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Retaining Wall System with Soldier Pile and Composite Resin Infused Lagging Elements

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Retaining Wall System with Soldier Pile and Composite Resin Infused Lagging Elements

Shane Phillip Moore

Problem Report submitted to the
College of Engineering and Mineral Resources at
West Virginia University in
partial fulfillment of the requirements
for the degree of

**Master of Science
in
Civil Engineering**

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Morgantown, West Virginia
2019

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ABSTRACT

Retaining Wall System with Soldier Pile and Composite Resin Infused Lagging Elements

Shane Phillip Moore

Retaining walls are used as an alternative to installing drainage and soil reinforcement, depending on site conditions. This report discusses the design of a retaining wall with steel soldier piles and lagging elements for rocky slopes. The design consists of six different options, three options with a span of eight feet and three options with a span of ten feet between soldier piles. The three options for both spans are the same concept but designed for different load conditions. These options include: timber with glass fiber reinforced polymers (GFRP) wraps, reinforced concrete with GFRP wraps, and an FRP Prodeck system resisting horizontal thrust as a beam supported by soldier piles. The two options with a GFRP wrap shell are recommended to be constructed using the resin infusion manufacturing process to achieve a better quality lagging element.

Resin infusion is an FRP manufacturing method in which an object is first wrapped with a dry fabric, placed in a bag and all air is removed from the bag with a vacuum pump. Once the object has reached the atmospheric vacuum pressure, resin is infused into the sample and cured while the plastic sheet provides a compressive force on the FRP which creates a better quality bond. The resin is then cured at room temperature.

A bending test is performed on seven railroad ties in this report. One control tie, three ties wrapped with GFRP using the hand lay-up method, and three resin infused ties with GFRP. The railroad ties were used in the field prior to testing. Along with performing a bending test on all seven railroad ties, a digital tap hammer was used to determine the bond quality on one railroad tie wrapped using the hand lay-up method and one tie wrapped with resin infusion. The results for this test suggest that resin infusion produces a higher peak bending stress, better bond quality, and a more consistent bond. The composite modulus of elasticity wasn't able to be differentiated between the samples due to the inconsistency with the railroad ties used.

The data herein proves that resin infusion produces a higher peak bending stress and better bond quality compared to hand lay-up. Resin infusion is a more expensive manufacturing process and it may create some complications such as resin not traveling through the entire specimen or the specimen not curing properly. Proper materials and experienced personnel must be present, but when used properly it is a more effective FRP reinforcing method.

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CHAPTER 1 INTRODUCTION

1.1 Background

Slope stabilization is a concept in the U.S. that is constantly evolving and being challenged. It is an important concept because it prevents debris on slopes from sliding or falling, which could injure people or damage vehicles. Properties affecting slope stabilization include: moisture content, permeability, shear strength of soil, particle size distribution, and soil density. A variety of methods are available to stabilize slopes by adding reinforcement or a drainage system throughout the slope but this report will discuss the design of a retaining wall for a project at the bottom of the steep rock slope to prevent debris from hitting people or vehicles.

Retaining walls are an option for retaining rocks and debris on unstable slopes from falling onto the road. There is a variety of retaining walls in use today and each system has unique advantages for each type of project. The preferred retaining wall system for this project is a soldier pile/lagging retaining wall. Cost and durability are two of the largest factors when determining which design to use for a structure. For durability purposes, wrapping lagging elements with glass fiber reinforced polymers (GFRP) is presented for four of the six lagging element designs. It is recommended that the elements wrapped with GFRP are wrapped using the resin infusion method vs the hand lay-up method as it achieves a better quality wrap. In addition to designing the retaining wall lagging elements with new materials, used railroad ties reinforced with GFRP are evaluated for cost efficient purposes.

Used railroad ties have undergone deterioration from weathering which results in lower mechanical properties. Reinforcing used railroad ties with GFRP increases the peak bending stress and prevents deterioration from weathering. Using the method of resin infusion will

maximize the quality of wrapping to ensure the best results are achieved for comparison purposes.

1.2 Objectives

- To present a steel pile with lagging element retaining wall design with multiple options
- To present practical lifting mechanisms and drainage systems for a contractor and owner to choose
- Incorporate resin infusion into retaining wall design
- Compare peak bending stresses between hand lay-up and resin infused railroad ties
- Evaluate bond quality and consistency between hand lay-up and resin infused railroad ties
- Determine the composite modulus of elasticity between hand lay-up and resin infused railroad ties
- Determine if used railroad ties reinforced with GFRP can be an option for lagging elements

1.3 Scope

- Retaining wall design with steel piles and lagging element options
- Testing of used railroad ties
- Comparison of 3 dimension needle punched fabric with 2 dimensional fabric using resin infusion
- Pull out testing of GFRP on a corroded steel plate

1.4 Report Organization

- Chapter 1 Introduction
- Chapter 2 Literature Review
- Chapter 3 Steel pile/lagging retaining wall design
- Chapter 4 Experimentation and testing of resin infusion on used railroad ties
- Chapter 5 Conclusions and future work
- Appendix A Retaining wall design calculations
- Appendix B Resin infused 3D needle fabric
- Appendix C FRP bond to corroded steel
- Appendix D Retaining wall drawings

CHAPTER 2 LITERATURE REVIEW

2.1 Retaining Wall Systems

Retaining walls are constructed to resist lateral pressures from soil. The walls can either be up against a slope resisting a permanent lateral load or near a slope to protect a road from falling rock debris. Similar to bridges, there is a large amount of retaining wall designs currently in use but retaining walls can be broken into lateral resistance systems and architectural retaining wall categories. Three of the most common lateral resistance systems for a retaining wall system is a gravity wall, a cantilever wall, and an anchored wall. Gravity walls resist a load by using its self-weight. Cantilever walls resist the load by having a foundation for the wall that creates a moment which acts like a cantilever beam. Anchored walls have anchors that attach to the wall and travel into the resisting soil to provide additional lateral resistance. Anchored systems are most commonly paired with gravity and cantilever walls. (Oregon Department of Transportation, 2018)

In addition to the three different lateral resistance systems, there is a variety of retaining wall system types: mechanically stabilized earth wall, prefabricated modular walls, soldier pile/lagging wall, sheet pile wall, masonry block wall, cast in place concrete wall, and tangent/secant walls (Oregon Department of Transportation, 2018). Not all retaining walls fit into one of these types but a majority of all retaining have some characteristic of these types. Architects and structural engineers make alterations to make the designs more elaborate but the designs typically fit into at least one of the categories.

2.1.1 Mechanically Stabilized Earth Wall

Mechanically stabilized earth walls are constructed by layering soil and reinforcement strips. A layer of a reinforced earth wall includes a precast concrete panel with soil reinforcing strips at the top and bottom of the panel (Figure 1) running behind the precast concrete panels, in the perpendicular direction to the plane of the panel. Once the panels are placed, the area between the existing slope and the panels is filled and compacted with soil. The purpose of the strips is to create a composite action between the soil and precast panels through friction between the strips and the soil. MSE wall do not work well on rocky slopes as MSE walls require a suitable amount of soil behind the precast face to develop enough frictional forces to support the wall. The friction between the strips and the soil is what the mechanically stabilized earth wall depends on and is how it resists horizontal and overturning forces. MSE walls provide better resistance to earthquake loading effects due to its flexibility and energy absorbing capacity. (Oregon Department of Transportation, 2018)

Figure 1

Figure 1 Mechanically Stabilized Earth Wall (Strohman & DiFiore, June)

2.1.2 Prefabricated Modular Walls

2.1.2.1 Gabion Wall

Gabion walls are a type of gravity structures that consist of large aggregate, usually between 4" to 8" in diameter, placed in a steel wire basket (3'x3'x3') to form a block. Multiple blocks are then stacked on top of each other to form a wall. The unit weight of the blocks is usually between 100 to 110 pcf. Due to using large aggregates, the void space in the blocks accounts for approximately 30% of the total space. Gabion walls also don't perform well on soft soils because without a foundation the self-weight of the blocks could lead to a settlement within the foundation. Figure 2 shows a gabion wall design from the elevation view. It can be seen that the blocks are stacked in a diagonal manner to ensure the weight of the blocks is against the slope. Additional gravel with a smaller aggregate size is placed behind the blocks to ensure that adequate drainage is obtained. (Oregon Department of Transportation, 2018)

Figure 2 Gabion Wall (Gabion Retaining Wall, 1969)

2.1.2.2 Crib Wall

Crib walls are constructed by interlocking stretcher and header elements to form a rectangle, similar to Lincoln Logs. The foundation must be prepared with the proper back slope angle so the wall can be layered at the proper angle. Once the wall is built, the hollow space is filled with compacted gravel to resist overturning and sliding forces by increasing its self-weight which classifies a crib wall as a gravity wall. The crib wall also angles toward the slope for increased strength, which is placed directly against the slope, shown in Figure 3. Loss of backfill material subjects crib walls to damage from uplift, cracking, bin modules being separated. (Oregon Department of Transportation, 2018)

Figure 3 Precast Concrete Crib Wall (UMR, 2019)

2.1.2.3 Precast Concrete Block Wall

Precast block walls are constructed as either a wet (with mortar joints) or dry (no mortar) block wall. Dry block systems require a shear key consisting of a bulge, bump, or lip to hold the layers in place. Since the retaining wall is a gravity wall system, the individual blocks are typically large which can be seen in Figure 4. These wall systems are stacked in a vertical manner due to interlocking sections of the block. This retaining wall system is also one of the easiest systems to construct in that a crane or forklift is the only machinery needed to place the blocks. (Oregon Department of Transportation, 2018)

Figure 4 Precast Block Wall (Elite Precast Concrete, 2019)

2.1.2.4 Precast Bin Wall

Precast bin walls look similar to the block walls from the exterior but the inside is hollow which allows for an easier construction process and the hollow space is filled with compacted gravel once an entire row has been placed. The bins are typically interlocking to ensure a proper connection is formed to resist lateral earth pressure and either steel or precast concrete can be used. (Oregon Department of Transportation, 2018)

2.1.3 Masonry Block Wall

A masonry block system is a cantilever retaining wall system with prefabricated rectangular hollow boxes. The size of the individual units can vary. This system is different than the precast concrete bin wall because it is a cantilever system with smaller boxes whereas the precast bin system is a gravity system with much larger boxes and the blocks are grouted together. The foundation of the system is placed below grade in a concrete footing with a vertical steel rod that runs through each masonry block which is what makes it a cantilever system (Figure 5). Backfilling against the retaining must wait until after the grout between blocks has achieved a sufficient strength and the backfill should be placed in 12 to 24 inch lifts. Each lift must then be compacted. A variety of different materials can be used but brown decorative masonry blocks are the most common material used for this system. (TEK, 2005)

Figure 5 Masonry Block Wall Diagram (TEK, 2005)

2.1.4 Cast in Place Concrete Wall

Cast in place (CIP) retaining walls are constructed by digging a foundation, placing formwork, and placing concrete at the site. The amount of steel reinforcement in the retaining wall determines the type of CIP retaining wall system.

Figure 6 shows the three types of cast in place retaining wall systems which is a gravity wall, semi gravity wall, and a cantilever wall. The cast in place retaining walls can either be placed against a slope or freestanding. Being placed against the slope will put a permanent horizontal load on the wall while making it a freestanding wall will put an impact load on it from falling rocks and soil. (Oregon Department of Transportation, 2018)

Figure 6 Cast in Place retaining Wall Systems (Mitchell, 2018)

2.1.4.1 CIP Gravity Wall

CIP gravity walls are constructed with a minimal amount of reinforcing steel and depend completely on its self-weight to resist overturning which is why it is a larger retaining wall system than the semi gravity and cantilever system. These retaining wall systems are most practical for shorter walls and not intended for softer soils. Due to how large the self-weight is of a gravity wall, it applies a large vertical force on the soil which could lead to a foundation settlement. (Oregon Department of Transportation, 2018)

2.1.4.2 CIP Cantilever Wall

Cantilever walls have the maximum amount of steel reinforcement which allows it to resist overturning and horizontal forces due to friction and the wall base reaction. Of the three retaining wall system

shown in

Figure 6, the cantilever wall system has the smallest width and can have the largest height since it doesn't have as large of a self-weight as the gravity wall systems. Similar to the gravity wall system, it also can't be placed in a soft soil because it is prone to foundation settlement. Cantilever wall systems may not be as heavy as the gravity wall system but it has a shallow foundation which is prone to settlement. Additional anchorage may be used for excessive tall walls to prevent additional lateral pressures. (Oregon Department of Transportation, 2018)

2.1.4.3 CIP Semi gravity Wall

A semi gravity wall is a combination between the cantilever and gravity wall system. It has more reinforcement than the gravity wall but not as much as the cantilever system. It also has more concrete than the cantilever wall system but not as much as the gravity wall system. These parameter allow for the system to resist overturning and horizontal forces from the wall base reaction, friction, and its self-weight. (Oregon Department of Transportation, 2018)

2.1.5 Tangent/Secant Pile Walls

Figure 7 Tangent/Secant Pile Wall (Geoexpertise, n.d.)

A tangent/secant pile wall is a row of cast in place cantilever or anchored piles. The circular piles are typically between 24 to 48 inches in diameter. It is different than a standard cast in place retaining wall because multiple circular piles are placed with either the circles overlapping or touching. The piles touching is a tangent wall and the piles overlapping is a secant wall. Figure 7 shows the plan view of how the retaining wall design appears. It can be noted that a variety of reinforcing steel can be used. Tangent/secant pile work best for deep excavation and are most commonly used in permanent excavation applications. (Oregon Department of Transportation, 2018)

2.1.1 Sheet Pile Wall

Figure 8 Sheet Pile Wall (Sheet Pile Walls, 2019)

Sheet pile walls are steel Z type piles welded or bolted together to make a wall and the sections are driven into the ground to form a cantilever wall design. Typically, approximately 50% of the wall is below grade and the other 50% is above grade retaining soil. Sheet pile walls can be used as either a temporary or permanent retaining wall for applications such as excavations, bulkhead walls, cofferdams and trenches. Areas with shallow bedrock or dense and coarse soils aren't ideal for sheet pile walls due to its flexibility. The wall can be used as a cantilever if it is below 15 feet tall and if it is above 15 feet then anchorage must be used for additional reinforcement. (Oregon Department of Transportation, 2018)

2.1.1.1 Cantilever Wall

A cantilever system is used when there is no excess lateral or vertical movements beyond the wall, no overstressing of individual elements, and the height of the wall is below 15 feet.

Figure 9 shows an elevation view of a typical cantilever system. The piles can be driven quickly which allows for a quicker construction process. The driven pile creates a moment that resist the lateral load on the wall in a similar manner to a cantilever. (Oregon Department of Transportation, 2018)

Figure 9 Cantilever Wall (Oregon Department of Transportation, 2018)

2.1.1.2 *Anchored Soldier Pile/Lagging Wall*

Anchorage is required when the height of the wall is above 15 feet. The cantilever action provides the bulk of the resistance, due to the additional lateral pressure on the wall, additional support is required which is the anchorage. Figure 10 shows an elevation view of the angle at which an anchor is installed. The anchors and tendons are placed in the backfill process during construction. Types of failures for anchoring include: tension failure of tendon, pullout failure of ground/tendon bond, pullout failure of tendon/grout bond, failure of wall in bending, failure of wall due to insufficient passive capacity, failure by forward rotation, failure due to insufficient axial capacity, failure by overturning, failure by sliding, and rotational failure of ground mass. (Oregon Department of Transportation, 2018)

Figure 10 Anchored Soldier Pile/Lagging Wall (Oregon Department of Transportation, 2018)

2.1.1 **Soldier Pile/Lagging Wall**

Soldier pile and lagging walls consist of steel wide flange or HP beams used as the piles and a variety of materials used as the lagging elements but most commonly either timber or reinforced concrete. The piles are either driven into the ground or placed into drilled shafts filled

with concrete. The lagging elements are placed between the flanges of the pile and provide lateral pressure resistance. Figure 11 shows the system being used on a residential property. There aren't any voids in the wall for drainage so a drainage system must be implemented. The most common type of drainage system is weep holes with a perforated pipe at the bottom of the wall. (Oregon Department of Transportation, 2018)

The typical span between soldier piles is between 6 to 10 feet. If the height of the wall is less than 15 feet than a cantilever system can be used but if the height is above 15 feet then it must be reinforced with anchorage. (Oregon Department of Transportation, 2018)

Figure 11 Soldier Pile/Lagging Retaining Wall (Build LLC, 2015)

2.1.2 Conclusions

The retaining wall systems presented herein offer solutions for a variety of scenarios for slopes. Some walls are similar to others with minimal differences but most slopes can be reinforced with one of the retaining wall systems presented. Not all retaining walls in use today are one of the systems presented because architects and engineers make alterations to existing retaining wall designs to fit the project at hand but characteristics of the walls are present in the design as the alterations are typically for aesthetics.

