Calculated Failure Moment, Test 4 Iteration 1

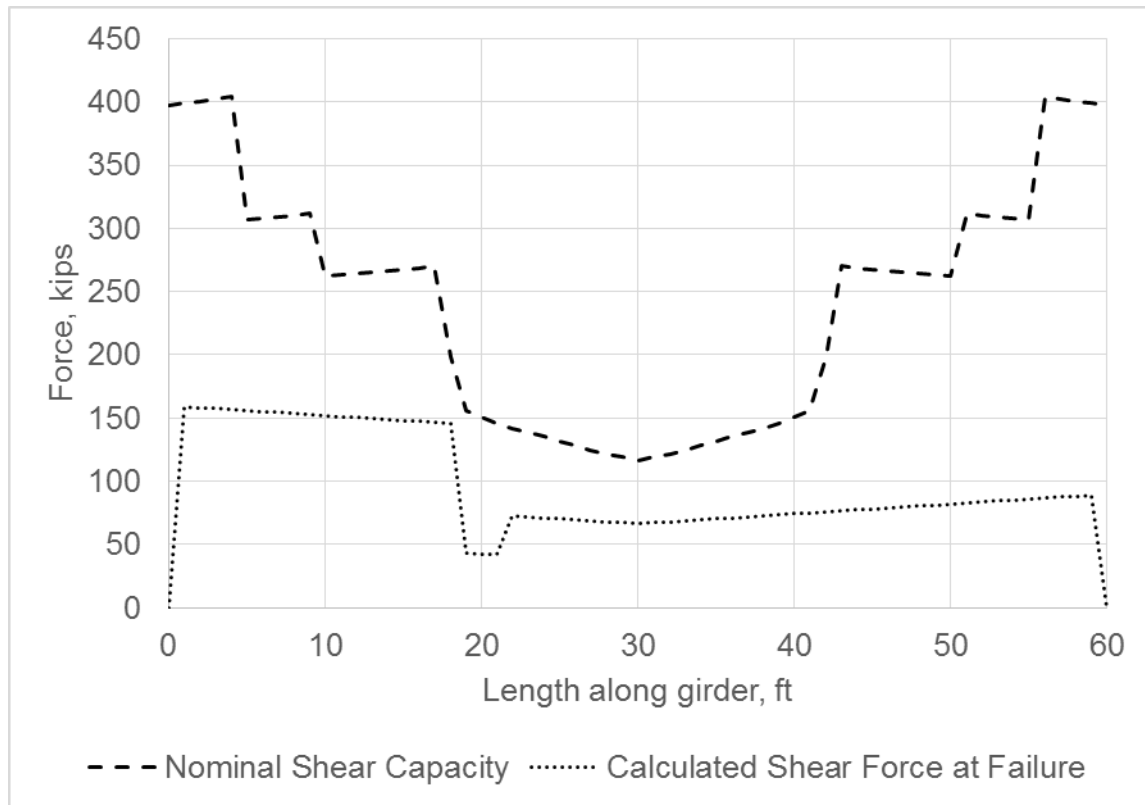


Figure 59- Nominal Shear Capacity versus Applied Shear, Test 4 Iteration 1

The first iteration of Test 4 was done on November 20, 2014. The load rate was managed to minimize the effects of sudden loading. At 20 kip increments until reaching 100 kips, the load was held steady and after pausing to ensure that the girder could be safely approached, the girder was inspected and cracks were marked. From 100 kips until 180 kips, the girder was inspected at 20 kip increments, at which point the load increment was decreased to 10 kips. At 200 kips of applied load, due to safety reasons, the girder was no longer closely inspected. The load increased to 241 kips when the actuator appeared as though it would slip out of plumb with the girder, and the test was stopped. This was caused by the rotation in the beam at the points of load relative to the actuator. For the second iteration, the span length was decreased to 50 ft to reduce the rotation at the points of loading. A diagram of the Test 4, Iteration 2 loading set-up is shown in Figure 60.

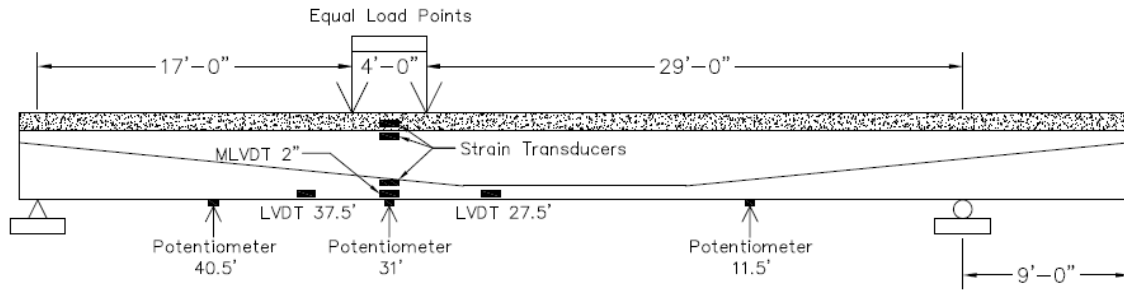


Figure 60- Test 4, Iteration 2 Test Set-up

The same assumptions about the concrete material properties and girder dimensions as well as using the same specifications for finding the moment and shear capacities were used for Iteration 2 of Test 4. For loading in this test, failure was anticipated at the second load point, 21 ft from the closest support. The nominal moment capacity at this point was 2744 k-ft, corresponding to an applied load of 228 kips after accounting for the distributed self weight of 0.78 kips/ft. The moment capacity versus the demand from the expected failure load, 228 kips, is shown in Figure 61. The shear capacity versus demand for this applied load is shown in Figure 62.

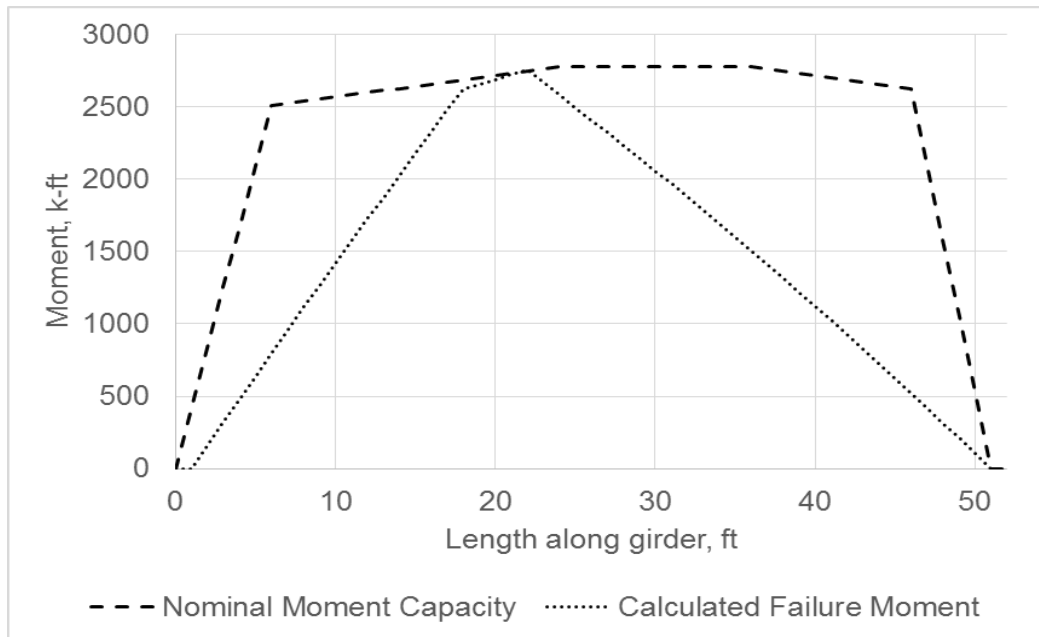


Figure 61- Nominal Moment Capacity versus Calculated Failure Moment, Test 4 Iteration 2

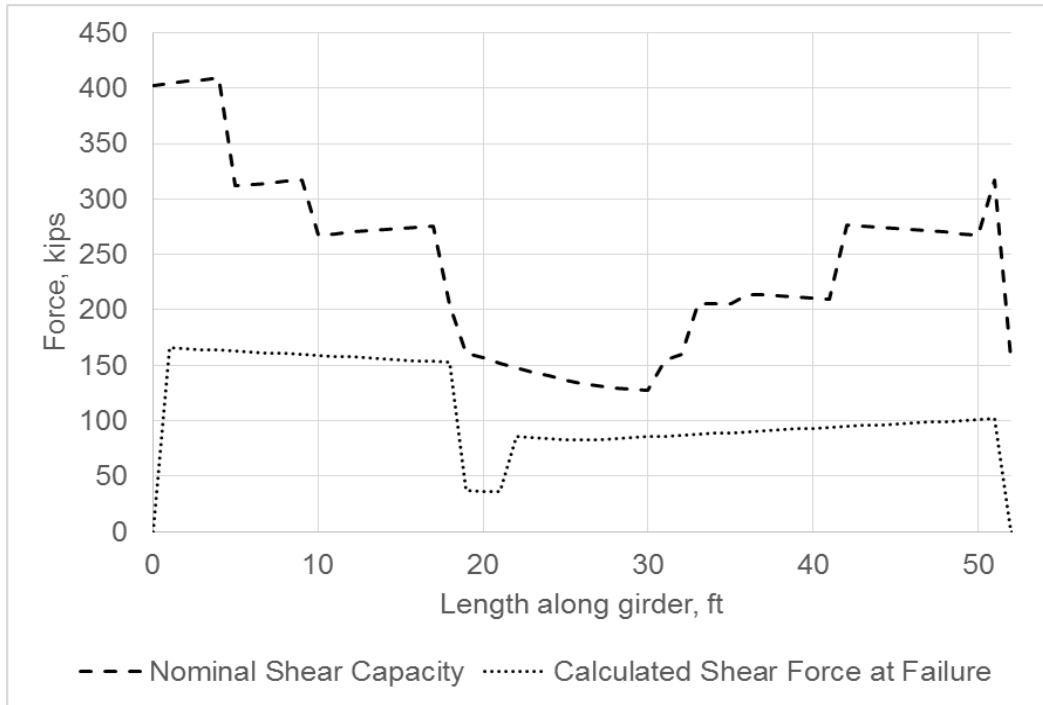


Figure 62- Nominal Shear Capacity versus Applied Shear, Test 4 Iteration 2

The second iteration of Test 4 was done on November 25, 2014. The load rate was managed to minimize the effects of sudden loading. At 50 kip increments until reaching 200 kips, the load was held steady and after pausing to ensure that the girder could be safely approached, the girder was inspected and cracks were marked. After reaching 200 kips, the load increment was decreased to 10 kips until reaching failure.

3.2.6: Test 5- FRCM Repaired Girder with Four Severed Strands

For Test 5, Girder D, a 60 ft long girder, was damaged for approximately 4 ft of length at the one-third and two-thirds points along the length of the girder. Four of the 48 prestressed strands in the bottom flange of the girder were severed at each of the two damage locations on Girder D. Having two damage locations on one girder allowed two different repair techniques to be tested on the same repair. Damage to each repair location was limited to 4 ft of damage and four strands severed to ensure that one repair would not influence the results of the other repair.

Figure 63 shows the damage locations along the length of the girder, and Figure 64 shows a cross section at the damage locations. At each location, two strands were severed in the bottom row of strands, one strand was severed in the second row from the bottom, and one strand was severed in the third row from the bottom. To damage the girder, it was set on its side and struck with a hydraulic hammer attached to a backhoe. After this damage from the hydraulic hammer, the concrete was sawed and chipped to provide a clean surface to repair. Saw cut lines are shown in Figure 65, with the cuts being performed in Figure 66. Before repairs, at least 1 in of concrete is chipped throughout the repair surface, shown in Figure 67.

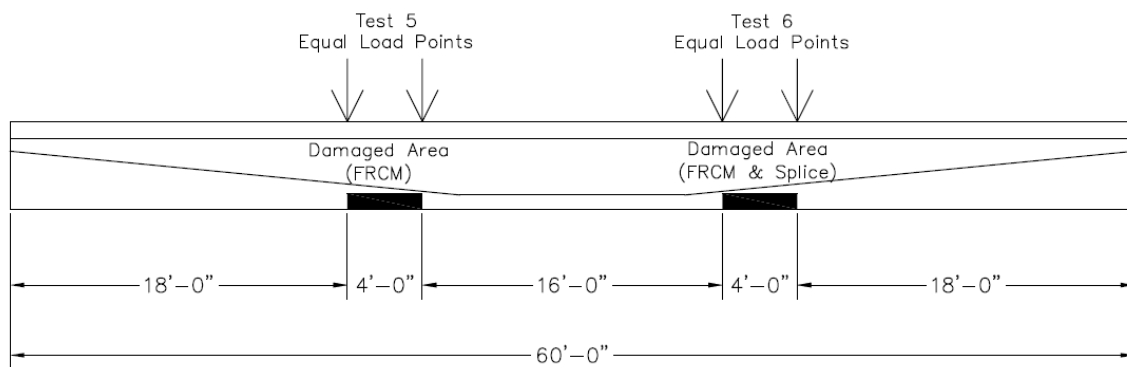


Figure 63- Girder D Damage Locations

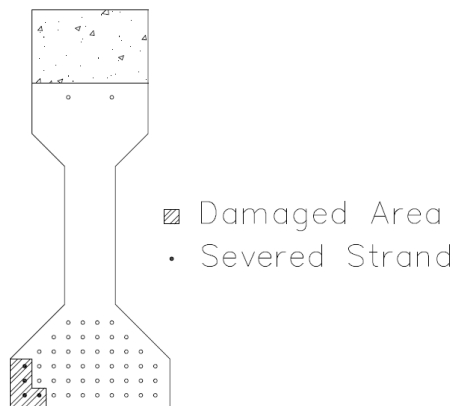


Figure 64- Girder D Cross Section Through Damaged Area



Figure 65- Girder D after Damage from Hydraulic Hammer



Figure 66- Cutting the Outline of the Damaged Area



Figure 67- Girder D after Chipping and Saw Cutting

The repairs on Girder D were performed by Structural Technologies. The repair for the damage location of Test 5 was done using Tstrata IRC as a patch material and the Ruredil X Mesh Gold FRCM system. Ruredil X Mesh Gold consists of a Polyparaphenylene benzobisoxazole (PBO, or commonly known as zylon) mesh and Ruredil X Mesh M750 mortar, a stabilized inorganic matrix to connect the mesh to the concrete substrate. After the concrete patch was placed and allowed to cure, the FRCM was installed. APPENDIX F: Common Repair Procedures provides the repair procedures followed for installation of the repair concrete. APPENDIX G: Fabric Reinforced Cementitious Matrix Procedures provides the repair procedures followed for the application of FRCM. The concrete patch repair, in addition to the strand splice and patch repair in Test 6, was performed in two days by two Structural Technologies workers, though the second day solely consisted of removing formwork following the concrete placement the day before. The FRCM repair, in addition to the FRCM repair in Test 6, was performed in two days by two Structural Technologies workers, though the first day

solely consisted of grinding the concrete substrate. Figure 68, Figure 69, Figure 70, Figure 71, Figure 72, Figure 73, and Figure 74 show the progress of the repair, including formwork placement and concrete patch placement, removal of formwork, surface preparation for the FRCM repair, and the FRCM installation.



Figure 68- Formwork for the Patch Material

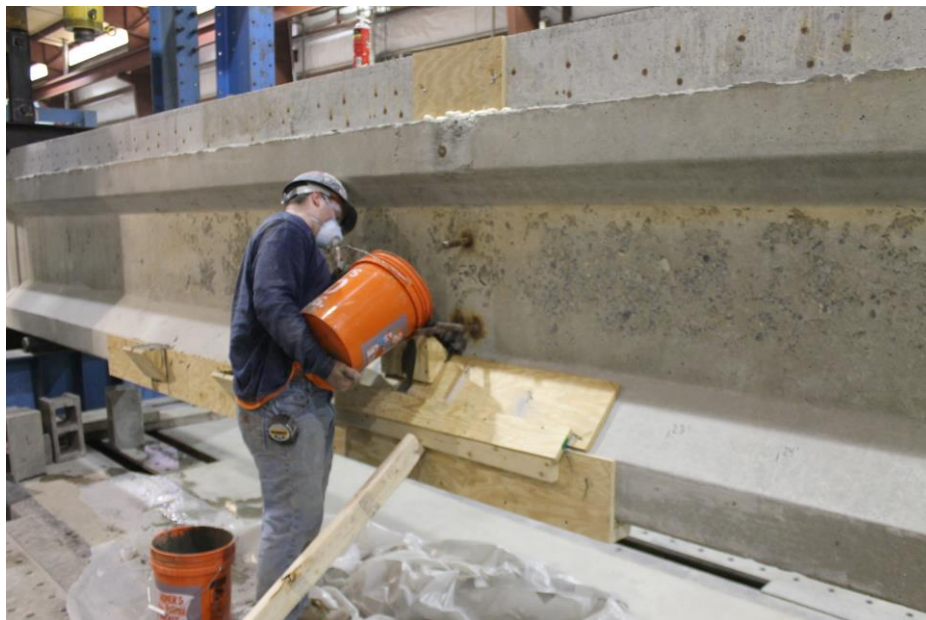


Figure 69- Placing the Patch Material

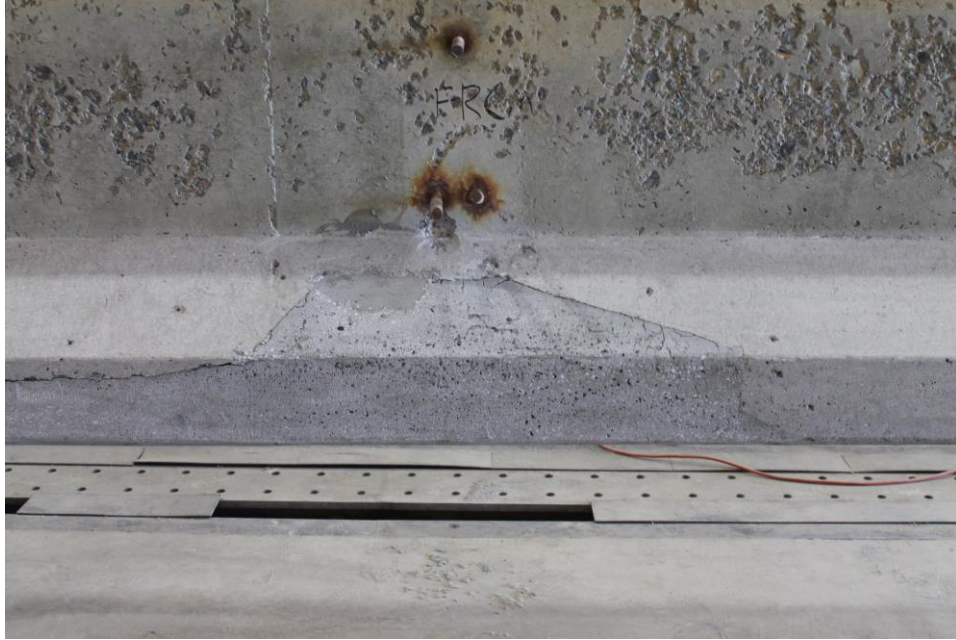


Figure 70- Repair Location after Formwork Removal



Figure 71- Repair Location after FRCM Surface Preparation



Figure 72- Ruredil FRCM Mesh



Figure 73- Installation of the FRCM



Figure 74- Completed FRCM Repair

Four layers of PBO mesh were applied to the bottom flange on the bottom face and each side up to the bottom of the web. The first layer of mesh was 15 ft long, the second layer was 13 ft long, the third layer was 11 ft long, and the fourth and final layer was 9 ft long. These layers were centered at 20 ft from the closest end of the girder. The length of each sequential layer of mesh was reduced to prevent debonding in the layers during testing. Between each mesh layer and after the final layer was installed, a layer of Ruredil X Mesh M750 mortar was troweled on. A diagram of the FRCM repair sequence and mesh lengths is shown in Figure 75.

