Determination of Optimum Tack Coat Application Rate for Geocomposite Membrane Use in Roads and Overlaid Bridge Decks

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(ABSTRACT)

Two critical components of the United States civil infrastructure, bridges and roads, have deteriorated in the past two decades at an accelerated rate and are in need of maintenance and rehabilitation. Geosynthetics may have the potential to provide a long-term solution to some of the problems that are present in these roads and bridges. When installed properly, some geosynthetics can act as both a moisture barrier and stress absorption layer. However, the tack coat application rate is critical as an excessive amount can cause eventual slippage, while too little may result in debonding. A new geocomposite membrane, which is comprised of a low modulus PVC layer sandwiched between two layers of nonwoven geotextile, has recently been introduced for use in highway systems for water impermeability and strain energy absorption. A laboratory testing program was conducted to determine the optimum asphalt binder tack coat rate that needs to be applied in the field. To accomplish this, a fixture was designed to allow the application of cyclic shear loading at the geocomposite membrane interface when used as an interlayer simulating one of two situations: a concrete bridge deck overlaid with the geocomposite membrane, and an HMA overlay or a flexible pavement with the geocomposite membrane sandwiched between an HMA base layer and an HMA wearing surface. The research concluded that 1.40 kg/m^2 of tack coat should be used when the geocomposite surface is in contact with an HMA base mix, 1.5

kg/m^2 should be used when it is in contact with an HMA surface mix, and 1.75 kg/m^2 should be used when it is in contact with concrete surfaces. However, these tack coat application rates are a function of the structural material type and the tack coat material type (binder performance grade). In addition, an analysis of the simulated bridge deck specimens with geocomposite membrane and the control samples, containing no membrane, shows distinct evidence that the membrane acts as a stress-absorbing material.

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CHAPTER ONE

INTRODUCTION

1.1. BACKGROUND

The current state of the United States infrastructure is rated as fair to poor. Approximately 40% of the 573,000 bridges across the country are deficient in some way (DiMaggio and Cribbs, 1996). Additionally, more than $78 billion is required to fix these deficient bridges. Similarly, about $212 billion is necessary to rehabilitate the deteriorated highways and roads of the nation's infrastructure (DiMaggio and Cribbs, 1996). Since the state of the infrastructure guides the economic growth of the country, it is imperative that these problems be remedied. Unfortunately, until recently, new construction was usually more important on the political agenda than rehabilitation.

Water is one of the major contributors to accelerated deterioration in roads and bridges. In the case of bridges, corrosion of the reinforcing steel is caused by the introduction of deicing salts. These salts contain chloride ions, which attack steel and cause rust to form in the presence of water and oxygen. Chloride ions reach reinforcing steel through cracks and/or by diffusion through the concrete pore system. When rust is formed as a result of an electro-chemical reaction, the resultant rust is six times the volume of the original iron and thus induces significant tensile stresses on the concrete (Mehta and Monteiro, 1993). Since concrete has a low tensile strength, this will result in cracking and the development of delamination and thus spalling.

In the case of roads, the infiltration of water also has detrimental effects. Most of recent pavement designs require a drainage layer to be incorporated into the pavement system to prevent any water entrapment in the structure. However, when water becomes trapped in the layers, due to the unavailability of a drainage layer or its ineffectiveness, freeze-thaw damage and spalling may occur in concrete pavements, and likewise stripping and spalling may occur in hot-mix asphalt (HMA) pavements. In addition, the pumping of fines (which is usually assisted by the presence of water) in concrete pavements will result in loss of support and thus faulting. In flexible pavements, this may result in a reduction in the resilient moduli of the pavement system layers, thus sacrificing the structural integrity of the pavement system.

The application of many types of geosynthetics as an interlayer in both roads and bridges has numerous benefits. These geosynthetics, also called membranes when used for water barriers, provide a solution to many of the problems inherent in the roads and bridges of today's infrastructure. The properties of membranes lend themselves to deal with the infiltration of water into bridge decks and pavement systems. First, a membrane is impermeable. When placed on a bridge deck in conjunction with a hot-mix asphalt (HMA) overlay, virtually no water is allowed into the bridge deck, given that it is installed properly, appropriate materials are used, and sufficient tack coat is applied. Consequently, a reduction in the chloride intrusion leads to lower chloride induced reinforcing steel corrosion. The use of a geosynthetic as an interlayer between a pavement's drainage layer and the underlying layer is also beneficial because of its impermeability. This interlayer will prevent the flow of water into the layers below the drainage material. Water will then flow through the drainage layer and run along the geosynthetic into drainage pipes or trenches in the shoulder.

Preventing the infiltration of water is not the only reason for the introduction of a membrane. The membrane also acts as another structural layer in a bridge deck or pavement system, providing additional stress absorption to the structural system. In a road, the membrane may provide a cushion, thus dissipating the stresses applied. In addition, the possibility of developing reflective cracking, the process of pre-existing cracks being reflected through an overlay to the surface of the pavement, may be significantly reduced, delayed, or eliminated by the use of a membrane.

The incorporation of a membrane may extend the service life of roads and bridges. Standard preformed membranes, or factory-manufactured sheets, can provide a service life increase of as much as 25 years (Al-Qadi *et al.*, 1992). However, the membrane system must have one other key property – durability – in order to produce such good results. Along with durability, the membrane must maintain an excellent bond with the surrounding layers throughout the service life. Unfortunately, one of the most important factors, the tack coat application rate, which controls the membrane permeability effectiveness and may cause slippage failure, has not been addressed. This research attempts to quantitatively define the tack coat application rate for a unique membrane – evaluated for the first time in this study.

The membrane, referred to in this research as a geocomposite membrane, consists of a polyvinyl chloride (PVC) geomembrane sheet backed on both sides with polyester non-woven geotextile. Prior applications of this geomembrane as a water barrier include dams, canals, reservoirs, floating covers, cofferdams and hydraulic tunnels. Excellent field performance has been reported in these applications (Scuero and Vaschetti, 1997). This geocomposite membrane has not been used on any roads or bridges in the United States; however, it has been used in pilot studies on two bridge decks in Italy.

1.2. PROBLEM STATEMENT

Shear stresses are magnified at the interfaces of different layers in pavement systems and overlaid bridges. In addition, water trapped in pavement systems or bridge decks may cause significant damage. Therefore, the geocomposite membrane is proposed as a water barrier and strain energy absorbent layer to be used in both structural systems. However, the tack coat application rate to be used on the geocomposite membrane system is not defined. Insufficient tack coat would cause debonding, while excess tack coat would cause slippage.

1.3. OBJECTIVES

To address the aforementioned problem statement, an optimized tack coat rate needs to be determined for the geocomposite membrane considered in this research as a first step in its successful application in roads and bridges. Therefore, it is the main objective of this research to determine the tack coat rate required for the newly developed geocomposite membrane for its use in flexible pavements and overlaid bridge decks. As part of the main objective, a new testing approach was developed to allow a quantitative determination of the effect of the tack coat rate.

1.4. RESEARCH SCOPE

In order to quantitatively determine the optimum tack coat rate for use with the geocomposite membrane, a testing approach and program was developed at Virginia Tech. The testing approach includes the design and construction of a testing fixture, and the preparation and testing of laboratory prepared specimens which simulated road and bridge cores.

Laboratory pavement specimens consisted of the geocomposite membrane sandwiched between a base HMA layer (SuperPave^{TM} mix design BM-25.0) and an HMA wearing surface (SuperPave^{TM} mix design SM-9.5A; A indicates a PG 64-22 asphalt binder). Simulated bridge deck specimens consisted of a geocomposite membrane sandwiched between a concrete cylinder cored from a concrete slab similar to that of a bridge deck in Virginia and a SM-9.5A HMA wearing surface. The parameter tested was the amount of tack coat rate applied on both sides of the geocomposite membrane, which was varied. Control specimens, with no geocomposite membrane, were also included. All specimens were tested in duplicate.

Specimens were tested in fatigue using the specifically designed fixture under a specific pattern of loading to simulate field vehicular loading. The optimum tack coat was determined as the amount of tack coat needed to provide the maximum number of cycles to failure.

In this thesis, Chapter 2 presents a review of the current literature available about geosynthetics used in both roads and bridges. Chapter 3 details the experimental program while Chapter 4 describes the data analysis and results. Chapter 5 presents findings and conclusions of this study. Finally, Chapter 6 outlines recommendations for future research.

CHAPTER TWO

LITERATURE REVIEW

2.1. INTRODUCTION

The efficient operation of the United States highway network is critical for the viability of national, state, and local economies. Two of the most important elements of this network are bridges and roads. Consequently, any time a bridge or road is closed either for maintenance, repair, or inspection, the closure has a detrimental impact on the economy. Since the state of the transportation infrastructure is responsible for approximately 20% of the economic growth of the country, it is imperative that the current problems with roads and bridges be remedied (DiMaggio and Cribbs, 1996).

To reduce the down time and inconvenience to travelers, and to build more reliable, longer lasting (based on life-cycle cost), and safer highway systems, a better use of advanced technologies in the transportation infrastructure is necessary. Among new materials used to improve pavement and bridge deck performance are geosynthetics. “Geosynthetics” is the collective term applied to thin and flexible sheets of synthetic polymer material incorporated in soils, pavements, and bridge decks (Koerner, 1990). The characteristics of the polymer from which a geosynthetic is made dictate its general performance. Its properties can be altered chemically by placing additives and fillers directly in the chain of compounds that make up the polymer, or mechanically by stretching the polymer during production.

Various geosynthetic types have recently been used in pavement systems and to protect bridge decks. For pavement systems, geosynthetics are thought to provide reinforcement (by increasing the tensile strength of a particular layer); stress absorption between pavement layers; separation (by maintaining the integrity of particular layers by preventing intermixing); drainage or filtration (by allowing the water to flow thereby dissipating pore water pressure while limiting soil movement); and/or a moisture barrier (by preventing water movement between layers). Currently, there are many types of geosynthetics available on the market. They can be divided into five main categories: geotextiles, geogrids, geonets, geomembranes, and geocomposites, which combine the best features of different geosynthetics in ways that can solve specific problems (Fluet, 1988*)*.