2.2 Composite Materials

Fiber reinforced polymer is a structural composite material consisting of three essential constituents: fibers, polymers, and additives. It is beneficial because it is lightweight, high strength, versatile, and resistant to corrosion along with a few other reasons. FRP is constructed by introducing fibers into a polymer matrix. The fiber volume fraction varies depending on the manufacturing method used the result of the two materials creates a fiber reinforced polymer. There are two types of resin, thermoset and thermoplastic. Thermoplastics melt and flow upon reheating but are not typically used in structural applications. Thermosets do not melt upon heating and have better structural properties. The five types of thermosets used are: polyester, epoxy, vinyl ester, urethane, and phenolics. The most common fabrics used are carbon, glass, aramid, and basalt. (GangaRao, Taly, & Vijay, Reinforced concrete design with FRP composites, 2007)

FRP can be manufactured through a variety of methods but this report focuses on the techniques discussed herein, including hand lay-up, pultrusion, and resin infusion.

2.2.1 Hand Lay-up

The hand lay-up process consists of applying resin to the prepared mold surface and placing the fabric into the mold and pressed against all the corners to ensure no voids are present. The hand lay-up process can also be used for reinforcing existing structural members such as beams, columns, concrete cylinders, railroad ties and any other object that could use reinforcement. This process consists of applying the saturated fabric directly to the member to ensure a strong bond. The quality of the wrap heavily depends on the workmanship of the personnel. The first step in bonding FRP to concrete is to prepare the concrete surface to make it even and uniform without excess cracking. A primer must then be applied to the concrete to

obtain a bond between the FRP and concrete. The third step is to apply a resin coat to the concrete which allows for the FRP to bond better. Finally, the FRP can be wrapped around the object being reinforced and all the voids must be removed to create an adequate bond. An additional layer or resin can be applied at the end to act as a protective coating. (GangaRao, Taly, & Vijay, Reinforced concrete design with FRP composites, 2007)

2.2.2 Pultrusion

The pultrusion process is used for creating longer objects such as rebar and other structural shapes. The term pultrusion comes from the pulling action that the equipment creates. The fibers/fabric are saturated in a resin bath and pulled through the pultrusion assembly, shown in Figure 12, and the fiber is shaped by the mold and cut to manageable lengths. This process creates a homogeneous material and is most commonly used to make FRP rebar along with other structural shapes. (GangaRao, Taly, & Vijay, Reinforced concrete design with FRP composites, 2007)

Figure 12 Schematic of Pultrusion Process (GangaRao, Taly, & Vijay, Reinforced concrete design with FRP composites, 2007)

2.2.3 Resin Infusion

Resin infusion, also referred to as vacuum infusion process (VIP), is an FRP manufacturing method that consists of using a vacuum to send resin through a fabric while applying a compressive force on the fabric. For molded components, a mold is first, acquired, then a release film on the mold. The desired number of dry fabric layers are placed on top of the release film, and the resin flow and vacuum lines are placed in appropriate locations to ensure the resin flows through the entire mold. Finally, a plastic sheet is placed over top and the edges are sealed. The air is vacuumed out to a minimum of 28 in.Hg, and the resin is sent through the mold. Figure 13 shows a schematic of the process described above. The filter jacket is placed when a T-fitting is needed. A T-fitting acts as a splitter for 2 resin feed or vacuum lines. A resin catch bucket is also implemented to ensure that when the resin has saturated the fabric throughout the entire mold, excess resin doesn't flow into the vacuum pump and goes into the bucket. The bucket is an air tight bucket with two resin lines, one to the pump and one to the mold. (Fibre Glast Developments Corporation, 2019)

Figure 13 Resin Infusion Set-up (Fibre Glast Developments Corporation, 2019)

Advantages of the resin infusion process include: better fiber to resin ratio, less wasted resin, consistent resin usage, controlled set up time, and it is a cleaner process. The hand lay-up process produces more waste and it isn't as consistent of a process so the same quality in projects is difficult to acquire. Disadvantages of resin infusion when compared to the hand lay-up process are: complicated set-up, easy to damage or ruin a part of the system, and it involves a trial and error process. It takes experienced personnel for resin infusion to be used correctly. (Fibre Glast Developments Corporation, 2019)

It should be noted that a mold is used for surfaces that aren't flat to create laminates but resin infusion can be used to reinforce structural members which is how resin infusion is implemented in this report. The same process is used except there isn't a mold. The resin flow and vacuum

lines will run along one direction of the member and additional resin flow lines may be needed for larger objects.

2.2.4 Conclusions

Composite materials offer benefits in a structural application and the FRP manufacturing methods presented can be used for different applications. Pultrusion is a process used for creating long homogeneous member while hand lay-up and resin infusion can be used to either create an object using a mold or reinforce an existing structural member. Hand lay-up is a simpler method but resin infusion produces a better fiber to resin ratio which leads to a stronger element (Fibre Glast Developments Corporation, 2019). These two methods will be compared directly later in this report.

CHAPTER 3 STEEL PILE/LAGGING RETAINING WALL

3.1 INTRODUCTION

The inspected site has a rocky slope that is approximately 63 feet high and runs along a 19 feet wide road that leads to a large plant. The slope has approximately a 2:1 ratio with a 10 feet wide ditch between the foot of the rocky slope and road. The ditch is able to catch some rockfall but rocks still manage to fall onto the road and rock as large as 8 feet in diameter have fallen onto the road. The owner of the road requires that rocks do not impact vehicles traveling on the road, thus a retaining wall must to be built between the ditch and the road to catch small and large rocks from falling onto the road, and also rocks that bounce after hitting the ditch. *Note: the specific site is not disclosed at the request of the owner.*

The retaining wall will be approximately 10 feet from the bottom of the slope. There are multiple retaining walls to choose from and an analysis was performed to determine the retaining wall most fitting for this project. A mechanically stabilized earth wall isn't adequate for this project because the slope is rocky and the wall is freestanding, not against the slope of soft or medium grade soil. A tangent/secant pile wall also isn't adequate because the foundation is rocky and tangent/secant piles are used for deep excavations. The remaining retaining wall options to pick from are: CIP concrete wall, masonry block wall, prefabricated modular block wall, sheet pile wall, and a steel pile/lagging wall. Due to the constant falling of rock debris from the slope, it is suggested that rocks from the back side of the wall are removed frequently to prevent buildup on the back side. This makes the steel pile/lagging wall the best option because while the steel piles are fixed at a location, the lagging elements can be removed easily.

The total length of the retaining wall is anticipated to be approximately 200 feet. Due to the rockiness of the soil, the piles aren't recommended to be driven into place because some areas may have rocks that prevent the piles from being driven to the appropriate depth. A circular concrete footer will be the most adequate and consistent design as a foundation for the steel piles. The piles are 8 feet in height, determined from a geotechnical report, which is to prevent the highest bouncing rocks from falling onto the road along with stopping the larger rocks. There is an additional 3 feet depth below grade for the foundation. In order to minimize the amount of lagging elements needed and the weight of each lagging element, each span between soldier piles will have 4 lagging elements approximately 2 feet in height. Figure 14 shows a 3 dimensional view of the proposed retaining wall.

Figure 14 3D View of Pile & Lagging Retaining Wall

Six options are presented for this design to allow for the owner and contractor to choose. Three options consist of having a 10 feet span between steel soldier piles and the other three with an 8 feet span. The first option is to use a timber with three layers of glass fiber reinforced polymers. The second option is to use a reinforced concrete lagging element with three layers of glass fiber reinforced polymers. The third option is to use an FRP Prodeck system rotated 90

degrees so it acts as a beam instead of a bridge. The other three options are the same as the first three but there is an 8 feet span between the steel soldier piles.

The resin infusion type of manufacturing process should be used for the designs that include reinforcing the lagging element with GFRP to ensure the GFRP has an adequate bond to improve the overall strength and prevent deterioration on the lagging element from weathering effects. The following test data in this report will prove why resin infusion is a more adequate manufacturing method for this application than hand lay-up.

3.2 Geotechnical Analysis

A geotechnical analysis was performed to determine the geometric parameters of the slope along with the soil properties. The slope was visually classified as a hard sandstone rock with a backslope angle of 60 degrees and a density of 155lb/ft³. The average size rocks on the slope was observed to be 3 feet x 3 feet from a field survey. A bounce analysis was performed and it determined that the max height a 1 foot x 1 foot rock would bounce is 7.1 feet. The ditch is 3.25 feet deep so the highest bouncing rocks would reach as high as 3.85 feet on the wall. An analysis was also performed assuming a full drainage ditch at an angle of 45 degrees. Some rocks for this analysis made it past the 2feet, 4feet, and 6 feet wall but zero rocks made it past the 8 feet wall. The 8 feet wall prevents any rockfall onto the road from all scenarios performed in the geotechnical report.

3.3 Design

Table 1 provides a description of what each lagging element option consists of. An explanation of the pile design, foundation, and each lagging element option is discussed herein

and Appendix A provides the calculations for each lagging element option along with the pile and foundation for a 10 and 8 feet span.

Table 1 Lagging Element Options

Option #	Pile Span (ft)	Material	FRP Reinforcement	Cross Section (in x in)
1	10	Timber	3 Layers GFRP	6.74 x 23.74
2	10	Reinforced Concrete	3 Layers GFRP	6.74 x 23.74
3	10	FRP Prodeck	NA	7 x 24
4	8	Timber	3 Layers GFRP	6.74 x 23.74
5	8	Reinforced Concrete	3 Layers GFRP	6.74 x 23.74
6	8	FRP Prodeck	NA	7 x 24

3.3.1 Pile Design

A soil density of $120 \frac{lb}{ft^3}$ was used because the soil on the slope is a sandstone rock which means that if a rockfall occurred there would be a large amount of void space between the fallen rocks against the slope, thus a full $155 \frac{lb}{ft^3}$ would not be in place. The pile acts as a cantilever with the foundation being the fixed end and the top of the wall is the free end. The load on the wall is a triangular distribution because rocks wouldn't pile in a consistent manner and the largest load from rocks would be at the base of the wall. The load on the piles was determined by tributary area of the wall that the pile is supporting. The controlling predominant failure criterion for the piles is the max moment. After calculating the peak moment, the AISC Steel Construction Manual was used to size the piles with a wide flange beam using the Allowable Strength Design (ASD) method. In addition to sizing the piles by determining the max moment, the bending stress of the flanges on the pile must be checked because the lagging elements will produce a local bending stress at the flanges on the side by the road when a rockfall occurs. The size of the

pile for the 10 feet span was determined to be a W8x48 using A572 Gr. 50 steel and the size of the pile for the 8 feet span was determined to be a W8x35 using A572 Gr. 50 steel.

The following notes are forces that can be neglected in the design:

1. Axial force effects on soldier piles are neglected because the lagging elements rest on the ground and not on the soldier piles.
2. Torsional forces would be from the piles not lining up perfectly which would result in the lagging materials having approximately a 5° angle between two soldier piles and their effects on pile are neglected. Each lagging design width is approximately 0.4 inches less than the distance from the inside of the two flanges of the wide flange pile (7.13 inches).
3. The impact force from falling rocks hitting the lagging elements is neglected because the planks would absorb impact which would have a negligible effect on the soldier pile. When soil is accumulated on the back side, the impact force would be minimal since the rocks would absorb part of the impact force.

3.3.2 Foundation

The soil at the location of the foundation is a type 1 bedrock consisting of mostly sandstone. The friction angle was not determined during the geotechnical analysis and it was assumed to be 30° with no cohesion (Wyllie & Norrish, 1996). The allowable bearing capacity is determined using Terzhagi's bearing capacity equation with a factor of safety of 3 (Das, 2016). Due to the shallowness of the foundation, the foundation will exert a horizontal pressure on soil through force transfer from the soldier pile except the concrete (footer) foundation's bearing capacity will resist the foundation from moving. A 50% moisture content was also assumed for the sandstone around the foundation due to rainfall from the slope. A 3 feet deep by 4 feet

diameter cylindrical footer was determined to be adequate for the 10 feet and 8 feet span between piles.

3.3.3 Timber with 3 Layers of GFRP

The predominant failure criterion for the timber lagging element option is bending stress. All lagging elements were designed for the element at the bottom of the wall since the largest load is at the bottom. The initial check for a timber lagging element for a 10 feet span had a bending stress too high for the timber to support it so three layers of GFRP is added in the calculations and the calculated bending stress is below the allowable bending stress. The allowable bending stress was calculated using the composite moment of inertia ($I_{transformed}$) which consists of using the modular ratio to multiple the width of the lagging element with. The composite moment of inertia calculations can be seen in Appendix A. The timber lagging element with an 8 feet span had a higher allowable bending stress than the calculated bending stress but it is still wrapped with 3 layers of GFRP to prevent deterioration from weathering effects. Figure 15 shows the cross section dimensions for the 8 feet and 10 feet pile span timber lagging elements. The ends of the lagging elements will also be covered with a GFRP end cap to keep debris from getting between the GFRP reinforcing wrap and timber. It should also be noted that the above induced stresses can be satisfied with glulam or LVL, but deflection limit state (0.5 inches) will not be satisfied unless a higher thickness than 6.5 inches is selected.

Figure 15 Cross Section of Timber Lagging Element w/ 3 Layers of GFRP

3.3.4 Reinforced Concrete with 3 Layers of GFRP

The reinforced concrete lagging elements were designed using the equations for a singly reinforced beam with the tension and compression side being the 24 inches side. The lagging elements are doubly reinforced though and singly reinforced equations give a more conservative moment capacity. The designs were calculated using the load resistance factor design method and the lagging elements were designed to withstand the induced stresses without the assistance of GFRP. GFRP is still added to prevent deterioration from weathering effects. In accordance with ACI, reinforced concrete lagging element will have a 1 inch cover on the shorter depth (6.5 inches) and a 2 inch cover on the larger depth (23.5 inches). The concrete won't be exposed to the earth which allows for it to have a 1 inch concrete cover.

Figure 16 shows the cross section of the reinforced concrete lagging element with 3 layers of GFRP for a 10 feet span. This option has five #6 bending reinforcement bars with #3 stirrups spaced at 12 inches c/c. Figure 17 shows the cross section of the reinforced concrete lagging element with 3 layers of GFRP for an 8 feet span. This option has four #5 bending reinforcement bars with #3 stirrups spaced at 12 inches c/c.

Figure 16 Cross Section of RC Lagging Element with 10 feet Span

Figure 17 Cross Section of RC Lagging Element with 8 feet Span

3.3.5 FRP Prodeck

FRP prodeck is most commonly used for bridge decks with the shorter side in the vertical direction, shown in Figure 18. As a beam bending element, it will be rotated 90 degrees so that it takes on the shape of a beam, shown in Figure 19. GFRP will be used and it has a modulus of elasticity around 4 msi. There are specific dimensions for some sizes but they can be constructed in which ever dimensions necessary, within reason. The hollow spaces in the prodeck will be filled with a foam to keep debris such as rocks, sand and water out; however, this is not a design or maintenance requirement. This system could be a bit more expensive than other systems. The distance between the inside of the flanges for both piles used is 7.13 inches. This means that the cross section of the FRP prodeck would be the same for both spans between soldier piles.

Figure 18 FRP Prodeck being Installed (GangaRao & Skidmore, 2017)

Figure 19 Cross Section of FRP Prodeck

3.4 Lifting Systems for Access to Back Side

Due to the rock falling, some sections of the proposed modular retaining wall (e.g. lagging) will be removed to remove the fallen rock debris from locations behind the proposed wall. The soldier pile and lagging elements modular system allows for individual sections to be removed between soldier piles. Four lifting systems are presented in this section. Two of the systems are systems that don't involve any additional work to the lagging elements and there are different options of picking the lagging elements between the soldier piles. The other two options require additional work on the lagging elements to make removal of the lagging elements easier. The potential owner and contractor will decide which lifting system is most suitable and if one of the two lifting systems require additional work on the lagging elements, then the owner and contractor will decide how often the systems will be implemented.

The weight of a 10 feet span reinforced concrete lagging element is approximately 1800 lbs and a forklift with the appropriate load capacity could pick it up over the top of the piles and remove it. Forklifts vary in load capacity vs height and as long as the forklift being used can lift 1800 lbs at 8 feet then a forklift is the most efficient for removing the lagging elements from the piles. A 2 inch tall block would be placed at the ends of the lagging elements to allow for access and easy removal. The height of the lagging element can be adjusted accordingly. The following options are for a scenario when either a forklift isn't present or a forklift with the proper loading capacity isn't present and a crane must be used.