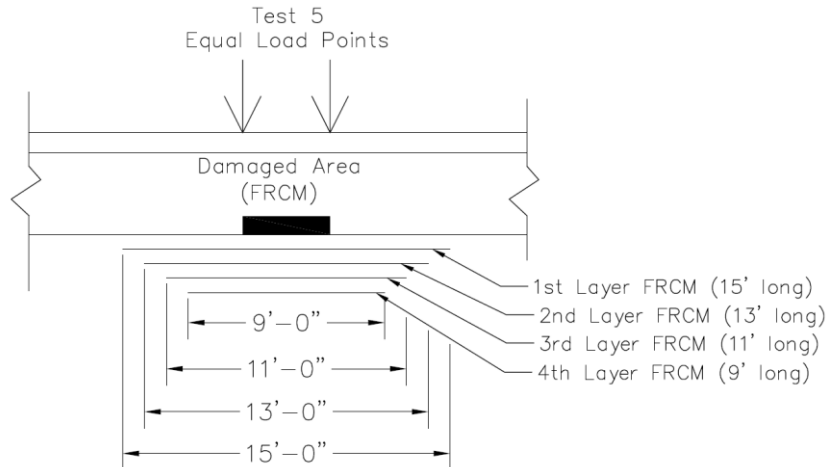


Figure 75- Test 5 FRCM Repair Sequence

Test 5 was conducted using a 52 ft span, with an overhang of 1 ft on the side closest to the repair and an overhang of 7 ft on the side farthest from the repair. A spreader beam and rubber pads were used to simulate two equal point loads spaced 4 ft apart, creating a constant moment region. This constant moment region was located 17 ft to 21 ft from one support. A 52 ft span was chosen to decrease the shear demand near the original midspan of the girder, where shear stirrups are further apart. To help prevent actuator slip, an additional load frame with another actuator was put in place 14 ft from one support. If the original actuator started to show signs of slipping, the additional actuator could be used to hold the load on the girder and allow the original actuator to be unloaded and straightened. A rubber pad was placed between the additional actuator and the girder to prevent direct contact. The additional load frame is shown in Figure 76.



Figure 76- Test 5 Additional Load Frame

Instruments were installed to measure and record deflection, applied load, and strain at different points along the girder. A load cell was placed above the actuator system to measure load applied by the actuator. Potentiometers were placed at the center of load as well as the halfway point from the center of load to the support on either side. In addition, a potentiometer was placed on the side of the girder near the center of load to measure lateral movement. Strain transducers were used to obtain a strain profile at the center of load and at the load point closest to the midspan, which was the point of maximum moment. At the center of load, a strain transducer was placed on the bottom flange on each side at 2 in from the bottom, on the top flange on each side at 39 in from the bottom, and on the deck on each side at 51.5 in from the bottom. At the load point closest to midspan, a strain transducer was placed on the bottom flange on each side at 2 in from the bottom and on the deck on each side at 51.5 in from the bottom. Additionally, an LVDT was placed at the end of the first FRCM layer at each corner to measure interface slip between the FRCM and the concrete substrate. The full test set-up and instrumentation plan for Test 5 is shown in a profile view in Figure 77, with a cross section at the center of load shown in Figure 78.

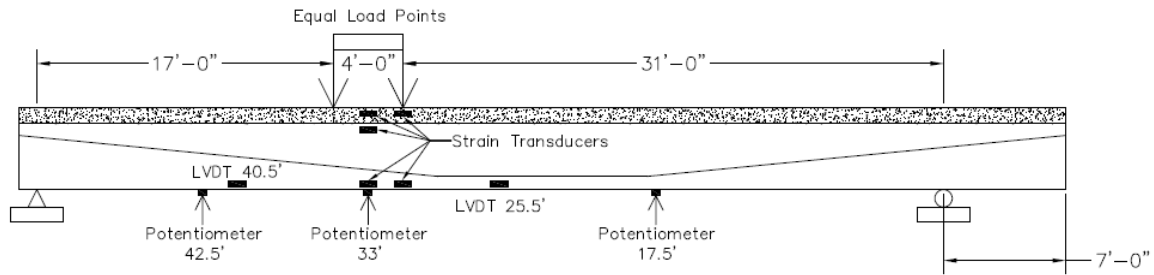


Figure 77- Test 5 Set-up and Instrumentation

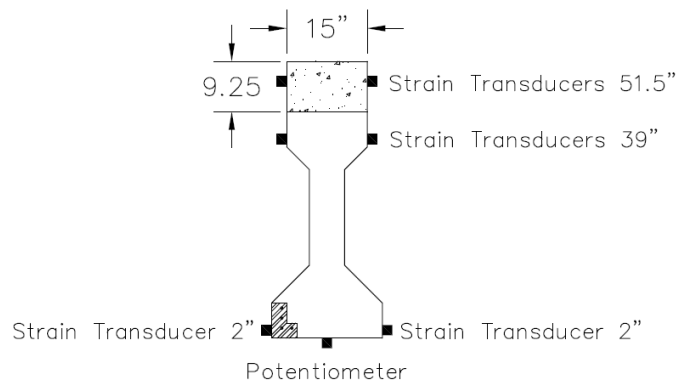


Figure 78- Test 5 Instrumentation Cross Section at Center of Load

The nominal moment and shear capacities of Girder D were determined using the provisions of AASHTO LRFD Bridge Design Specifications (American Association of State Highway and Transportation Officials, 2012) and guidelines from ACI 549 (American Concrete Institute, 2013). The dimensions of the concrete deck were approximately 9.25 in deep and 15 in wide. Gaps in the deck from demolition of the bridge were not filled in before Test 5, shown in Figure 79. These gaps were about 0.5 in wide across the entire width and depth of the deck and there were eight of them spaced along the length of the girder.



Figure 79- Saw Cut Gaps in the Deck Concrete of Girder D

The minimum compressive strength values specified in the plans, 5000 psi for the girder concrete and 4000 psi for the deck concrete were used in this analysis instead of the compressive strength values obtained from taking core samples of concrete. For loading in this test, failure was anticipated at the second load point, 21 ft from the closest support. The nominal moment capacity at this point was 2634 k-ft, corresponding to an applied load of 210 kips after accounting for the distributed self weight of 0.78 kips/ft. The moment capacity versus the demand from the expected failure load, 210 kips, is shown in Figure 80. The shear capacity versus demand for this applied load is shown in Figure 81. The jumps in the shear capacity along the girder are due to the variable spacing of shear stirrups, shown in Figure 8.

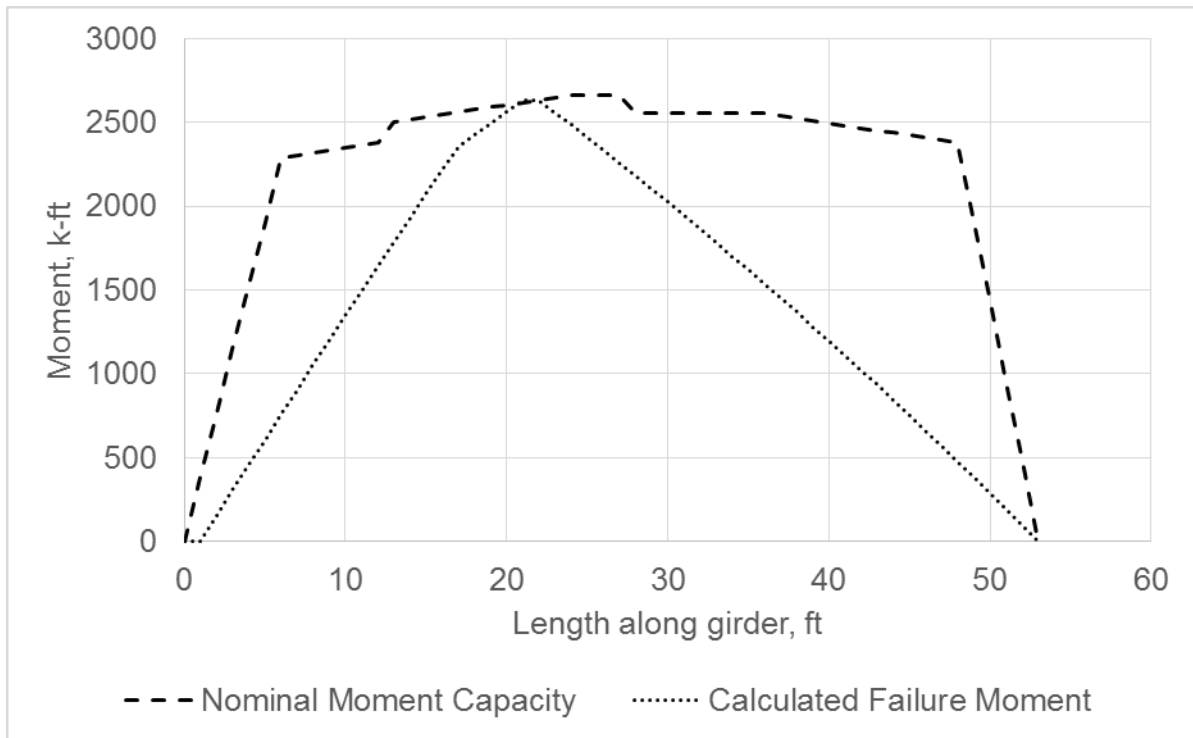


Figure 80- Test 5 Nominal Moment Capacity versus Calculated Failure Moment

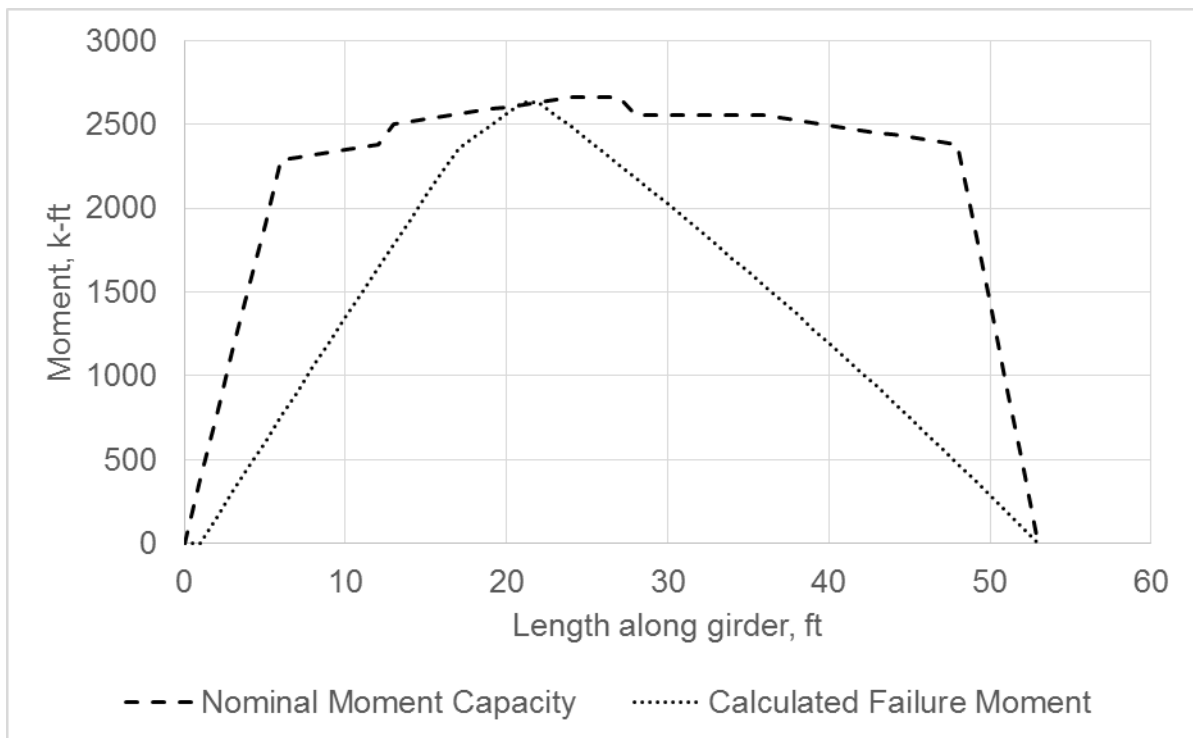


Figure 81- Shear Capacity versus Calculated Shear at Failure

3.2.7: Test 6- FRCM Repaired Girder with Four Severed/Re-Tensioned Strands

For Test 5 and Test 6, Girder D, a 60 ft long girder, was damaged for approximately 4 ft of length at the one-third and two-thirds points along the length of the girder. Four of the 48 prestressed strands in the bottom flange of the girder were severed at each of the two damage locations on Girder D. Having two damage locations on one girder allowed two different repair techniques to be tested on the same repair. Damage to each repair location was limited to 4 ft of damage and four strands severed to ensure that one repair would not influence the results of the other repair. Figure 82 shows the girder after damage from the hydraulic hammer as well as chipping and saw cutting to clean up the repair area. This repair included both the strand splice method and an externally bonded FRCM wrap. Since strand splices are an active, internal repair and FRCM wrap is a passive, external repair, they are easily combinable into one repair. Since Test 4 and Test 5 have similar damage and the same number and layout of strands severed, the results of Test 6 can appropriately be compared to the results from Test 4 and Test 5 to see how the combined repair performed versus the stand-alone strand splice and FRCM repairs.



Figure 82- Test 6 Repair area after Damage

The repairs on Girder D were performed by Structural Technologies. The repair for the damage location of Test 6 was done using Grabb-it cable splices for the strand splice repair, Tstrata IRC as a patch material, and the Ruredil X Mesh Gold FRCM system. Ruredil X Mesh Gold consists of a PBO mesh and Ruredil X Mesh M750 mortar, a stabilized inorganic matrix to connect the mesh to the concrete substrate. After the strands were re-tensioned and the concrete patch was placed and allowed to cure, the FRCM was installed. APPENDIX F: Common Repair Procedures provides the repair procedures followed for installation of the repair concrete. APPENDIX G: Strand Splice Repair Procedures provides the repair procedures followed for the installation of strand splices. APPENDIX H: Fabric Reinforced Cementitious Matrix Repair Procedures provides the repair procedures followed for the application of FRCM. The strand splice and concrete patch repair, in addition to the concrete patch repair in Test 5, were performed in two days by two Structural Technologies workers, though the second day solely consisted of removing formwork following concrete placement on the previous day. The FRCM repair, in addition to the FRCM repair in Test 5, was performed in two days by two Structural Technologies workers, though the first day solely consisted of grinding the concrete substrate. Figure 83, Figure 84, Figure 85, Figure 86, Figure 87, Figure 88, and Figure 89 show the progress of the repair, including installation and tensioning of the strand splices, concrete patch placement, surface preparation for the FRCM repair, and the FRCM installation.



Figure 83- Strand Splice Installation

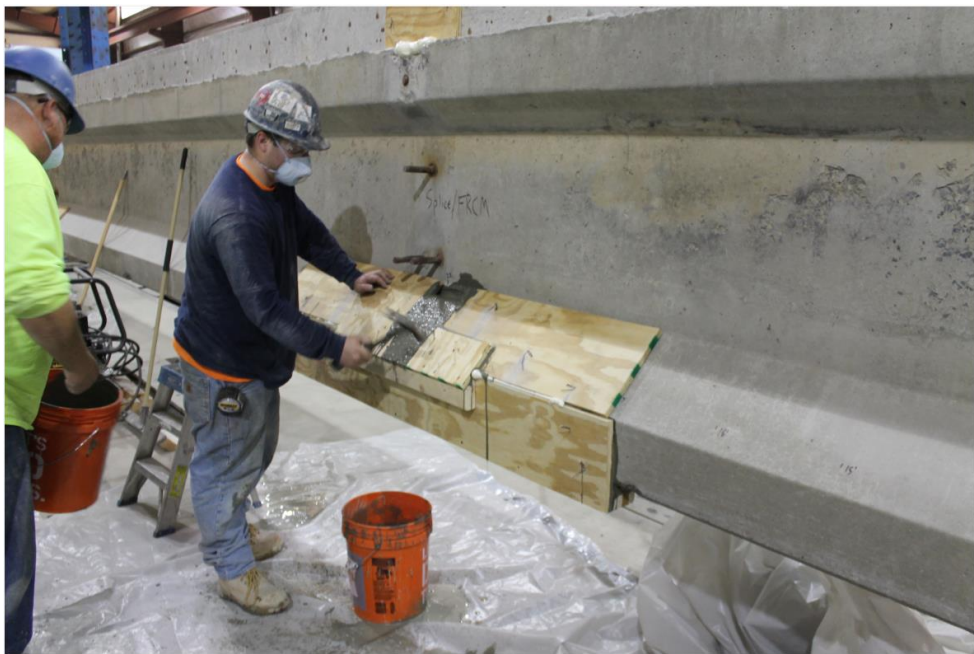


Figure 84- Placing the Patch Material



Figure 85- Damage Location after Removal of Formwork



Figure 86- Application of the First Layer of Mortar for FRCM Installation



Figure 87- Installation of the First Layer of FRCM Mesh on the Bottom Face



Figure 88- Installation of the Second Layer of FRCM Mesh on the Side of the Girder



Figure 89- Completed FRCM Repair

Four layers of PBO mesh were applied to the bottom flange on the bottom face and each side up to the bottom of the web. The first layer of mesh was 15 ft long, the second layer was 13 ft long, the third layer was 11 ft long, and the fourth and final layer was 9 ft long. These layers were centered at 20 ft from the closest end of the girder. The length of each sequential layer of mesh was reduced to prevent debonding in the layers during testing. Between each mesh layer and after the final layer was installed, a layer of Ruredil X Mesh M750 mortar was troweled on. A diagram of the FRCM repair sequence and mesh lengths is shown in Figure 90.

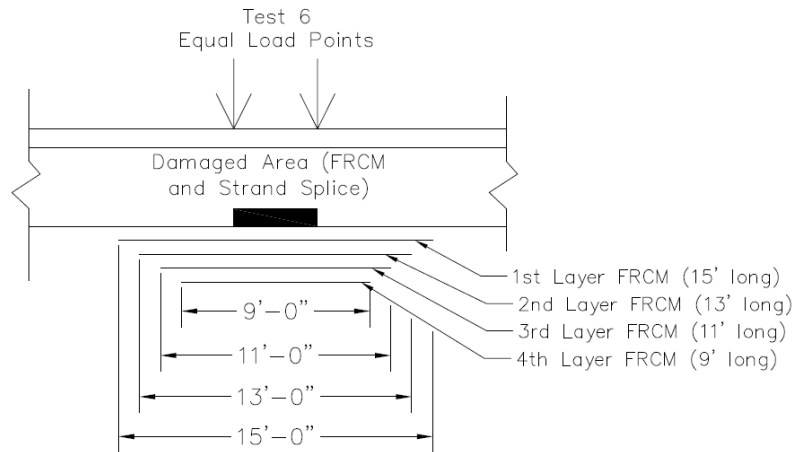


Figure 90- Test 6 FRCM Repair Sequence

Test 6 was conducted using a 52 ft span, with an overhang of 1 ft on the side closest to the repair and an overhang of 7 ft on the side farthest from the repair. A spreader beam and rubber pads were used to simulate two equal point loads spaced 4 ft apart, creating a constant moment region. This constant moment region was located 17 ft to 21 ft from the closest support. A 52 ft span was chosen to decrease the shear demand near the original midspan of the girder, where shear stirrups are further apart. Since Test 5 failed due to an unexpected shear transfer failure due to the discontinuous deck, the transverse gaps in the deck were filled in with a high early strength, low shrinkage grout, with a filled-in gap shown in Figure 91. To help prevent actuator slip, an additional load frame with another actuator was put in place 14 ft from one support. If the original actuator started to show signs of slipping, the additional actuator could be used to hold the load on the girder and allow the original actuator to be unloaded, straightened, and reloaded. A rubber pad was placed between the additional actuator and the girder to prevent direct contact. The additional load frame is shown in Figure 92.