Although significant amounts of geosynthetics were used in the 1970s to protect bridge decks and in the 1980s in pavement rehabilitation, it is only recently that their true potential has been recognized. There are advantages and disadvantages to using geosynthetics in pavements at the interface of old flexible pavement and new overlay on top of the aggregate base layer or between new HMA layers. The advantage of geosynthetics is its potential to function as moisture barriers, stress-relieving interlayers, and/or reinforcement. Its use may reduce reflection cracking, water infiltration, and development of fatigue cracking. Potential disadvantages may occur due to improper asphalt distribution (which causes slippage) and during HMA recycling. Although geosynthetic membranes may be used to abate crack propagation from bridge deck surfaces into HMA overlays, its main use in the United States, so far, is to protect bridge decks from water and chloride intrusion.

2.2. GEOSYNTHETICS USE IN CIVIL INFRASTRUCTURE

Geosynthetics are used for applications such as soil stabilization, soil reinforcement, liquid drainage, and leak-proof barrier systems (Koerner and Soong, 1997). Geosynthetics are intended to do the job better and more economically. They provide the structure with an impermeable and durable layer, which increases the service life. The increase in service life and durability reduces the maintenance costs, making the structure more economical. The use of geosynthetics in transportation applications is a relatively recent development. However, this development is a result of earlier attempts in other functions.

In the past, items such as tree trunks and small bushes were used for soil stabilization. However, due to the lack of strength in those materials, it is likely that the attempts to stabilize the soil were not always successful (Koerner and Soong, 1997). As time and knowledge progressed, more systematic approaches were used in an attempt to solve this problem. Logs were lashed together to form mattressed surfaces. Evidence of this can be seen as early as 3000 BC with split-log corduroy roads. Further progressions include smoothing of ridged surfaces and paving with stone blocks. However, because timber was still the primary material used, avoiding deterioration of the system was impossible (Koerner and Soong, 1997).

It was not until the 20th century that another material was used as soil reinforcement. In 1926 the South Carolina Highway Department placed a cotton fabric over a primed earth base and applied hot asphalt over the fabric. Additionally, a thin layer of sand was spread on top of the asphalt layer. The highway department performed field experiments over a period of eight years and concluded that the roads were in good condition (Koerner and Soong, 1997). The fabric reduced cracking, raveling and other localized road failures. The results of this project indicated the potential of geosynthetics in transportation applications.

2.2.1. Types of Geosynthetics

Geosynthetics are classified into five major types: geotextiles, geogrids, geocomposites, geonets, and geomembranes. These five types perform several applications including reinforcement, separation, drainage, filtration, and moisture barrier.

Geotextiles are perhaps the most widely used types of geosynthetics. They are formed from synthetic fibers that are weaved together to form a flexible, porous fabric (Koerner, 1990). Woven, non-woven and knit types comprise the main facets of geotextiles. Their applications include all five of the main applications of geosynthetics: reinforcement, separation, drainage, filtration, and moisture barrier. As a reinforcing agent, the purpose of a geotextile is to add tensile properties to the soil (Fluet, 1988). When used as a separator between two solid materials, the geotextile prevents blending. For example, geotextiles are commonly placed between the aggregate base course and the existing soil subgrade within pavement systems preventing loss of aggregate to the subgrade or contamination of the aggregate by the infiltration of subgrade soil (Fluet, 1988). In some cases, a geotextile may act as both a reinforcing material as well as a separator. When a geotextile is used as a filter, it prevents the infiltration of particles from one material into the pores of a neighboring material. For drainage, a geotextile allows the flow of fluid laterally within the plane of the geotextile itself (Fluet, 1988). Lastly, geotextiles can act as moisture barriers, preventing moisture from penetrating through when impregnated with asphalt, for example, when used underneath an HMA overlay (Koerner, 1990).

Geogrids are primarily used for reinforcement. They are usually made from a plastic material, typically high-density polyethylene, polyester, or polypropylene that is formed into a grid with large openings (Koerner, 1990). The openings will typically be large enough to allow soil to pass through, but not so large as to forfeit the main purpose of these geosynthetics. The strength of the ribs and junctions must be maximized in order to allow the geogrid to function well as reinforcement. Examples of geogrids in reinforcement applications include beneath aggregate in unpaved roads, as asphalt reinforcement in pavements, and to reinforce landfills to allow for vertical expansion (Koerner, 1990).

Geonets are formed with polymeric ribs placed at acute angles to each other (Koerner, 1990). Polyethylene typically accounts for approximately 97% of the overall composition. Their appearance is similar to that of geogrids, with an open, grid-like pattern. However, the geonets are unlike geogrids in their function. While geogrids are used for reinforcement, geonets are mainly used for drainage purposes. Geonets would also be suitable for use as reinforcement since they are very strong when confined in soil (Fluet, 1988).

Geomembranes are the second most commonly used type of geosynthetic. They are composed of thin rubber or plastic sheets and their main function is primarily as a liquid or vapor barrier (Koerner, 1990). For all intents and purposes, these materials may be considered impermeable. Because of this property, geomembranes are often used in the lining systems of landfills (Fluet, 1988). Other applications include waterproofing within tunnels and beneath asphalt overlays.

A geocomposite is the combination of any two of the other four types of geosynthetics. As with geonets, the primary use of geocomposites is for drainage. However, since this type of geosynthetics combines two of the other types, many other purposes are served. For example, the geotextile-geonet provides separation and filtration while vastly improving the drainage properties (Koerner, 1990). When used in a horizontal fashion, they prevent water from moving upward, thus reducing the possibility of frost heave. Also, a geotextile is often laminated on one or both sides with a geomembrane. This allows for an increased drainage function since the lateral drainage through the geotextile prevents water from coming in direct contact with the geomembrane.

2.2.2. Membrane Classification

Membrane systems have been used as a preventive, as well as rehabilitative, technique to prevent the deterioration of bridge decks. The purpose of membranes is to provide protection, specifically in the 25 snow-belt states in the United States, against chloride intrusion in concrete bridge decks. A typical membrane consists of a material, either preformed or cured from a liquid, that is applied to a concrete bridge deck surface along with an overlay, typically HMA, that acts as a wearing surface (Galagedera, 1991). Preformed membranes may increase the life of bridge decks by as much as 25 years when installed on new concrete bridge decks and overlaid with 76 mm of HMA (Al-Qadi *et al.*, 1992).

Membranes are classified into two broad systems, liquid and sheet. Sheet systems typically include preformed sheets that are bonded to the concrete surface to form a continuous membrane. The sheet systems are well constructed since the quality of the material is controlled during manufacturing. However, there are some disadvantages to sheet systems. They often require more labor than liquid systems since they are not typically self-adhesive. Sheet systems are also difficult to install on curved, super-elevated or rough decks; thus, workmanship is critical. Also, sheet systems tend to be more expensive than liquid systems (Manning and Rydell, 1976).

Liquid systems consist of one or two components of latex or chemical curing solutions that are applied to the concrete surface, acting as a waterproofing agent. Because liquid systems are applied in place and therefore heated on site, it is difficult to ensure that proper adhesion is achieved. Furthermore, careful inspection is required to control the thickness of the membrane and to detect the presence of pinholes. In contrast to the sheet systems, these are easy to install on curved, super-elevated and rough decks. Additionally, because the membrane is self-adhesive, it is less vulnerable at critical locations such as joints and curbs (Manning and Rydell, 1976).

A more specific classification of membranes includes the following five types: preformed, thermoplastic, polyurethane, epoxy, and tar emulsion. Epoxy, polyurethane and tar emulsion are liquid systems. Epoxy and polyurethane systems are both considered to be thin membrane systems (OECD Road Research Group, 1972). These types of systems adhere well to the deck, providing a high degree of protection against water penetration. However, when the bond between the deck and membrane is not adequate, slippage may occur over time. Also, epoxy systems tend to pinhole or blister and have low flexibility along with high cost (Frascoia, 1976). Tar emulsion that has coal tar added to the waterproofing layer has properties similar to epoxy and polyurethane. Preformed and thermoplastic are sheet systems, which are factory manufactured and applied with an asphalt tack coat to ensure an adequate bond to the bridge deck. Standard preformed systems are the most widely used of the five types and may provide a service life increase of up to 25 years. Standard preformed membranes also tend to rate the highest among all types of membranes because of their ease of application, relative cold temperature flexibility and low cost per cubic yard (Larsen, 1993).

2.3. USE OF MEMBRANES ON CONCRETE BRIDGE DECKS

Membranes are introduced into concrete bridge deck systems in order to decrease the deterioration of the concrete due to chloride intrusion. Chloride intrusion typically occurs after the deck has been exposed to deicing salts or salt water. The corrosion process of concrete slabs is an electrochemical process (Mehta and Monteiro, 1993). There are four main steps involved in this process:

1. Iron changes into an iron ion ($Fe \rightarrow Fe^{2+} + 2e^-$).
2. Iron ion combines with a chloride ion to form iron chloride complex ($Fe^{2+} + 2Cl^- \rightarrow FeCl_2$).
3. Iron hydroxide is then formed when the iron chloride complex reacts with water and the hydroxyl ion ($FeCl_2 + H_2O + OH^- \rightarrow Fe(OH)_2 + H^+ + 2Cl^-$).
4. Rust and water are formed when the iron hydroxide reacts with oxygen ($4Fe(OH)_2 + 2O^- \rightarrow 2Fe_2O_3 + 4H_2O$).

The resulting rust (which has up to six times its original volume) eventually causes a deterioration of the bond between the concrete and reinforcing steel, thus cracking the surrounding concrete.

Many factors influence the amount of chloride intrusion that will take place during the lifetime of a concrete bridge. These factors include: the permeability of the membrane; the degree of bonding between the membrane and the concrete; the degree of joint bonding; the extent of damage to the membrane during construction; the amount of water the membrane has absorbed; and the thickness of the membrane (Galagedera, 1991). Since HMA is both permeable and porous, chloride-laden water can be transmitted through the HMA layer and become trapped between the layer and the membrane (Manning and Rydell, 1976). If there are any holes in the membrane, the chloride-laden water will eventually come in contact with the concrete surface, initiating the chloride intrusion process.

With the problem of chloride intrusion so severe in the bridges of the United States, it was clear that efforts must be made to prevent this problem. The Strategic Highway Research Program (SHRP) was a $150 million, five-year effort to address some of the major problems facing highway engineers in the United States. Of the money allotted, $10 million was spent on the multi-billion dollar problem of the corrosion of reinforcing steel in concrete due to exposure to deicing salt and sea salt (Broomfield, 1994). This work was divided into three areas: bridge assessment, rehabilitation methods, and a methodology for bridge rehabilitation. The bridge assessment area addressed the need for determining efficient ways of assessing a corroding reinforced concrete bridge. Proper and efficient assessment of the bridge allowed for decisions to be made about the amount of repair needed and the rehabilitation techniques that should be used.