3.4.1 Suction Cups Lift Assembly

Suction cup lift assemblies are most commonly used on flat objects such as glass sheets and have sufficient capacity for the lagging elements to be lifted. Figure 20 shows a suction cup system for this application. The suction cups would be a two to six inches in diameter to attach to the top surface of the lagging elements which is about seven inches wide. This assembly works by placing the suction cups to the top surface of the lagging elements. Vacuuming out the air to achieve near perfect suction (~14psi or 28in.Hg), then lifting the lagging elements out of the piles with a crane. Advantages of using this system are that no additional lifting holes or hooks need to be created on the lagging elements, which is more aesthetically pleasing and the lagging elements can be lifted directly out of the soldier piles very efficiently without the use of a forklift. A disadvantage for this system is that in order for the suction cups to work properly, they need to be attached to a flat surface.

Figure 20 Suction Cup Lift Assembly (Anver, 2019)

3.4.2 Choker & Chain Lift Assembly

This method is the most common one to lift objects which involves wrapping a choker around a beam at two separate locations, evenly spaced, and attaching the chokers to a hook

before a crane lifts the lagging element. Figure 21 is a schematic of a choker-chain lift assembly. This system would be used in this application by lifting a lagging element approximately three inches with a forklift and attaching the chokers around the elements and lifting them with a crane. An advantage of this system is that it can be used on a variety of larger elements and it doesn't need a flat surface. A disadvantage for this system is that each lagging element has to be pryed and then lifted first with a forklift in order for a choker to go underneath the lagging element to wrap around. Such procedure (maneuver) could lead to complications in terms of ease of lifting out perfectly straight lagging element.

Figure 21 Choker and Chain Lift Assembly (Peerless Chain, 2015)

3.4.3 Embedded/Bonded Hooks on Front and Back Surface

This system involves embedding hooks into the reinforced concrete or timber lagging elements or bonding hooks onto the FRP bridge deck (Figure 22). A crane with four chains would be needed to lift the elements from the piles. Two hooks would be placed on each side to allow for uniformity. Advantages to this system are that the lagging elements can be lifted and placed straight. The disadvantage of this system is that hooks would be placed on the sides of the lagging elements which are not aesthetically pleasing and they may be damaged in field over the repeated use.

Figure 22 Hooks on Front and Back

3.4.4 Holes Drilled Through the Middle of Lagging Element

With holes drilled through an entire element, it would allow for a dowel to be placed through the element for lifting purposes. The holes would be drilled after the timber and reinforced concrete options were wrapped in GFRP and the diameter of the holes would be covered in a resin (becomes matrix after cure) to prevent local deterioration (Figure 24). No additional resin would need to be added to the FRP deck system after the holes are drilled through the lagging element. Advantages of this system is that it is easy to implement, but has potential issues long term related to weathering of the holes. It is more aesthetically pleasing and the holes can act as “weep holes” to drain any water that might accumulate behind the wall.

Figure 23 shows an alternative lifting mechanism for lifting lagging elements with holes drilled through the middle. The system involved placing a steel C shaped piece onto the hole with a rod running through the hole. This system allows for only two chains to be needed. The holes would have to be drilled within the top half of each lagging element in order for this system to be implemented.

Figure 23 Lifting Mechanism for Holes Drilled Through the Middle (Schultes, 2019)

Figure 24 Holes Drilled Through Middle

3.4.5 Small Holes Drilled Through Bottom of Lagging Element

If the reinforced concrete option is used, then the largest radius of the circles drilled out can be 0.5 inches because of the 1 inch concrete cover and it is recommended that at least five 0.5 inch dowels are used when lifting the lagging elements out of the soldier piles. If the timber or FRP prodeck options are used then the radius of the drilled circle can be 1 inch. The holes would either be drilled after the FRP was added or built in the concrete form. The holes would be coated with a resin to protect the elements from weathering affects. An advantage of this system

is that the holes are small and the bottom of the lagging elements so the holes wouldn't be as visible as the other options. A disadvantage is that if the reinforced concrete option is used then the radius can't be larger than 0.5 inches which means that at least 5 dowels would have to be used when lifting the lagging elements and small holes may be difficult to drill at the end of an object. Figure 25 shows 0.5 inch radius holes spaced at 6 inches but the diameter and spacing can vary depending on the option used for the lagging element.

Figure 25 Small Holes Drilled Through the Bottom

3.5 Drainage Systems

After the installation of the retaining wall the ABL site will consist of the rock wall slope, a ditch, the retaining wall, and the road. Water from the rock wall slope will go into the ditch and flow downhill, but water from the road will flow into the retaining wall which is why a drainage system must be put in place. Roads are typically constructed in a triangular cross section with the top of the triangle (crown) in the center of the road which allows for the water from each half of

the road to flow to the sides which prevents ponding on the road. Two drainage options are presented in this report.

3.5.1 Raised Lagging Elements w/ Gravel

This option involves raising the plank wall by 4 inches and either filling the area underneath with coarse gravel or leaving it empty so water can flow freely. This would be constructed by using 4 inch tall blocks at the bottom of the lowest lagging element, shown in Figure 26. The blocks on both sides of the lagging element would support the weight of all four planks. This method allows for easy maintenance and it also allows for water to be able to evenly flow under the wall.

The calculations to support this design are presented in Appendix A. A reinforced concrete lagging element is used for the calculations because it is the heaviest material considered for this project. The bending stress on the bottom lagging element is checked because there are three other elements on top of it which contribute a dead load constantly acting on it. The pressure of the foundation on the sandstone rock is also checked to ensure the vertical load doesn't produce a higher stress than the allowable bearing capacity.

Figure 26 Raised Lagging Elements w/ Gravel Drainage

3.5.2 Raised Concrete Foundation

Raising the concrete foundation to 3'-4" inches or removing the top 4 inches of the rocks around the foundation would allow for 4 inches of the foundation to be above grade. The lagging elements would be sitting on the foundation rather than on blocks. The advantage of using this system is that a block isn't needed and the lagging elements rest on the foundation. The disadvantage of this system is that the top section of the foundation is visible.

3.6 Conclusions

The retaining wall designs presented herein will prevent any rockfall from traveling to the road and withstand any inducing stresses larger rocks put on the wall. A total of six designs are presented herein to include options for steel pile spacing and lagging elements. In addition to the design options, lifting and drainage system options are available. The most practical lifting system would be to use a forklift to remove the lagging elements but if the proper forklift isn't available then the other options may be considered. Options are presented to allow for the owner to pick which is preferred and minimal changes to the design don't affect the load capacities.

CHAPTER 4 EXPERIMENTATION AND TESTING OF RESIN INFUSION ON USED RAILROAD TIES

4.1 Introduction

GFRP wrapped timber members the size of the lagging are not currently commercially produced in large scale, thus an experimental program was conducted to evaluate their strength and compare to theory. To simplify the process, used timber railroad ties were used, and both resin infusion and hand lay-up used to strengthen the ties with GFRP wraps. The railroad ties are 7 inches thick and 8.5 feet long, which is nearly the same as the thickness and width used with the 8 foot pile spacings. A total of seven samples were tested and reported herein. One control sample without any FRP, three samples with seven layers of GFRP wrapped using hand lay-up, and three samples with seven layers of GFRP using resin infusion. Table 2 shows details of which manufacturing process, resin type, and static testing that was used for all the samples tested.

Table 2 Railroad Tie Testing Samples

Sample No.	FRP Manufacturing Process	Resin Used	Static Test Used
1	No FRP (Control)	NA	3 Point
2	Hand Lay-up	Vinyl Ester	3 Point
3	Hand Lay-up	Vinyl Ester	3 Point
4	Hand Lay-up	Vinyl Ester	3 Point
5	Resin Infusion	Polyester	3 Point
6	Resin Infusion	Vinyl Ester	4 Point
7	Resin Infusion	Vinyl Ester	4 Point

4.2 Methods

4.2.1 FRP Reinforcement on Railroad Ties Manufacturing Process

Discarded timber railroad ties were used as core material for all the samples and all had obvious signs of wear and tear such as holes, cracks and warping. Figure 27 shows the typical condition of a used railroad tie. After procuring used ties, each tie was examined to ensure that each railroad tie was in “good enough” condition to be used as a core for refurbished railroad ties with GFRP wrap. “Good enough” condition consisted of railroad ties not having cracks larger than 3 inches in width and not having any visible warping.

Figure 27 Used Railroad Tie

Since the ties were exposed to environmental conditions and stored outside, a considerable amount of debris was present on the surface of the ties. In order to attain a good bond, the debris was then removed with a brush and the edges of the wood core were rounded (~1/2” diameter) to reduce stress concentrations and improve bonding around corners. Valspar

bonding water based primer was then applied before wrapping to ensure better FRP bond to the wood. A tie with the Valspar primer applied is shown in Figure 28.

Figure 28 Railroad Tie after Application of Polyurethane Primer

The fabric used for all samples is a Vectorply E-LTM 3615 0/90 glass fabric with a density of 17.92 oz/sq.yd. The fabric was wrapped around the railroad ties in the same manner for all cross ties. Five of the railroad ties used Hetron D 1222 vinyl ester resin, and one used a low viscosity, resin infusion grade poly ester resin, Polynt.

4.2.1.1 Hand Lay-up

The hand lay-up manufacturing process followed typical wet lay-up procedures wherein involves wrapping the railroad ties with a glass fabric that was previously saturated with a vinyl ester resin. The first step after applying a polyurethane primer is to add two layers of GFRP to the end of the ties covering the end grain to act as an end cap, then drive gang nails into the end

surface, as shown in Figure 29. Before working with resin, safety precautions must be taken to follow Material Safety Data Sheet (MSDS) procedures. These precautions include: manufacturing in an area with good ventilation or wearing a respirator/mask, using gloves, and wearing safety glasses. The resin used consists of Vinyl Ester resin with a catalyst and an accelerator (Methyl ethyl ketone peroxide (MEKP) and Cobalt Naphthenate). 0.75% and 0.25% by weight is the proportion of MEKP and Cobalt that is added to the resin. MEKP is added first to the resin and it is mixed with a mixer until an opaque amber color forms. The Cobalt is then added and mixed. At this point, at approximately 70°F, there is approximately 30 minutes of working time for the resin to harden. The amount of MEKP and Cobalt added to the resin can be increased or decreased to account for the different plant or lab environment temperature conditions.

Figure 29 Railroad Tie w/ End Cap and Gang Nail

After resin preparation including promoters and accelerators, the glass fabric was cut to provide seven wraps around the cross tie and laid on a plastic sheet. The resin was then applied with a roller in a uniform manner to ensure the proper fiber volume fraction is attained. The cross was placed on a rotating system, shown in Figure 30. The glass fabric was attached to one face of the pile with staples, then the tie was rotated with the fabric being pulled tight and wrapped as tightly as possible. Once the seventh layer was in place, the fabric was stapled to the tie and it is removed and placed on a flat surface to cure. It should be noted that the proper procedure for curing is to leave the railroad tie on the rotating apparatus and constantly rotate the tie at a slow pace until the resin has partially cured so that sagging of wet wrap can be eliminated but that method was not able to be performed for this project. Also, this allows for resin to be uniformly distributed throughout the cross tie and prevent resin accumulation at the bottom.

Figure 30 Railroad Tie Rotation Set-up

4.2.1.2 Resin Infusion

. Since the used railroad ties had multiple cracks and holes throughout, resin and gravel was mixed together to form a putty that was applied over the surface of the cross tie to ensure that resin doesn't travel into the core which would cause for an excessive amount of resin to be used. Figure 31 shows the completion of this process.

Figure 31 Railroad Tie after Application of Resin & Gravel Putty

The Valspar primer is also added to the used railroad ties and 12 inches of glass fabric is dry wrapped at the ends, as shown on the tie on the left of the pallet in Figure 32. The next step was to cut the glass fabric and wrap the tie, as shown in Figure 33. Staples were also used to hold the fabric in place. It should also be noted that it was more difficult to wrap the railroad ties with dry fabric compared to pre saturated fabric since pre saturated fabric allows for the layers to bond together throughout the process whereas dry fabric doesn't bond to the wood core.

Figure 32 Dry End Caps added to Railroad Tie

Figure 33 Wrapping Railroad Tie with Dry Fabric

The two VE resin infused samples were set up the same way which is shown in Figure 34. After dry wrapping seven layers of glass fabric around the core, the samples were wrapped with peel ply and a resin flow medium (mesh) was stapled around the entire sample. The peel ply allows for the resin flow medium to peel off of the sample after the resin has cured instead of bonding to the fabric. The samples were then placed on a table with a 3 mil plastic sheet lying on it. The 2 resin feed lines (bottom) and 1 vacuum line (top) were both spiral tubing, as shown at the bottom of the sample in Figure 34. Once the spiral tubing was placed, a solid tube was attached to them to either be placed in the resin bucket or attached to the vacuum pump. The vacuum bag was sealed with sealant tape, and the vacuum turned on until the pressure reached a minimum of 28 in.Hg. At this point, the resin tubes were opened and the resin was allowed to saturate the entire sample, which takes approximately 5 to 10 minutes. After the entire sample is saturated in resin the ends were sealed to prevent air from entering the vacuum bag and the sample is left to cure which takes approximately 20 more minutes.

Figure 34 Resin Infusion Set-up

The sample with the low viscosity poly ester resin was set up in the same manner as the other two samples, except no peel ply or resin flow medium was used. The low viscosity of the resin allowed it to flow through the voids in the fabric faster. The vacuum line used was also a mesh material rather than spiral tubing, which doesn't affect the infusion process.

Trial and error is a large part of resin infusion because even if the proper materials and conditions are met, trial and error must be conducted to determine spacing and amount of resin flow and vacuum lines. Figure 35 shows one of the first railroad ties resin infused and a dry streak is shown in the middle because the proper amount of resin flow lines wasn't used so the resin wasn't able to flow far enough on the railroad tie before it cured. A glass fabric and high viscosity resin was used for this trial. Another trial was performed on a carbon fabric with a low viscosity resin. The resin wasn't able to flow through the carbon fabric with and without a resin flow medium. The carbon fibers have a diameter of approximately 7 microns and glass fibers have a diameter of approximately 20 microns. This means that the void space in the glass fabric is approximately three times as large as the void space in the carbon which is why resin wasn't able to flow through the carbon fabric.

Figure 35 Resin Infusion Trial and Error

4.2.2 Static Testing

4.2.2.1 3 Point Bending Test

Five of the seven tests performed were three point bending test with 7 feet test spans, shown in Figure 36. A steel spacer beam was placed at mid span to cover the distance between actuator and the railroad tie. A Linear Variable Differential Transformer (LVDT) was set up to measure the deflection for samples 2, 3, 4 and 5. A strain gage was used for the tension side for samples 4 and 5. Once the samples have been set-up, load was applied until either a debond occurred in the core or between the core and GFRP. Due to the excessive deterioration in the railroad ties, a majority of the failures occurred from a debond in the core.

Figure 36 3 Point Bending Test Set-up for Sample 5

4.2.2.2 4 Point Bending Test

The 4 Point Bending Test was used for samples 6 and 7 to limit the amount of shear acting on the sample. This test was set up with a 7 feet span and the loads were applied at each third of the sample. Figure 37 shows the set-up for the 4 point bending test. An LVDT and a strain gage was used on both samples. Unlike the three point bending test, an obvious debond didn't occur in a 4 point bending test before failure because a deflection less than half of the three point bending test was produced which causes for less buckling on the top surface. The failure was determined from the peak load, i.e. before the test sample stopped taking any extra load, but additional deformation continued.

Figure 37 4 Point Bending Test Set-up for Sample 6

4.2.2.3 Coupon Tension Test

One coupon was removed from sample 5 and tested under tension in accordance with ASTM D 3039 Standard Test Method for Tensile Properties of Polymer Matrix Composite Materials. A 1"x15"x0.16" sample was cut from sample 5 after testing and it was loaded at a rate of 0.05 in/min until rupture.

4.2.3 Nondestructive Digital Tap Hammer Testing

A nondestructive digital tap hammer was used to determine the void space between materials to compare the hand lay-up vs resin infusion process. The digital tap hammer works by tapping a section of the sample and a value in microseconds (μS) will display. An accelerometer is in the head of the hammer and when it taps an object it translates a force-time into a voltage pulse. The pulse width is then computed and the value is displayed on the screen. Figure 38 shows the

digital tap hammer used for testing. Higher values attained from the digital tap hammer indicate that a delamination is present in the FRP or a disbond between the core and FRP is present. (Georgeson, Lea, & Hansen, 1996)

Figure 38 Digital Tap Hammer

Two samples (Samples 2 and 5) were tested in this report, A total of ten taps were performed on each sample at evenly spaced locations, five on the compression and five on the tension side. Sample 5 buckled on the compression side near the location of the applied load and this location was tested as well.