Figure 91- Filling of Saw Cuts in Deck with Grout



Figure 92- Test 6 Additional Load Frame

Instruments were installed to measure and record deflection, applied load, and strain at different points along the girder. A load cell was placed above the actuator system to measure load applied by the actuator. Potentiometers were placed at the center of load as well as the halfway point between the center of load to the support on either side. In addition, a

potentiometer was placed on the side of the girder near the center of load to measure lateral movement. Strain transducers were used to obtain a strain profile at the center of load and at the load point closest to the midspan at 21 ft, which was the point of maximum moment. At the center of load, a strain transducer was placed on the bottom flange on each side at 2 in from the bottom, on the top flange on each side at 39 in from the bottom, and on the deck on each side at 51.5 in from the bottom. At the load point closest to midspan, a strain transducer was placed on the bottom flange on each side at 2 in from the bottom and on the deck on each side at 51.5 in from the bottom. Additionally, an LVDT was placed at the end of the first FRCM layer at each corner to measure interface slip between the FRCM and the concrete substrate. The full test set-up and instrumentation plan for Test 6 is shown in a profile view in Figure 93, with a cross section at the center of load shown in Figure 94.

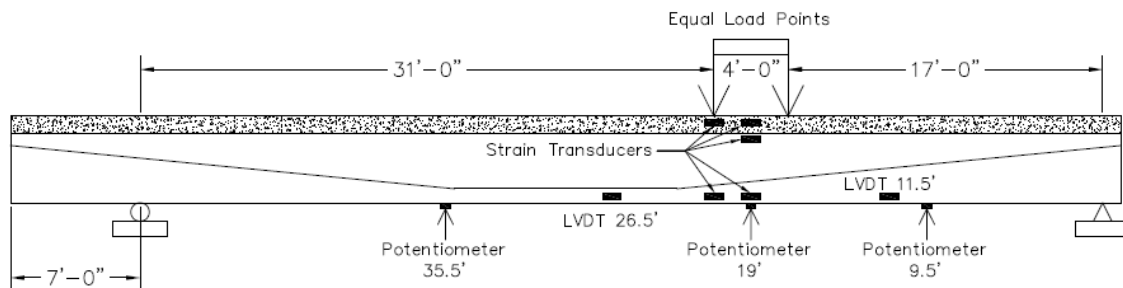


Figure 93- Test 6 Set-up and Instrumentation

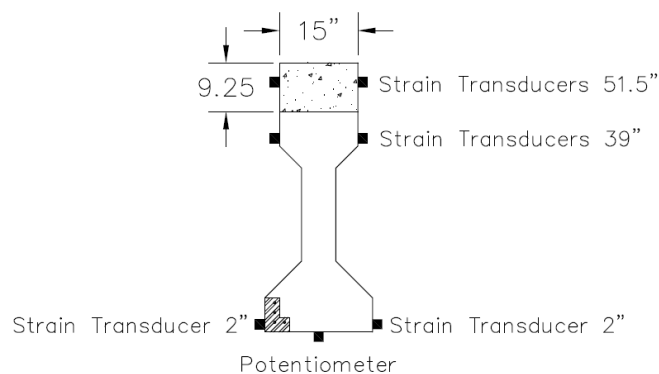


Figure 94- Test 6 Instrumentation Cross Section at Center of Load

The nominal moment and shear capacities of Girder D were determined using the provisions of the AASHTO LRFD Bridge Design Specifications (American Association of State Highway and Transportation Officials, 2012) and guidelines from ACI 549 (American Concrete Institute, 2013). The dimensions of the concrete deck were approximately 9.25 in deep and 15 in wide. The minimum compressive strength values specified in the plans, 5000 psi for the girder concrete and 4000 psi for the deck concrete, were used in this analysis instead of the compressive strength values obtained from taking core samples from the girders. For loading in this test, failure was anticipated at the second load point, 21 ft from the closest support. The nominal moment capacity at this point was 2804 k-ft, corresponding to an applied load of 225 kips after accounting for the distributed self weight of 0.78 kips/ft. The moment capacity versus the demand from the expected failure load, 225 kips, is shown in Figure 95. The shear capacity versus demand for this applied load is shown in Figure 96. The jumps in the shear capacity along the girder are due to the variable spacing of shear stirrups, shown in Figure 8.

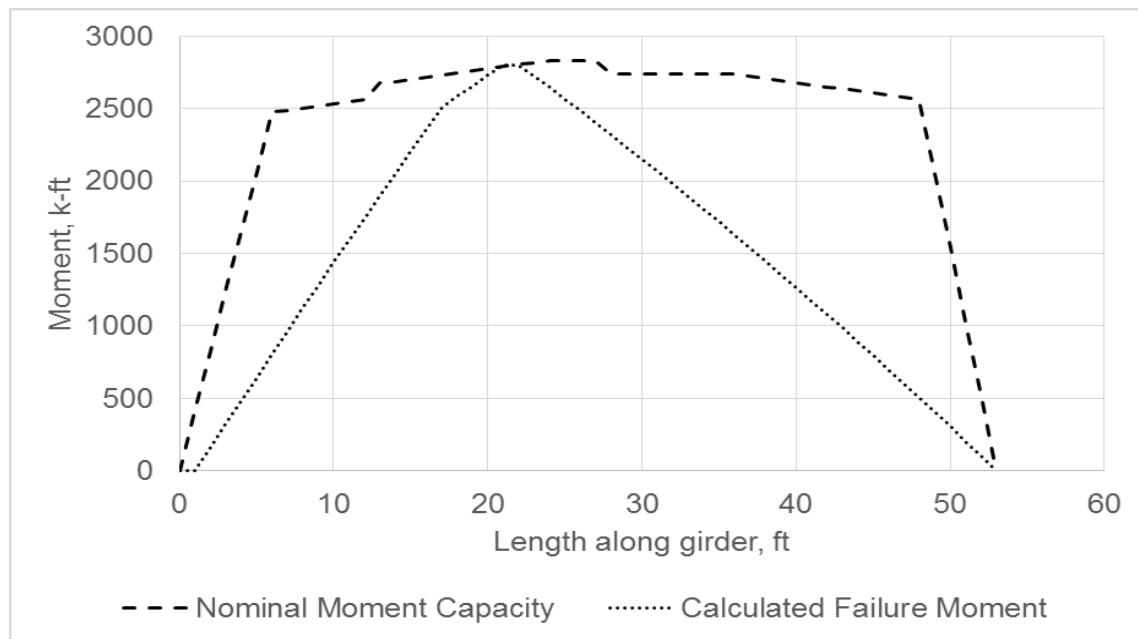


Figure 95- Test 6 Moment Capacity versus Calculated Failure Moment

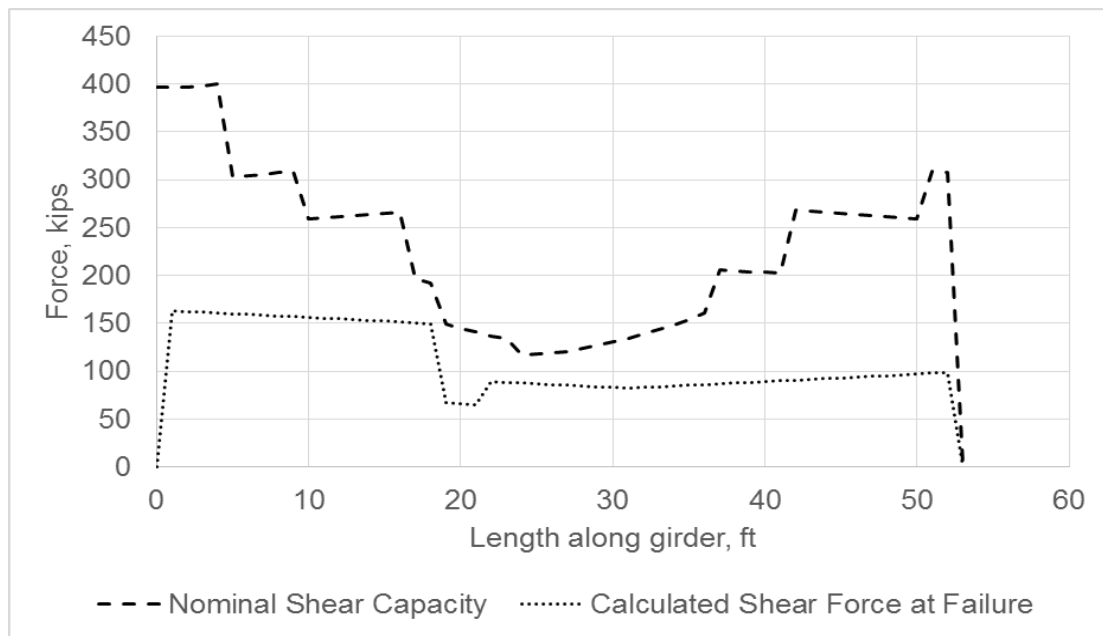


Figure 96- Shear Capacity versus Calculated Shear at Failure

CHAPTER IV: Results and Discussion

4.1: Introduction

This section compares the data recorded from the testing with calculated or theoretical data. Important values obtained from the tests such as cracking moment, failure moment, and deflection are evaluated and compared to predictions of the girder behavior using AASHTO formulas, moment-curvature analyses, and non-linear finite element modeling. The material property tests are evaluated as well. This section also includes assessments of the performance of the different repair methods in restoring strength and ductility of the girder.

4.2: Material Properties

4.2.1: Girder Concrete

Core samples were taken from the girder and deck concrete to determine material properties of the concrete. Table 6 summarizes the ultimate compressive strengths obtained for the girder and the deck. Table 7 summarizes the splitting tensile strength of cores taken from the girder. The average compressive strength of the girder concrete was 6,650 psi, which is greater than 5,000 psi, the minimum ultimate compressive strength required by the bridge plans, which are provided in APPENDIX A. The average compressive strength of the deck concrete was 6,020 psi, which is greater than 4,000 psi, the minimum ultimate compressive strength required by the bridge plans. The average tensile strength of the girder concrete was 424 psi, only about 6.5% of the compressive strength. The tensile strength of the girder concrete was lower than expected, though similar to results found using design equations. Table 8 compares the tested tensile strength of the girder concrete with a conservative formula for finding tensile strength, using both the design compressive strength of the concrete and the average tested compressive strength of the concrete.

Table 6- Girder and Deck Ultimate Compressive Strength

Sample	Cylinder #	Load (lbs)	Diameter (in)	Area (in ²)	Strength (psi)	Average (psi)
Girder	1	39,000	2.75	5.94	6570	6650
	2	40,000	2.75	5.94	6730	
Deck	3	35,000	2.75	5.94	5890	6020
	4	36,500	2.75	5.94	6150	

Table 7- Girder Concrete Ultimate Tensile Strength

Cylinder #	Load (lbs)	Diameter (in)	Length (in)	Strength (psi)	Average (psi)
5	7,500	2.75	4.50	386	424
6	10,000	2.75	5.00	463	

Table 8- Comparison of Tested Girder Concrete Tensile Strength with Design Equations

Tested Tensile Strength (psi)	$6 * \sqrt{f'_c}$ Design Compressive Strength (psi)	$6 * \sqrt{f'_c}$ Tested Compressive Strength (psi)
424	424	484

4.2.2: Tstrata IRC Repair Concrete

For the repairs performed by Structural Technologies in both July 2014 and March 2015, cylinder samples were taken for the Tstrata IRC Repair Concrete and tested for ultimate compressive strength. Results are summarized in Table 9. The average compressive strength at 24 hours was 3,820 psi, which is less than 4,500 psi, the specified strength for design listed by the manufacturer (Structural Group Incorporated, 2015). The average compressive strength at 48 hours was 4,970 psi, with no manufacturer-specified strength listed at that time. At 7 days, the average compressive strength was 6,070 psi, which is less than 7,700 psi, the specified strength for design listed by the manufacturer (Structural Group Incorporated, 2015).

Table 9- Repair Concrete Compressive Strength

Time	Repair #	Cylinder #	Failure Load (lbs)	Area (in ²)	Strength (psi)	Average per Repair (psi)	Average (psi)	Design (psi)
24 hr	Jul-14	1	45000	12.57	3580	4060	3820	4500
		2	57000	12.57	4540			
	Mar-15	3	50000	12.57	3980	3580		
		4	40000	12.57	3180			
48 hr	Jul-14	5	69000	12.57	5490	5730	4970	-
		6	75000	12.57	5970			
	Mar-15	7	46000	12.57	3660	4220		
		8	60000	12.57	4770			
7 day	Jul-14	9	82000	12.57	6520	6250	6070	7700
		10	75000	12.57	5970			
	Mar-15	11	83000	12.57	6600	5890		
		12	65000	12.57	5170			

4.2.3: Girder Mild Reinforcement

Reinforcing steel bars were collected from the girder and tested in tension for yield strength and ultimate strength, with the results shown in Table 10. These samples, thought to be Grade 40 reinforcing steel though not listed in the bridge plans, had an average yield strength of 49 ksi and an average tensile strength of 78.5 ksi. The tensile stress-strain relationship for these samples is shown in Figure 97.

Table 10- Reinforcing Steel Tensile Testing

Bar #	Load (kips)	Diameter (in)	Area (in ²)	Yield (ksi)	Average Yield (ksi)	Strength (ksi)	Average Strength (ksi)
1	16,000	0.5	0.20	52.3	49.3	81.4	78.5
2	14,900	0.5	0.20	46.2		75.6	

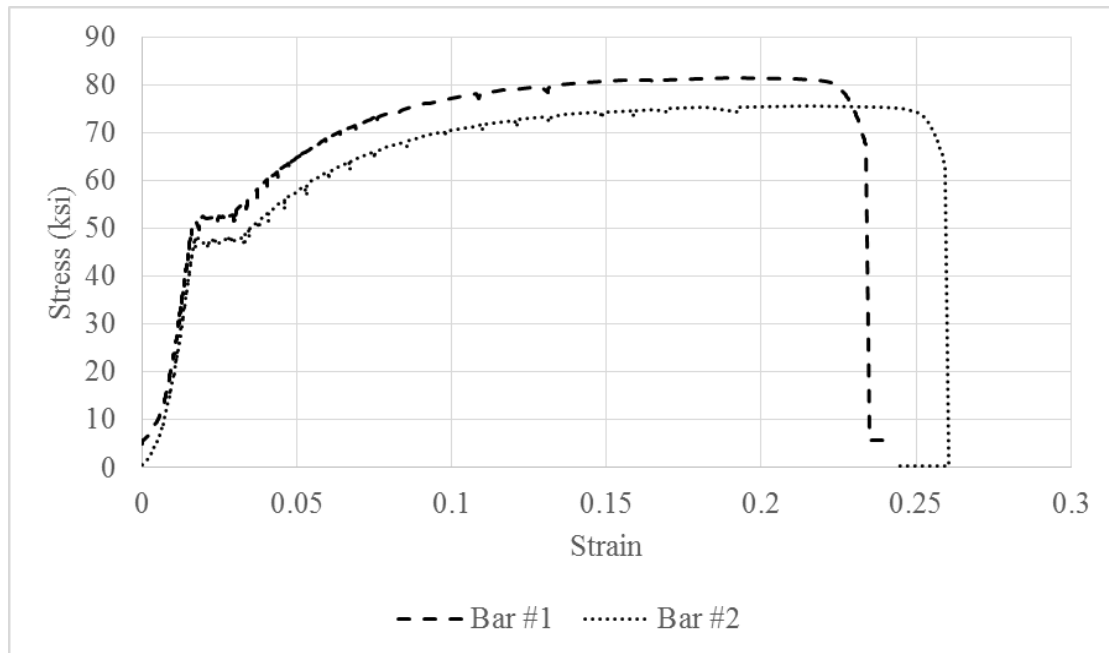


Figure 97- Stress-Strain Relationship for Reinforcing Steel Samples

4.2.4: Prestressing Strand

Prestressing strand samples were collected from the girder and tested in tension for yield strength and ultimate strength. These tests are summarized in Table 11. The bridge plans, shown in APPENDIX A: Route 614 Bridge Blueprints, specify grade 250 stress-relieved prestressing strands. The average yield strength from the samples was 214 ksi and the average ultimate tensile strength was 262 ksi. Figure 98 shows a failure of a strand sample, with the failure mode being the rupture of one wire.

Table 11- Prestressing Strand Tensile Testing

Strand #	Load (lbs)	Diameter (in)	Area (in ²)	Yield (ksi)	Average Yield (ksi)	Strength (ksi)	Average Strength (ksi)
1	20,500	0.375	0.08	218	214	257	262
2	21,400	0.375	0.08	210		267	



Figure 98- Prestressing Strand Rupture at Ultimate Tensile Strength

Table 12 summarizes the results of the effective prestress tests from cutting the strands and measuring the change in length over a portion of the strand to find strain and stress. The results of the second, third, fourth, and sixth strands provide unrealistic strain values, indicating that the extensometer may have slipped. The average stress value from the first and fifth tests was approximately 132 ksi.

Table 12- Effective Prestress Test

Strand #	Initial Length (in)	Length Change (in)	Strain	Effective Prestress Force (ksi)
1	2.0104	0.0102	0.0051	137
2	2.0085	0.0191	0.0095	257
3	2.0057	0.031	0.0155	417
4	2.0131	-0.0012	-0.0006	-16.1
5	2.0071	0.0095	0.0047	128
6	2.0059	0.0254	0.0127	342

The effective prestress was also calculated using both AASHTO Lump Sum Losses and a detailed evaluation of the prestress losses. Table 13 shows a comparison of the experimental

value of the effective prestress with values from both calculation methods. As shown in the table, the experimental results match up well with the lump sum estimates of prestress losses, though the effective prestress value found from the detailed estimation of prestress losses is lower than the experimental value. The predictions show that the experimental value found for the effective prestress, 132 ksi, is a reasonable value and one that is appropriate for use in analyses on the girders.

Table 13- Effective Prestress Results and Predictions

Experimental Value (ksi)	Lump Sum (ksi)	Detailed Estimate (ksi)
132	130	114

4.2.5: Grabb-it Splice Chuck

Tests were performed on Grabb-it splice chucks to verify the relationship between torque and restored prestress. Strands used for these tests were Grade 270 strand with a diameter of 0.375 in, while the strands in the girders were 0.375 in diameter Grade 250 strand. Table 14 shows the results from the splice chuck torque tests (Bao, 2015). The theoretical values are provided by the manufacturer and are found in Table 2. The torque values showed a poor correlation to stress values in the strands as predicted by the manufacturer at high tensile strengths, as shown in Figure 119. In the installation of strand splices for repairs on Test 2, Test 4, and Test 6, the applied torque was 150 ft-lbs with a desired stress of 200 ksi, or 80% of the ultimate tensile strength of the strand. As shown in Figure 99, the prestress restored in these installations likely reached about 163 ksi, or 65% of the ultimate tensile strength of the strand. Following each torque test, the strand splice was stressed in tension until rupture to determine the failure mode. In all three tension tests, failure occurred in the strands prior to failing in the splice chuck, with results shown in Table 15. While it is conservative to assume that the strand splice

system can reach 85% of the ultimate tensile strength of the strand, these results show that the splices can reach 100% of the ultimate tensile strength of the strand.