SHRP also looked at the use of waterproof membranes in the evaluation of bridge decks (Al-Qadi *et al.*, 1993). It ascertained that the detection of damage through the HMA overlay was a problem. Two approaches were used to broach this problem. First, SHRP proposed using ground penetrating radar (GPR) to look for delamination and other damage under the HMA layer. Second, the development of an ultrasonic pulse velocity technique was planned to assess the condition of the membrane (Broomfield, 1994). The GPR technique was found to be adequate for “rating” the amount of damage on a bridge. However, using this technique for accurately locating and quantifying the damage was not recommended. The ultrasonic technique allowed for the determination of an empirical relationship between the membrane condition and the pulse velocity. This technique was used to create a database and was adopted as part of the Federal Highway Administration Demonstration Project.

2.3.1. Requirements for an Ideal Waterproofing System

In order for a membrane system to be effective, it must act as an excellent waterproofing system. Bridge deck preparation is a key factor to successful membrane performance. Before the membrane is applied, the surface of the bridge deck should be accurately finished and free from any obstructions. All membrane curing material and oil or other materials should already be cleared from the deck. The texture of the concrete surface is critical because it controls the degree of bonding between the deck and the membrane. Next, the concrete edges must be waterproofed in order to allow a smooth surface to which the membrane is applied. Additionally, the concrete itself should be sound and durable. Finally, the concrete must be swept clean and surface dry.

In addition to bridge deck preparation, there are several other requirements for an ideal waterproofing system. The first is impermeability; the whole system must be naturally impermeable to water under all conditions. The membrane system should also maintain its waterproofing capabilities under service conditions, notable temperature extremes and vehicular loading. The second requirement is that the membrane must have the mechanical properties necessary to separate the HMA overlay from the bridge deck surface. Next, the membrane should provide resistance to cracks propagating from the deck. If cracking of the concrete deck occurs, the membrane must be able to function properly despite the presence of those cracks.

Durability of the membrane is another key requirement since replacement would be costly and difficult. The material should not show any aging effects, such as brittleness. The membrane itself must be compatible with all of the other materials such as the concrete bridge deck, the wearing course, and any adhesives and prime coats that may be applied. It is essential that all of these components be physically and chemically compatible to ensure that the membrane will bond well to both the concrete deck and the wearing course.

Lastly, the installation of the waterproofing system should be easy and relatively simple, such that workmen of moderate skills can install it. The material should be suitable for application to a bridge deck under site conditions, sufficiently durable to resist damage during the installation and construction of the wearing surface, and resistant to blistering as long as proper precautions are taken. Application of the membrane should also be at the correct air-shade temperature, typically not less than 4°C (OECD Road Research Group, 1972).

There are several problems that may be encountered when installing a membrane system. If water penetrates the membrane and comes in contact with the concrete, chloride intrusion will occur and the membrane becomes useless. However, the most serious problem associated with membrane systems is blistering. Blistering affects all types of membranes, including liquid and sheet systems. Blisters form as a result of poor installation of the membrane system onto the concrete surface, causing air and other gases to become trapped (Larsen, 1993). Once these vapors and gases are trapped between the membrane and bridge deck, expansion occurs which in turn compromises the performance of the membrane. Blistering may also occur when the deck surface is heated by solar radiation and water is evaporated beneath the waterproofing membrane (Al-Qadi *et al.*, 1992).

In addition to a strong bond between the membrane and concrete, there are several ways to reduce the possibility of blistering. Prefabricated sheets should be secured sufficiently to resist the vapor pressure that may build up, requiring the use of venting layers or ventilating pipes (OECD Road Research Group, 1972). Since blistering generally occurs long after the membrane has been placed, either one of these methods will help to release any entrapped air. Covering the waterproofing immediately with at least 50-mm of the wearing course or another protective surface will also prevent the formation of blisters. Furthermore, waterproofing should be installed with careful attention paid to the curbs and joints.

2.3.2. Field Performance of Bridge Deck Membrane Systems

With the requirements for membrane systems stated along with problems that may be encountered along the way, the next step is to see how these membrane systems actually perform in field applications. The results of five studies on the field performance of bridge deck membrane systems are summarized below.

Fifteen bridge decks with waterproof membranes were evaluated as part of a SHRP study in three New England states – New Hampshire, Maine and Vermont. Five bridge decks were evaluated in New Hampshire. The membranes had been installed on these decks at the time of construction. Four of these bridge decks had the same kind of membrane system – a nonwoven fibrous mat between two layers of bituminous and synthetic resins. The fifth was made of a bottom layer of rubberized asphalt with adhesive qualities, a polypropylene barrier sheet, and a top layer of rubberized asphalt/wax (Al-Qadi *et al.*, 1993). From V–meter ultrasonic pulse measurements and core samples, it was found that four of the five membranes were performing satisfactorily. The fifth showed evidence of shoving and rutting. This problem was attributed to the deteriorated and distressed overlay. In terms of bonding, the membranes proved to be good in adhering to the concrete and the overlay. On one bridge, the bonding was poor in one location because the membrane had been installed on a dirty concrete surface. Also, the poorest bonding was located at the shoulder of the road where many membranes had deteriorated or had small holes (Al-Qadi *et al.*, 1993). This was explained by the presence of accumulated salt buildup on the side of the road.

Similar to the bridges in New Hampshire, five bridge decks were evaluated in Maine. Again, the membranes were installed on newly constructed bridges. There were three different types of membranes installed on these bridge decks. The first two were the same as the two types in New Hampshire. The third consisted of an impregnated fiberglass mesh between layers of bituminous mastic (Al-Qadi *et al.*, 1993). From the V–meter ultrasonic pulse measurements, it was found that all of the bridges were satisfactory except one. Similar to the bridge in New Hampshire, the membrane was damaged due to the presence of a deteriorated overlay.

The final five bridge decks evaluated in that study were in Vermont. Of those bridges, two had localized cracks and satisfactory overlays. Another showed evidence of severe rutting throughout the bridge. The most recently constructed bridge had severe shoving problems. The last bridge was found to be in good condition. From the 15 bridges evaluated, a regression model was derived from the chloride contents of each bridge in order to determine the factors that influence the degree of chloride contamination. The most important factors were found to be the salt application rate and the membrane type (Al-Qadi *et al.*, 1993).

In the state of Vermont, previous studies concluded that membrane waterproofing and removal of chloride intruded areas would not significantly reduce the further corrosion of the bridge decks (Frascoia, 1987). In an effort to validate these studies, the Vermont Department of Highways conducted a field study on 69 new bridge decks with 33 different membrane systems. The study began in 1975 and took place over the course of 16 years (Frascoia, 1987). Of the 33 membrane systems, 15 were preformed, seven were epoxy, five were thermoplastic, four were polyurethane, and two were tar emulsion.

The field evaluation procedure consisted of samples taken at three distances off the curb line. The first sample was taken at 305 mm due to the potential for leakage at the curb line area. The second was at a 1.5 m offset. This was located in the breakdown lane where satisfactory performance would be expected if the membrane was not damaged during paving or internal leakage did not occur. The last sample was taken at an offset of 4.6 m in order to establish membrane performance in the wheel path area which is subjected to continuous traffic, thereby causing aggregate to punch through the membrane (Frascoia, 1987).

The results of this study are based on the percent of chloride-contaminated concrete samples and the level of contamination. Concrete samples are considered contaminated when the chloride content is 50 ppm over the base chloride levels recorded on the specific bridge decks following construction. The presence of chloride contamination does not indicate that the membrane is a total failure. It only indicates that the membrane is not 100% effective. Test results show that the majority of the membrane systems have performed well over 14 years of exposure. Projections based on these performance results infer that a significant number of the membrane systems would provide chloride protection for 50 years or longer (Frascoia, 1987).

Interlayer membranes were installed on six different bridge decks in Kansas and have been monitored for 20 to 25 years. Three different types of membranes were used on these bridge decks. The first type, preformed, consisted of a pliable sheeting construction of polypropylene and coal tar placed over a primer with an HMA overlay covering the membrane (Wojakowski and Hossain, 1994). Liquid/preformed was the second type of membrane, an applied in place nonwoven polypropylene fabric with cationic emulsified asphalt. In this case, either HMA or an aggregate overlay covers the membrane. The last type of membrane was a liquid type made of a coal tar modified polyurethane, which was then covered with an asphalt roofing sheet and a layer of HMA.

The data on the bridge decks was collected in three ways: resistivity measurements, visual distress survey, and condition survey and maintenance history data. The general performance of bridge decks installed between 1967 and 1971 has decreased since 1982 in terms of electrical resistivity measurements and visual distress survey results. Overall, the liquid/preformed systems had the best performance over the 25 year period. The poorest performance was obtained from the preformed system and a liquid membrane (Wojakowski and Hossain, 1994).

2.3.3. Prediction of Membrane Service Life

In general, the service life of bridge decks protected with membranes is governed by the deterioration of the HMA overlay. Standard preformed membranes with HMA overlays should theoretically provide 50 years of protection against chloride contamination; however, the service life for a wearing surface with a membrane is typically less than 15 years (Larsen, 1993). Even with this dramatic reduction in service life, the addition of a membrane to a bridge deck is still beneficial.

Several states across the United States have incorporated membranes into bridge deck overlays with much success. Case studies in Wyoming and Texas both have shown membrane systems that have been in place for 20 years. In both cases, there is reason to believe that the membrane will be in place for decades (Steinberg, 1998). Studies performed by the Bureau of Reclamation also confirmed and reinforced these positive results (Steinberg, 1998).

2.4. USE OF GEOSYNTHETICS IN FLEXIBLE PAVEMENTS AND OVERLAYS

As stated earlier, geotextiles are used in pavements as an intermediary between the old pavement, HMA or concrete, and HMA overlay. The HMA overlay is applied as a means of increasing the structural integrity of the pavement. However, if the HMA overlay alone was used as a means of repairing the pavement, problems similar to the ones encountered with the old pavement would resurface on the overlay. For example, the cracks that were present in the old pavement would be reflected through the HMA overlay to the top of the new pavement. The use of a geosynthetic may reduce, if not eliminate, the possibility of this reflective cracking.