4.3 Results

4.3.1 Static Testing

4.3.1.1 3 and 4 Point Bending Tests

Table 3 shows a summary of all the testing data collected from the 7 tests performed. Figure 39 shows the stress vs strain diagrams for the 4 samples that had strain gages and Figure 40 shows the load vs deflection diagrams for all of the samples except Sample 1. Sample 1 failed at approximately 6 kips and the other six samples had nonlinearities around 6 - 8 kips in the load vs deflection diagrams. These nonlinearities can be attributed to the actuator transitioning from a low pressure to high pressure. The slope then decreases and levels out.

Samples 2-7's peak bending stress is calculated using the composite moment of inertia. The composite moment of inertia is calculated using the modular ratio which is the same method used for the design in Chapter 3. The design uses the same fabric configuration as used for wrapping the samples which gives the same modulus of elasticity values for the GFRP. The modulus of elasticity used for the timber is 0.55×10^6 psi, which was attained from previous testing (Abhari, 2007). The GFRP thickness used for the calculations is 0.16 inches since this was the value attained from the sample cut for the coupon testing.

Table 3 Static Testing Summary of Results

Sample No.	Static Test Used	Peak Load (lb)	Deflection at Peak Load (in)	Strain at Peak Load (microstrain)	Max Moment (lb x ft)	I_{comp} (in ⁴)	Peak Bending Stress (psi) $\frac{Mc}{I}$	Average Bending Stress (psi)	E (10 ⁶ psi)
1	3 Point	5,894	NA	NA	10,315	NA	275	275	NA
2	3 Point	16,515	1.03	2,154	28,901	338.6	277.4	289	1.42
3	3 Point	16,905	2.23	NA	29,584	400.3	258.7		NA
4	3 Point	21,537	1.99	NA	37,690	400.3	329.5		NA
5	3 Point	24,004	1.30	5,198	42,007	412.1	356.8	284	0.98
6	4 Point	16,798	0.60	1,943	19,598	400.3	171.4		1.67
7	4 Point	27,659	0.90	1,998	36,878	400.3	322.4		1.75

Figure 39 Stress vs Strain of Samples 2, 5, 6, and 7

Figure 40 Load vs Deflection of Samples 2 - 7

Both sets of FRP manufacturing methods, hand lay-up and resin infusion, had one peak bending stress outlier in the sets tested. The load vs deflection diagram shows that all three resin infused samples had a higher stiffness but the modulus of elasticity is inconsistent between the resin infused samples for the stress vs strain diagram. Sample 4 had a peak bending stress approximately 25% higher than the other two hand lay-up samples tested and Sample 6 had a peak bending stress approximately 50% less than the other two resin infusion samples tested. If these two samples were discarded, the average peak bending stresses for the two sets of samples would be 268.1 psi and 339.6 psi. This would indicate that the vacuum assisted resin infusion produces stronger bond, allowing the samples to reach a higher bending stress to failure. It can be assumed that the outliers were from the wide range of weathering the railroad ties had previously undergone, causing some to be in better condition than others.

No visible debonds occurred on sample 2 and the load vs deflection diagram shows that it stopped taking a load while continuing to deflect around 16 kips. After 16 kips it can be seen that the load was continuously applied but it never passed 16 kips. The failure can be assumed to be in the timber since no visible debond occurred.

Figure 41 and Figure 42 show the 3 point bending test performances of samples 3 and 4. Local buckling occurred on the top surface near the location of applied load. This buckling effect of FRP is visible in sample 3 but not in sample 4. There is a slight discoloration at 4 locations where local buckling occurred for sample 3 and it reached failure shortly after this debond. It can be assumed that a nonvisible debond occurred on Sample 4, which depends on quality of wood samples. Both samples behaved similarly in that at a deflection of approximately 1.6 to 1.7 inches the debond occurred and the samples continued to take a smaller amount of load before failing.

Figure 41 Sample 3 after Testing

Figure 42 Sample 4 after Testing

Sample 5 has the most distinguishable debond occur from local buckling effects near the applied load. Figure 43 shows the debond that occurred near the applied load of 21 kips. The stress vs strain diagram shows that the initial FRP buckling occurs at approximately 4,000 psi. After the debond occurred it continued to take a small amount of load then failed at the moment when any additional load on the sample was no longer able to be applied.

Figure 43 Sample 5 Debond

Samples 6 and 7 were the only two samples under 4 point bending and these two samples didn't have as noticeable of a failure as the other samples on the load vs deflection and stress vs strain diagram. Since a 4 point bending test produces smaller deflection, there were no local buckling effects on the samples and no debonds were visible. Neither the load vs deflection nor stress vs strain diagram suggest that a debond occurred as in samples 3 and 4. Both samples reached a point that any increase in load was possible. Sample 6 levels out on both diagrams, load vs deflection and stress vs strain, which is when the load stops being applied. At approximately 27,500 lbs on the load vs deflection diagram (Figure 40), sample 7 levels out in terms of carrying any extra load. This aspect is not able to be seen on the stress vs strain diagram (Figure 39).

4.3.2 Coupon Tension Test

The coupon tension test was performed to determine the modulus of elasticity of GFRP without any timber and the results are shown in Figure 44. The modulus of elasticity calculated

for the coupon tension test is 2.32×10^6 psi and it was determined from the slope between 10% and 40% of peak stress. The modulus of elasticity for used timber was recorded in a different report to be 0.509 and 0.589×10^6 psi (Abhari, 2007). This value is low because of the weather related deterioration of used railroad ties as a core material

Figure 44 Coupon Tension Test Stress vs Strain of FRP from Sample 5

4.3.1 Nondestructive Tap Hammer Testing

Table 4 shows the tap hammer based results for samples 2 and 5 being tested at ten separate locations. The average pulse width and standard deviation are displayed at the bottom. Sample 5 was resin infused and sample 2 was wrapped using the hand lay-up method. The average pulse width for Sample 5 is 200 μS lower than Sample 2's and it also has a standard deviation of 93 μS less. The resin infusion samples had a visibly better bond and the nondestructive tap hammer testing shows that there is a smaller void space between the 7 layers of GFRP and timber. This

shows that the resin infused sample has a more consistent and better bond than the sample wrapped using the hand lay-up method.

Table 4 Digital Tap Hammer Testing Results

	Sample 2 Hand Lay-up	Sample 5 Resin Infusion
Compression	1169	1118
	1186	1081
	1110	1047
	1427	1100
	1504	1039
Tension	1314	1102
	1379	1078
	1218	1119
	1411	1169
	1244	1101
Average (μS)	1296.2	1095.4
STDEV (μS)	130.2	37.4

Sample 5 was the only sample that had a true debond occur between the GFRP and timber. Figure 45 shows the area near the location of the applied load buckled, creating a debond. This section was tested and the pulse width attained was 1350 μS . This value is only 50 μS larger than the average pulse width of Sample 2. This shows that the bond between GFRP and timber of sample 2 was similar to the bond that the resin infused sample had between the timber and GFRP after its debond occurred.

Figure 45 Sample 5 Debond

4.4 Discussion

The values attained from these tests are heavily dependent on the quality of railroad tie. Each set of three tests has one outlier. There is one sample wrapped with the hand lay-up process that has a peak bending stress of ~ 25% higher than the average of the other two samples. Of the three resin infused samples, one sample was approximately 50% lower than the average of the other two samples. This large variance is attributed to the difference in the level of deterioration of the railroad ties; in other words quality of the used tie employed as a substrate in our experiments.

Sample #5 was resin infused with low viscosity (~100 cps) resin which makes the resin infusion process easier. Due to the low viscosity of the polyester resin, the properties may have been affected because admixtures are used to achieve to lower viscosities for resins.

4.5 Conclusions

From the test data, the resin infusion process produced a better and more consistent bond because of vacuum pressure (~28in.Hg); thus results in higher peak bending stress values compared to hand lay-up. The average peak bending stress produced of the resin infused sample would be 75 psi (25%) higher than the bending stress for the samples wrapped using hand lay-up. All samples had a similar “jagged” pattern at the beginning of the test which was assumed to be from the actuator transitioning from low pressure to high pressure. It should also be noted that this “jagged” pattern could be from an initial failure in the timber core and the load continues to increase due to confinement of GFRP on the core. Only 2 samples, samples 3 and 5, had a visible debond and it was suggested that no debond occurred for the samples under a 4 point bending test since 4 point testing minimizes stress concentrations and deflections.

More accurate results could be determined by wrapping new railroad ties, which have more consistent cross sections and mechanical properties, and determining if resin infusion still produces higher values, as suggested herein. The disadvantages of using resin infusion is the cost, time for manufacturing, and time for trial and error adjustments before perfecting the process. The pros and cons can be vary depending on the project and if time and cost are not considerations for efficiency, then resin infusion is a more beneficial manufacturing process than hand lay-up.

Section A.4 in Appendix A shows calculations for using a 7” x 9” used railroad tie as a lagging element for the retaining wall design and the peak bending stress is approximately 600 psi for the lagging element. These calculations include the composite moment of inertia. The average peak bending stress obtained from testing is approximately 285 psi. This shows that the used railroad ties have undergone too much deterioration, thus decreasing the timber modulus of

elasticity by almost a factor of 3. However, the allowable peak bending stress for Southern Pine timber is 1.5 ksi and new 7" x 9" railroad tie wrapped with GFRP, which makes it an adequate option for the retaining wall design.

CHAPTER 5 CONCLUSIONS AND FUTURE RESEARCH

5.1 Conclusions

A site with a rocky slope with rockfall issues was evaluated and multiple retaining wall options were considered for the site. Due to the 7.1 feet bounce height of small rocks and limited space between the road and ditch by the slope, a soldier pile/lagging wall was determined to be the most adequate options. This design consists of steel wide flange piles with a cylindrical concrete footer and four 2 feet tall lagging elements for each span. Six options were presented to allow for the owner to choose from. These designs consisted of the following:

- 10 feet span between soldier piles
 - Timber with GFRP wrap
 - Reinforced concrete with GFRP wrap
- 8 feet span between soldier piles
 - Timber with GFRP wrap
 - Reinforced concrete with GFRP wrap
 - FRP prodeck

Four of the six options involve reinforcing a lagging element with GFRP. Hand lay-up is the more common method for reinforcing an object but resin infusion creates compressive forces on the object being infused which creates a better fiber to resin ratio, creating a stronger element. It is recommended that the lagging elements reinforced with GFRP are resin infused to create the best quality material.

In addition to the retaining wall designs, options were presented for water drainage systems instead of weep holes and lifting the lagging elements to allow for removal of debris

behind the wall. The first drainage system options involve lifting the lagging elements with a 4 inch block between the foundation and bottom lagging elements. The second drainage option involves leaving 4 inches of the foundation above the ground to allow for the bottom lagging element to rest on it rather than on the ground. The simplest lifting method for removing the lagging elements is to use a forklift. The heaviest lagging element option is approximately 1800 lbs which a forklift with the proper loading capacity can carry. There are five options that can be implemented if either a forklift isn't available at the site or a forklift with the proper lifting capacity isn't at the site.

Currently, there aren't any structural members with the dimensions listed in the design commercially available so experimental testing was performed on used railroad ties with dimensions of 7" x 9". Timber is the least expensive option of the three materials presented which makes it the most probable option to be used. Used railroad ties are easily attainable as railroad ties are replaced when deterioration from weathering has become too severe for use in the field.

Seven railroad ties were tested under bending. One railroad tie was a control tie, three were wrapped using the hand lay-up process and three were wrapped using the resin infusion process. The purpose of testing the used railroad ties with GFRP was to determine if the ties could be used as a lagging element option and to see how resin infusion compared to hand lay-up for direct comparisons. The following were the conclusions determined from the testing:

- Used railroad ties reinforced with GFRP are not an adequate lagging element option for the retaining wall design as the bending stress for the design is 300 psi higher than the average bending stress of the tested railroad ties.
- Resin infusion produces a more consistent and better bond than hand lay-up.

- Resin infusion produces higher peak bending stress values if timber core samples have uniform mechanical properties.
- Resin infusion is a more delicate process and needs a better appreciation for specific materials and conditions to get successfully infused samples.
- Tests performed couldn't determine whether resin infusion produced a higher modulus of elasticity due to non-uniform mechanical properties of wood cores that have been used in the testing.

5.2 Future Research

A majority of this report presented a retaining wall design and the only future work for the design would be to study the design after construction to determine how effective it work. However, determining the effectiveness of resin infusion in structures has more research to be done for infusing wood core with FRP fabric as a shell material. The report herein compared resin infusion with hand lay-up on used and creosote treated railroad ties which affects the bond strength and bond surface area between the GFRP and timber. The following studies should be conducted to get a better understanding on the effectiveness of resin infusion:

- Ensure resin infused and hand lay-up samples are constructed with as much care as possible to ensure a minimal amount of void space between FRP and core material
- Perform all tests as a distributed load to simulate load acting on a lagging element for retaining wall design

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APPENDIX A RETAINING WALL DESIGN CALCULATIONS

A.1 10 ft span between soldier piles

A.1.1 Soldier Pile Design

$$\text{Soil Pressure} = 120 \frac{\text{lb}}{\text{ft}^3} \times 8 \text{ ft} = 960 \frac{\text{lb}}{\text{ft}^2}$$

*8ft = height of steel pile above ground (Pressure at bottom)

$$\text{Pressure on Pile} = 960 \frac{\text{lb}}{\text{ft}^2} \times 10 \text{ ft} = 9600 \frac{\text{lb}}{\text{ft}}$$

Pressure Distribution (Figure 46):

Figure 46 Triangular load distribution on 10' Span

* Triangular load distribution because pressure peaks at bottom due to soil accumulation

(Note: Since the wall height is modest, we did not account for soil resistance of concrete circular footers below the ground line)

Try W8x48 A572 Gr. 50 Steel

Given ASD Properties (AISC Manual):

$$Z_x = 49 \text{ in}^3 \quad \text{Allowable Section Modulus}$$

$$\frac{V_{nx}}{\Omega_v} = 68 \text{ k} \quad \text{Allowable Shear Capacity}$$

$$\frac{M_{px}}{\Omega_b} = 122 \text{ k} \times \text{ft} \quad \text{Allowable Moment Capacity}$$

$$I = 184 \text{ in}^4$$

$$A = 14.1 \text{ in}^2$$

$$f_y = 50 \text{ ksi}$$

Figure 47 W8x48 Cross Section

$$\text{Max } M_{\text{on soldier pile}} = 9600 \frac{\text{lb}}{\text{ft}} \times \frac{8\text{ft}}{2} \times \frac{8\text{ft}}{3} = 102,400 \text{lb} \times \text{ft}$$

$$102.4\text{k} \times \text{ft} < 122\text{k} \times \text{ft} \quad \mathbf{OK}$$

$$Z_x = \frac{M}{\frac{f_y}{\Omega}} = \frac{102.4\text{k} \times \text{ft} \times 12 \frac{\text{in}}{\text{ft}}}{\frac{50 \text{ksi}}{1.67}} = 41.04 \text{in}^3$$

$$\Omega = 1.67 = \text{ASD Safety Factor (AISC)}$$

$$41.04 \text{in}^3 < 49 \text{in}^3 \quad \mathbf{OK}$$

$$\text{Max } V = 9600 \frac{\text{lb}}{\text{ft}} \times \frac{8\text{ft}}{2} = 38.4\text{k}$$

$$38.4\text{k} < 68\text{k} \quad \mathbf{OK}$$

$$\Delta_{\text{allowable}} = \frac{L}{180} = \frac{8\text{ft} \times 12 \frac{\text{in}}{\text{ft}}}{180} = 0.533 \text{in} \quad (\text{ASCE Cantilever Beam}) \quad (\text{BS EN 1990 Cantilever Beam})$$

$$\Delta_{\text{actual for cantilever beam}} = \frac{wl^4}{30EI} = \frac{9600 \frac{\text{lb}}{\text{ft}} \times \frac{\text{ft}}{12\text{in}} \times (8\text{ft} \times 12 \frac{\text{in}}{\text{ft}})^4}{30 \times 29,000,000 \text{psi} \times 184 \text{in}^4} = 0.424 \text{in} \quad (\text{AISC Manual})$$

$$0.424 \text{in} < 0.533 \text{in} \quad \mathbf{OK}$$

$$f_{\text{allowable}} = \frac{f_y}{\Omega} = \frac{50 \text{ksi}}{1.67} = 29.94 \text{ksi}$$

$$f_b = \frac{M_c}{I} = \frac{102.4 \text{lb} \times \text{ft} \times 12 \frac{\text{in}}{\text{ft}} \times 4.25 \text{in}}{184 \text{in}^4} = 28.4 \text{ksi}$$

$$28.4 \text{ksi} < 29.94 \text{ksi} \quad \mathbf{OK}$$

$$c_{\text{top half}} = \frac{A_1 y_1 + A_2 y_2}{A_1 + A_2} = \frac{.685 \text{in} \times 8.11 \text{in} \times 3.9 \text{in} + 0.4 \text{in} \times 3.57 \text{in} \times 1.79 \text{in}}{.685 \text{in} \times 8.11 \text{in} + .4 \text{in} \times 3.57 \text{in}} = 3.47 \text{in}$$

$$Q = Ac = \frac{14.1 \text{in}^2}{2} \times 3.47 \text{in} = 24.46 \text{in}^3$$

$$\tau = \frac{VQ}{It} = \frac{38.4\text{k} \times 24.46 \text{in}^3}{184 \text{in}^4 \times 0.4 \text{in}} = 12.76 \text{ksi}$$

$$F_{v \text{ allowable}} = 0.3F_u = 0.3 \times 65 \text{ksi} = 19.5 \text{ksi} \text{ (AISC)}$$

$$12.76 \text{ksi} < 19.5 \text{ksi} \quad \mathbf{OK}$$

A.1.1.1 Flange bending due to load from 1 ft deep (along the wall depth) lagging

$$\text{Load on a 1 ft deep section} = 120 \frac{\text{lb}}{\text{ft}^3} \times 1 \text{ft} \times 8 \text{ft} = 960 \frac{\text{lb}}{\text{ft}}$$

$$\text{Load on a 1 ft deep flange section} = 960 \frac{\text{lb}}{\text{ft}} \times \frac{10 \text{ft}}{2} = 4800 \text{lb}$$

$$\text{Distributed load across 3 inches, i.e. half flange} = \frac{4800 \text{lb}}{3 \text{in}} = 1600 \frac{\text{lb}}{\text{in}}$$