Table 14- Strand Splice Torque Test Results

Torque (ft-lbs)	Test 1 (ksi)	Test 2 (ksi)	Test 3 (ksi)	Average (ksi)	Theoretical (ksi)	% Average Tested vs. Theoretical
40	57	69	78	68	54	127%
50	68	87	86	80	67	120%
75	87	101	110	99	101	99%
100	114	126	130	123	134	92%
125	130	144	145	140	168	83%
150	145	172	171	163	201	81%

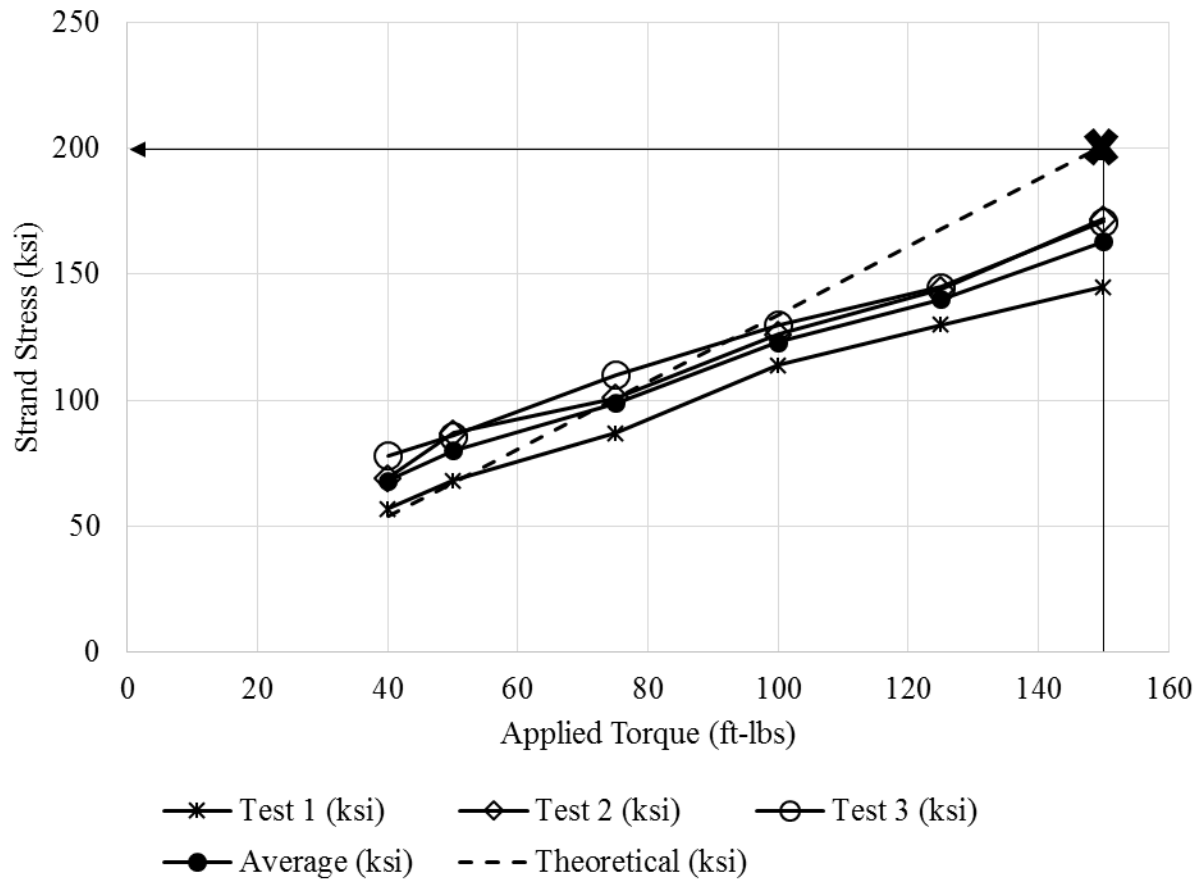


Figure 99- Strand Splice Applied Torque versus Tension Relationship

Table 15- Strand Splice Tension Test

Test #	Load (lbs)	Length Chuck-to-Chuck (ft)	Stress (ksi)	Failure
1	23300	16	291	Strand Fracture
2	22800	6	285	Strand Fracture
3	24000	4	299	Strand Fracture
Average:	23300		292	

4.3: Analytical Modeling

4.3.1: Calculations Based on AASHTO Guidelines

The nominal moment and shear capacities of each girder were determined using the provisions of AASHTO LRFD Bridge Design Specifications (American Association of State Highway and Transportation Officials, 2012). The nominal moment capacity of a prestressed concrete girder is determined in AASHTO by using equilibrium to determine the depth of the neutral axis and the resulting forces in the concrete and the strands, with the compression of the concrete compression block equaling the tensile force in the strands. The tensile force of each of the prestressing strands is assumed to act as a single force acting at the centroid of the strands, calculated using AASHTO equation 5.7.3.1.1-1. The compressive force in the concrete is determined using the Whitney Stress Block approximation, which simplifies the stress distribution as a rectangular block. After finding the neutral axis depth, the nominal moment is calculated with AASHTO equation 5.7.3.2.2-1.

The cracking moment is an important consideration in the design of prestressed concrete bridge girders. The concrete in the tensile region is designed to remain uncracked at service loads, as this efficiently uses the entire concrete cross section and helps prevent corrosion of the prestressing strands and mild reinforcement. The cracking moment is found using AASHTO

equation 5.7.3.3.2-1. Table 16 summarizes the predictions based on AASHTO equations for the cracking and nominal moments.

Table 16- AASHTO Calculation Summary

Test	Description	Number of Severed Strands	M_{cr} (k-ft)	M_n (k-ft)
1	Control Test	0	1954	2952
2	8-Strand Splice	8	1709	2776
4	4-Strand Splice	4	1816	2744
5	4-Strand FRCM	4	1578	2672
6	4-Strand FRCM with Splice	4	1711	2844

To model the damage for Test 2, Test 4, Test 5, and Test 6, the total area of prestressing strands was reduced by the number of strands severed and the centroid of the prestress force was moved. If the strands were spliced and re-tensioned, an additional tensile force based on the manufacturing prestress of 160 ksi was added at the centroid of the splices. For the FRCM repairs, a tensile force was added at the centroid of the FRCM repair, with the effective force based on the strain in proportion to the neutral axis and the strain in the prestressing strands.

The shear capacity of each girder was determined using the simplified procedure from the AASHTO LRFD Bridge Design Specifications (American Association of State Highway and Transportation Officials, 2012). The simplified procedure, similar to the method used in ACI 318-11 (American Concrete Institute, 2011), involves calculating the shear strength in the concrete based on flexure-shear strength (V_{ci}) and web-shear strength (V_{cw}), with the lower of the two values controlling at a particular section. V_{ci} is found using AASHTO equation 5.8.3.4.3-1, and V_{cw} is found using equation 5.8.3.4.3-3. The total shear strength at a section is found by adding the steel contribution to the shear strength from stirrups with the concrete contribution. If some of the prestressing strands are harped at a particular section, the vertical

component of the prestressing force, V_p , also contributes to the shear strength, though in the simplified procedure, V_p is included in the V_{cw} formula. The contribution to the shear strength from the steel stirrups, V_s , is found using AASHTO equation 5.8.3.3-4

4.3.2: Moment-Curvature Based on Strain Compatibility

A moment-curvature analysis was performed for each girder to determine how the girder would perform as the load was applied. This analysis included six points: no applied moment, decompression of the concrete at the bottom flange, first flexural cracking, and the top fiber concrete strain of 0.001, 0.002, and 0.003, the assumed point at which the concrete at the top of the compression block crushes. The stress-strain relationship of the prestressing strands was obtained from material property testing, shown in Figure 100. An idealized bilinear approximation of the stress-strain results was developed by Gangi (2015) and used for the finite element modeling. The modified Hognestad model was used for the concrete stress-strain relationship (Hognestad, 1956).

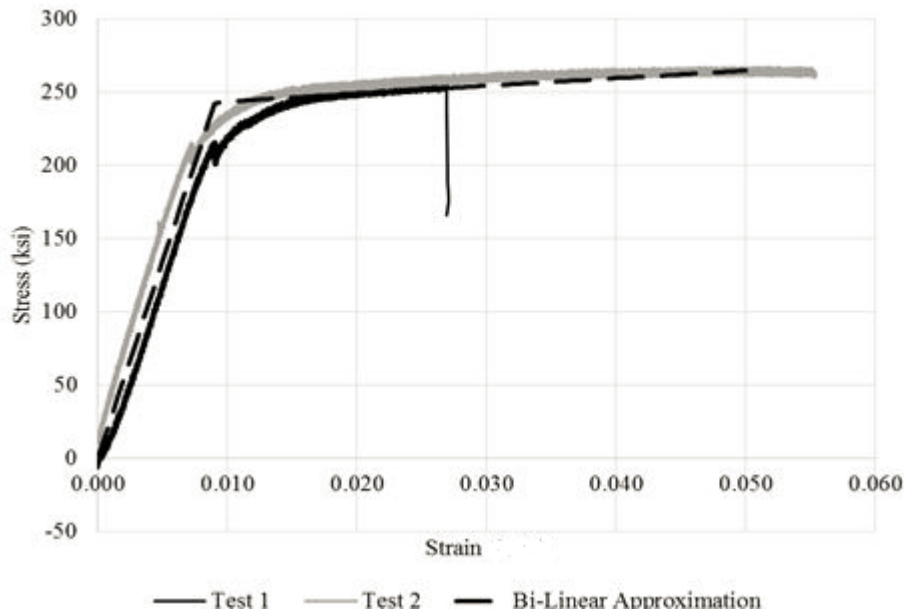


Figure 100- Prestressing Steel Stress-Strain Relationship

The nominal moments calculated using strain compatibility are summarized in Table 17. The moment-curvature behavior for each test is shown in Figure 101. Example moment-curvature calculations are found in APPENDIX C: Moment-Curvature Calculations. For these analyses, instead of using the tested girder concrete tensile strength, equation 9-10 in ACI 318 was used. Since the tested tensile strength was lower than anticipated with a small sample size, it was thought that the ACI equation may be more accurate.

Table 17- Moment-Curvature Calculation Summary

Test	Description	Number of Severed Strands	M_{cr} (k-ft)	M_n (k-ft)
1	Control Test	0	2150	2928
2	8-Strand Splice	8	2045	2721
4	4-Strand Splice	4	2057	2707
5	4-Strand FRCM	4	1868	2712
6	4-Strand FRCM with Splice	4	2028	2888

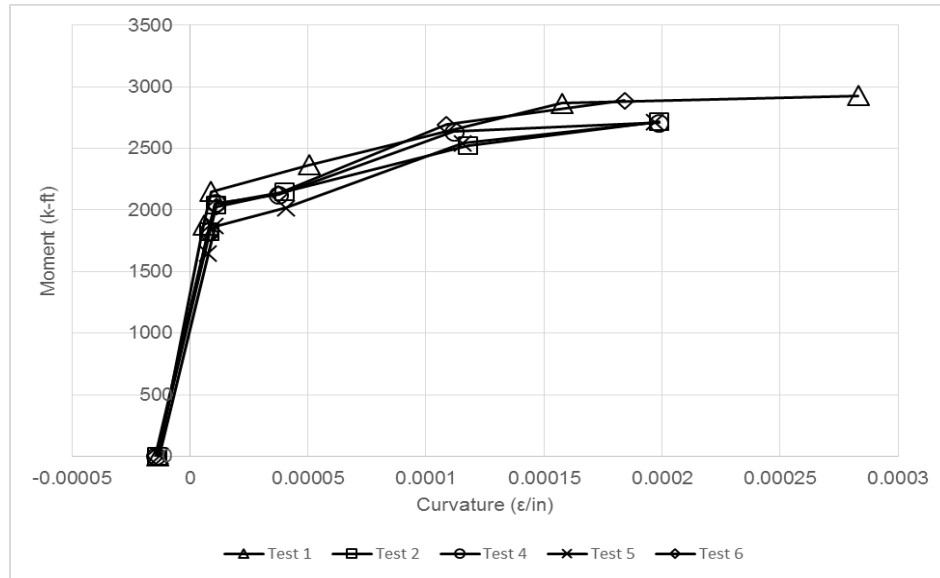


Figure 101- Calculated Moment-Curvature Behavior

One way to measure ductility is by using curvature, using the ratio of the curvature at nominal moment capacity to the curvature at first yield of the reinforcing steel. As seen in Figure 100, prestressing strand has less of a well-defined yield point as mild reinforcement, though a strain of 0.01 can be assumed to be the strain at yield. From the moment curvature analysis, the curvature ductility for each test was calculated, as shown in Table 18. A typical curvature ductility value for a tension controlled failure is between 2 and 2.5. This table shows that each of the tests, with the exception of the Control Test, was expected to be compression controlled. This is because each girder did not have a full composite deck, though the Control Test had the largest deck.

Table 18- Curvature Ductility of each Test

Test	Description	Number of Severed Strands	Curvature at Yield, ϕ_y (in ⁻¹)	Curvature at Ultimate, ϕ_u (in ⁻¹)	Curvature Ductility (ϕ_u/ϕ_y)
1	Control Test	0	$133 * 10^{-6}$	$283 * 10^{-6}$	2.13
2	8-Strand Splice	8	$140 * 10^{-6}$	$199 * 10^{-6}$	1.42
4	4-Strand Splice	4	$140 * 10^{-6}$	$199 * 10^{-6}$	1.41
5	4-Strand FRCM	4	$142 * 10^{-6}$	$197 * 10^{-6}$	1.38
6	4-Strand FRCM with Splice	4	$144 * 10^{-6}$	$184 * 10^{-6}$	1.28

4.3.3: Non-Linear Beam Analysis

A non-linear finite element analysis of the girder for each test was performed using the finite element program OpenSees (Gangi, 2015). The material properties used for the non-linear finite element analyses were obtained from material testing. For each test, model simulations were run for the undamaged and damaged conditions in addition to the repaired girder. The nominal moment capacities obtained from OpenSees are shown in Table 19, along with the predicted failure mode (Gangi, 2015). The girder failed in flexure for each test, with compressive strain in the top fiber of the concrete deck exceeding 0.003 being the predicted failure mode for Test 1, Test 4, Test 5, and Test 6. In Test 2, a flexural tension failure was predicted with a strand splice rupture at a stress of 225 ksi (Gangi, 2015).

Table 19- Finite Element Analysis Results

Test	Tested Repair	Number of Severed Strands	M_{nFF} (k-ft)	Failure Mode
1	Control Test	0	3093	Concrete Crushing at 0.003 in/in
2	8-Strand Splice	8	2798	Strand Splice Rupture at 225 ksi
4, Iteration 1	4-Strand Splice	4	3180	Concrete Crushing at 0.003 in/in
4, Iteration 2	4-Strand Splice	4	3193	Concrete Crushing at 0.003 in/in
5	4-Strand FRCM	4	2900	Concrete Crushing at 0.003 in/in
6	4-Strand FRCM & Strand Splice	4	3130	Concrete Crushing at 0.003 in/in

4.4: Laboratory Tests

4.4.1: Test Results Overview

The results of the flexural tests on the girders are summarized in Table 20. The tested moment capacity is included for each test along with the predicted moment capacity using AASHTO, moment-curvature, and nonlinear finite element analysis. The results for the cracking

moment for each girder test is shown in Table 21, along with the predicted cracking moment using AASHTO, moment-curvature, and nonlinear finite element analysis.

Table 20- Nominal Moment Summary

Test	Description	Number of Severed Strands	M_n , AASHTO (k-ft)	M_n , M- ϕ (k-ft)	M_n , Finite Element (k-ft)	M_n , Actual (k-ft)
1	Control Test	0	2952	2928	3093	3223
2	8-Strand Splice	8	2776	2721	2798	2798
4	4-Strand Splice	4	2744	2707	3193	3163
5	4-Strand FRCM	4	2672	2712	2900	2798
6	4-Strand FRCM with Splice	4	2844	2888	3130	3304

Table 21- Cracking Moment Summary

Test	Description	Number of Severed Strands	M_{cr} , AASHTO (k-ft)	M_{cr} , M- ϕ (k-ft)	M_{cr} , Actual (k-ft)
1	Control Test	0	1954	2150	2197
2	8-Strand Splice	8	1709	2045	1077
4	4-Strand Splice	4	1816	2056	1753
5	4-Strand FRCM	4	1578	1868	2292
6	4-Strand FRCM with Splice	4	1711	2028	2180

4.4.2: Test 1- Control Test

The control test, with a span of 35 ft, reached an applied load of 401 kips, for a maximum applied moment of 3070 k-ft and a moment (including self weight of the girder and deck, approximately 1.0 kips/ft) of 3223 k-ft. The maximum deflection was 1.85 in on the second trial, not including 0.20 in of residual deflection after the first trial. The cracking load was approximately 267 kips, for an applied cracking moment of 2050 k-ft and a total cracking moment of 2197 k-ft.

At 401 kips, the actuator reached its maximum capacity, so the control test had to be concluded without a girder failure. Cracking patterns between and around the load points were consistent with impending flexural failure, with crack patterns outside the load points consistent

with a shear failure, though not necessarily an impending one. Flexural cracks are shown in Figure 102. The failure load predicted from non-linear finite element analysis was 403 kips with concrete crushing as the failure mode, and a deflection of 1.81 in (Gangi, 2015). Figure 103 shows the measured moment versus deflection curve and compares it to the predicted moment capacities found using AASHTO and moment-curvature. Since the girder had already cracked during the first trial, the second test trial has less stiffness as it nears the original cracking moment. Table 22 compares the cracking moment and the nominal moment obtained from the test with predictions using AASHTO, moment-curvature, and finite element modeling. This table also includes a ratio of the predicted moments divided by the experimental moments. The predictions based on each analysis method are conservative in estimating both the ultimate moment capacity and the cracking moment. The moment capacity calculated from this test was used for comparison between an undamaged girder and that damaged girders that were repaired in ensuing tests.



Figure 102- Flexural Cracking in the Bottom Flange and into the Web

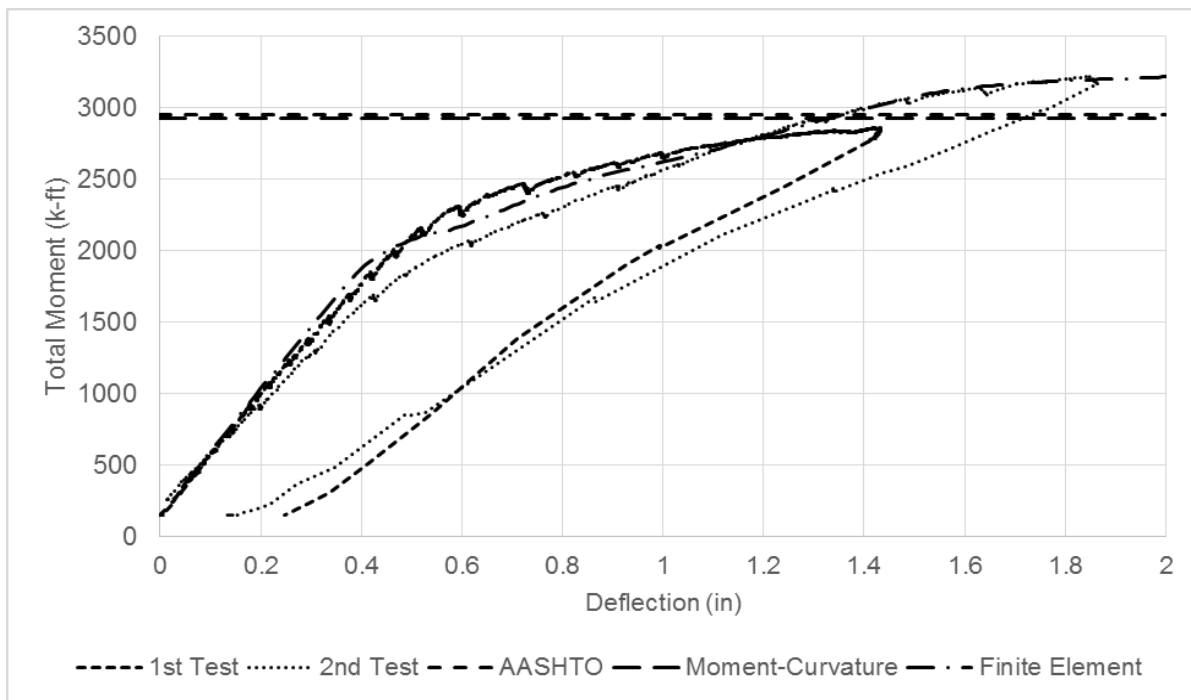


Figure 103- Test 1 Moment versus Deflection Behavior

Table 22- Control Test Cracking and Ultimate Moment Experimental Data versus Analytical Data

	Cracking Moment (k-ft)	Ultimate Moment (k-ft)	Mcr Ratio	Mn Ratio
Tested	2197	3223	-	-
AASHTO	1954	2952	0.89	0.92
Moment-Curvature	2150	2928	0.98	0.91
Finite Element	-	3093	-	0.96

4.4.3: Test 2- Girder with Eight Severed/Re-Tensioned Strands

Test 2, with a span of 58 ft, reached an applied load of 197 kips, for a maximum applied moment of 2470 k-ft and a moment (including self weight of the girder and deck, approximately 0.82 kips/ft) of 2798 k-ft. The maximum deflection was 4.12 in. The cracking load was approximately 60 kips, for an applied cracking moment of 750 k-ft and a total cracking moment of 1077 k-ft.