Reflective cracking in both flexible and concrete pavements may occur due to several reasons. Thermal stresses in old pavements (low temperature cracking) and jointed concrete pavements may cause reflective cracking (Wright and Guild, 1996). Shrinkage cracking in concrete or cement treated base is another cause. The acceleration of reflective cracking is caused by traffic stresses and high viscosity (stiff) binder. Another cause of reflective cracking could be any construction on or around the pavement. For example, the widening of the road could result in later reflective cracking. Finally, any settlement that occurs in the underlying layers may contribute to eventual reflective cracking.

2.4.1. Field Performance of Flexible Pavements and Overlays

Reflective cracking through HMA overlays is considered a severe problem. The use of a geosynthetic interlayer would theoretically help reduce, if not eliminate, the reflection of existing cracks. However, actual field experiments need to be conducted to reinforce this hypothesis. The results of several field studies that incorporated the use of a geosynthetic interlayer in flexible pavements are discussed below.

In North America alone, there are over 15,000 lane miles of paving fabrics installed each year (Carmichael and Marienfeld, 1999). Due to the increasing use of geotextiles in the transportation industry, the Geotextile Division of the Industrial Fabrics Association International (IFAI) formed a Paving Fabrics Task Group. The purpose of this group was to summarize the uses and benefits of nonwoven geotextiles used for pavement membrane interlayer systems. The group identified four main areas in which nonwoven geotextiles are used.

One of the simplest ways to integrate a geotextile into a pavement system is to use it along with a chip seal or any other surface treatment. Seal coats are thin asphalt surface treatments, with or without aggregate, that are used to waterproof the road and provide skid resistance (Huang, 1993). When the membrane was used as a layer between an unbound soil layer, such as the aggregate base layers or in-situ subgrade, and the surface treatment, the results were generally favorable. For this reason several countries, including Australia, France, and Canada, have used this application since 1985. In all of these countries, the sections with a geotextile performed better than the control sections with no fabric interlayer (Carmichael and Marienfeld, 1999). Subgrade deformation was also a problem with most of the control sections. However, the sections with a paving fabric showed little or no subgrade deformation. In these sections, the moisture content in the subgrade remained more uniform than in control sections.

A case study performed in Australia demonstrated the importance of uniform moisture content in the subgrade. The results from the study were compiled after about 20 months of service life and a period of two months of wet weather. The moisture contents in the subgrade under the paving fabric were on average approximately 20% lower than those under the control sections with no paving fabric (Carmichael and Marienfeld, 1999). Considering that the measurements were taken after two months of heavy rainfall, it was evident that the fabric provided protection against the infiltration of surface water.

Since 1985, chip seals have also been used in conjunction with membranes to repair existing asphalt pavements. Case studies were performed in both the United States and the United Kingdom. In this application, the installation of both the membrane and chip seal is critical to the long-term performance of the overall system. If the system is not carefully installed, there will be a bond loss between the stone and the fabric. However, in cases where proper installation procedures were followed, the sections with a geotextile exhibited less cracking and distress than control sections with no geotextile (Carmichael and Marienfeld, 1999). For the most part, the fabric was still intact, but there were some indications of stone loss.

The use of paving fabrics with overlays for existing pavements is perhaps the most common application. A geotextile used in conjunction with an HMA overlay for a deteriorating concrete pavement has been a very successful treatment. Such systems have been evaluated by the IFAI at sites in the United States, Belgium and Austria. The systems were evaluated between three and ten years of service life. The results of the study show that the use of a geotextile greatly reduced the evidence of reflection cracking (Carmichael and Marienfeld, 1999). The paving fabric was also helpful with another common problem in concrete pavements, popouts. However, the results of the study were not all positive. The use of a membrane interlayer did not prevent the reflection of existing transverse cracks or joints, particularly when there was a high deflection or large vertical movement between the joints (Carmichael and Marienfeld, 1999).

The findings of this survey were reinforced by one particular case study in Georgia (Carmichael and Marienfeld, 1999). Overlays, including one with a geotextile and the other without any fabric, were studied in order to show the benefits of using a membrane. More specifically, the findings were based on overlay thickness and its effect on the development of reflection cracking. It was concluded that for a 50 mm HMA overlay without a paving fabric, a 60% level of reflection cracking could be expected after just one year. The same level of reflection cracking could also be anticipated after three years for a 100 mm overlay. For an even thicker asphalt overlay of 150 mm, the 60% level of cracking would not likely occur until nine years after the beginning of the service life (Carmichael and Marienfeld, 1999). In contrast, for the overlays placed in conjunction with a geotextile, the 60% level of reflection cracking was not present until after three years for the 50 mm overlay and six years for the 100 mm overlay. The increase in retardation of reflective cracking with the use of a geotextile was quantified by a factor of three. These results also showed that 50 mm of HMA with a geotextile is equivalent to 100 mm of the same HMA with no geotextile.

Case studies in California provided some useful information on the use of interlayer fabrics. The study advised that the geotextile should only be considered to replace 30 mm of the thickness of the HMA overlay and should not be used to prevent the reflective cracking of any preexisting thermal crack (Carmichael and Marienfeld, 1999). The study also recommended that these fabrics only be used for full lane coverage (Carmichael and Marienfeld, 1999). There was also some evidence that the membranes may have an effect on the overall performance of the HMA overlay, and further field investigation studies are recommended.

Field studies were conducted on various existing HMA pavements in the United States, South Africa, Belgium, Germany, Spain, and Austria. All of the pavements were between five and ten years old when tests were conducted. Similar to the results of the studies on concrete pavements, it was evident that the use of a membrane interlayer greatly reduced reflective cracking (Carmichael and Marienfeld, 1999). However, for HMA overlays of less than 37 mm, there was no visible reduction in the reflection cracking. Despite this discouraging data, there was still a relationship between the thickness of the overlay and the amount of reduction in cracking.

In a study in Texas, 10% reflection cracking was present after approximately one year in HMA overlays 19 – 50 mm thick, 20% occurred within one to one and a half years and 40% was evident after just two years. By comparison, HMA overlays with a geosynthetic interlayer of the same thickness range displayed 10% reflective cracking after three years, 20% cracking after three and a half years and 40% after nine years (Carmichael and Marienfeld, 1999). For a 25% level of cracking, the use of a geotextile increased the performance of HMA overlays, ranging from 19 – 50 mm thick, by at least a factor of 1.75. In some cases, the performance was increased by a factor as large as three (Carmichael and Marienfeld, 1999).

The reconstruction of a high-speed test track at the Motor Industry Research Association (MIRA) in England incorporated a geotextile interlayer in an attempt to reduce reflective cracking. The original pavement, consisting of concrete slabs from an old airfield had experienced difficulties with reflective cracking as soon as 18 months after an HMA overlay (Wright and Guild, 1996). The reconstruction process included stripping the track of all previous overlays and adding a regulating layer, base course, and wearing surface. The regulating layer was incorporated to level out the original concrete slabs. At the interface of this regulating layer and the base course, a geotextile was used to reduce the possibility of reflective cracking. Because of the need for stress relief and reinforcement, a woven polypropylene was chosen as the geotextile interlayer. In addition, a control section was constructed. The sections were monitored for a period of 17 months with no sign of reflective cracking (Wright and Guild, 1996).

In Belgium, the use of a geotextile underneath an HMA overlay has been incorporated into several deteriorated pavements (Wright and Guild, 1996). After the original pavement was sufficiently repaired and smoothed and all surface debris had been cleared, the tack coat was applied at a uniform rate. The coat was either emulsion or asphalt binder mixed with polymers. The rate of spray was typically between 1 – 1.5 kg/m^2. However, this varied with the type of membrane that was used. For a woven textile or grid, an emulsion tack coat was applied at a rate of 400 g/m^2 (Vanelstraete and Francken, 1996). In the actual application of the geotextile, the most important factor was that it be laid perfectly flat with no wrinkles or air bubbles. Additionally, a 100 – 150 mm overlap was provided at the seams to prevent any future problems.

Six projects were constructed and evaluated in Belgium. All of the overlays were placed on existing concrete slabs. Steel reinforcement nettings and polyester nonwoven geotextiles impregnated with bitumen were the two types of interlayers used. Five of the sites include steel reinforcement while only one used the nonwoven geotextile. The sites with steel reinforcement, used in conjunction with overlays ranging from 40 – 140 mm, performed very well after three to six years of service life (Vanelstraete and Francken, 1996). The nonwoven geotextile was placed with an extremely thin overlay of 40 mm. Two years after the repair, just 23% of the cracks were reflected and there were no other major problems reported with the geotextile. These results suggested that a lower percentage of cracks would have been reflected if the slabs had been cracked and seated or the thickness of the overlay had been increased (Vanelstraete and Francken, 1996). The results of all six sites also showed a relationship between the overlay thickness and the amount of reflective cracking. As the overlay thickness increased, the amount of reflective cracking decreased. For example, a 40 mm thick overlay showed the highest percentage of cracks; whereas a 140 mm thick overlay exhibited no reflective cracking (Vanelstraete and Francken, 1996).

Significant research has been conducted in Greece on the use of geotextiles to prevent reflective cracking in pavements. The first application of such a system was attempted in 1988 on the rehabilitation of a provincial road. The subgrade of that pavement was an expansive clay soil; thus the old HMA wearing surface showed signs of cracking (Collios, 1993). This could also have been a result of the heavy traffic loads and high temperature variations. The rehabilitation of the pavement included the use of a non-woven polypropylene geotextile as an interlayer between the original surface and an HMA overlay. The thickness of the overlay was calculated using fracture mechanics theory and found to be 100 mm. This thickness correlated to a maximum crack width of 20 mm and failure at 50,000 cycles of loading.

Before construction, all cracks with at least a 0.5 mm width were sealed with asphalt and larger cracks of up to 20 mm were filled with resins (Collios, 1993). A tack coat of 1.5 L/m^2 was applied before the geotextile was placed and all wrinkles were manually smoothed out. Another tack coat was sprayed on top of the fabric at a rate of 0.5 L/m^2. The asphalt overlay was placed in two lifts of 50 mm each. The pavement has been monitored since rehabilitation and the results were characterized as excellent. No reflective cracking appeared on the road after four years of use at doubling of traffic loads (Collios, 1993). The pavement was also subjected to two severe winters with no major problems. The design life expectancy of 50,000 cycles seems reasonable indicating that such an application could be duplicated for similar roads throughout Greece.