*flange width is 3.2425 inches but use 3 inches for workability purposes

b=12in 12 in deep section along the pile depth – Design depth for pressure computation

d=0.685in Thickness of flange

$$\text{Max } M_{\text{flange}} = 1.6 \text{kips} \times 3 \text{in} \times \frac{3 \text{in}}{2} = 7.2 \text{k} \times \text{in}$$

$$f_{\text{allowable}} = \frac{f_y}{\Omega} = \frac{50 \text{ksi}}{1.67} = 29.94 \text{ksi}$$

$$f_b = \frac{Mc}{I} = \frac{7.2 \text{k} \times \text{in} \times \frac{0.685 \text{in}}{2}}{\frac{12 \text{in} \times 0.685^3}{12}} = 7.67 \text{ksi}$$

$$7.67 \text{ksi} < 29.94 \text{ksi} \quad \mathbf{OK}$$

$$f_v = \frac{V}{A} = \frac{4.8 \text{k} \times 1.5}{12 \text{in} \times 0.685 \text{in}} = 0.876 \text{ksi}$$

$$0.876 \text{ksi} < 19.5 \text{ksi} \quad \mathbf{OK}$$

Hence, a W8x48 A572 Gr. 50 Steel Pile is adequate (Figure 47)

A.1.2 Foundation

Type of soil: sandstone rock

$$\text{Unit Weight } (\gamma) = 155 \frac{\text{lb}}{\text{ft}^3}$$

Typical Friction Angle of Soil (ϕ') = 30°

Cohesion (c') = 0

Terzaghi's Bearing Capacity Factors –

(Table 4.1 Principles of Foundation Engineering (8th Edition) by Braja M. Das)

$$N_c = 37.14$$

$$N_q = 22.46$$

$$N_\gamma = 19.13$$

$$\begin{aligned} q_u &= 1.3c'N_c + qN_q + 0.4\gamma BN_\gamma = 155 \frac{lb}{ft^3} \times 3ft \times 22.46 + 0.4 \times 155 \frac{lb}{ft^3} \times 4ft \times 19.13 \\ &= 15188 \frac{lb}{ft^2} \end{aligned}$$

$$q_{all} = \frac{q_u}{FS} = \frac{15188 \frac{lb}{ft^2}}{3} = 5062.7 \frac{lb}{ft^2}$$

*3 is the minimum Factor of Safety that can be used (Principles of Foundation Engineering (8th Edition) by Braja M. Das)

Assume 50% Moisture Content

$$\text{Unit Weight of Water } (\gamma) = 62.4 \frac{lb}{ft^3}$$

$$\gamma_{50\% MC} = 155 \frac{lb}{ft^3} \times 0.5 + 62.4 \frac{lb}{ft^3} \times 0.5 = 108.7 \frac{lb}{ft^3}$$

$$\begin{aligned} q_u &= 1.3c'N_c + qN_q + 0.4\gamma BN_\gamma = 108.7 \frac{lb}{ft^3} \times 3ft \times 22.46 + 0.4 \times 108.7 \frac{lb}{ft^3} \times 4ft \times 19.13 \\ &= 10651.3 \frac{lb}{ft^2} \end{aligned}$$

$$q_{all} = \frac{q_u}{FS} = \frac{10651.3 \frac{lb}{ft^2}}{3} = 3550.4 \frac{lb}{ft^2}$$

$$q = \frac{\frac{9600 \frac{lb}{ft}}{2} \times 8ft}{3ft \times 4ft} = 3200 \frac{lb}{ft^2}$$

$$3200 \frac{lb}{ft^2} < 3550.4 \frac{lb}{ft^2} \quad \textbf{OK}$$

A.1.3 Timber with 3 Layers of GFRP Wrap

*Designed using ASD Approach

GFRP wrap used: Sikawrap Hex-100-G (GFRP)

$$E = 3.97msi$$

$$t = 0.04in$$

Southern Pine Select Structural

NDS manual Table 4D

$$F_b = 1,500 \text{ psi}$$

$$F_v = 165 \text{ psi}$$

$$E = 1,500,000 \text{ psi}$$

After 3 layers of GFRP

Timber Dimensions = 6.5" x 23.5"

$$\begin{aligned} \text{Uniform Soil Pressure Load on 2' Wide Lagging Element} &= 120 \frac{lb}{ft^3} \times 8ft \times 2ft \\ &= 1920 \frac{lb}{ft} \end{aligned}$$

2 ft = Height for each plank

8 ft = Height of wall (Pressure at bottom)

$$Max M = \frac{wl^2}{8} = \frac{1920 \frac{lb}{ft} \times 10^2 ft}{8} = 24k \times ft = 288k \times in$$

$$\eta = \frac{3.97}{1.5} = 2.65$$

$$c = \frac{6.5in + 2 \times 0.12in}{2} = 3.37in$$

$I_{transformed(top\&bottom)}$

$$= \frac{23.5in \times (6.5in)^3}{12} + 2\left[\frac{62.3in \times (0.12in)^3}{12} + 0.12in \times 62.3in \times (3.31in)^2\right]$$

$$= 701.6in^4$$

*GFRP is wrapped on all four sides. Sides are neglected

$$f_{w/GFRP} = \frac{Mc}{I_{transformed}} = \frac{288k \times in \times 3.37in}{701.6in^4} = 1.38ksi$$

1.38ksi < 1.50ksi (NDS Manual) **OK**

$$\Delta_{allowable} = \frac{L}{240} = \frac{10ft \times 12\frac{in}{ft}}{240} = 0.5in \text{ (American Wood Council - All Other Structural Members)}$$

$$\Delta_{actual} = \frac{5wl^4}{384EI} = \frac{5 \times 1920\frac{lb}{ft} \times \frac{ft}{12in} \times (10ft \times 12\frac{in}{ft})^4}{384 \times 1,500,000\frac{lb}{in^2} \times 701.6in^4} = 0.41in$$

1,500,00psi = timber modulus of elasticity

701.6in⁴ = Composite modulus of elasticity using transformation method

0.41in < 0.5in

A.1.4 Reinforced Concrete with 3 Layers of GFRP

Designed without GFRP, GFRP is added for weathering effects

*Designed using LRFD

Dimensions = 6.5in x 23.5in

$$f'_c = 5ksi$$

$$f_y = 60ksi$$

$$d_t = 6.5in - 1in - 0.375in - 0.375 = 4.75in$$

Assume:

#3 stirrups

#6 bending reinforcement

In accordance w/ ACI, reinforced concrete lagging element will have a 1 in cover on the shorter depth (6.5in) and a 2 in cover on the larger depth (23.5in). The concrete won't be exposed to the earth which allows for it to have a 1 in concrete cover.

$$\text{Uniform Load on Lagging Element} = 120 \frac{lb}{ft^3} \times 8ft \times 2ft = 1920 \frac{lb}{ft}$$

$$M_u = \frac{1.6wl^2}{8} = \frac{1.6 \times 1.92 \frac{k}{ft} \times (10ft)^2}{8} = 38.4k \times ft = 460.8k \times in$$

$$M_n = \frac{M_u}{\phi} = \frac{460.8k \times in}{0.90} = 512k \times in$$

$$c_b = d_t \left(\frac{87,000}{87,000 + f_y} \right) = 4.75in \left(\frac{87,000}{87,000 + 60,000psi} \right) = 2.81in$$

$$c = 0.5 \times 2.81in = 1.41in$$

$$a = 0.85 \times 1.41in = 1.19in$$

$$C = 0.85f'cba = 0.85 \times 5,000psi \times 23.5in \times 1.19in = 119,327lb$$

$$A_s = \frac{C}{f_y} = \frac{119,327lb}{60,000psi} = 1.99in^2$$

Use 5 #6 bars on both sides (Double Reinforced)

$$A_s = 2.21in^2$$

$$a = \frac{A_s f_y}{0.85 f' c b} = \frac{2.21in^2 \times 60,000psi}{0.85 \times 5,000psi \times 23.5in} = 1.33in$$

$$c = \frac{1.33in}{0.85} = 1.56in$$

$$\epsilon_f = 0.003 \left(\frac{d_t - c}{c} \right) = 0.003 \left(\frac{4.75in - 1.56in}{1.56in} \right) = \frac{0.0061in}{in} > \frac{0.005in}{in} \quad \textbf{OK}$$

$$M_n = A_s f_y \left(d_t - \frac{a}{2} \right) = 2.21in^2 \times 60,000psi \left(4.75in - \frac{1.33in}{2} \right) = 541,671lb \times in$$

$$542k \times in > 512k \times in \quad \textbf{OK}$$

$$E = 57,000\sqrt{f'_c} = 57,000\sqrt{5,000psi} = 4,038,508psi$$

$$I = \frac{bd^3}{12} = \frac{23.5in \times (6.5in)^3}{12} = 537.8in^4$$

$$\Delta = \frac{5 \times 1.6 \times 160 \frac{lb}{in} \times (120in)^4}{384 \times 4,030,508psi \times 537.8in^4} = 0.32in$$

$$0.32in < 0.5in \quad \textbf{OK (i.e. elastic design w/ no concrete cracking)}$$

Shear Reinforcement

$$V_u = \frac{1.6wl}{2} = \frac{1.6 \times 1920 \frac{lb}{ft} \times 10ft}{2} = 15,360lb$$

$$V_c = 2\sqrt{f'_c} b d_t = 2\sqrt{5000} \times 23.5 \times 4.75in = 15,786.2lb$$

Check for Adequacy

$$= 8\sqrt{f'_c} b d_t = 8\sqrt{5000psi} \times 23.5in \times 4.69in = 63,144.6lb$$

$$V_n = \frac{V_u}{\phi} = \frac{15,360lb}{0.75} = 20,480lb$$

$$V_n > \frac{1}{2}V_c \quad \text{Use Stirrups}$$

Try #3 two legged stirrup

$$A_v = 0.11in^2 \times 2 = 0.22in^2$$

$$S = \frac{A_v f_y d_t}{V_n - V_c} = \frac{0.22in \times 60,000psi \times 4.75in}{20,480lb - 15,786lb} = 13.4in$$

Use #3 stirrups spaced at 12 inches c/c

A.2 8 ft span between soldier piles

A.2.1 Steel Soldier Pile

$$\text{Soil Pressure} = 120 \frac{\text{lb}}{\text{ft}^3} \times 8\text{ft} = 960 \frac{\text{lb}}{\text{ft}^2}$$

*8ft = height of steel pile above ground

$$\text{Pressure on Pile} = 960 \frac{\text{lb}}{\text{ft}^2} \times 8\text{ft} = 7680 \frac{\text{lb}}{\text{ft}}$$

Pressure on Pile Distribution (Figure 48):

Figure 48 Triangular Load Distribution on 8' Span Soldier Pile

* Triangular load distribution because pressure peaks at bottom due to soil accumulation (Note: Since the wall height is modest, soil resistance of concrete circular footers below the ground line was not accounted for)

Try W8x35 A572 Gr. 50 Steel

Given ASD Properties (AISC Manual):

$$Z_x = 34.7 \text{ in}^3 \quad \text{Allowable Section Modulus}$$

$$\frac{V_{nx}}{\Omega_v} = 50.3 \text{ k} \quad \text{Allowable Shear Capacity}$$

$$\frac{M_{px}}{\Omega_b} = 86.6 \text{ kxft} \quad \text{Allowable Moment Capacity}$$

Figure 49 W8x35 Cross Section

$$I = 127in^4$$

$$A = 10.3in^2$$

$$f_y = 50ksi$$

$$\text{Max } M_{on\ soldier\ pile} = 7680 \frac{lb}{ft} \times \frac{8ft}{2} \times \frac{8ft}{3} = 81,920lb \times ft$$

$$81.9k \times ft < 86.6k \times ft \text{ OK}$$

$$Z_x = \frac{M}{\frac{f_y}{\Omega}} = \frac{81.92k \times ft \times 12 \frac{in}{ft}}{\frac{50 ksi}{1.67}} = 32.83in^3$$

$$\Omega = 1.67 = \text{ASD Safety Factor}$$

$$32.83in^3 < 34.7in^3 \text{ OK}$$

$$\text{Max } V = 7680 \frac{lb}{ft} \times \frac{8ft}{2} = 30.1k$$

$$30.1k < 50.3k \text{ OK}$$

$$\Delta_{allowable} = \frac{L}{180} = \frac{8ft \times 12 \frac{in}{ft}}{180} = 0.533in \quad (\text{ASCE Cantilever Beam}) (\text{BS EN 1990 Cantilever Beam})$$

$$\Delta_{actual\ for\ cantilever\ beam} = \frac{wl^4}{30EI} = \frac{7680 \frac{lb}{ft} \times \frac{ft}{12in} \times (8ft \times 12 \frac{in}{ft})^4}{30 \times 29,000,000psi \times 127in^4} = 0.492in \text{ (AISC Manual)}$$

$$0.492in < 0.533in \text{ OK}$$

$$f_{allowable} = \frac{f_y}{\Omega} = \frac{50ksi}{1.67} = 29.94ksi$$

$$f_b = \frac{M_c}{I} = \frac{81.92k \times ft \times 12 \frac{in}{ft} \times 4.06in}{127in^4} = 31.43ksi$$

$$31.43ksi > 29.94ksi \text{ NOT OK}$$

The bending stress is 1.49 ksi larger than the allowable stress. Due to the additional bending resistance that the lagging elements provide on the soldier pile. The W8x35 beam can be used. The additional bending resistance that the lagging elements provide is neglected in all of the calculations.

$$c_{top\ half} = \frac{A_1 y_1 + A_2 y_2}{A_1 + A_2} = \frac{.495in \times 8.02in \times 3.813in + 0.31in \times 3.57in \times 1.79in}{.495in \times 8.02in + .31in \times 3.57in} = 3.37in$$

$$Q = Ac = \frac{10.3in^2}{2} * 3.37in = 19.04in^3$$

$$\tau = \frac{VQ}{It} = \frac{30.1k * 19.04in^3}{127in^4 \times 0.31in} = 14.57ksi$$

$$F_{v\ allowable} = 0.3F_u = 0.3 \times 65ksi = 19.5ksi \text{ (AISC)}$$

$$14.57ksi < 19.5ksi \quad \textbf{OK}$$

A.2.1.1 Flange bending due to load from 1 ft deep (along the wall depth) lagging

$$\text{Load on a 1 ft deep section} = 120 \frac{lb}{ft^3} \times 1ft \times 8ft = 960 \frac{lb}{ft}$$

$$\text{Load on a 1 ft deep flange section} = 960 \frac{lb}{ft} \times \frac{8ft}{2} = 3840lb$$

$$\text{Distributed load across 3 inches, i.e. half flange} = \frac{3840lb}{3in} = 1280 \frac{lb}{in}$$

*flange width is 3.20 inches but use 3 inches for workability purposes

b=12in 12 in deep section along the pile depth – Design depth for pressure computation

d=0.495in thickness of flange

$$Max\ M_{flange} = 1.28kips \times 3in \times \frac{3in}{2} = 5.76k \times in$$

$$f_{allowable} = \frac{f_y}{\Omega} = \frac{50ksi}{1.67} = 29.94ksi$$

$$f_b = \frac{Mc}{I} = \frac{5.76k \times in \times \frac{0.495in}{2}}{\frac{12in \times 0.495^3}{12}} = 11.79ksi$$

$$11.79ksi < 29.94ksi \quad \text{OK}$$

$$f_v = \frac{V}{A} = \frac{3.8k \times 1.5}{12in \times 0.495in} = 0.959 ksi$$

$$0.959ksi < 19.5ksi \quad \text{OK}$$

Hence, a W8x35 A572 Gr. 50 Steel Pile is adequate (Figure 49)

A.2.2 Foundation

Section A.1.2 shows the computations for the foundation with a 10 feet span which is adequate. The same size foundation must be used for the 8 feet span retaining wall as the pressure created is 250 psi too high for a 3ft x 3ft foundation. The bearing capacity is 3550 psi for a 3ft x 4ft foundation and the load on the soldier pile for an 8 feet span is less than the load on a soldier pile for a 10 feet span. Hence, a 3ft x 4ft wide circular concrete foundation is adequate for an 8 feet span retaining wall.