The failure mode of Test 2 was a flexural tensile failure in which one repaired strand slipped out of its splice chuck and one strand ruptured. Cracks formed and widened in the repair area, showing signs of failure prior to reaching the ultimate load and providing warning of an imminent failure. Figure 104 shows the repair concrete at failure. After completion of the test, the repair concrete was chipped out to investigate the performance of the strand splices and observe the failure. Figure 105 shows the failure, with one repaired strand slipping out of the splice chuck and another prestressed strand rupturing. The picture was taken after the girder was removed from the testing area and stored outside for a period of time, causing rust to form. Many of the spliced strands experienced significant yielding, with permanent deformation evident.



Figure 104- Flexural Cracks in Patch Concrete at Failure



Figure 105- Test 2 Failed Splice and Ruptured Strand

The moment versus deflection curve of the test is shown in Figure 106 and compared with predictions using AASHTO, moment-curvature, and finite element modeling. Table 23 compares the cracking moment and the nominal moment obtained from the test data with predictions using each analysis method. Each method accurately predicted the nominal moment capacity of the girder. The lower cracking moment in the experimental data is likely due to the

repair concrete not being precompressed during installation. Adding a positive moment during installation of the repair material and then releasing it adds a precompressive force to the repair material and prevents premature cracking.

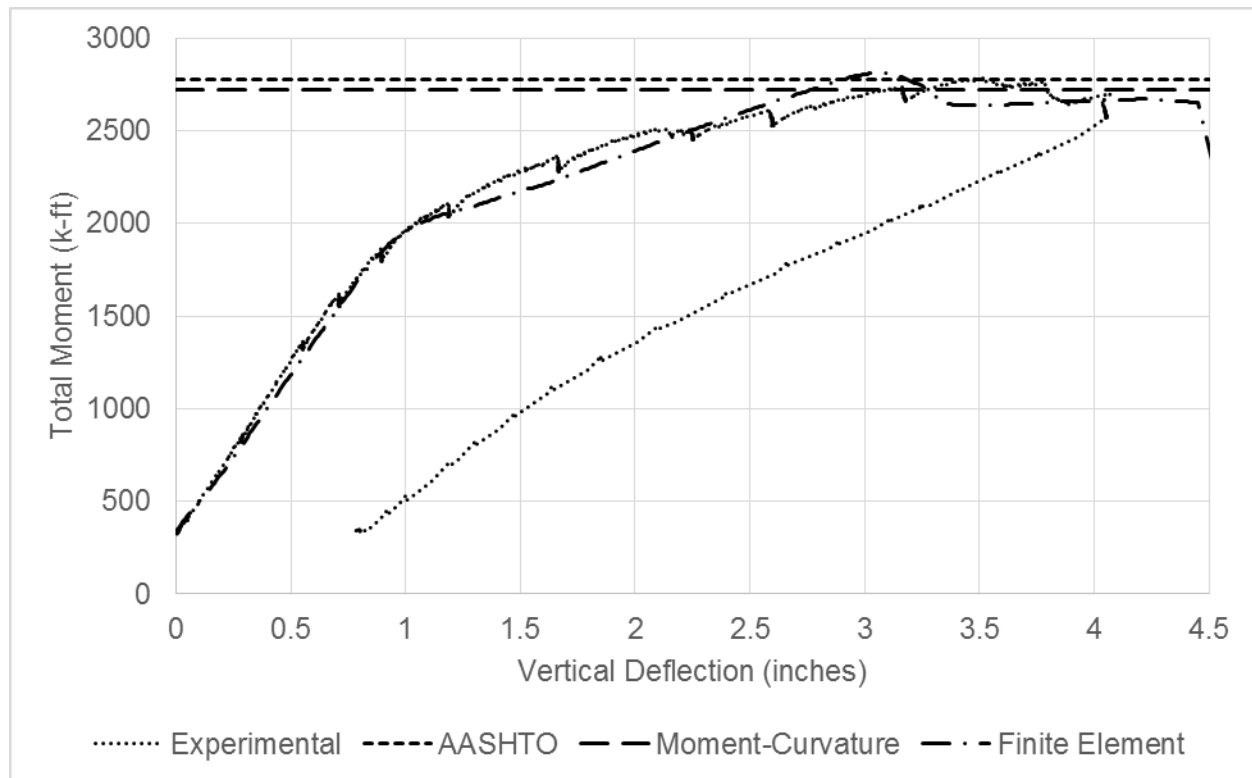


Figure 106- Test 2 Moment versus Deflection Behavior

Table 23- Test 2 Cracking and Ultimate Moment Experimental Data versus Analytical Data

	Cracking Moment (k-ft)	Ultimate Moment (k-ft)	Mcr Ratio	Mn Ratio
Tested	1077	2798	-	-
AASHTO	1709	2776	1.59	0.99
Moment-Curvature	2045	2720	1.90	0.97
Finite Element	-	2798	-	1.00

4.4.4: Test 4- Girder with Four Severed/Re-Tensioned Strands

Test 4 was executed with a span of 58 ft for the first iteration and 50 ft for the second iteration. The first iteration of the test reached an applied load of 241 kips and a deflection of

4.12 in, for an applied moment of 2921 k-ft and a moment of 3213 k-ft, including a self weight of approximately 0.78 kips/ft. The second iteration of the test reached an applied load of 266 kips and a deflection of 3.61 in, for an applied moment of 2931 k-ft and a moment of 3163 k-ft including self weight. Cracking was observed for the first iteration at an applied load of approximately 120 kips for an applied cracking moment of 1450 k-ft and a total cracking moment of 1753 k-ft.

The failure mode of Test 4 was a flexural compression failure in which the concrete in the deck between the load points crushed. This failure is shown in Figure 107. Flexural cracking was observed along the bottom flange, with widening cracks as the load increased near the ultimate capacity, as shown in Figure 108 and Figure 109. After completion of the test, the repair concrete was chipped out to investigate the performance of the strand splices. The splices showed no observable slipping, rutpuring, or excessive yielding.

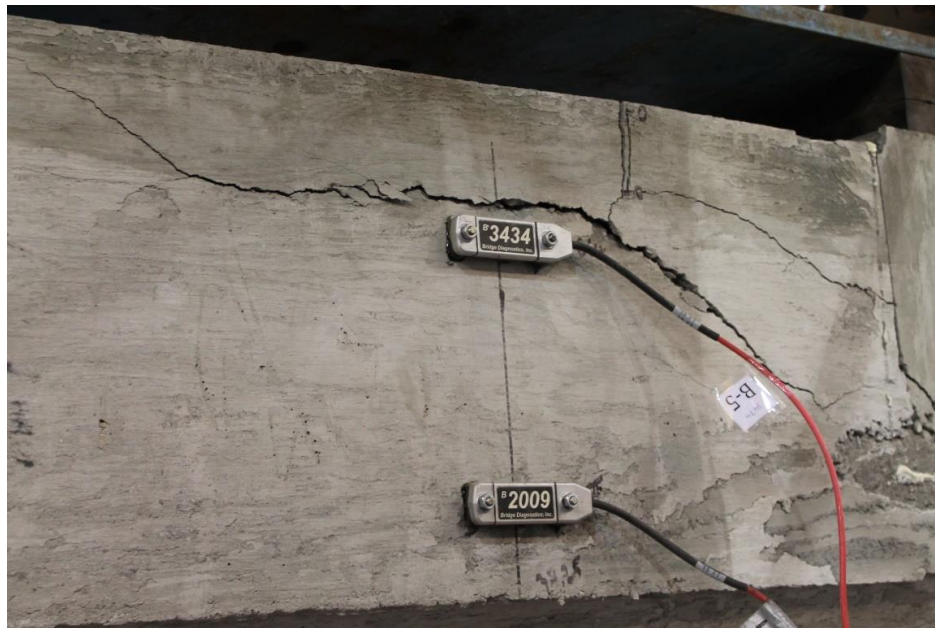


Figure 107- Test 4 Flexural Compression Failure in the Deck



Figure 108- Test 4 Cracking Pattern at the Load Points during Iteration 1

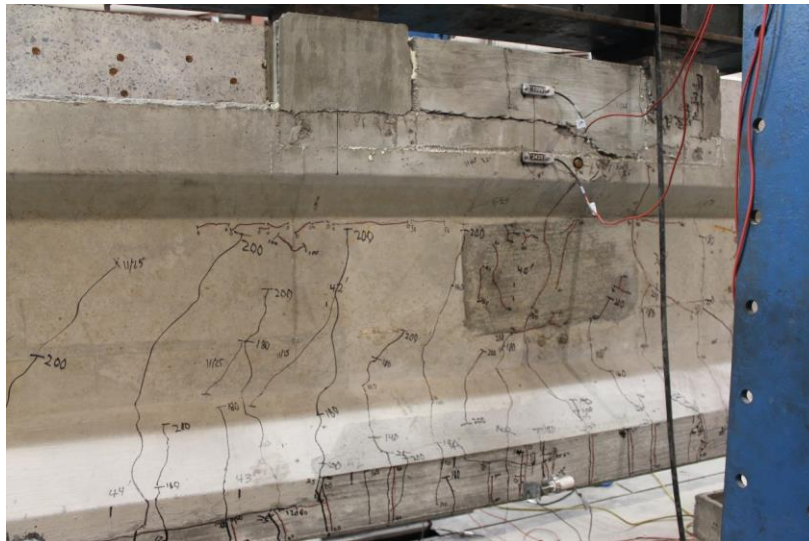


Figure 109- Test 4 Cracking Patterns near Maximum Load

The moment versus deflection curve for the first iteration is shown in Figure 110 and compared with predictions using AASHTO, moment-curvature, and finite element modeling. Figure 111 shows the moment versus deflection curve for the second iteration and compares it to

predictions. Table 24 compares the cracking moment and nominal moment obtained from test data with predictions using each analysis method. The finite element analysis method accurately predicted the nominal moment capacity of the girder, while the AASHTO and moment-curvature methods were conservative. Similar to Test 2, each method overpredicted the cracking moment due to the lack of precompression in the repair concrete.

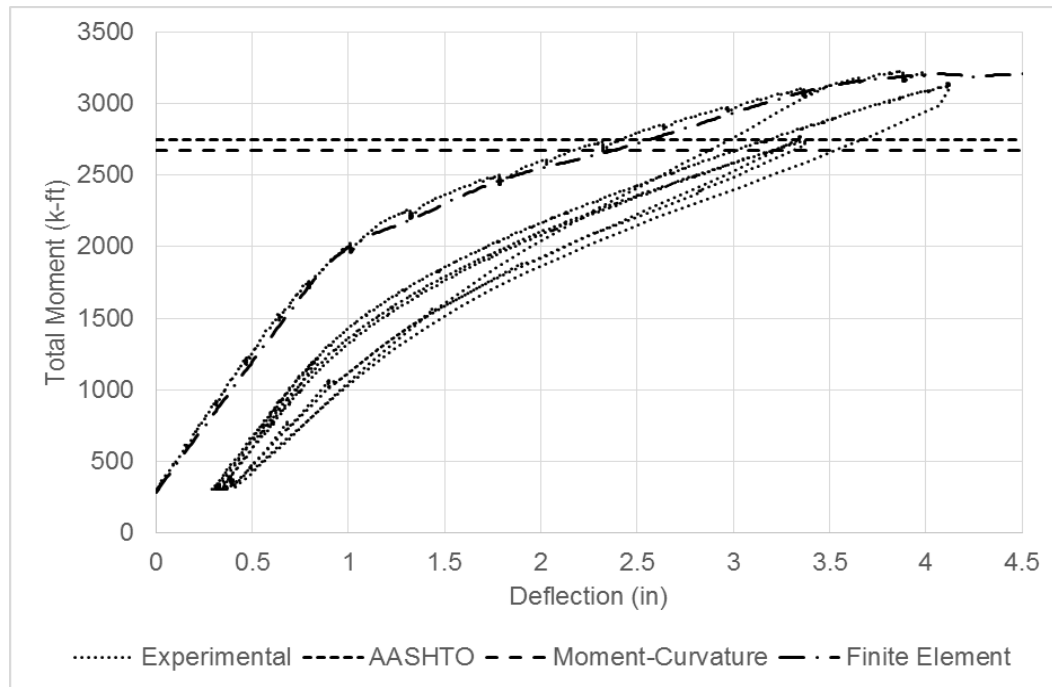


Figure 110- Test 4, Iteration 1 Moment Deflection Behavior

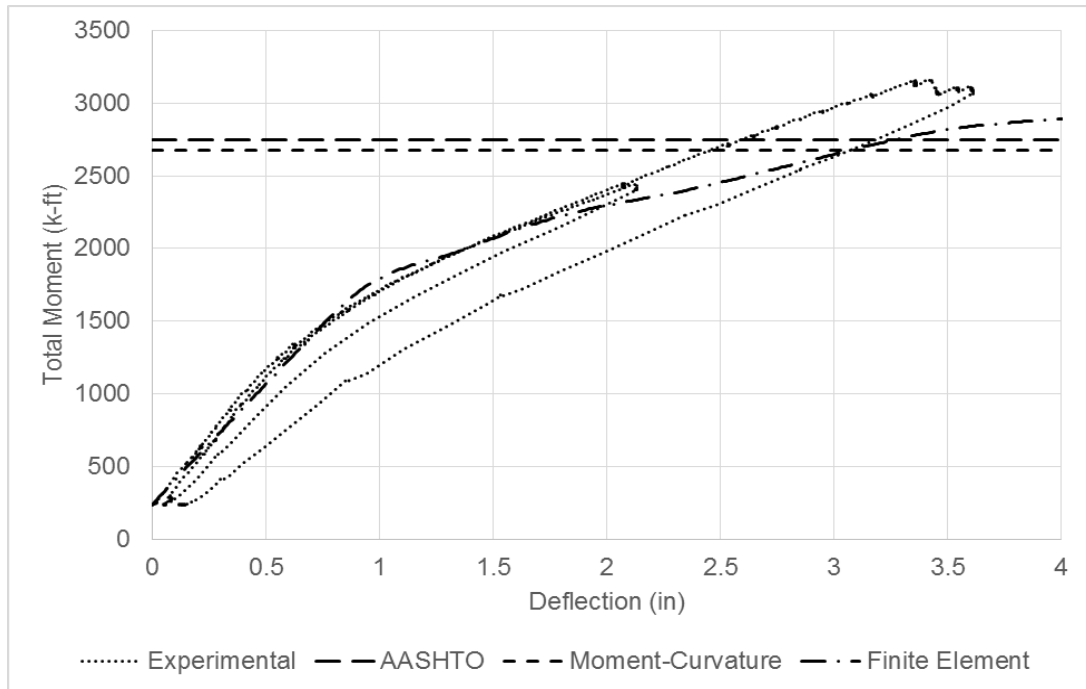


Figure 111- Test 4, Iteration 2 Moment Deflection Behavior

Table 24- Test 4 Cracking and Ultimate Moment Experimental Data versus Analytical Data

	Cracking Moment (k-ft)	Ultimate Moment (k-ft)	Mcr Ratio	Mn Ratio
Tested	1753	3163	-	-
AASHTO	1816	2744	1.04	0.87
Moment-Curvature	2056	2707	1.17	0.86
Finite Element	-	3193	-	1.01

4.4.5: Test 5- FRCM Repaired Girder with Four Severed Strands

Test 5, with a span of 52 ft, reached an applied load of 222 kips, for a maximum applied moment of 2514 k-ft and a moment of 2768 k-ft including the self weight of the girder, 0.78 kips/ft. The total deflection at the center of load was 3.02 in. Cracking was observed at an applied load of approximately 180 kips for an applied cracking moment of 2038 k-ft and a total cracking moment of 2293 k-ft.

The girder failed in horizontal shear transfer due to saw cut gaps along the length of the girder from demolition of the bridge. These saw cuts split the composite deck into different lengths and restricted the deck's ability to act compositely with the girder. Cracking occurred in the deck but cracks retreated below the saw cuts into the girder, creating a stress concentration. The failure occurred suddenly with concrete crushing in the interface of the deck and the girder, shown in Figure 112.



Figure 112- Test 5 Horizontal Shear Transfer Failure

The moment versus deflection curve for the Test 5 is shown in Figure 113 and compared with predictions using AASHTO and finite element modeling. Table 25 compares the cracking moment and nominal moment obtained from test data with predictions using each analysis method. The finite element analysis method overpredicted the nominal moment capacity of the girder, while the AASHTO method was conservative. Possibly due to the FRCM material covering up cracks in the girder, the estimate of the cracking load from the AASHTO method was very low.

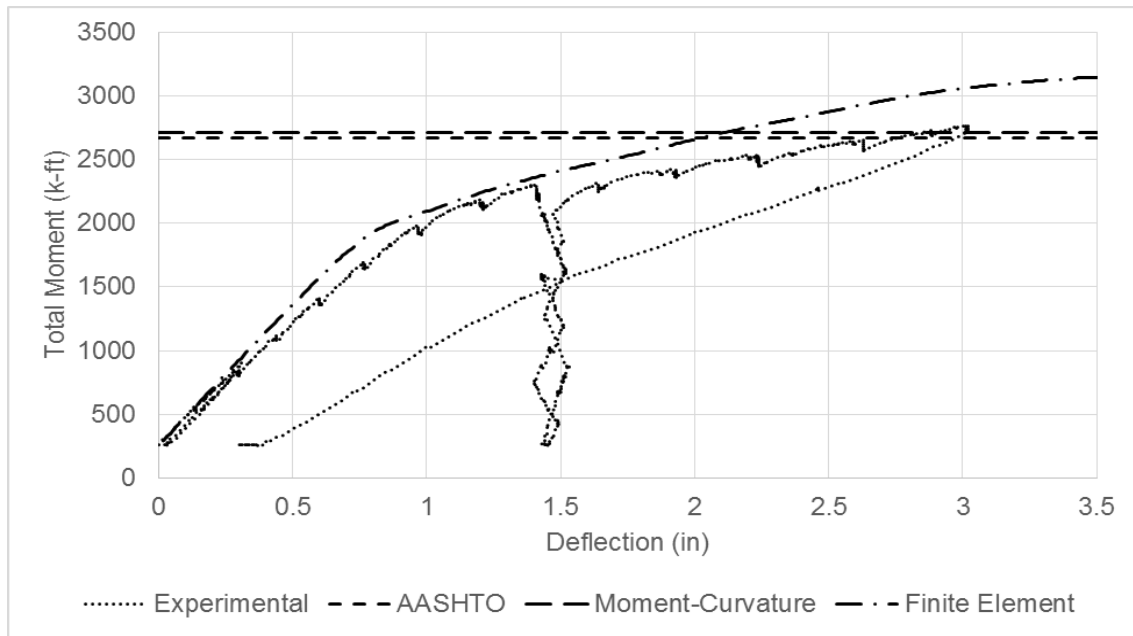


Figure 113- Test 5 Moment versus Deflection Behavior

Table 25- Test 5 Cracking and Ultimate Moment Experimental Data versus Analytical Data

	Cracking Moment (k-ft)	Ultimate Moment (k-ft)	Mcr Ratio	Mn Ratio
Tested	2292	2768	-	-
AASHTO	1578	2672	0.69	0.97
Moment-Curvature	1868	2712	0.82	0.98
Finite Element	-	2900	-	1.05

Using data from the strain transducers on the girder, moment-curvature results were developed at the point of maximum moment, 22 feet from the end of the girder. These results were compared with the predicted moment-curvature, found using strain compatibility. The zero point in the moment-curvature results included the curvature from the self weight and prestress, so the results were adjusted to include the initial curvature at the zero point. Figure 114 shows the moment-curvature results versus the prediction. The strain transducers were removed prior to failure due to safety concerns. In the strain transducer data, there was no significant change in slope at the cracking moment, though near the failure moment, the data showed a loss of

stiffness. The strain transducers, measuring change in length, can be influenced by nearby cracking.