The Netherlands has conducted research on Structofers, a polyester fabric with a square weave impregnated with bitumen. The research centered on the bond between the geotextile and the HMA. This research incorporated the use of a slip layer in the waterproofing system. This slip layer allows a large horizontal slip to occur in order to prevent reflection cracks (van Zanten, 1986). However, the use of this slip layer was never tested due to laboratory limitations. A tack layer of approximately 1 kg/m was suggested to provide a good bond between a nonwoven geotextile of 200 g/m^2 and the existing pavement (van Zanten, 1986).

The California Department of Transportation (Caltrans) has conducted field studies on paving fabrics used in overlay applications. The findings indicated that the incorporation of a paving fabric could provide an increase in service life equivalent to placing an extra 30 mm of overlay without a fabric (Marienfeld and Baker, 1999). The additional service life is attributed to two factors, stress absorption and waterproofing. The stress absorption provided by the paving fabric may prevent the reflection of cracks. The waterproofing function lowers the water content beneath the fabric; thus protecting the structural integrity of the pavement.

2.4.2. Laboratory Testing

The broad range of products available for use as interlayers suggested a great need for laboratory testing in order to further analyze and understand the performance of geosynthetics, and thus select the appropriate product for a specific project.

2.4.2.1. *Thermal Cracking Laboratory Testing*

Francken and Vanelstraete did significant laboratory testing on overlay membrane systems, especially for thermal stress related fracture, in Belgium (Tschegg *et al.*, 1998). The specimens were composed of a cracked concrete base, a geotextile interlayer and an HMA overlay. The thermal cracking tests were performed on systems with five different interlayers, SAMI, nonwoven and woven textiles, grids and steel reinforcement nettings. The study was developed to investigate the effect of vertical displacements on the overlay system. The study concluded that interface systems might be used to prevent reflective cracking that may result from horizontal thermal movements in the underlying layers. Also, when comparing the results from the four different geotextiles used, it was evident that only the nonwoven textile fully impregnated with modified binder was effective in the prevention of cracks (Vanelstraete and Francken, 1996). The best results from the testing occurred with the steel reinforcement netting as an interlayer. No cracks were exhibited at any time during the experiment. Therefore, it is inferred that the amount and severity of reflective cracking is determined by the type of interlayer system and the degree of bonding to the underlying concrete slab (Vanelstraete and Francken, 1996).

Vanelstraete and Francken (1996) also simulated another cause of reflective cracking in a laboratory setting – the rocking of concrete slabs. This phenomenon can lead to reflective cracking in the overlay shortly after rehabilitation. The specimens were similar to those from the thermal-cracking test. They were also comprised of a cracked concrete slab, interlayer, and a bituminous overlay. However, these specimens were placed on a system with flexible support on one side and a rigid support on the other. The testing involved subjecting the specimen to vertical movements of about 1 mm at the crack edges (Tschegg *et al.*, 1998). This was a relatively new experiment that had only just begun. It is anticipated that this testing procedure will be used for a range of geotextiles, similar to the thermal cracking tests.

Also, it should be noted that the amount of rocking that occurs in concrete slabs is ultimately determined by the properties of the soil and base courses that lie below it. The laboratory testing can not be expected to completely simulate the field performance since these properties vary greatly among pavement locations. The laboratory study did recommend that the concrete slabs be cracked and seated before any rehabilitation begins (Vanelstraete and Francken, 1996). It was also concluded that the interlayer systems would help to decrease slab rocking close to the crack tip. However, the thickness of the asphalt overlay will ultimately decide the performance of the system (Vanelstraete and Francken, 1996).

2.4.2.2. Wedge Splitting Laboratory Test

Determining the strength of the bond between the geotextile and the surrounding layers is difficult. The bond strength has previously been determined by the pull-off test. This involves taking cores from the field and literally pulling the interlayer until a shearing takes place. There are many disadvantages to this method. The strength reading from the test gives no measure of the fracture properties or propagation of cracks (Tschegg *et al.*, 1998). Also, it cannot be determined from such a test whether the fracture took place in a brittle or ductile manner. In order to solve the problems with the pull-off test, a new procedure called the wedge-splitting test was developed to analyze the fracture behavior of overlay systems.

Cores, taken from the field, are placed on a linear support in a compression-testing machine. A wedge is inserted at the interlayer of the specimen. The wedge is then loaded transmitting a force to the specimen and causing its ultimate failure (Tschegg *et al.*, 1998). The loading system must be stiff during the entire test in order to simulate crack propagation. A load-displacement curve is generated from the test and this is used to characterize the fracture behavior of the material tested (Tschegg *et al.*, 1998).

The wedge-splitting test was performed on specimens with three different types of geotextiles, a nonwoven fabric (PGM 14), a non-woven fabric with glass grid reinforcement (PGM-G), and a nonwoven fabric with polypropylene grid reinforcement (TENSAR). The specimens consisted of a bituminous base material, one of the interlayers, and a 30 to 40 mm thick asphalt overlay (Tschegg *et al.*, 1998). A control specimen was also tested for the sake of comparison. Three to five identical specimens were tested for each type of geotextile and a variety of temperatures were also simulated.

The results of the tests show that the highest resistance against crack propagation occurs with the PGM 14 and PGM-G interlayers (Tschegg *et al.*, 1998). The cracks occurred in all of the specimens between the interlayer and the base course, where a lower resistance against crack propagation exists. The reason for this was that the asphalt overlay was applied hot, increasing the bond strength between itself and the geotextile. At the interface between the base course and the geotextile, the tack coat was applied to a worn surface that tends to be lack binder and has aggregates protruding (Tschegg *et al.*, 1998). The PGM 14 is useful in the case where the base course is particularly deteriorated because of its flexibility, which results in a better bond.

Conclusions of the laboratory testing indicate that the wedge splitting method is useful for the understanding of the interface and reflective cracking in overlay systems. The testing procedure is simple and can be performed at a relatively low cost. The load deflection curve generated from the testing is sufficient data to characterize the fractures of the specimens (Tschegg *et al.*, 1998). Also, fabrics with low stiffness and good bonding to the asphalt layer are crucial to preventing the propagation of reflective cracks. Further, a membrane that exhibits good bonding to the asphalt layer shows the highest resistance against cracking. Finally, this method can be used along with field studies to determine the proper construction quality needed to eliminate the propagation of reflective cracks (Tschegg *et al.*, 1998).

2.4.2.3. Permeability Tests

Laboratory investigations have been conducted on both field and laboratory prepared samples. The permeability of the field samples was determined from a variety of tests including gravity head, constant head, and vacuum procedures. Bushey (1976) determined, using a vacuum technique on field cores, that the waterproofing benefit provided by the paving fabric was greater than two orders of magnitude improvement. Guram (1983) used a gravity head test to show that cores containing paving fabric had permeabilities of 10^{-4} to 10^{-6} mm/s where cores without paving fabric had permeabilities of 10^{-3} to 10^{-4} mm/s (Marienfeld and Baker, 1999). Similar to the previous study, this showed improvements of one to two orders of magnitude in the cores with paving fabric.

Research was also conducted, using permeability tests, to determine the amount of tack coat appropriate for paving fabrics. Smith (1984) used a melt-through technique to show that the acceptable tack coat amount for nonwoven paving fabrics was between 0.9 –

1.4 L/m^2 (Marienfeld and Baker, 1999). Further, in order to provide an adequate waterproofing system, the amount of asphalt tack coat must saturate the fabric and bond the interlayer system. The amount of tack coat that will generally provide this bond is about 1.04 – 1.13 L/m^2 (Marienfeld and Baker, 1999). The use of less tack coat will significantly lower the waterproofing effects while more will tend to cause slippage in the interlayer system. It should also be noted that field installation is of great importance and the tack coat must be applied uniformly.

CHAPTER THREE

EXPERIMENTAL PROGRAM

A research program was developed to identify an optimized tack coat rate to be used with the geocomposite membrane in roads and bridges. This chapter details the experimental program.

3.1. TESTING FIXTURE

Simulated bridge and road samples including a geocomposite membrane were tested for shear failure at the geocomposite membrane interface under dynamic loading using a Materials Testing System (MTS) housed in the Structures and Materials Laboratory at Virginia Tech. A fixture (Figures 3.1 and 3.2) was designed and constructed specifically for this research. The purpose of this fixture is to transfer load that may simulate the shear stresses developed by vehicular loading during the life of the highway structure.

The fixture consists of two independent parts, each with a chamber. The two chambers are used for housing either concrete or HMA specimens. The first part, a chamber (100-mm inside diameter) connects to a loading shaft, which is connected to a load cell that measures the applied vertical loading and the MTS actuator. The chamber (98.4-mm inside diameter) of the second part connects to a load cell and a built-in control device. This chamber is connected to the lower part of the MTS system and is completely stationary. The two parts of the fixture are completely isolated. However, for stability purposes, a sleeve with linear bearings connects the two parts of the fixture while still allowing free vertical movement. Clamps are used to tighten the specimen into each chamber, thus reducing the possibility of torque or rotation during the test.