A.2.3 Timber with 3 Layers of GFRP Wrap

*Designed using ASD Approach

GFRP wrap used: Sikawrap Hex-100-G (GFRP)

$$E = 3.97msi$$

$$t = 0.04in$$

Southern Pine Select Structural

NDS manual Table 5A

$$F_b = 1,500 psi$$

$$F_{vx} = 165 \text{ psi}$$

$$E = 1,500,000 \text{ psi}$$

Designed without GFRP, GFRP is still used for weathering effects

Timber Dimensions = 6.5" x 23.5"

$$\text{Uniform Load on Lagging Element} = 120 \frac{\text{lb}}{\text{ft}^3} \times 2\text{ft} \times 8\text{ft} = 1920 \frac{\text{lb}}{\text{ft}}$$

2 ft = height of individual lagging element

8 ft = height of retaining wall (Pressure at bottom)

$$\text{Max } M = \frac{wl^2}{8} = \frac{1920 \frac{\text{lb}}{\text{ft}} \times 8^2 \text{ft}}{8} = 15.3\text{k} \times \text{ft} = 184\text{k} \times \text{in}$$

$$I = \frac{bd^3}{12} = \frac{23.5 \times 6.5^3}{12} = 538 \text{in}^4$$

$$f_b = \frac{Mc}{I} = \frac{184\text{k} \times \text{in} \times \frac{6.5\text{in}}{2}}{537.8 \text{in}^4} = 1.11 \text{ksi}$$

$$1.11 \text{ksi} < 1.50 \text{ksi} \quad \textbf{OK}$$

$$\Delta_{allowable} = \frac{L}{240} = \frac{10\text{ft} \times 12 \frac{\text{in}}{\text{ft}}}{240} = 0.5 \text{in} \text{ (American Wood Council All Other Structural Members)}$$

$$\Delta_{actual} = \frac{5wl^4}{384EI} = \frac{5 \times 1920 \frac{\text{lb}}{\text{ft}} \times \frac{\text{ft}}{12 \text{in}} \times (8\text{ft} \times 12 \frac{\text{in}}{\text{ft}})^4}{384 \times 1,500,000 \frac{\text{lb}}{\text{in}^2} \times 538 \text{in}^4} = 0.22 \text{in}$$

$$0.22 > 0.5 \text{in} \quad \textbf{OK}$$

A.2.4 Reinforced Concrete with 3 Layers of GFRP

*Designed using LRFD

Dimensions = 6.5in x 23.5in

$$f'_c = 5 \text{ksi}$$

$$f_y = 60ksi$$

$$d_t = 6.5in - 1in - 0.375in - \frac{0.625in}{2} = 4.81in$$

Assume:

#3 stirrups

#5 bending reinforcement

In accordance w/ ACI, the reinforced concrete lagging element will have a 1 in cover on the shorter depth (6.5in) and a 2 in cover on the larger depth (23.5in). The concrete won't be exposed to the earth which allows for it to have the preceding depths.

$$Uniform\ Load\ on\ Lagging\ Element = 120 \frac{lb}{ft^3} \times 2ft \times 8ft = 1920 \frac{lb}{ft}$$

$$M_u = \frac{1.6wl^2}{8} = \frac{1.6 \times 1.92 \frac{k}{ft} \times (8ft)^2}{8} = 24.6k \times ft = 295k \times in$$

$$M_n = \frac{M_u}{\phi} = \frac{295k \times in}{0.90} = 328k \times in$$

$$c_b = d_t \left(\frac{87,000}{87,000 + f_y} \right) = 4.81in \left(\frac{87,000}{87,000 + 60,000psi} \right) = 2.85in$$

$$c = 0.5 \times 2.85in = 1.42in$$

$$a = 0.85 \times 1.42in = 1.21in$$

$$C = 0.85f'cba = 0.85 \times 5,000psi \times 23.5in \times 1.21in = 120,848lb$$

$$A_s = \frac{C}{f_y} = \frac{120,848lb}{60,000psi} = 2.01in^2$$

Use 4 #5 bars on both sides (Double Reinforced)

$$A_s = 1.23in^2$$

$$a = \frac{A_s f_y}{0.85 f' c b} = \frac{1.23in^2 \times 60,000psi}{0.85 \times 5,000psi \times 23.5in} = 0.74in$$

$$c = \frac{0.74in}{0.85} = 0.87in$$

$$\epsilon_f = 0.003 \left(\frac{d_t - c}{c} \right) = 0.003 \left(\frac{4.75in - 0.87in}{0.87in} \right) = \frac{0.013in}{in} > \frac{0.005in}{in} \quad \mathbf{OK}$$

$$M_n = A_s f_y \left(d_t - \frac{a}{2} \right) = 1.23in^2 \times 60,000psi \left(4.81in - \frac{0.74in}{2} \right) = 327,672 \text{ lb} \times in$$

$$327,672lb \times in > 295,000lb \times in \quad \mathbf{OK}$$

$$E = 57,000\sqrt{f'_c} = 57,000\sqrt{5,000psi} = 4,038,508psi$$

$$I = \frac{bd^3}{12} = \frac{23.5in \times (6.5in)^3}{12} = 537.8in^4$$

$$\Delta = \frac{5 \times 1.6 \times 160 \frac{lb}{in} \times (96in)^4}{384 \times 4,030,508psi \times 537.8in^4} = 0.13in$$

$$0.13in < 0.5in \quad \mathbf{OK}$$

Shear Design

$$V_u = \frac{1.6wl}{2} = \frac{1.6 \times 1920 \frac{lb}{ft} \times 8ft}{2} = 12,288lb$$

$$V_c = 2\sqrt{f'_c} b d_t = 2\sqrt{5000} \times 23.5 \times 4.81in = 15,985.6lb$$

Check for Adequacy

$$= 8\sqrt{f'_c} b d_t = 8\sqrt{5000psi} \times 23.5in \times 4.81in = 63,942lb$$

$$V_n = \frac{V_u}{\phi} = \frac{12,288lb}{0.75} = 16,384lb$$

$$V_n > \frac{1}{2}V_c \quad \text{Use Stirrups}$$

Try #3 two legged stirrup

$$A_v = 0.11in^2 \times 2 = 0.22in^2$$

$$S = \frac{A_v f_y d_t}{V_n - V_c} = \frac{0.22in \times 60,000psi \times 4.81in}{16,384lb - 12,288lb} = 15.5in$$

Use #3 stirrups spaced at 12 inches c/c

A.3 Drainage Block Calculations

Bottom Lagging Element in Bending

$$\gamma_{concrete} = 150 \frac{lb}{ft^3}$$

$$W_{individual\ lagging\ element} = 150 \frac{lb}{ft^3} \times 2ft \times 7in \times \frac{ft}{12in} = 174.9 \frac{lb}{ft}$$

$$W_{all\ 4\ lagging\ elements} = 174.9 \frac{lb}{ft} \times 4 = 699.6 \frac{lb}{ft}$$

$$M = \frac{wl^2}{8} = \frac{699.6 \frac{lb}{ft} \times (10ft)^2}{8} = 8,745lb \times ft = 104,940lb \times in$$

$$I = \frac{7in \times (2ft \times 12in)^3}{8} = 12096in^4$$

$$E = 57,000\sqrt{5,000psi} = 4,030,509psi$$

$$f_b = \frac{104,940lb \times in \times 1ft \times 12 \frac{in}{ft}}{12096in^4} = 104.1psi \quad \textbf{OK}$$

$$\Delta = \frac{5wl^4}{384EI} = \frac{5 \times 699.6 \frac{lb}{ft} \times \frac{ft}{12in} \times (120in)^4}{384 \times 4,030,509psi \times 12096in^4} = 0.003in \quad \textbf{OK}$$

Foundation

$$P = 699.6 \frac{lb}{ft} \times 10ft = 6996lb$$

$$\sigma = \frac{P}{A} = \frac{6996lb}{\pi(2ft)^2} = 556.7psi$$

$$556.7 \text{ psi} < 3550 \text{ psi} \quad \textbf{OK}$$

A.4 Railroad Tie Calculations

$$\text{Uniform Load on Lagging Element} = 120 \frac{\text{lb}}{\text{ft}^3} \times 2\text{ft} \times 0.75\text{ft} = 720 \frac{\text{lb}}{\text{ft}}$$

0.75 ft = height of individual lagging element (9")

8 ft = height of retaining wall (Pressure at bottom)

$$\text{Max } M = \frac{wl^2}{8} = \frac{720 \frac{\text{lb}}{\text{ft}} \times 8^2\text{ft}}{8} = 5.76\text{k} \times \text{ft} = 69.12\text{k} \times \text{in}$$

$I_{transformed} = 400.3\text{in}^4$ – 7"x9" Railroad Tie Composite Moment of Inertia

$$f_b = \frac{Mc}{I} = \frac{69.12\text{k} \times \text{in} \times \frac{7\text{in}}{2}}{400.3\text{in}^4} = 0.604 \text{ ksi}$$

$$0.604\text{ksi} > 0.284\text{ksi} \quad \textbf{Not OK}$$

0.284 ksi is the average bending stress of the resin infused railroad ties

APPENDIX B RESIN INFUSED 3D NEEDLE FABRIC

B.1 Summary

This study investigates the tensile and bending properties of through thickness needle punched glass fabrics. The tests are compared with a two dimensional glass fabric to have a basis for comparison. The resin amount in each laminate was controlled by using the resin infusion method to allow for a minimal amount of parameters while comparing the results. The fiber volume fraction was also computed for each laminate to make calculation adjustments for stresses if applicable but each laminate had the same fiber volume fraction.

The hypothesis for these tests is that the through thickness needle punched glass fabric was going to have higher tensile and flexural properties but the needle punched fibers would provide additional interlaminar shear strength. The results are the opposite of what was predicted with the needle punched fabric having a 37% lower peak tensile stress than the two dimensional glass fabric. The flexure tests showed similar results but the results from the flexure tests can't be trusted because there was such a high deviation between the results. The needle punched fibers in the fabric damaged the two dimensional fibers during testing which is what caused a lower peak stress.

B.2 Introduction

Fiber Reinforced Polymers composites are gaining acceptance in the structural engineering industry for reinforcing or retrofitting structural elements. There are many production methods which include: hand lay-up, pultrusion, filament winding, resin infusion and others. There are also many types of fabrics used which include glass, carbon, aramid, boron and also many type of fabric configurations and densities. The fibers in these fabrics can also be oriented in many different directions. Along with the multiple types of fabric and production methods, there are multiple types of resins that can be used in the production of fiber reinforced polymer composites and these resins include vinyl ester, polyester, urethanes, epoxy, and others.

There have been attempts to improve in each of the elements of the fiber reinforced polymer composites and this study focuses on testing glass fibers with third dimensional needle poked fibers (poking perpendicular to the 2-D fabric) using vinyl ester in the resin infusion process. The hypothesis is that the three dimensional fibers will increase the interlaminar shear strength which will increase the tensile and bending strength by delaying interlaminar shear failure. Both tests are performed and a standard two dimensional glass fiber is tested to compare the results. The same resin infusion techniques and resin are used to ensure that the results can be directly compared.

B.3 Methods

Resin infusion is a process that uses vacuum pressure to drive resin into fabric layers. The process is tedious involving multiple steps. The first step is to acquire the desired amount of layers of fabric for the laminate to be made and cut each layer of fabric to the same size. Then the layers of fabric are to be wrapped with a 3 mil plastic sheeting and the edges are to be sealed with a sealant tape to get an airtight seal. Before completely sealing off the laminate, two spiral tubes must be placed horizontally across the laminate under the plastic sheeting with each tube attached to a tube that continues outside of the seal. One set of tubing is used to pull the resin through a bucket into the laminate and the other set of tubing is attached to a vacuum pump and it is used to get a uniform vacuum pressure across the laminate to allow for the resin to flow at a steady and uniform rate. The resin set of tubing is placed at one end of the laminate and the vacuum pump set of tubing is placed approximately 6" off the laminate on the other side of the laminate. Figure 50 below shows a laminate that is in the resin infusion process with all of the components labeled accordingly. The sample has been under the resin infusion process for approximately 30 seconds and the resin has traveled approximately 3 inches across the laminate.

Figure 50 Resin Infusion on a Laminate

A total of three laminates were made with two different fabrics and the same vinyl ester resin was used for all three laminates. The first laminate made was a four layers with 0/90 fibers and no out of plane needle punched fibers. The second laminate was also four layers but the fibers were primarily oriented in the ± 45 direction with some 0/90 fibers and this laminate had the three dimensional needle punched fibers in it. The third laminate was the same fabric as the second but twelve layers were made.

Once each laminate was made, multiple strips were cut out of the middle of the laminate and used for tension and flexure tests. The first set of testing on the second and third laminate was done with the fibers oriented in the ± 45 direction which had extraordinarily low results so the laminates were cut out again in the 0/90 direction to get more accurate results. A total of fifteen 1"x14", four 1"x7", and six 0.5"x7" strips were cut out for further mechanical testing.

The fiber volume weight fraction was also calculated and the calculations are shown in Section B.9. The calculations were done by cutting out a 3"x3" sample from each laminate and

weighing it. The dimensions for each side were taken at three different locations and the average was used. The thickness dimensions were taken at each corner and the average dimension was used for this as well.

B.3.1 Tension Testing

15 tensile tests were performed and ASTM D 3039 Standard Test Method for Tensile Properties of Polymer Matrix Composite Materials provided all of the parameters to be used for this test. The first set of samples tested was three samples from the first laminate. These samples had the approximate dimensions of 1"x14"x0.129". The second set of samples was 6 samples from the second laminate with three samples in the primarily +/-45 direction and the other three in the primarily 0/90 direction with the approximate dimension of 1"x14"x0.126". The third set of samples was 6 samples from the third laminate with three samples in the primary +/-45 direction and the other three samples in the primary 0/90 direction with the approximate dimension 1"x14"x0.289". These tests were performed by placing approximately 3 inches on each side in an FRP grip and applying a tensile load to rupture at a 0.05 in/min rate.

Figure 51 Tension Test Set-up

B.3.2 Flexure Testing

Ten flexural tests were performed and ASTM D 790 Standard Test Methods for Flexural Properties of Unreinforced and Reinforced Plastics and Electrical Insulating Materials provided all of the parameters to be used for this test. The first set of samples was two samples from the first laminate that had the dimensions of approximately 0.5"x7"x0.113". The second set of samples was four samples from the second laminate with two samples oriented primarily in the +/-45 direction and the other two samples primarily oriented in the 0/90 direction with the approximate dimensions of 0.5"x7"x0.121". The third set of samples was four samples from the third laminate with two samples oriented primarily in the +/-45 direction and the other two samples primarily oriented in the 0/90 direction with the approximate dimensions of 1"x7"x0.275". ASTM states that the span length for each sample must be sixteen times the thickness of the sample. For the first laminates' samples, the span length was 2" and for the third

laminates' samples the span length was 4.8". The tests were performed by placing the samples on two rollers with the required span length apart and applying a point load at mid span with the calculated load rate until rupture or debond occurs in FRP. If neither of these occurred then the load was stopped after it reached its peak and started to decrease with an increase in deflection or strain.

$$\text{Load Rate} = \frac{0.01 \times 2^2}{6 \times 0.12} = 0.056 \text{ in/min}$$

$$\text{Load Rate} = \frac{0.01 \times 4.8^2}{6 \times 0.275} = 0.140 \text{ in/min}$$

Figure 52 Flexural Test Set-up for 1st & 2nd Laminate

Figure 53 Flexural Test Set-up for 3rd Laminate

B.4 Results

A total of 25 different samples were tested under this study, fifteen tension tests and ten flexure tests. All of the tension tests were performed on an Instron 1000HDX instrument and the flexure tests were performed on an Instron 8500 instrument. Of the 25 samples tested, 17 had strain gages to obtain strain values as a function of applied load. Two out of each set of three for the tension tests and one out of each set of two for the flexure tests had strain gages bonded to plot load versus strain diagrams. The flexure test samples had a strain gage on both, the tension and compression side while the tension test samples only had one strain gage. The load and position were the only values obtained for the samples without strain gages.