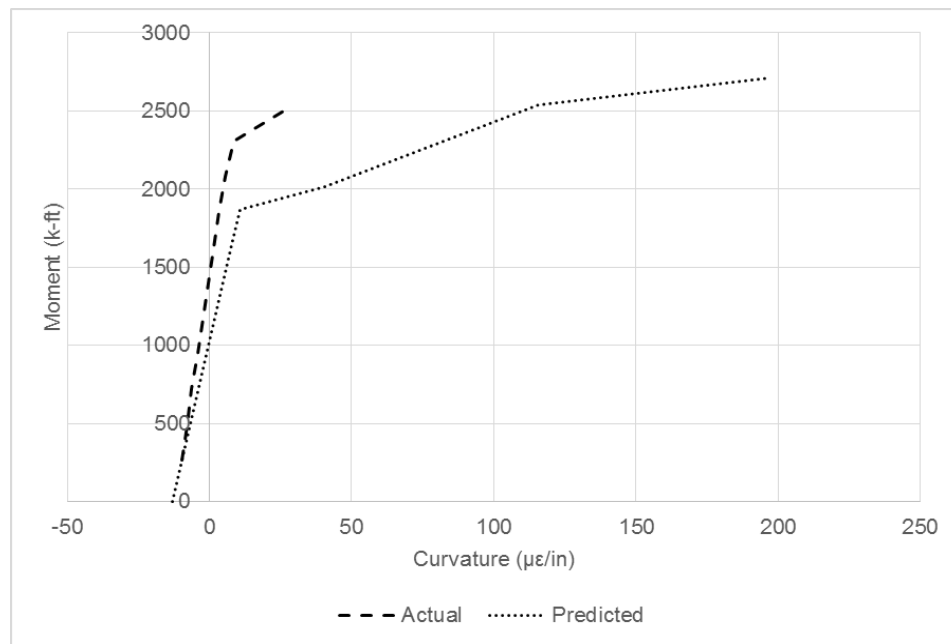


Figure 114- Test 5 Moment-Curvature Data at the Point of Maximum Moment

Strain transducers were placed on the bottom flange to record the behavior of the FRCM during the test. The moment versus measured micro-strain for the two strain transducers on the bottom flange at the point of maximum moment are shown in Figure 115. The figure shows a similar slope for each transducer before cracking and a similar sharp jump after cracking. The maximum strain reached in the transducers was approximately 1400 microstrain.

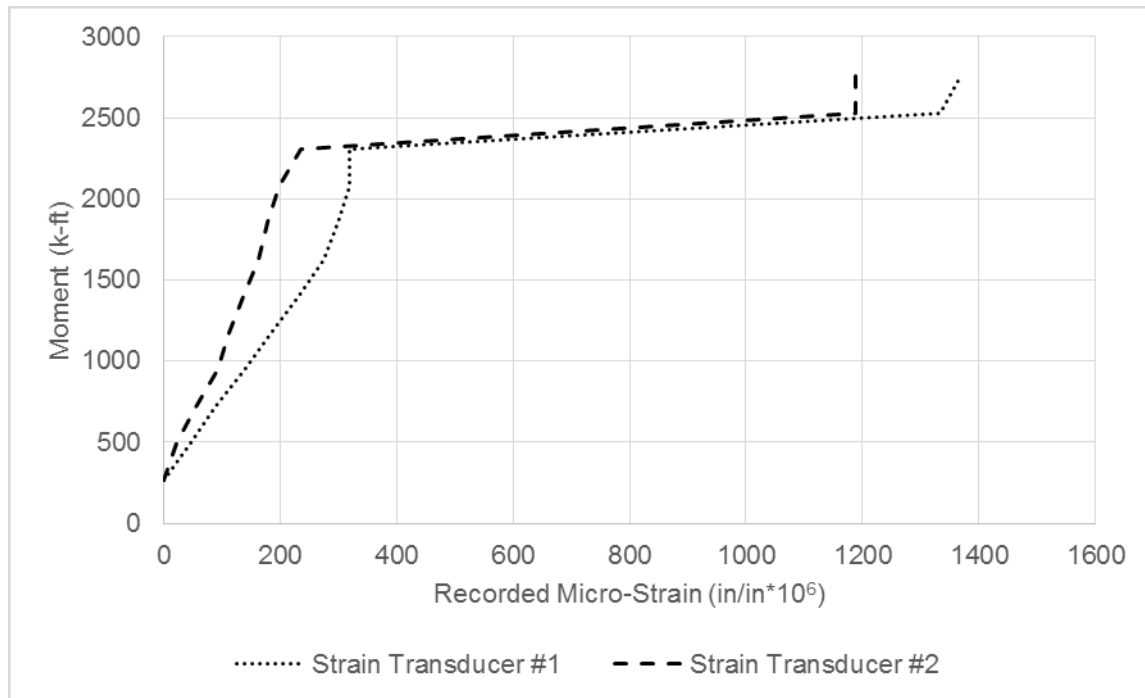


Figure 115- Test 5 Bottom Flange Strain Transducer Data

4.4.6: Test 6- FRCM Repaired Girder with Four Severed/Re-Tensioned Strands

Test 6, with a span of 52 ft, reached an applied load of 269 kips, for a maximum applied moment of 3050 k-ft and a moment of 3304 k-ft including self weight. The total deflection was 3.62 in. Cracking was heard and then observed at an applied load of approximately 170 kips for an applied cracking moment of 1926 k-ft and a total cracking moment of 2180 k-ft.

The failure mode of Test 6 was a flexural compression failure in which the concrete in the deck between the load points crushed. This failure is shown in Figure 116. Flexural cracking was observed along the bottom flange, with widening cracks as the load increased near the ultimate capacity, as shown in Figure 117.



Figure 116- Compression Failure in the Composite Deck

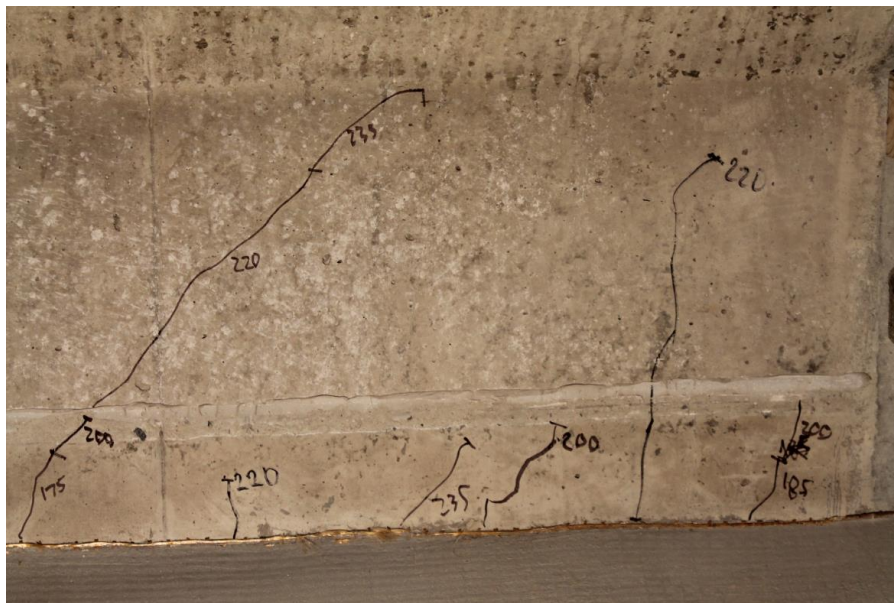


Figure 117- Flexural Cracks Progressing up the Web

The moment versus deflection curve for the Test 6 is shown in Figure 118 and compared with predictions using AASHTO and finite element modeling. Table 26 compares the cracking moment and nominal moment obtained from test data with predictions using each analysis method. Both the finite element analysis method and the AASHTO method underpredicted the

nominal moment capacity of the girder. Possibly due to the FRCM material covering up cracks in the girder, the estimate of the cracking load from the AASHTO method was very low.

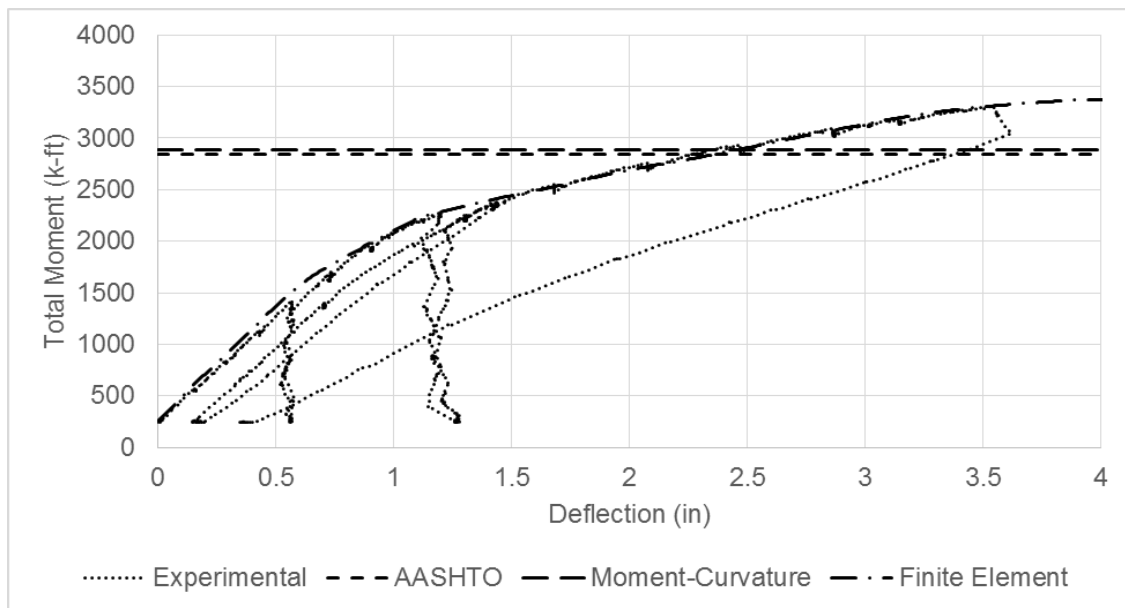


Figure 118- Test 6 Moment Deflection Behavior

Table 26- Test 6 Cracking and Ultimate Moment Experimental Data versus Analytical Data

	Cracking Moment (k-ft)	Ultimate Moment (k-ft)	Mcr Ratio	Mn Ratio
Tested	2180	3304	-	-
AASHTO	1711	2844	0.78	0.86
Moment-Curvature	2028	2888	0.93	0.87
Finite Element	-	3130	-	0.95

Using data from the strain transducers on the girder, moment-curvature results were developed at the point of maximum moment, 22 feet from the end of the girder. These results were compared with the predicted moment-curvature, found using strain compatibility. The zero point in the moment-curvature results included the curvature from the self weight and prestress, so the results were adjusted to include the initial curvature at the zero point. Figure 119 shows the moment-curvature results versus the prediction. The strain transducers were removed prior

to failure due to safety concerns. The results show a loss of stiffness near the cracking moment, but no significant loss of stiffness as the moment increases toward failure. The strain transducers, measuring change in length, can be influenced by nearby cracking.

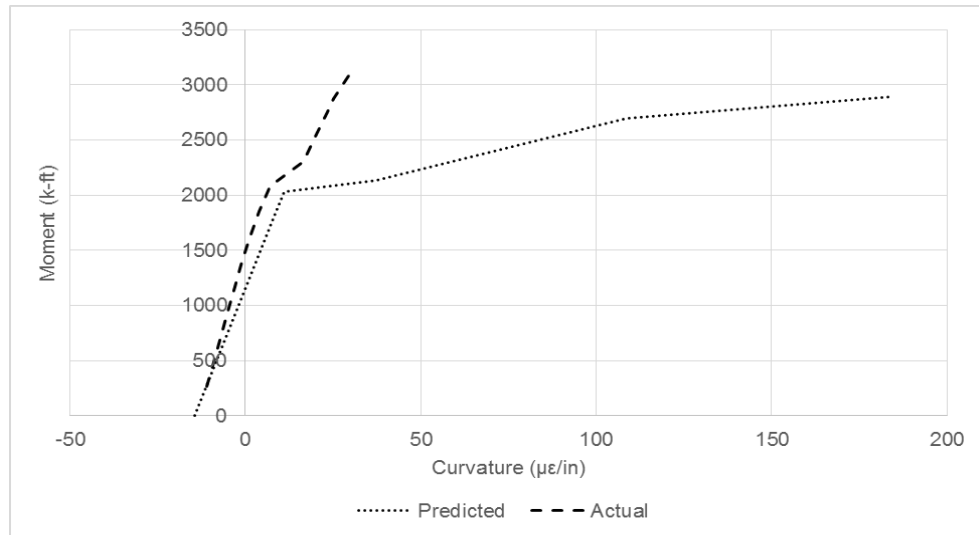


Figure 119- Test 6 Moment-Curvature Data at the Point of Maximum Moment

Strain transducers were placed on the bottom flange to record the behavior of the FRCM during the test. The moment versus measured micro-strain for the two strain transducers on the bottom flange at the point of maximum moment are shown in Figure 120. The figure shows a consistent slope for each transducer before cracking and then a divergence after cracking. The maximum strain reached in the transducers was approximately 800 microstrain.

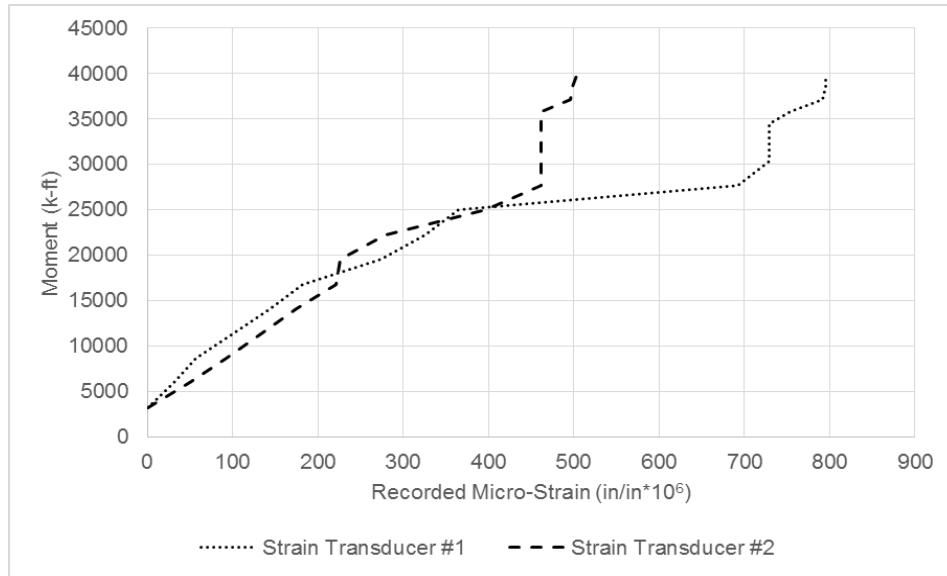


Figure 120- Test 6 Bottom Flange Strain Transducer Data

4.5: Data Analysis

4.5.1: Adjustment of Undamaged Strength to Account for Girder Dimension Variations

The dimensions of the composite deck were not the same for each girder, and therefore not the same for each test. In addition, each test with the exception of Test 2 featured load points that were not between the harping points of the harped strands, meaning that the centroid of the prestressing force was not consistent for each test. These variations, which affect the overall capacity of each test, were accounted for by finding the undamaged nominal capacity using the AASHTO LRFD Bridge Design Specifications for the specific girder dimensions of each test, shown in Table 27. This undamaged capacity was found using the minimum compressive strength of the girder and deck, as specified in the APPENDIX A: Route 614 Bridge Blueprints. This provides a conservative value of the undamaged capacity for each test. The tested moment capacity for each test exceeds the undamaged capacity, showing that each test did successfully restore the capacity to an acceptable level.

Table 27- AASHTO Undamaged Capacity for each Test

Test	Description	Number of Severed Strands	Undamaged Nominal Moment Capacity (k-ft)	Tested Moment Capacity (k-ft)	% of the Undamaged Capacity
1	Control Test	0	2952	3223	109.2%
2	8-Strand Spliced	8	2773	2798	100.9%
4	4-Strand Spliced	4	2744	3163	115.3%
5	4-Strands Severed with FRCM	4	2710	2768	102.1%
6	4-Strands Spliced with FRCM	4	2710	3304	121.9%

4.5.2: FRCM Repaired Girder with Severed/Re-Tensioned Strands and Full Composite

Deck

To analyze the performance of FRCM repaired girders with re-tensioned strands in a realistic girder repair scenario, the moment capacity of a girder with a full composite deck was calculated using the provisions of AASHTO LRFD Bridge Design Specifications for the undamaged case, damaged case, and repaired case for four strands, eight strands, and 12 strands severed. A 60 ft girder was analyzed with the same dimensions and strand layout as Girder B, Girder C, and Girder D with the exception of having deck dimensions of 9.25 in deep and 88 in (7 ft and 4 in) wide, with 88 in being the center to center spacing of girders in the original Route 614 Bridge, shown in APPENDIX A: Route 614 Bridge Blueprints. For the four strands damaged case, the same damage pattern that was used for Test 3, Test 4, Test 5, and Test 6 was used in this analysis, shown in Figure 121. For the eight strands damaged case, the same damage pattern that was used for Test 2 was used in this analysis, shown in Figure 122. For the 12 strands damaged case, a similar damage pattern was used, with four strands severed on each of the first two rows, three strands severed on the third row, and one strand severed on the fourth row, shown in Figure 123. Damage with four strands severed represents an 8% loss of

reinforcement, which would be considered severe damage, but would likely not require an active repair. Damage with eight strands represents a 16 to 17% loss of reinforcement, which may require an active repair to restore capacity in both strength and service limit states. Damage with 12 strands represents a 25% loss of reinforcement, which would require an extensive repair to restore capacity in strength and service limit states. At that level of damage, repair may become impractical and girder replacement may be the best option.