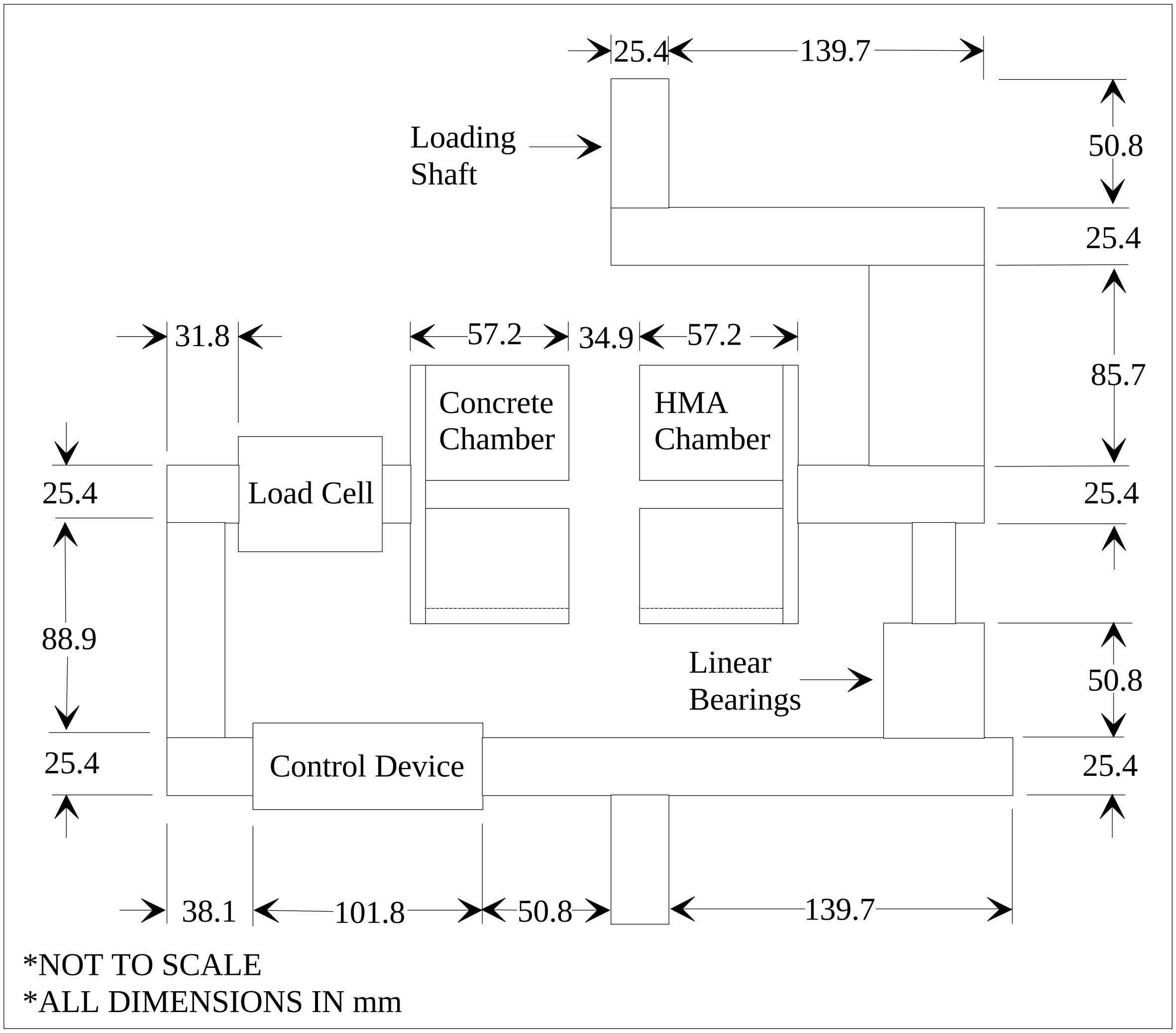

Figure 3.1. A Schematic of the Fixture Developed at Virginia Tech.

Testing using this fixture can be performed as either stress- or deflection-controlled. To properly simulate field conditions, a deflection-controlled test was used. The loading shaft connects to an MTS model 810 servo-hydraulic system that controls the vertical deflection. The servo-hydraulic system is controlled by an ATS data acquisition system that controls the characteristics of the dynamic vertical loading. A built-in load cell and LVDTs continuously measure the applied vertical load and deflection during the testing.

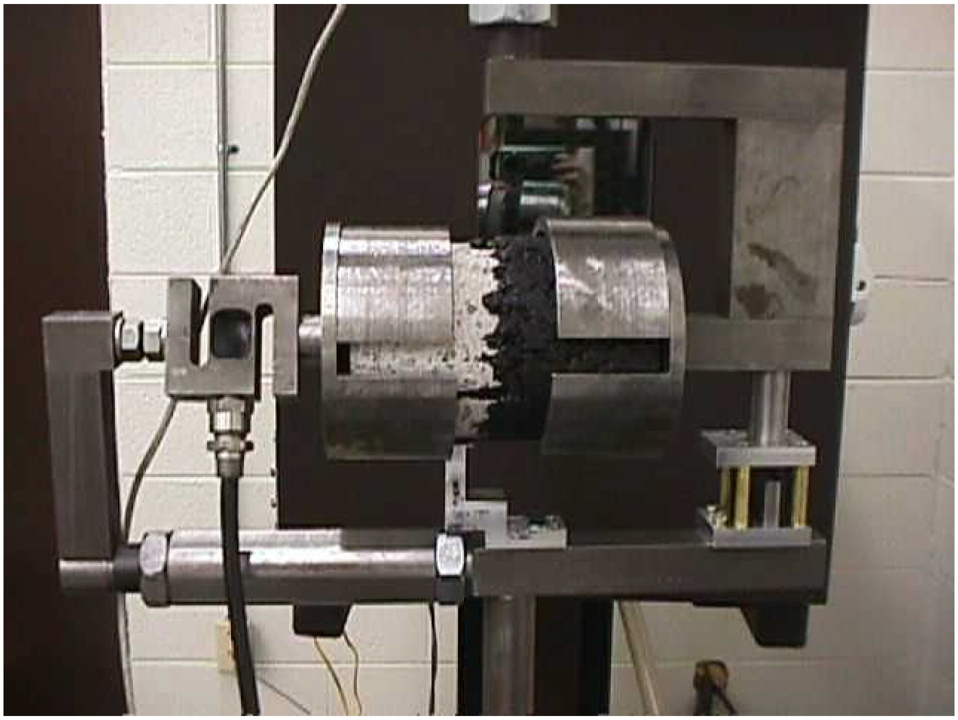

Figure 3.2. A Picture of the Fixture Developed at Virginia Tech.

3.2. MATERIALS PROPERTIES

3.2.1. Bridge Deck Specimens

For bridge deck simulation samples, large concrete slabs were cast using Type A1 mix, which is used in Virginia for bridge decks. The concrete mix design is presented in Appendix A. Cores were obtained after 28 days of concrete curing. Each concrete core was placed in a gyratory mold and geocomposite membrane was placed on the core. A wearing surface was compacted on top of the membrane using the gyratory compactor. The following is a description of the materials used and the specimen preparation process.

For concrete slab preparation, forms were built for three concrete slabs. Each slab was 1.14-m-long x 1.52-m-wide x 133-mm-thick. Wire mesh (2.5-mm in diameter) was placed in each form at the middle of the slab, providing temperature and shrinkage reinforcement during the curing time. The concrete mix (type A1) used is a typical mix used for bridge decks in the Commonwealth of Virginia. During the pouring of the concrete, quality control tests were performed, including slump, unit weight, and air content. The concrete slump was 16.5 cm, the unit weight was 2,220 kg/m^3, and the concrete had an air content of 5%. The seven-day compressive strength was 14.0 MPa and the 28-day compressive strength was 28.7 MPa.

After the concrete was poured, the slabs were surface-finished similar to that of a bridge deck, using a trowel. The slabs were cured for 28 days and then removed from the forms. These slabs were then cored in order to obtain cylinders that could be used in specimen preparation. The concrete core diameter was 93.7 mm. The concrete core was also cut to a length of 63 mm to fit into the testing fixture. The original surface of the core, similar to one on a bridge deck, was preserved to be used as the top surface in the specimen preparation.

SuperPave™ mix SM-9.5A was selected to be used as the HMA; “A” indicates that PG 64-22 was used as a binder. Approximately 5.6% asphalt binder was used in the mixture. The aggregates used in the SM-9.5A HMA are: #8 quartzite (50% by weight) with a maximum aggregate size of 12.5 mm; #10 quartzite (30%) with a maximum aggregate size of 4.75 mm; concrete sand (10%), which passes a 2.36 mm sieve; and processed RAP (10%). Sieve analyses were performed on two representative samples of each aggregate, as well as on two samples of the aggregate batch. The results of the sieve analyses, shown in Appendix A, were found to be within the specified range. The

#8 and #10 aggregates were both obtained from a quarry in Sylvatus, Virginia, the concrete sand was obtained from a quarry in Wythe, Virginia, and the RAP from an asphalt plant in Blacksburg, Virginia. The aggregate gradation and mix design are presented in Appendix A.

3.2.2. Road Specimens

The 150 mm HMA samples were prepared using the Troxler gyratory compactor. SuperPave^{TM} mix BM-25.0 was selected to be used as the HMA. As in the SM-9.5A mix, PG 64-22 was used as a binder. Approximately 4.6% asphalt binder was used in the mixture. The aggregates used in the BM-25.0 HMA are #357 limestone (20%), #68 limestone (30%), #10 limestone (30%), concrete sand (10%) and processed RAP (10%). Sieve analyses were performed on two representative samples of each aggregate, as well as on two samples of the aggregate batch. The results of the sieve analyses were found to be within the specified range and are presented in Appendix A. The #357, #68 and #10 limestone aggregates were obtained from a quarry in Blacksburg, Virginia, the concrete sand was obtained from a quarry in Wythe, Virginia, and the RAP from an asphalt plant in Blacksburg, Virginia. 150 mm HMA samples were cored to obtain samples. Each has a diameter of 93.7 mm in order to fit inside the testing fixture.

3.3. SPECIMEN PREPARATION

The preparation of road and bridge deck specimens was basically the same. The only difference was the base material used – concrete for the bridge deck specimens and BM-25.0 HMA for the road specimens. The overall process of preparing each batch of specimens took approximately 40 hr. This process began with the preparation of the aggregate for the HMA. Each sample contained approximately 1200 g of HMA. Since three samples were prepared at once, 3600 g of aggregate were weighed and placed in an oven heated to 135°C (300°F). This aggregate was heated overnight to ensure that all moisture was evaporated.

The asphalt binder was placed in the same oven and heated until it reached the mixing temperature of 126°C (285°F). While the asphalt binder was heating, the mixing bowl, whip, spoon, and spatulas were also placed in the oven to be heated to the mixing temperature. The asphalt binder was also stirred approximately every 20 min. to ensure even heating throughout the asphalt binder. When the binder was heated to the mixing temperature, the mixing bowl and aggregate were removed from the oven. The aggregate was placed in the bowl and a "crater" was formed in the middle to prepare for the addition of the binder. The mixing bowl and aggregate were placed on a 22,000 g scale. Next, the scale was zeroed to ensure that the exact amount of binder was added. For the SM-9.5A asphalt mix, 5.6% asphalt binder (by weight of the mix) was added. More specifically, 213.6 g of asphalt binder was added to the 3600 g of aggregate to prepare the HMA desired. The asphalt binder was then placed back in the oven to be used later for the tack coat. The mixing whip was then removed from the oven and placed on the mixer along with the bowl. The aggregate and asphalt binder were then mixed until the aggregate was thoroughly coated with asphalt, typically two min. The HMA was then placed in a shallow pan and heated in the same oven heated to 135°C (300°F) for approximately four hr. The purpose of this heating was to short term age the mixture before compaction, similar to field conditions.