B.4.1 Tension Testing

Table 5 shows the results and calculated values under tension and Figure 54 shows the stress vs. strain graph for each sample tested with a strain gage. Since the resin infusion process was able to compress twelve layers at a higher proportion than four layers, the thickness of the twelve layers wasn't three times the thickness of the four layer laminates. This caused a discrepancy in the stress values for the third laminate which is why a "theoretical thickness" value was created. The theoretical thickness just gives a uniform thickness for each layer of the laminates which allowed for the third laminate to be exactly three times the thickness of the first and second laminate. Even when using a theoretical thickness, the 1st laminate without any three dimensional needle poked fibers had the highest peak stress out of all the samples. The third laminate, which was a twelve layer laminate with the three dimensional needle poked fibers, had the highest modulus of elasticity out of all the samples. This is due to the lower peak strain at the peak stress.

Table 5 Tension Test Summary Values

Tension Tests Summary											
# Layers	Fiber Orientation	Sample	Width (in)	Thickness (in)	Theoretical Thickness (in)	Area (in ²)	Peak Load (lbf)	Peak Stress (lbf/in ²)	Theoretical Peak Stress (lbf/in ²)	Peak Strain	E (10 ⁶ psi)
4	0/90	1-1T	0.973	0.126	0.125	0.1226	6423	52391	52810	NA	NA
		1-2T	1	0.133	0.125	0.133	6601	49632	52808	21687	2.038
		1-4T	1	0.128	0.125	0.128	6349	49602	50792	20729	2.267
	+/-45	2-3T	0.968	0.124	0.125	0.12	1087	9056	8983	23116	0.774
		2-4T	0.976	0.129	0.125	0.1259	1128	8959	9246	11587	1
		2-5T	0.967	0.127	0.125	0.1228	1101	8965	9109	NA	NA
	0/90	2-6T (0-90)	1.01	0.12	0.125	0.1212	4117	33969	32610	26059	1.679
		2-7T (0-90)	1.02	0.122	0.125	0.1244	3983	32007	31239	20017	1.576
		2-8T (0-90)	1.02	0.135	0.125	0.1377	4312	31314	33820	NA	NA
		3-1T	0.97	0.303	0.375	0.2939	3375	11483	9278	NA	NA
12	+/-45	3-2T	0.97	0.303	0.375	0.2939	3444	11718	9468	5910	0.928
		3-3T	0.976	0.288	0.375	0.2811	3486	12402	9525	48775	0.793
		3-6T (0-90)	1.02	0.278	0.375	0.2836	12918	45556	33773	19987	2.415
	0/90	3-7T (0-90)	1.01	0.279	0.375	0.2818	12977	46052	34263	18443	2.471
		3-8T (0-90)	1.01	0.282	0.375	0.2848	11395	40008	30086	NA	NA

Figure 54 Tension Test Stress vs. Strain Diagrams

The first nine samples tested were three from the first laminate oriented in the 0/90 direction, and three from the second and third laminate oriented primarily in the +/-45 direction. Figure 56 through Figure 58 in section B.7 show the rupture pattern for each sample. The first laminates' samples rupture pattern was generally parallel to the tensile force while the second and third laminates' samples rupture patterns were in the +/-45 degree direction to the tensile force applied. Due to the extraordinarily low peak load results from the second and third laminates, the laminates were cut out again but in the perpendicular direction so that the fibers were primarily oriented in the 0/90 direction to get a more true peak load value. Figure 59 in section B.7 shows the rupture pattern for the 0/90 oriented second and third laminates and each samples' rupture pattern was generally parallel to the tensile force applied with the exception of 3-8 failing at the grip.

B.4.2 Flexure Testing

Table 6 shows the results for all of the flexure tests performed and Figure 55 shows the stress vs. strain diagram for each flexure sample tested with a strain gage. The results show that the third laminate oriented generally in the 0/90 direction had the highest average peak stress values by 15% but the first laminate oriented in the 0/90 direction had the highest modulus of elasticity values by 55%. These results are the opposite of the tension test results where the stresses and modulus of elasticity values were opposite.

Table 6 Flexural Test Results

Flexure Tests Summary																
Sample	MA	Width	Thickness	Theoretical Thickness	Area	Span	Peak Load	Peak Stress	Theoretical Peak Stress	Peak Strain (Comp)	Peak Strain (Tension)	Max Moment	Moment of Inertia (10 ⁻⁵)	Theoretical Moment of Inertia	E (10 ⁶ psi)	Theoretical E (10 ⁶ psi)
1-3F	N/A	0.51	0.115	0.125	0.05865	2	83.3	37052.5	31361.3	N/A	N/A	41.652	6.464	8.301	N/A	N/A
1-5F	0.5	0.514	0.11	0.125	0.05654	2	130.4	62917.1	48723.0	-13243	8677	65.218	5.701	8.366	6.82	5.28
2-1F	0.5	0.525	0.122	0.125	0.06405	2	27.4	10520.4	10021.5	-7870	8945	13.701	7.944	8.545	1.58	1.51
2-4F	N/A	0.512	0.121	0.125	0.06195	2	32.9	13159.8	12331.1	N/A	N/A	16.441	7.559	8.333	N/A	N/A
2-9F (0-90)	0.5	0.51	0.12	0.125	0.0612	2	24.2	9885.6	9110.6	3426	-3762	12.100	7.344	8.301	2.28	2.16
2-10F (0-90)	N/A	0.515	0.121	0.125	0.06232	2	12.0	4774.5	4473.8	N/A	N/A	6.000	7.603	8.382	N/A	N/A
3-4F	N/A	0.934	0.275	0.375	0.25685	4.8	213.9	21801.2	11724.2	N/A	N/A	256.650	161.869	410.449	N/A	N/A
3-5F	0.5	0.942	0.276	0.375	0.25999	4.8	206.1	20675.6	11199.9	-26526	38396	247.272	165.043	413.965	1.21	0.66
3-9F (0-90)	0.5	1.014	0.272	0.375	0.255	4.8	732.6	70311.1	36991.2	-12463	14880	879.120	170.045	445.605	3.12	1.64
3-10F (0-90)	N/A	1.011	0.276	0.375	0.255	4.8	501.16	46853.2	25380.2	N/A	N/A	601.392	177.132	444.287	N/A	N/A

Figure 55 Flexural Test Stress vs. Strain Diagrams

When performing the flexure test, only one sample had a debond and the rest of the samples had minor rupture on the compression and tension sides at the bending point. Due to minor the ruptures, there was never a true rupture point during the test so the tests were stopped when the load started to drop.

The first 6 flexure tests were two from the first laminate in the 0/90 direction and two from the second and third laminate in the +/-45 direction. Figure 60 through Figure 62 in section B.10 show the first six samples tested and each sample had a similar pattern with a small rupture at the bending points on the compression and tension sides. The flexure tests had similar results to the tension tests in that the values were extraordinarily low for the second and third laminate results so more samples were cut in the perpendicular direction so that the fibers were primarily oriented in the 0/90 direction. Figure 63 and Figure 64 in section B.8 show that the second laminate had a debond in one of its samples and the third laminate had the same pattern as the other samples with a small rupture in the bending point on the compression and tension side.

B.5 Discussion

The resin infusion process is preferred over hand layup because each laminate, that is resin infused, gets the same amount of resin because the resin can only fill the void spaces during the process. It can't be over or under saturated. Because of the uniform resin distribution, there was a very small amount of variance between the resin to fabric ratio for all three laminates. The resin infusion process was able to condense the twelve layer laminate down more than the four layer laminates but it was still able to have the same resin to fabric ratio due to resin only

traveling into the void spaces. Each sample was also cut out of the middle of each laminate made to ensure the most uniform resin distribution and that the fabric wasn't damaged before the resin infusion process.

Each sample in the tension test had a small deviation which shows that the values obtained were consistent and as true as possible. None of sample sets had any outliers and the largest difference between two numbers in any sample set is less than 15%. Because of the low deviation, there aren't any discussion points for the tension tests and the results are as true as possible.

The flexure tests had a much higher deviation than the tension tests and multiple discussion points for why the results weren't as clear as the tension tests' results. The smallest difference between two samples' peak stresses in a sample set is 5% but the largest difference between two samples in a sample set is 41%. Sample 2-10F (0-90) was tested but the only results obtained were from the Instron 8500 which had unreadable results but it is still in the table with the others to compare the dimensions.

The first discussion point for the flexure tests is that a strain gage was on each side of the sample so the gages weren't on the center where the bending point was. The gages were 0.5" off the center. This was required so that the gages weren't broken under the actuator. This affected the results because the strains were lower for one sample in each sample set which would have affected the modulus of elasticity values. The second discussion point is that the samples never had a true rupture point during the tests so the tests were stopped after the peak load was reached. The last discussion point is that the Instron 8500 has a capacity of 22,000 lbs. and lowest and highest peak loads reached were 24 lbs. and 501 lbs. This affects the stress values because the highest peak load reached was only 2.3% of the instrument's capacity which affects

the accuracy of the reading. This had a large effect on the 2nd laminate's samples because of how low each of the samples' peak loads were. The stress vs. strain diagrams for each of the 2nd laminate's samples were not linear, each diagram looked like steps and a trend line had to be made to get a modulus of elasticity value. For more accurate results, a smaller instrument would have to be used.

B.6 Conclusions

The results from these tests indicate that the two dimensional fabric with no out of plane needle punched fibers performed the best when compared to three dimension needle-punched fabric in tension and flexure which also suggests that it performed better in shear. An argument could be made that the three dimensional fabric had a lower weight and it wasn't a true comparison between the two fabrics but that is why the fiber volume fraction was calculated for all three laminates. The FVF calculations show that each laminate had the same FVF within 2%. If the FVF was 50% lower for the first laminate then it would have made a difference because the FVF is multiplied by the peak stress and the peak stress would have been lower for the first laminate in that case in the tension test. Since the FVF values are the same, the values obtained in the tension test are as true as possible. The values in the flexure tests similar results but due to the inconsistency, these results can't be trusted. It can be reasoned that the results from the needle punched fabric are much lower because the needle punched fibers damage the two dimensional fibers during the tests which results in a lower peak stress value.

Future work for this topic would include performing the flexure tests on a loading instrument with a smaller capacity to get more accurate results. Loading instruments can only be calibrated to the nearest percent in most cases and the max load reached in all of the flexure tests

was only 2.3% of the instrument's capacity. In addition to modifying the flexure test, a shear test could also be performed to reinforce that the shear strength of the needle punched fabric is weaker than the two dimensional fabric.

B.7 Tension Test Photographs

Figure 56 Laminate 1(0/90) Tension Test Ruptures

Figure 57 Laminate 2 (+/-45) Tension Test Ruptures

Figure 58 Laminate 3 (+/-45) Tension Test Ruptures

Figure 59 Laminate 2 & 3 (0/90) Tension Test Ruptures

B.8 Flexural Test Photographs

Figure 60 Laminate 1 (0/90) Flexure Test Results

Figure 61 Laminate 2 (+/-45) Flexure Test Results

Figure 62 Laminate 3 (+/-45) Flexure Test Results

Figure 63 Laminate 2 (0/90) Flexural Test Results w/ Debond

Figure 64 Laminate 3 (0/90) Flexure Test Results

B.9 Fiber Volume Fraction Calculations

$$Total\ Weight\ (lb) = \frac{Measured\ Weight\ (g)}{453.6\ \frac{g}{lb}}$$

$$Weight\ of\ Fabric\ (lb) = \#Layers \times \frac{fabric\ weight\ \frac{oz}{yd^2}}{16\ \frac{oz}{lb}} \times \frac{2D\ Dimensions}{36\ \frac{in}{yd} \times 36\ \frac{in}{yd}}$$

$$Weight\ of\ Resin\ (lb) = Total\ Weight - Weight\ of\ Fabric$$

$$Fiber\ Volume\ Fraction = \frac{\frac{Weight\ of\ Fabric}{Total\ Weight}}{Density\ of\ Fiber\ (188)}$$

B.9.1 Laminate #1 Weight Sample

#Layers = 4

Dimensions: 3.013"x2.847"x0.115"

Measured Weight = 26g

Fabric Weight = 22 oz/yd²

$$\mathbf{FVF = 42.2\%}$$

B.9.2 Laminate #2 Weight Sample

$$\#Layers = 4$$

$$\text{Dimensions: } 2.950'' \times 2.867'' \times 0.128''$$

$$\text{Measured Weight} = 22\text{g}$$

$$\text{Fabric Weight} = 18.14 \text{ oz/yd}^2$$

$$\mathbf{FVF = 40\%}$$

B.9.3 Laminate #3 Weight Sample

$$\#Layers = 12$$

$$\text{Dimensions: } 2.915'' \times 2.889'' \times 0.281''$$

$$\text{Measured Weight} = 64\text{g}$$

$$\text{Fabric Weight} = 18.14 \text{ oz/yd}^2$$

$$\mathbf{FVF = 41.9\%}$$

APPENDIX C FRP BOND TO CORRODED STEEL

C.1 Summary

This study investigates the bond strength of glass fiber reinforced polymers to corroded steel. When steel corrodes it initially increases in cross section and produces a brown/red dust layer on the outer surface. The cross section is then reduced as the rust flakes away. Ideally the rust would be completely removed with sandblasting, grinding, etc. to allow the FRP to bond to bare steel. In this study, the steel was only pressure washed to evaluate the bond using a more cost effective preparation.

Three different tests were performed to compare the bond strength: tensile, flexural, and pull out tests. Each of these test a different of properties in a material and it allows for comparisons. These three tests indicated that excellent bond strength was attained.

C.2 Introduction

Corrosion is an electrochemical process that requires the coexistence moisture and oxygen. This generates oxidization in the iron portion of the steel which increases its cross section due to rusting. Rust in the steel is a dusty layer on the surface. Once the rust has formed it is nearly impossible to form a bond between the steel and another material.

FRPs are commonly used to reinforce materials such as steel, concrete and timber but reinforcing corroded steel with FRP is a difficult project as the corrosion doesn't allow for a proper bond between the steel and FRP. Being able to reinforce corroded steel with FRP would be beneficial the restoration industry of steel structures as deficient and corroded steel elements with FRP could be reinforced rather than replaced.

C.3 Methods

Before adding a layer of glass fabric to corroded steel, a corroded steel plate is pressure washed to remove as much of the rust as possible. An ATPRIME 2 component primer was used to coat the surface where FRP composite was to be bonded and six layers of glass fabric were then added with a urethane ester resin using the hand lay-up method. The first two sets of six samples for the pull out tests were prepared this way. In lieu of hand layup method, vacuum assisted resin infusion method can be adopted for highly consistent results.

The remaining five samples were also pressure washed and the same ATPRIME 2 component primer was applied along with six layers being used. The only difference between the samples is that a vinyl ester resin was used rather than a urethane ester resin.

C.3.1 Pull Out Testing

A total of 15 Pull out tests were performed and ASTM D 7522 Pull-Off Strength for FRP Laminate Systems Bonded to Concrete Substrate provided all of the parameters for this test. This test was done by applying multiple FRP layers to a corroded steel plate and bonding a steel circular loading fixture (dolly) to the top surface of the FRP. An adhesion testing device was then attached to the dolly and it produces a perpendicular tensile load on the steel plate. This test was performed until the FRP layers were pulled out from the steel plate.

The first set of tests performed was a set of six samples with a 1” diameter steel circular loading fixture. JB weld epoxy was the adhesive used for this set of tests which resulted in a pull out failure in the adhesive agent and not the FRP to steel area. This is a failure of the test method and does not reflect the material strength; hence the test results and photographs are not shown in this report. Plexus MA310 was used for the second set of six pull out tests (Figure 65 and Figure 66), along with the same primer and FRP, and it allowed for a pull out failure to occur in the FRP

to steel area rather than the steel loading fixture to the FRP area which is what is needed for a successful pull out test of this type

Figure 65 2nd Pull out Test

Figure 66 2nd Pull out Test (Side View)

The third set of tests performed was a set of three samples with a 2" diameter steel circular loading fixture. The only difference in this set other than the loading fixture was the adhesive agent. Devcon 5 minute epoxy gel cream was used as the adhesive agent and it resulted in a pull out failure in the FRP to steel area. Figure 67 and Figure 68 show the debond from the third set.