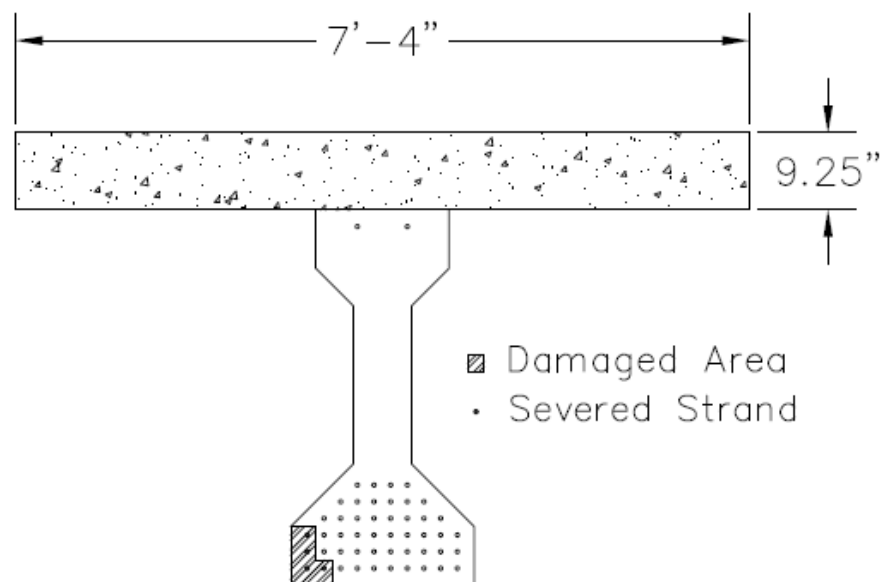


Figure 121- Strand Damage Pattern for Four Strand Composite Deck Analysis

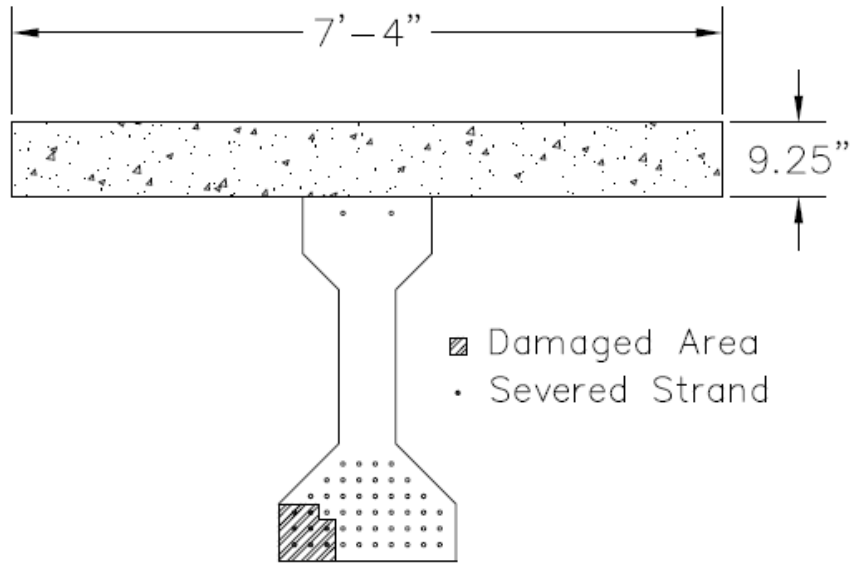


Figure 122- Strand Damage Pattern for Eight Strand Composite Deck Analysis

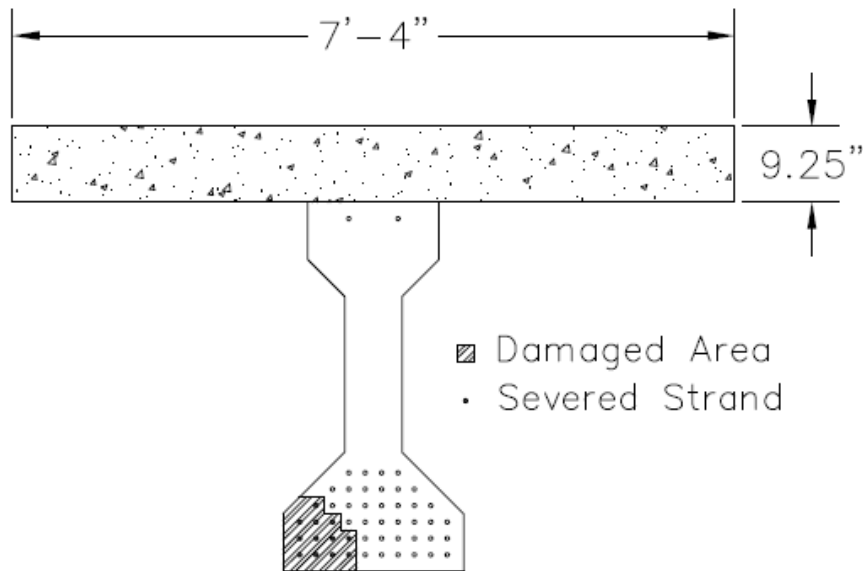


Figure 123- Strand Damage Pattern for 12 Strand Composite Deck Analysis

The FRCM repair modeled in these analyses is similar to the FRCM repairs for Test 5 and Test 6. The model includes four layers of FRCM mesh with the same cross-sectional area as the repairs in Test 5 and Test 6. The girder in this model is repaired with FRCM in the middle one-third of the 60 ft length, for a length of 20 ft. Table 28 summarizes the results, showing the capacity lost by damage and re-gained by the strand splice and FRCM repair. In each case, the

controlling failure mode was debonding of the FRCM material. In the design guidelines of ACI 549, the effective tensile strain level in the FRCM reinforcement is conservatively limited to a strain of 0.012. The undamaged, damaged, and repaired moment capacities along the length of the girder are shown in Figure 124 for the four strand damage pattern, Figure 125 for the eight strand damage pattern, and Figure 126 for the 12 strand damage pattern. The four strand repair restored the capacity significantly beyond the original undamaged capacity. The eight strand repair restored the capacity to slightly less than the undamaged case, restoring nearly all of the moment capacity. The 12 strand repair restored the capacity to 91% of the undamaged case. Each repair restored approximately 16% of the undamaged moment capacity.

Table 28- FRCM and Strand Splice Composite Deck Analysis Summary

Number of Strands Severed	Undamaged Capacity (k-ft)	Damaged Capacity (k-ft)	% Capacity Lost by Damage	Repaired Capacity (k-ft)	% Capacity Regained	% of Undamaged Capacity after Repair
4	3611	3310	8.3%	3889	16.0%	107.7%
8	3611	3010	16.6%	3594	16.2%	99.5%
12	3611	2713	24.9%	3302	16.3%	91.4%

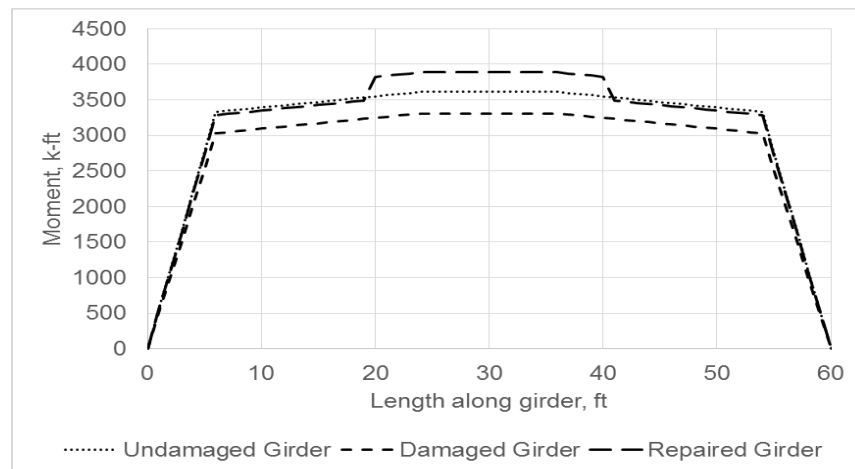


Figure 124- Moment Capacity along the Length of the Girder for the Four Strand Damage Pattern



Figure 125- Moment Capacity along the Length of the Girder for the Eight Strand Damage Pattern



Figure 126- Moment Capacity along the Length of the Girder for the 12 Strand Damage Pattern

CHAPTER V: Conclusions and Recommendations

5.1: Conclusions and Recommendations

This thesis has presented the experimental tests on two prestressed concrete girder repair methods: strand splicing and fabric reinforced cementitious matrix (FRCM). The tests were part of a research project with an overall goal to evaluate repair techniques for impact damaged prestressed concrete bridge girders. Girder A, the Control Test, was a section from an undamaged girder. Girder B, used for Test 2, had eight severed strands that were spliced and re-tensioned with Grabb-it strand splices. Girder C, used for Test 4, had four severed strands that were spliced and re-tensioned with Grabb-it strand splices. Girder D was used for Test 5 and Test 6. Test 5 had four severed strands and was repaired using an FRCM mesh. Test 6 had four severed strands that were spliced and re-tensioned with Grabb-it strand splices and repaired using an FRCM mesh.

The strand splice repairs were successfully able to restore the strength lost by damage when four strands were severed. When eight strands were severed, the strand splice repair was not able to fully restore the strength lost by damage, but was able to restore the girder to a minimum design strength, according to the AASHTO LRFD Bridge Design Specifications, though this was mostly due to the concrete in the girder and deck far exceeding the design compressive strength. Therefore, strand splice repairs are not recommended as the sole repair method if more than 10% of the strands are damaged. This is due to the reduced tensile strength of strands that are spliced. If more strength restoration is required, strand splices can easily be combined with an external repair such as FRP or FRCM. A combined repair with strand splices and an external passive repair is especially useful if both strength and service limit states are a concern.

Concrete cracking is an important consideration when addressing a repair. When performing a concrete patch repair, preloading can be a useful tool to prevent premature cracking of the patch material. Preloading to put the area of the patch in tension during concrete placement, then releasing the preload after the repair concrete reaches a certain strength allows the repair concrete to be precompressed, preventing the early formation of cracks due to loading or other factors such as shrinkage. Premature cracking allows moisture to reach the steel and cause corrosion. This is particularly a concern with strand splice repairs, as the increased diameter of the splice chucks reduces the concrete cover. In testing, the FRCM material shows no noticeable cracking during testing, even at high moments. This means that the FRCM repair would help prevent corrosion of the substrate material.

Externally bonded FRP and FRCM are similar repair methods with a few distinct differences. Both materials have a linear stress-strain curve leading up to failure, though it should be noted that FRCM mesh has a bilinear stress-strain curve with the change of slope at low stresses (though this initial slope this is ignored in design). In design of each repair, debonding may be the controlling failure mode. Debonding between the repair and the substrate is a sudden, brittle failure, and strain limits are conservatively set in the design equations for each material to decrease the likelihood of debonding. Though installation time was similar for each repair (roughly 4 hours for each repair location), FRCM has many advantages in installation over FRP. FRCM installation involves simple troweling techniques and no personal protective equipment (PPE) beyond what is worn for typical concrete application. In addition, the repair is reversible and provides some fire protection.

The amount of damage required to necessitate complete girder replacement depends on state-specific guidelines. This testing showed that, for multiple repair methods, full undamaged

moment capacity can be restored when 8% of the prestressing strands are lost. Though a strand splice repair was unable to restore the girder to undamaged capacity when 16% of the strands were severed, analysis has shown a combined repair such as strand splice and FRCM can restore a girder to undamaged capacity for this damage. Analysis of a girder with 25% of the strands severed showed that a repair is possible to restore the strength and serviceability, though the repair would be extensive and potentially impractical.

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APPENDIX A: Route 614 Bridge Blueprints