The concrete and BM-25.0 samples were labeled with a permanent marker. The method of identification will be discussed further in Chapter 3, Section 4, Testing Program. Also, to allow for easier removal from the compaction molds, and to keep asphalt off the side of the concrete, the bottom half of the concrete sample was covered with duct tape. The geocomposite was then cut into a circle with approximately the same diameter as the concrete cylinder and BM-25.0 (93.7 mm). This was done using a band saw. A pair of scissors was then used to trim any excess fabric from the sides of the circle.

The next step was to apply the first layer of tack coat to the concrete or BM-25.0. This process was similar to the weighing of the asphalt binder in the mixing procedure. First, the concrete was placed on the scale and the scale was zeroed. Then, the tack coat was applied to the concrete or BM-25.0 with a small paintbrush. Extreme care was taken while applying the tack coat to ensure an even distribution of the binder. An even distribution was necessary to keep the application of the tack coat similar to field conditions where the tack coat is sprayed. Care was also taken to ensure that the exact amount of tack coat was applied. After the tack coat was applied to the concrete or BM-25.0, the geocomposite was placed on top of the tack coat. Another layer of tack coat was applied using the same small paintbrush.

The compaction mold was then removed from the oven. A hydraulic lift was used to raise the puck inside the mold. The sample was placed on the puck and slowly lowered into the mold. This was done to reduce the amount, if any, of tack coat that melted onto the sides of the mold. The mold was then placed on the scale and the scale was zeroed. The HMA was removed from the oven and approximately 1200 g was placed in the compaction mold on top of the sample. The mixture was leveled and a paper disk was placed on top. The mold was then placed inside the gyratory compactor. The SM-9.5A mix design specifies the design number of gyrations to be 86. The gyratory was set to a 1.25° angle of gyration and the sample was compacted. After the compaction was completed, the mold was removed from the gyratory and set aside to cool. The sample was cooled inside of the mold. After the sample was removed from the mold, the duct tape was removed and the sample was ready for testing.

3.4. TESTING PROGRAM

The specimen, which consists of concrete (or BM-25.0) core, geocomposite membrane, and SM-9.5A, was placed in the loading fixture with the SM-9.5A part in the moving chamber. First, the concrete (or BM-25.0) side of the specimen was held firmly in place using two clamps similar to O rings. Then, the other chamber, which houses the SM-9.5A part of the specimen, was slowly raised up until it was in contact with the specimen. The SM-9.5A side was then clamped down using two more clamps. This procedure was followed to ensure that the sample was completely leveled in the loading fixture. Since the SM-9.5A diameter was slightly larger than the diameter of the concrete, this was the only way to make sure that the specimen was level. After the sample had been secured in the fixture, the test was started.

A 0.40-mm deflection was applied to each specimen in the form of a 0.10-sec half sine wave, followed by a relaxation period of 0.9-sec (Figure 3.3). The test was run until failure. Failure was identified when the slope between applied load and the logarithm of number of cycles reached zero; at this point the test was considered complete. The raw data was recorded for the first ten cycles, then every ten until 100 cycles, every 50 until

300 cycles, every 100 until 1,000 cycles, every 1,000 until 10,000 cycles, and finally every 10,000 cycles after that. For each cycle, 50 measurements were taken, with 98% of these measurements in the first 0.1-sec. This procedure ensures that the maximum load is recorded during the cycle.

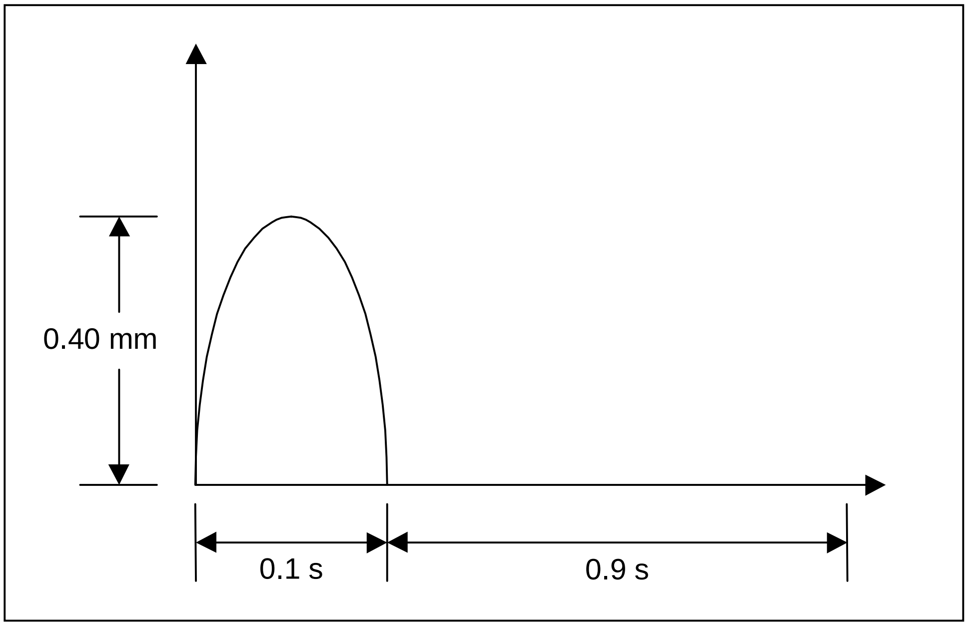

Figure 3.3. Loading Cycle for Testing.

As the main objective of this research was to optimize the rate of tack coat application, the focus was on varying the tack coat rate while keeping the concrete and HMA compositions constant. Initially, preliminary testing was conducted to determine five different tack coat rates to be used for testing. Various tack coat rates were applied to small samples of the geocomposite membrane. Seven different tack coat rates, ranging from 0.5 kg/m^2 to 3.0 kg/m^2, were applied to 76 x 76-mm geocomposite samples. This range was selected based on the rate of 2.5 kg/m^2 used between the concrete bridge deck and the geocomposite membrane. 3.0 kg/m^2 between the geocomposite membrane and HMA overlay has been used in Italy. However, a tack coat rate of approximately 1.00 kg/m^2 is usually used with membranes in the United States. In cases where just an HMA overlay was placed on a concrete bridge deck, the recommended amount of asphalt binder to be used as a tack coat rate is in the range of 0.22 to 0.45 kg/m^2.

Visual evaluating the 76 x 76-mm geocomposite membrane tack coated samples indicated that a 0.5 kg/m^2 tack coat rate was not sufficient. Most of the asphalt binder was absorbed into the fabric, leaving little tack coat to act as a bonding agent. Also, it was evident that 3.0 kg/m^2 was extremely high. Eliminating these two tack coat rates, five different rates, 1.0, 1.5, 1.75, 2.0, and 2.5 kg/m^2 were considered for evaluation.

For the bridge deck scenario, a total of 29 different specimens were prepared to address tack coat rate variations. Five different tack coat rates, one control, and two with a polymerized tack coat (PG 76-22) were tested. A tack coat rate of 0.45 kg/m^2 was used for the control specimens. Tack coat rates of 2.0 and 2.5 kg/m^2 were used for the PG 76-22 specimens. All of the specimens were prepared at least in replicates to obtain repeatable data. The same amount of tack coat was used on both sides of the geocomposite membrane in each specimen.

For the road scenario, a total of twelve specimens were tested. Four different tack coat rates and control samples at two tack coat rates were tested. Tack coat rates of 1.25, 1.5, 1.75 and 2.0 kg/m^2 were used for the geocomposite specimens. For the control specimens, tack coat rates of 0 and 0.45 kg/m^2 were tested. All of the specimens were prepared in duplicate.

The nomenclature used to identify the specimen is fairly simple. An example for the bridge deck samples is B1_0B. The first letter, “B”, indicates that this is a bridge deck specimen. If the specimen were to be a control, a “C” would be added after this first letter. The next part of the identification, “1_0”, indicates that 1.0 kg/m^2 of tack coat was applied to the specimen. The last letter, “B”, indicates the duplicate identification, meaning that this is the second sample for this tack coat.

CHAPTER FOUR

DATA ANALYSIS AND RESULTS

Based on the experimental procedure discussed in Chapter Three, the analysis of the collected data is presented in this chapter. In general, data were analyzed by plotting the maximum shear stress (the machine loading applied divided by the surface area at the interface of the geocomposite membrane) against the number of cycles on a semi-logarithmic scale. This data is used to identify the number of cycles to reach failure for each specimen.

4.1. ANALYSIS OF SIMULATED BRIDGE DECK SPECIMENS

Specimens containing a concrete core, tack coat, geocomposite membrane, and an HMA wearing surface were tested using a fixture, designed and assembled at Virginia Tech as discussed in Chapter Three, allowing the simulation of shear stresses on bridge decks resulting from vehicular loading. The tack coat application rate was varied from 1.0 kg/m^2 to 2.5 kg/m^2, with each specimen prepared at least in duplicate. Control specimens containing concrete core, tack coat, and HMA wearing surface were also tested using the same fixture. As stated above, a plot of shear stress vs. number of cycles was generated from each tested specimen. The results of this data analysis are summarized below.

As described in Chapter Three, fifty data points were collected during each 1-sec cycle. The data was then filtered to obtain the maximum shear stress during each of these cycles. This maximum stress was then plotted on a semi-logarithmic scale versus the number of cycles. Since the first 1,000 loading cycles were attributed to seating, they are omitted from each plot. A typical relation between shear stress and number of loading cycles for a 1.75 kg/m^2 tack coat rate is shown below in Figure 4.1. Plots of all other specimens are shown in Appendix C.