Figure 67 3rd Set Pull out Test (Side View)

Figure 68 3rd Set Pull out Tests

C.3.2 Flexure Testing

One flexural test was performed and ASTM D 790 Standard Test Methods for Flexural Properties of Unreinforced and Reinforced Plastics and Electrical Insulating Materials provided all the parameters to be used for this test. Figure 69 and Figure 70 show the rupture in the FRP and debond of the FRP to steel. The FRP section of this sample was 0.217" thick and the steel section was 0.1" thick. Both sections had a width of 1.02". ASTM states that the span length for each sample must be 16 times the thickness of the sample. For this case, each sample's span length is 5.1" long. The tests were performed by placing the samples on two rollers that are 5.1

inches apart under a point load at mid span with the calculated load rate until an FRP rupture or debond between the FRP and steel plate occurred.

$$\text{Load Rate} = \frac{0.01 \times 5.1^2}{6 \times 0.32} = 0.14 \text{ in/min}$$

Figure 69 Flexure Test FRP Rupture

Figure 70 Flexure Test FRP Debond

C.3.3 Tension Testing

One tensile test was performed and ASTM D 3039 Standard Test Method for Tensile Properties of Polymer Matrix Composite Materials provided all of the parameters to be used for

this test. The FRP section of this sample was 0.217" thick and the steel section was 0.11" thick. Both sections had a width of 0.92. The fibers used were 0/90/+/- 45 which required the overall length to be 10 inches. These tests were performed by placing approximately 2 inches on each side in an FRP Grip and applying a tensile load at a 0.05 in/min rate until either a rupture in the FRP or a debond between the FRP and steel place occurred. Figure 71 and Figure 72 show the FRP rupture and debond between FRP and steel.

Figure 71 Tension Test Debond

Figure 72 Tension Test FRP Rupture

C.4 Results

A total of seventeen different samples were successfully tested and reported in this report, nine pull out tests, one flexure test, and one tension test. Six of the pull out tests were performed on an instrument that was able to record the position as the load was applied, while three with a different loading instrument that only recorded the maximum load. The tension and flexure tests were performed on the same loading apparatus as the third set of pull out tests and these samples each had a strain gage on the steel layer. The tension test had one strain gage on the FRP layer and the flexure test had two gages on the FRP layer.

The first two sets of six pull out tests were manufactured and tested by Rick Pauer. The three other tests were manufactured by Rick Pauer and tested at the West Virginia University Construction Facilities Center Lab by Mark Skidmore, PE.

C.4.1 Pull-Out Testing

Table 7 and Figure 73 are the results and load vs position diagram for the six pull out tests using the urethane ester resin. The debond for this set of tests was between the FRP and steel which resulted in a true pull out test failure.

Table 7 Results for Plexus MA310 Adhesive and Urethane Ester Resin Samples

Specimen #	Diameter in	Load at Peak lbf	Stress at Peak Psi	Load at Break Lbf	Stress at Break psi	Energy To Break in*lbf
1	1.000	418.269	532.6	418.269	532.555	10.113
2	1.000	297.852	379.2	297.852	379.236	4.484
3	1.000	323.792	412.3	323.792	412.263	5.136
4	1.000	526.662	670.6	526.662	670.565	11.502
5	1.000	404.901	515.5	404.901	515.534	6.668
6	1.000	277.039	352.7	277.039	352.737	2.845
Mean	1.000	374.752	477.1	374.752	477.148	6.791
Std. Dev.	0.000	93.742	119.4	93.742	119.355	3.373
Mean-2 Std. Dev.	1.000	187.269	238.4	187.269	238.437	0.046
Mean+2 Std. Dev.	1.000	562.236	715.9	562.236	715.859	13.537

Figure 73 Load vs Position Diagram for MA310 Adhesion and Urethane Ester Resin Samples

Table 8 shows the results and calculated values for the third set of samples. Three samples were used in this test and the vinyl ester resin was used along with a 2” diameter circular loading fixture. The instrument used didn’t display the displacement used throughout the tests. Hence, no load vs. position diagrams was created. The debond for this set of tests occurred between the FRP and steel which results in a true pull out test failure but the values are much lower because of the vinyl ester resin used.

Table 8 Results for Devcon 5 Minute Epoxy Gel Cream Adhesive and Vinyl Ester Resin Samples

Pull Out Tests					
Sample	Diameter (in)	Load at Peak (lb)	Stress at peak (psi)	Load at Break (lb)	Stress at Break (psi)
1	2	321	122	321	122
2	2	407	154	407	154
3	2	742	282	742	282
Mean		490	186	490	186
Std. Dev.		222.4	84.7	222.4	84.7

C.4.2 Flexure Testing

Table 9 and Table 10 shows the results and calculated values for the flexural test performed. The equations above show how the stresses were calculated in both the FRP and steel along with how the total stress was calculated and the modulus of elasticity values were carried over from the tension tests. Figure 74 is the stress vs. strain diagram that shows the behavior of the individual layers for the flexure test. The two FRP strain gages were placed side by side at the bending point to compare the strains at the two locations.

Table 9 Flexure Test Results for Steel

Flexure Test - Steel							
Width (in)	Steel Thickness (in)	Steel Moment of Inertia (in ⁴)	Load at Peak (lb)	Moment (lbxin)	Steel Stress at Peak (psi)	Steel Strain at Peak	E (Steel) (10 ⁶ psi)
1.02	0.11	0.000000	508.57	610.28	0.00	23284.00	29.00

Table 10 Flexure Test Results for FRP

Flexure Test - FRP											
Width (in)	FRP Thickness (in)	FRP Moment of Inertia (in ⁴)	Load at Peak (lb)	Moment (lbxin)	FRP Stress at Peak (psi)	FRP 1 Strain at Peak	FRP 2 Strain at Peak	E (FRP 1) (10 ⁶ psi)	E (FRP 2) (10 ⁶ psi)	η1	η2
1.02	0.22	0.000000	508.57	610.28	0.00	39685.00	39558.00	4.71	4.71	8398.73	8398.73

$$\sigma_s = \frac{PL}{4} \chi \frac{c_s}{I_s} \quad (\text{Approximation})$$

$$\sigma_{FRP} = \frac{PL}{4} \chi \frac{c_{FRP}}{I_{FRP}} \quad (\text{Approximation})$$

$$\sigma_{Total} = \sigma_s + \sigma_{FRP} \quad (\text{Approximation})$$

Figure 74 Stress vs. Strain Diagram for Flexure Test

C.4.3 Tension Testing

Table 11 shows the results and calculated values for the tension test performed. Figure 75 is the stress vs strain diagram for the individual layers of the sample for the tension test. The FRP plot in Figure 75 is used to calculate the modulus of elasticity for the FRP layer. It should be noted that the operating stresses in field use will be of order of 6 to 7 ksi. Steel has a modulus of elasticity of 29,000,000 psi but in this test it is approximately 15 Msi lower because of its composite action with FRP composites so its actual value is displayed in the table.

Table 11 Results for Tension test Sample

Tension Test										
Width (in)	Steel Thickn ess (in)	FRP Thickness (in)	Area (in ²)	Load at Peak (lb)	Stress at Peak (psi)	Steel Strain at Peak	FRP Strain at Peak	E (FRP) (10 ⁶ psi)	E (Steel) (10 ⁶ psi)	η
0.92	0.1	0.217	0.292	9355	19066	37566	35828	4.71	29	6.16

Figure 75 Stress vs. Strain Diagram for Tension Test

C.5 Discussion

With a proper adhesive agent, the pull out testing with the 1 inch samples had a smaller variance with one outlier. The 2 inch pull out tests had similar peak load values to the second set but the peak stresses were significantly smaller. There are multiple factors that have a role in this which are: a 2 inch loading fixture was used instead of a 1 inch fixture and, a vinyl ester resin was used instead of the urethane ester resin. The vinyl ester resin isn't as high of a grade as the urethane ester resin used on the first two so this created a weaker bond between all layer of the FRP and the bond between the FRP and steel plate. Since there was only one sample tested for tension and flexure test there isn't anything to compare the results to. These samples were all cut out from the same plate each sample had a similar bond strength compared to the others.

C.6 Conclusions

The lab testing results for all of the tests were as expected with excellent bond strength in the tension, flexure and pull out testing. The third set of pull out testing had much lower results than the first two but that was due to the lower quality materials used. These results are a strong start to implementing FRP composites to reinforce corroded steel elements with the exception of using the proper quality materials to ensure proper bond strength is attained. Future testing that can be done to reinforce these results would be shear, compression, and torsional testing. It is assumed that the bond strength would still be of excellent quality. In addition to more lab testing, a larger sample could be tested to ensure that the bond strength remains excellent along with some field testing to test the durability of the bond strength over a certain amount of time.

APPENDIX D RETAINING WALL DRAWINGS

WEST VIRGINIA UNIVERSITY

CONSTRUCTED FACILITIES LABORATORY

SOLDIER PILE/LAGGING RETAINING WALL DESIGN

NO.	DESCRIPTION
1	TITLE SHEET
2	TYPICAL SECTION
3	GENERAL NOTES
4	PLAN VIEW (10')
5	ELEVATION VIEW (10')
6-8	SPECIAL DETAILS (10')
1	
1	
1	
1	
1	
1	

SIGNED: _____

DATE: _____

RECOMMENDED:.....
PROJECT ENGINEER

RECOMMENDED:.....
FOR APPROVAL STATE HIGHWAY ENGINEER

APPROVED:.....
COMMISSIONER OF HIGHWAYS

TYPICAL SECTION

- ① Item 1 Asphalt Material
- ② Item 2 Compacted Base Gravel/Asphalt
- ③ Item 3 Portland Cement Circular Concrete Footing
- ④ Item 4 A572 Gr. 50 Steel WF 8x48 Pile
- ⑤ Item 5 Lagging Material (Timber w/ FRP or Reinforced Concrete w/ FRP or FRP 4" Prodeck)
- ⑥ Item 6 Loose Sandstone Rock Cut

WEST VIRGINIA UNIVERSITY DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING				
TYPICAL SECTION				
REVISION NUMBER	SHEET NUMBER	REVISION	DATE	BY

GENERAL NOTES

GOVERNING SPECIFICATIONS

The West Virginia Department of Transportation Division of Highways Standard Specifications, Roads and Bridges adopted 2017, as amended by the West Virginia Department of Transportation, Division of Highways, supplemental specifications, dated January 2019; the contract plans; and the contract documents are the governing specifications for this project.

TRAFFIC CONTROL

"Traffic Control for Street and Highway Construction and Maintenance Operation", 2006 Edition shall apply to this project

STANDARD DETAILS

The West Virginia Department of Transportation, division of Highways, Standard Details Book, Volume 1 dated May, 2016 & Volume II dated January 1, 1994 shall apply to this project

PRE-CONSTRUCTION DATA

Prospective bidders may review files from West Virginia University Constructed Facilities Center. These files may contain additional information not included in the contract documents including but not limited to, old plans, shop drawings, geotechnical information, environmental documents, permit applications, and other data. Copies may be obtained upon request and payment of printing fees.

UTILITIES

The information shown on these plans concerning type and location of utilities is not guaranteed to be accurate or comprehensive. The contractor is responsible for making his own determination as to the type and location of underground utilities as may be necessary to avoid damage there to. Any damage incurred shall be at the contractors expense.

The contractor shall coordinate his operations with local utility companies.

CONTRACTOR CO-ORDINATION

The contractor is hereby informed that other contractors may be working in the area. The contractor shall coordinate his maintenance of traffic and signing with other contractors so that there are no conflicting, repetitious, or confusing signs.

DRAINAGE

All bridges, inlets, and other drainage items on this project have been located as available information allows. The contractor will be responsible for final location and elevation of all drainage items and make any necessary adjustments in the field.

SHOULDER & DITCH

N/A

WASTE MATERIAL

All material removed and not reused in the construction of the project will be removed from the project and disposed of by the contractor after discussions with project site field engineers.

BRIDGES

N/A

ASPHALT PAVEMENT

N/A

ROAD REPAIR

N/A

EXISTING DELINEATORS AND SIGNS

Existing delineators, posts, and signs which are to be removed for the completion of this project shall be stored by the contractor and then reinstalled at the existing locations upon completion of the project. There will be no additional payment for this work.

RETAINING

All costs for laying out the retaining wall, including any surveying, shall be included in the unit bid price of the piling. The tops of all piles of the retaining wall shall be cut off flush with the final shoulder elevation.

All piling holes shall be backfilled with concrete. All cost shall be included in the unit bid price of the piling

SEEDING AND MULCHING

N/A

MAINTAINING TRAFFIC

1. The maintenance of traffic is only a recommendation. Should the contractor desire to adopt another method of maintaining traffic during construction, it shall be submitted to the engineer for review and approval at least 14 days prior to commencement of work in the affected areas.

2. Work in any one area can be combined with another operation in the same area or with another area to utilize more efficiently the excavated material, providing combined operation does not cause undue delay to traffic

3. Reserving and rehandling of any unclassified excavation material as a result of the maintenance of traffic scheme or necessitated on the project during construction shall be at the expense of the contractor.

4. All temporary work including incidental drainage to items that may be required for maintaining traffic shall be provided and installed by the contractor and shall be included in the unit bid prices.

5. One lane two direction traffic shall be maintained during work hours using case A6B of "Manual of Temporary Traffic Control for Streets and Highways", 2006 Edition with a maximum delay of 30 minutes to drivers. Emergency Vehicles shall be allowed through at all times. Contractor shall be responsible to ensure that both lanes are suitable for traffic at the end of each work day.

[illegible]

PLAN VIEW (10' SPACING)

EXISTING ROADWAY

STEEL W8X48

-10'

-10'

—10'

LAGGING MATERIAL/

EXISTING DITCH

-TOTAL SPAN = 150-200' (TBD)-

REVISION NUMBER	SHEET NUMBER	REVISION	DATE	

WEST VIRGINIA UNIVERSITY DEPARTMENT OF
CIVIL AND ENVIRONMENTAL ENGINEERING

PLAN VIEW (10')

ELEVATION VIEW (10' SPACING)

PILE/LAGGING CONNECTION DETAILS (10')

SOLDIER PILE & FRP PRODECK LAGGING CONNECTION

SOLDIER PILE & TIMBER/REINFORCED CONCRETE (W/ FRP) LAGGING CONNECTION

WEST VIRGINIA UNIVERSITY DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING			
SPECIAL DETAILS 1 (10')			
REVISION NUMBER	SHEET NUMBER	REVISION	DATE BY

LAGGING MATERIAL CROSS SECTION DETAILS (10')

REINFORCED CONCRETE W/ GFRP WRAP CROSS SECTION

FRP PRODECK CROSS SECTION

WEST VIRGINIA UNIVERSITY DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING				
SPECIAL DETAILS 2 (10')				
REVISION NUMBER	DATE	BY	REVISION	DATE

LAGGING MATERIAL CROSS SECTION DETAILS (10')

TIMBER W/ CFRP CROSS SECTION

						WEST VIRGINIA UNIVERSITY DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING
						SPECIAL DETAILS 3 (10')
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PLAN VIEW (8' SPACING)

REVISION	SHEET				
NUMBER	NUMBER				

ELEVATION VIEW (8' SPACING)

WEST VIRGINIA UNIVERSITY DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING			
ELEVATION VIEW (8')			
REVISION NUMBER	DATE	BY	

PILE/LAGGING CONNECTION DETAILS (8')

SOLDIER PILE & FRP PRODECK LAGGING CONNECTION

SOLDIER PILE & TIMBER/REINFORCED CONCRETE (W/ FRP) LAGGING CONNECTION

						WEST VIRGINIA UNIVERSITY DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING
						SPECIAL DETAILS 1 (8')
REVISION NUMBER	SHEET NUMBER	REVISION	DATE	BY		

LAGGING MATERIAL CROSS SECTION DETAILS (8')

REINFORCED CONCRETE W/ GFRP WRAP CROSS SECTION

FRP PRODECK CROSS SECTION

WEST VIRGINIA UNIVERSITY DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING				
SPECIAL DETAILS 2 (8')				
REVISION NUMBER	SHEET NUMBER	REVISION	DATE	BY

LAGGING MATERIAL CROSS SECTION DETAILS (8')

TIMBER W/ CFRP CROSS SECTION

						WEST VIRGINIA UNIVERSITY DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING
						SPECIAL DETAILS 3 (8')
REVISION NUMBER	SHEET NUMBER	REVISION	DATE	BY		