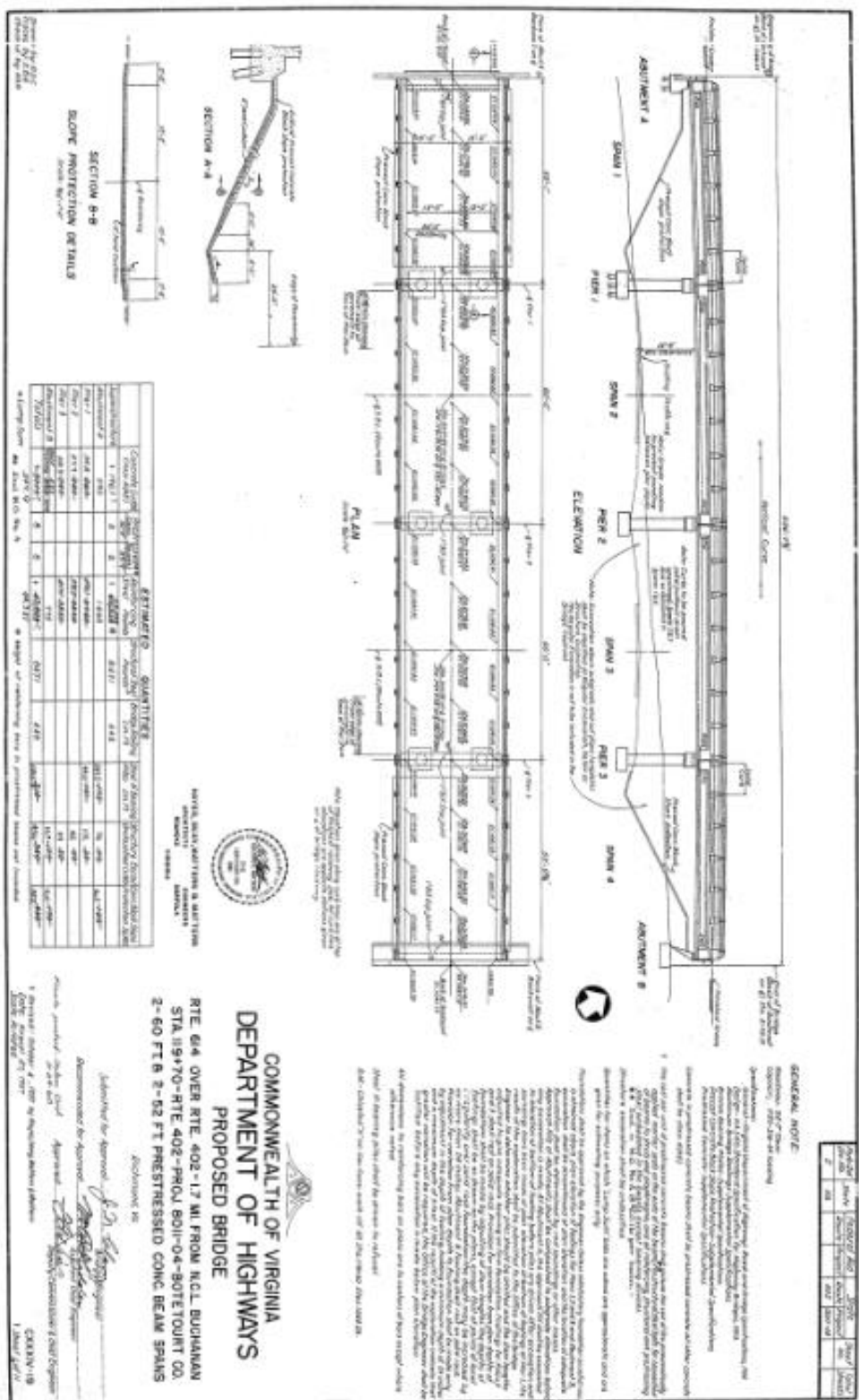


Figure 127- Route 614 Bridge Blueprint Page 1

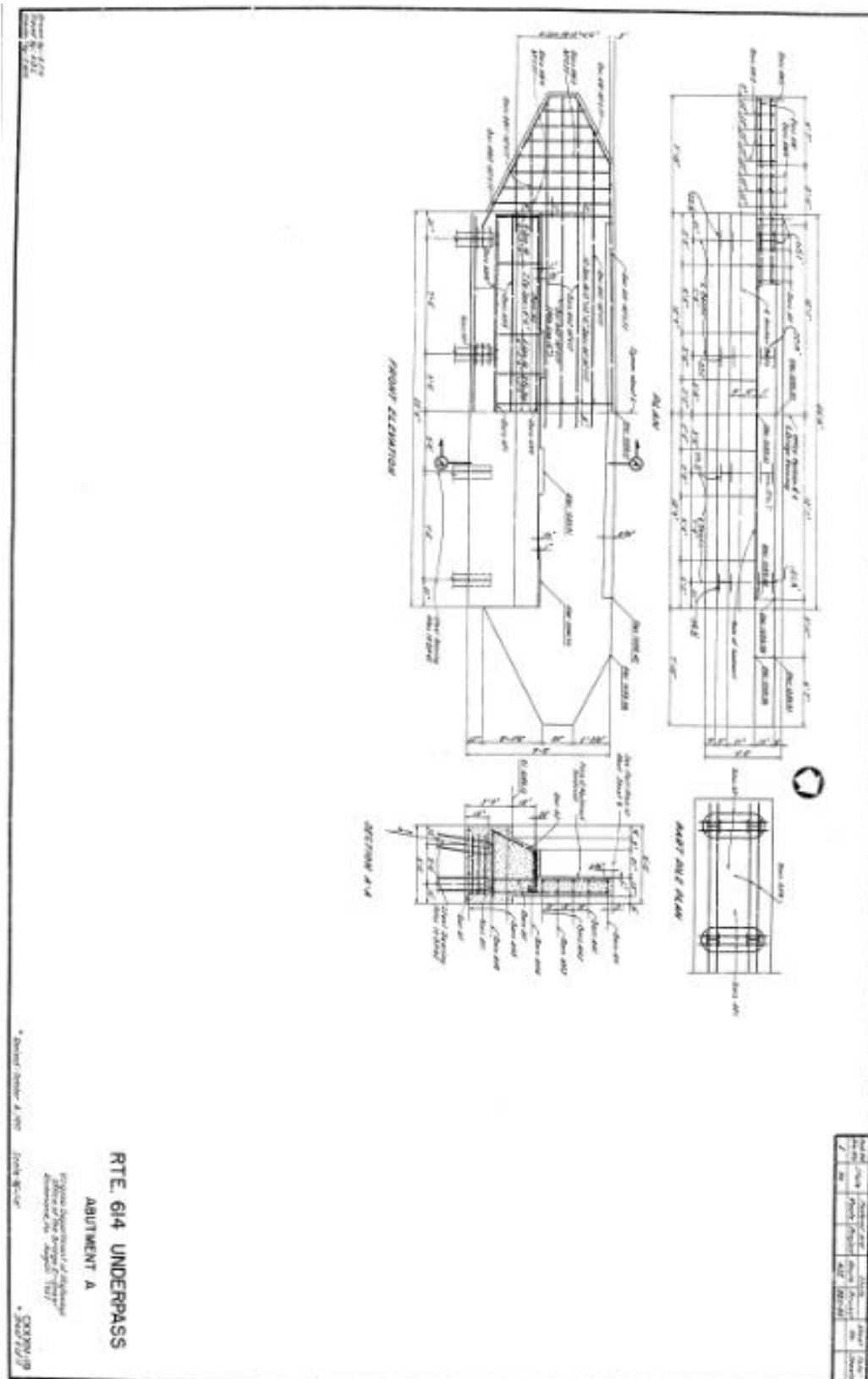


Figure 130- Route 614 Bridge Blueprint, Page 4

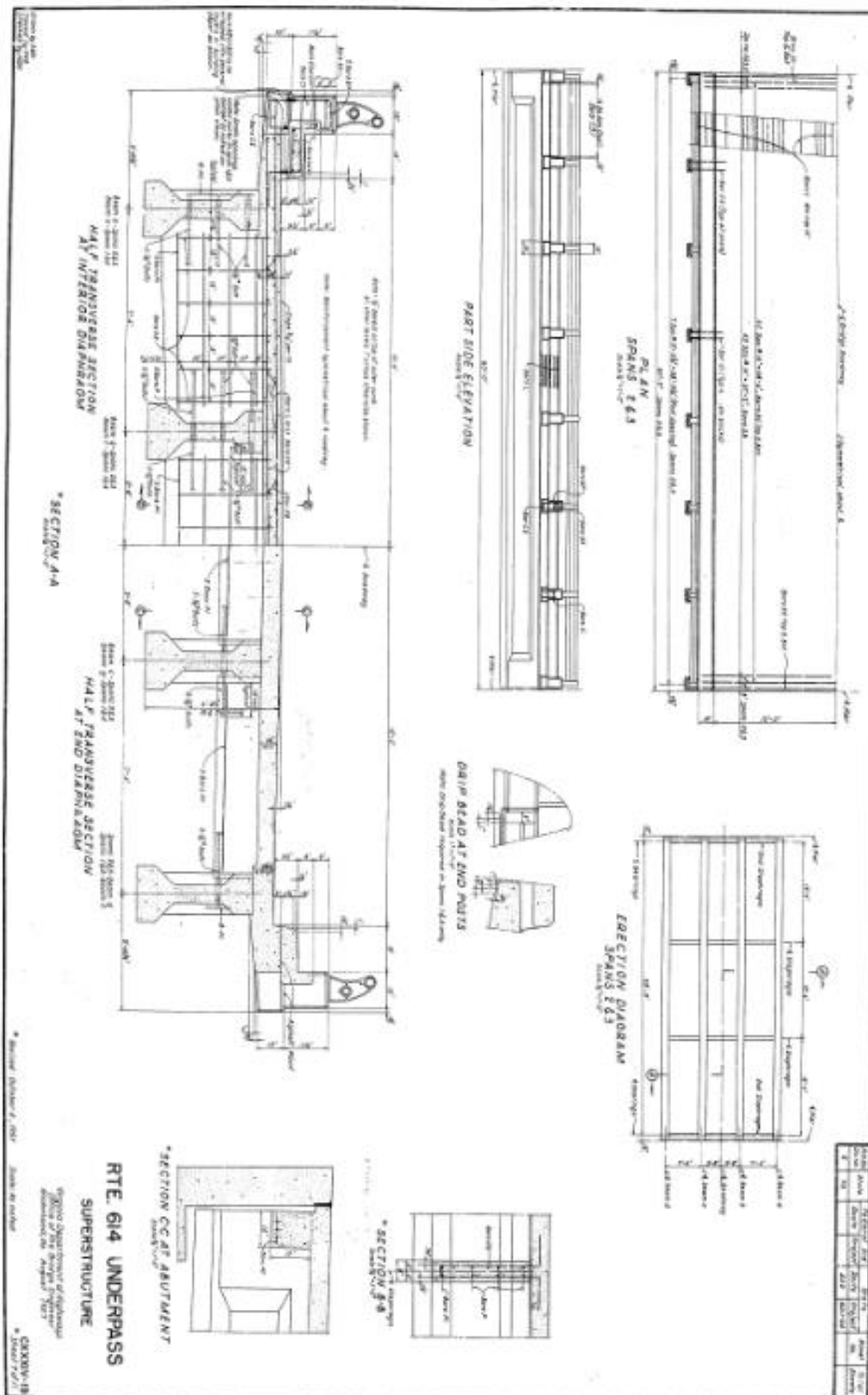


Figure 133- Route 614 Bridge Blueprint, Page 7

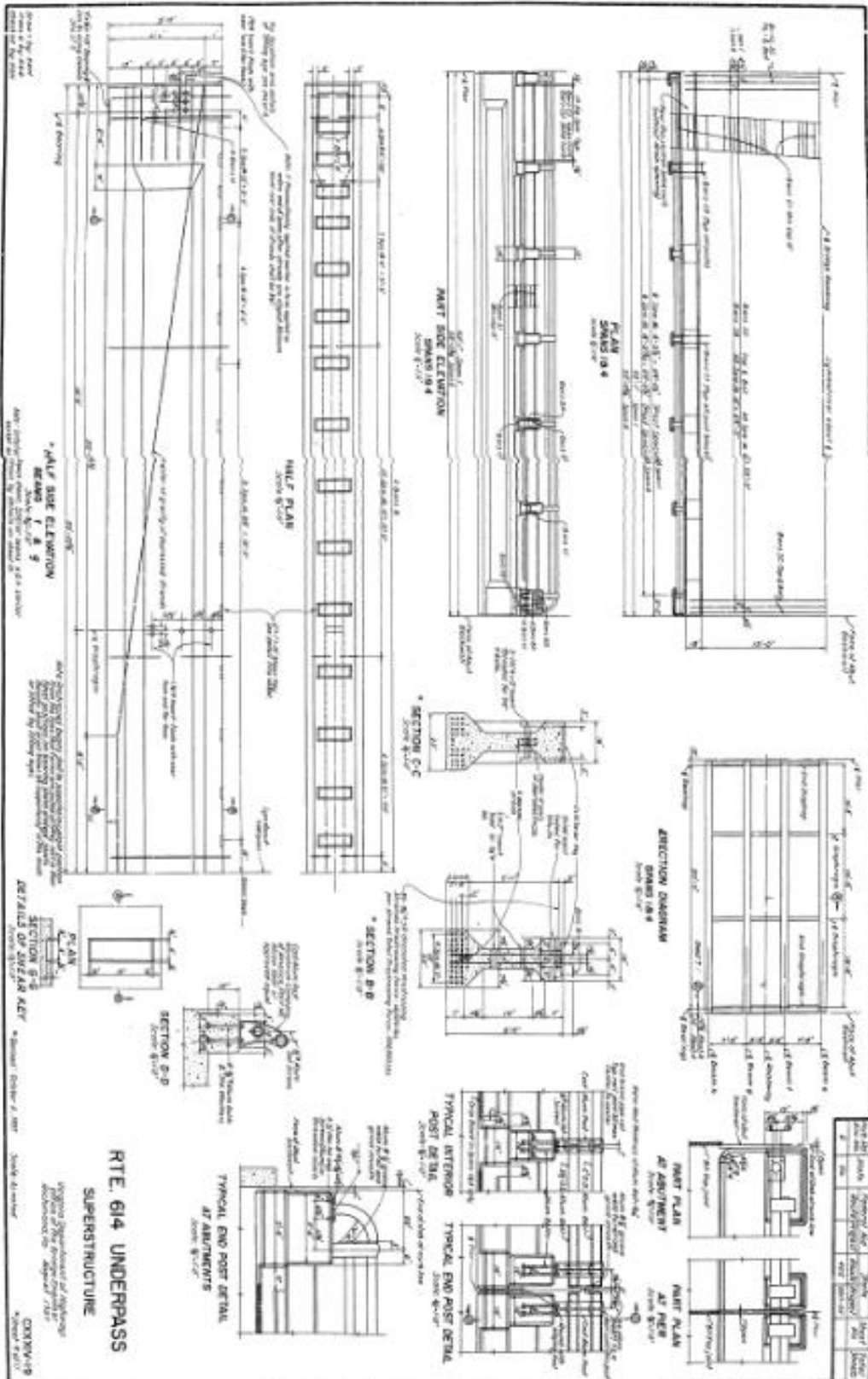


Figure 135- Route 614 Bridge Blueprint, Page 9

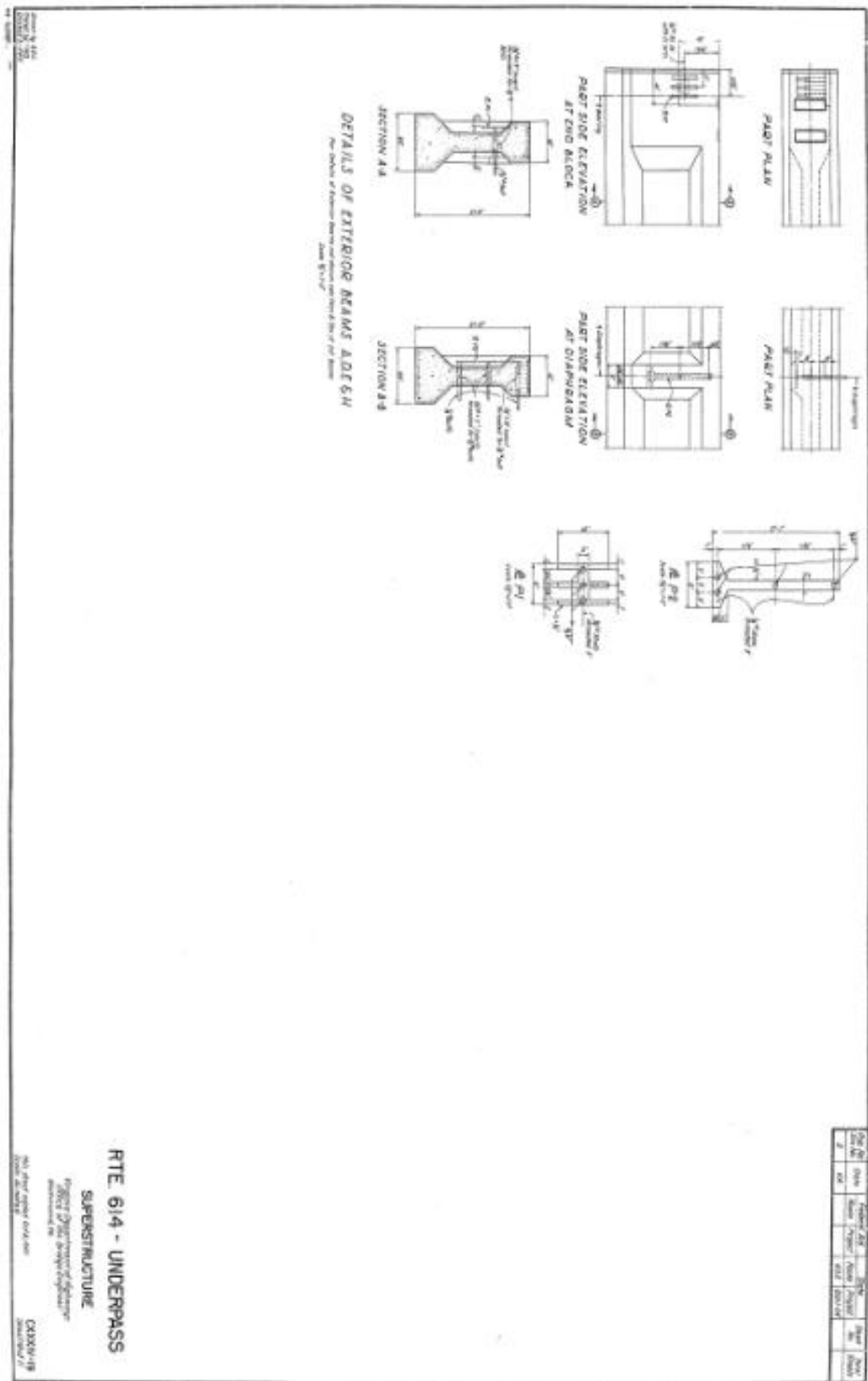


Figure 136- Route 614 Bridge Blueprint, Page 10

Figure 137- Route 614 Bridge Blueprint, Page 11

APPENDIX B: AASHTO Flexural Calculations

AASHTO Equation 5.7.3.2.2-1 for Nominal Moment

$$M_n = A_{ps}f_{ps} \left(d_p - \frac{a}{2} \right) + 0.85f'_c(b - b_w)h_f \left(\frac{a}{2} - \frac{h_f}{2} \right)$$

Where:

M_n = Nominal Moment, in-k

A_{ps} = Total Area of Prestressing Strands, in²

$$f_{ps} = f_{pu} \left(1 - k \frac{c}{d_p} \right)$$

f_{pu} = Strand Ultimate Stress, ksi

$$k = 2 \left(1.04 - \frac{f_{py}}{f_{pu}} \right)$$

f_{py} = Strand Yield Stress, ksi

c = Neutral Axis, in

d_p = Prestressing Depth, in

$$a = \frac{A_{ps}f_{ps}}{0.85f'_cb} = \beta_1 c$$

f'_c = Concrete Compressive Strength, ksi

b = Width of Compression Block, in

β_1 = Stress Block Factor

b_w = Web Width, in

h_f = Compression Flange Depth, in

AASHTO Equation 5.7.3.3.2-1 for Cracking Moment

$$M_{cr} = \gamma_3 \left[(\gamma_1 f_r + \gamma_2 f_{cpe}) S_c - M_{dnc} \left(\frac{S_c}{S_{nc}} - 1 \right) \right]$$

Where:

- S_c = Composite Tensile Section Modulus, in³
- f_r = Concrete Modulus of Rupture, ksi
- f_{cpe} = Compressive Stress from Prestress, ksi
- M_{dnc} = Unfactored Dead Load Moment on Non-Composite Section, in-k
- S_{nc} = Non-Composite Tensile Section Modulus, in³
- γ_1 = Flexural Cracking Variability Factor = 1.6
- γ_2 = Prestress Variability Factor = 1.1
- γ_3 = Reinforcement Ratio of Yield to Tensile Strength = 1.0

APPENDIX C: AASHTO Shear Calculations

$$V_n = V_c + V_s + V_p \quad (\text{AASHTO eq. 5.8.3.3-1})$$

$$V_c = \min(V_{ci}, V_{cw})$$

$$V_{ci} = 0.02\sqrt{f'_c}b_vd_v + V_d + \frac{V_iM_{cre}}{M_{max}} \geq 0.06\sqrt{f'_c}b_vd_v \quad (\text{AASHTO eq. 5.8.3.4.3-1})$$

Where:

V_d = Shear force at section due to unfactored dead load (kips)

V_i = Factored shear force at section due to externally applied loads
occurring simultaneously with M_{max} (kips)

M_{cre} = Moment causing flexural cracking at section due to externally
applied loads (kip-in)

$$M_{cre} = S_c * (f_c + f_{cpe} - \frac{M_{dnc}}{S_{nc}}) \quad (\text{AASHTO eq. 5.8.3.4.3-2})$$

Where:

f_{cpe} = compressive stress in concrete due to effective prestress only
at extreme fiber of section where tensile stress is caused by
externally applied loads (ksi)

M_{dnc} = total unfactored dead load moment acting on the monolithic
or noncomposite section (kip-in)

S_c = section modulus for the extreme fiber of the composite section
where tensile stress is caused by externally applied loads (in³)

S_{nc} = section modulus for the extreme fiber of the noncomposite
section where tensile stress is caused by external loads (in³)

M_{max} = Maximum factored moment at section due to externally
applied loads (kip-in)

$$V_{cw} = \left(0.06\sqrt{f'_c} + 0.30f_{pc} \right) b_v d_v + V_p \quad (\text{AASHTO eq. 5.8.3.4.3-3})$$

Where:

f_{pc} = compressive stress in concrete (after prestress losses) at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange (ksi). In a composite member, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of web and flange, due to both prestress and members resisted by the prestress member acting alone

$$V_s = \frac{A_v f_y d_v (\cot \theta + \cot \alpha) \sin \alpha}{s} \quad (\text{AASHTO eq. 5.8.3.3-4})$$

Where $V_{ci} < V_{cw}$:

$$\cot \theta = 1.0$$

Where $V_{ci} > V_{cw}$:

$$\cot \theta = 1.0 + 3 \left(\frac{f_{pc}}{\sqrt{f'_c}} \right) \leq 1.8$$

$V_p = 0$ (included in V_{cw} equation)

APPENDIX D: Moment-Curvature Calculations

$$f_c = f_c' \left[\frac{2\varepsilon}{\varepsilon_0} - \left(\frac{\varepsilon}{\varepsilon_0} \right)^2 \right]$$

Where:

f_c = Concrete Compressive Strength, ksi

f_c' = Ultimate Compressive Strength, ksi

ε = Strain

ε_0 = Ultimate Strain

APPENDIX E: FRCM Repair Calculations

$$\varepsilon_{fe} = \varepsilon_{fd} \leq 0.012 \quad (\text{ACI 549 eq. 11.1a})$$

Where:

ε_{fe} = Effective tensile strain in the FRCM reinforcement attained at failure

ε_{fd} = Design tensile strain of the FRCM composite material

$$f_{fe} = E_f \varepsilon_{fe} \quad \text{where } \varepsilon_{fe} \leq \varepsilon_{fd} \quad (\text{ACI 549 eq. 11.1b})$$

Where:

f_{fe} = Effective tensile stress level in the FRCM reinforcement at failure

E_f = Tensile modulus of elasticity of cracked FRCM

$$\varphi_m M_n = \varphi_m (M_s + M_f) \quad (\text{ACI 549 eq. 11.1c})$$

Where:

M_n = Nominal flexural strength

M_s = Contribution of steel reinforcement to the nominal flexural strength

M_f = Contribution of FRCM reinforcement to the nominal flexural strength

$$\varphi_m = \begin{cases} 0.90 & \text{for } \varepsilon_t \geq 0.005 \\ 0.65 + \frac{0.25(\varepsilon_t - \varepsilon_{sy})}{0.005 - \varepsilon_{sy}} & \text{for } \varepsilon_{sy} < \varepsilon_t < 0.005 \\ 0.65 & \text{for } \varepsilon_t < \varepsilon_{sy} \end{cases} \quad (\text{ACI 549 eq. 11.1d})$$

Where:

ε_{sy} = Steel tensile yield strain

ε_t = Net tensile strain in extreme tension steel reinforcement

APPENDIX F: Common Repair Procedures

Equipment Required: Grinder with diamond blade, 15 lb chipping gun, sand blaster, pressure washer

Step 1- Saw cut around perimeter of damage 0.5 in down using grinder and diamond blade, avoiding inward corners to reduce stress concentrations in the patch material, shown in Figure 138.



Figure 138- Saw Cutting around Damage Perimeter

Step 2- Chip away concrete inside damage perimeter 1.0 in down throughout damage area using light chipping gun. To improve bond between the patch material and the substrate concrete, chip around each strand located nearest to the end of the damage perimeter on the three sides to expose the strand for the entire length, shown in Figure 139.



Figure 139- Chipping Concrete with Light Chipping Gun

Step 3- Sand blast damage area, then pressure wash, to clean damage surface, shown in Figure 140.



Figure 140- Sand Blasting to Clean Concrete Surface

Step 4- Construct formwork, mix concrete, wet substrate surface, and place concrete, shown in Figure 141.



Figure 141- Placing Repair Concrete

APPENDIX G: Strand Splice Repair Procedures

Equipment Required: light chipping gun, torque wrench, adjustable wrench (2x), Grabb-it Cable Splice

Step 1- During chipping of the concrete, ensure that, for strands that will be spliced, the concrete is chipped around the strand to clear a 1 in radius around the strand for at least 30 in of length.

Step 2- Install strand splice. Place an adjustable wrench on each splice chuck and turn the turnbuckle using the calibrated torque wrench. Using the Grabb-it torque tables, torque the strand to 70% of the ultimate tensile stress, shown in Figure 142.



Figure 142- Tensioning of Strand Splice

APPENDIX H: Fabric Reinforced Cementitious Matrix Repair Procedures

Equipment Required: sanding wheel, scissors, trowel

Step 1- Cut mesh sheets to prescribed lengths using scissors, shown in Figure 143.



Figure 143- Cutting the FRCM Mesh

Step 2- At least 24 hours after the concrete patch material has set, prepare surface with sanding wheel, smoothing rough edges, shown in Figure 144.



Figure 144- FRCM Surface Preparation using Sanding Wheel

Step 3- Wet concrete surface, mix matrix mortar, and apply first layer using a trowel, shown in Figure 145.



Figure 145- First Layer of FRCM Mortar

Step 4- After the first layer of mortar has been applied, roll on the first layer of FRCM mesh, shown in Figure 146.



Figure 146- Installation of the First Layer of FRCM Mesh

Step 5- Mix matrix mortar and apply an additional layer using a trowel. Follow this with another layer of FRCM mesh and repeat until final layer of FRCM mesh is installed, shown in Figure 147.



Figure 147- Installation of the Second Layer of FRCM Mesh

Step 6- Apply final layer of mortar to smooth repair surface, shown in Figure 148.



Figure 148- Final Layer of Mortar