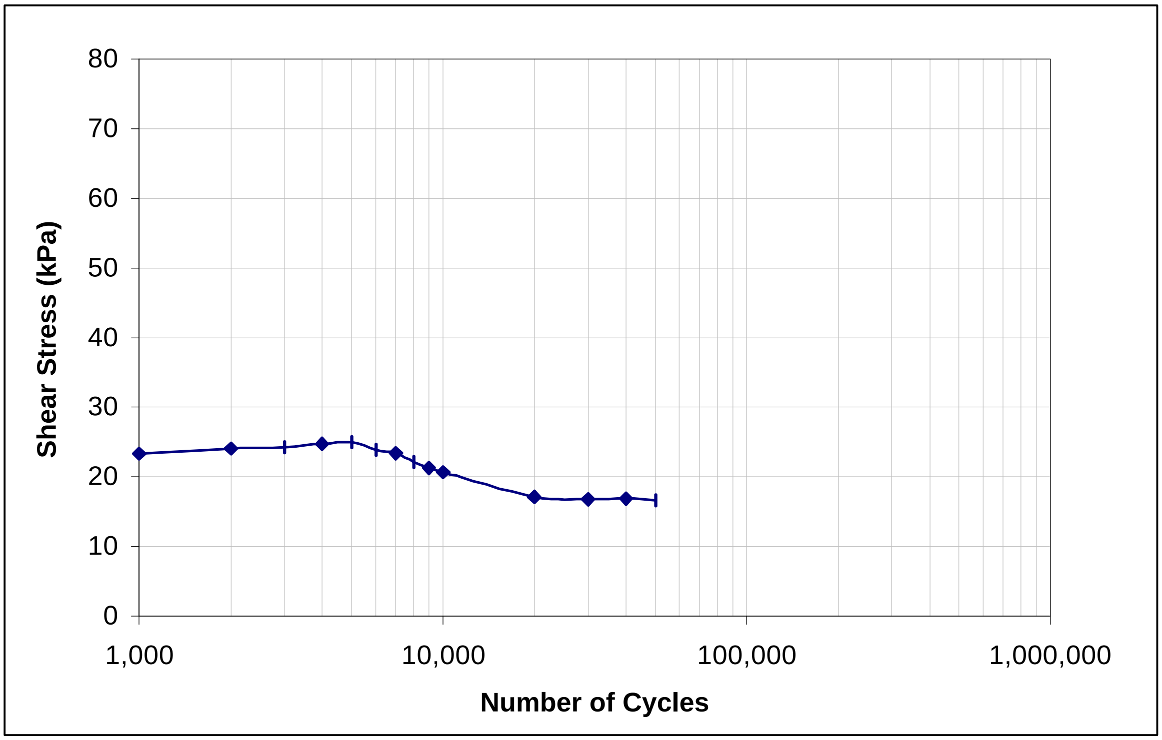

Figure 4.1. Typical Data for a Simulated Bridge Deck Specimen.

The above data is typical for most of the samples tested to failure. Also, duplicates of each tack coat were tested to ensure accuracy and repeatability in the results obtained. However, during the data analysis process, some of the samples were deemed to be outliers due to their inconsistency with other duplicate specimens that resulted from sudden machine failure (i.e. due to a power failure) in most cases. Table 4.1 shows the results of all consistent specimens along with the number of cycles to failure for each data analysis. The failure point was determined to be the point at which the slope of the shear stress vs. number of cycles line became zero. At this point, the test was considered complete and all data past that point was eliminated.

Table 4.1. Bridge Deck Specimen Test Results.

Sample	Amount of Tack Coat (kg/m^2)	Type of Asphalt Tack Coat	Number of Cycles to Failure	Average Number of Cycles to Failure	Standard Deviation	90% Confidence Limits: Upper Limit	90% Confidence Limits: Lower Limit
B1_0B	1.00	PG 64-22	20000	12333	6658	21379	3288
B1_0C	1.00	PG 64-22	8000				
B1_0D	1.00	PG 64-22	9000				
B1_5C	1.50	PG 64-22	10000	27500	17078	45705	9295
B1_5D	1.50	PG 64-22	50000				
B1_5E	1.50	PG 64-22	20000				
B1_5F	1.50	PG 64-22	30000				
B1_75A2	1.75	PG 64-22	40000	50000	10000	59011	40989
B1_75B	1.75	PG 64-22	60000				
B1_75C	1.75	PG 64-22	60000				
B1_75D	1.75	PG 64-22	40000				
B1_75E	1.75	PG 64-22	50000				
B2_0A	2.00	PG 64-22	40000	40000	0	40000	40000
B2_0B	2.00	PG 64-22	40000				
B2_0C	2.00	PG 64-22	40000				
B2_5A	2.50	PG 64-22	30000	35000	7071	49600	20400
B2_5C	2.50	PG 64-22	40000				
BC_A	0.45	PG 64-22	2000	2000	0	2000	2000
BC_B	0.45	PG 64-22	2000				
B2_0P1	2.00	PG 76-22	50000	45000	7071	59600	30400
B2_0P2	2.00	PG 76-22	40000				
B2_5P1	2.50	PG 76-22	50000	65000	21213	108800	21200
B2_5P2	2.50	PG 76-22	80000				

From the average value and 90% confidence limits of the number of cycles to failure for each tack coat application rate, a plot (Figure 4.2) was generated and used to determine the optimum tack coat application rate. The relationship between tack coat rate and the number of loading cycles to failure suggests that a tack coat rate of 1.75 kg/m^2 may provide the optimum performance.

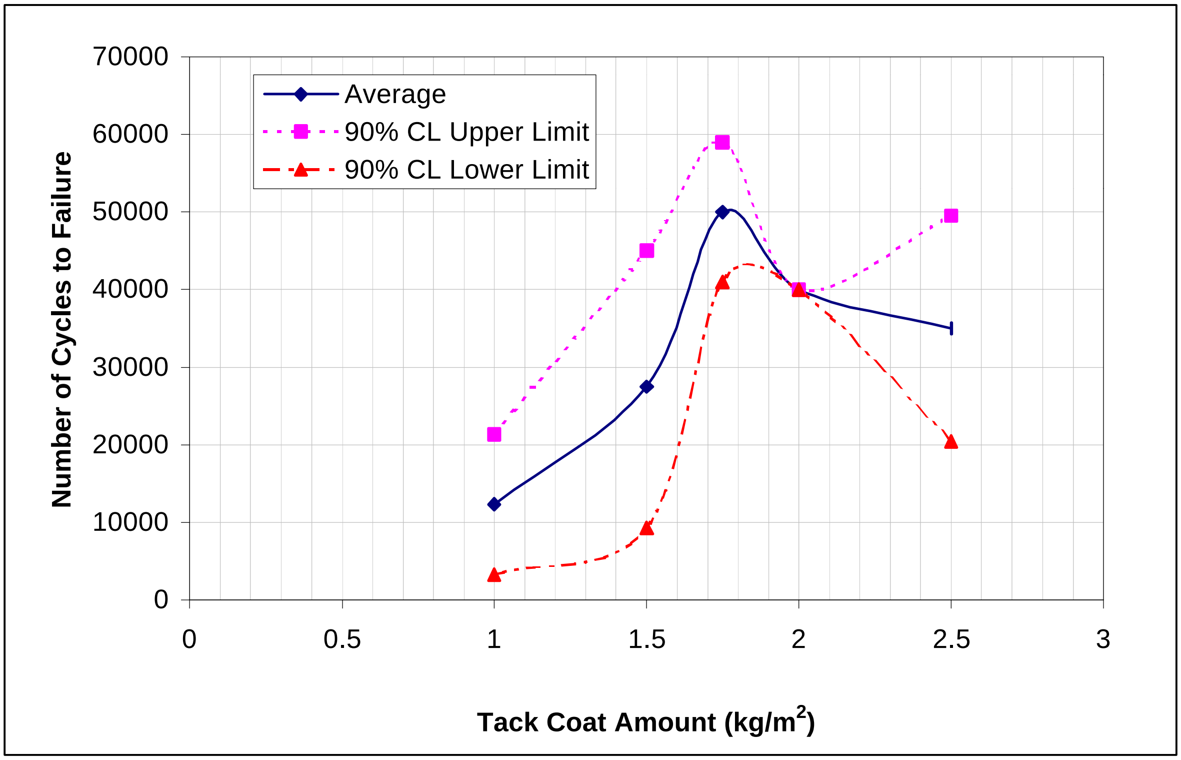

Figure 4.2. Optimum Tack Coat Determination for Bridge Deck Specimens.

The variation in the number of loading cycles to failure can be attributed to several factors, including specimen preparation. It was evident that some of the tack coat was lost in the process of preparing specimens. This was due to the fact that the gyratory compactor mold was heated to mixing temperature when the concrete core and tack-coated geocomposite membrane were lowered into it. This caused some of the binder tack coat to soften and adhere to the side of the mold. Also, during the compaction process, additional tack coat was squeezed down onto the sides of the specimen in some cases. There was no way to accurately determine the amount of tack coat lost during the specimen preparation process but it is expected to be less than 10% for high tack coat rates and very minimum for the low tack coat rates.

The data analysis revealed important observations. When a geocomposite membrane was used, all specimens failed at the geocomposite membrane-HMA interface. The failure reduction in shear stress was gradual in the case where the geocomposite was used. However, a sudden and significant reduction in shear stress (more than 50%) was noted in the case of the control specimens; this reduction was proceeded by a slight increase in shear stress. In addition, the resultant initial shear stress was significantly lower when the geocomposite membrane was used, which may suggest the apparent characteristic of the geocomposite membrane as a stress absorber.

As would be expected, binder type has a significant effect on the system performance when used as a tack coat. Table 4.1 shows that four specimens were prepared and tested with a polymerized tack coat, PG 76-22 (containing approximately 3% of a styrene-butadiene-styrene block copolymer). These specimens show a slight increase in the number of cycles to failure. However, the increase in cost associated with this binder may not justify the slight increase in system performance. It appears that the use of PG 64-22 binder will be sufficient in achieving the optimum system performance. Figure 4.2 depicts an optimum tack coat rate of 1.75 kg/m^2. This tack coat application rate will provide a sufficient bond with the concrete bridge deck while minimizing the potential for slippage attributed to excess tack coat. For the upper surface of the geocomposite membrane, a rate of 1.5 kg/m^2 may be applied if an HMA surface mix is used.

4.2. ANALYSIS OF SIMULATED FLEXIBLE PAVEMENT SPECIMENS

Specimens containing an HMA base layer (BM-25.0), tack coat, geocomposite membrane, and an HMA wearing surface (SM-9.5A) were tested using the same fixture. The tack coat application rate was varied from 1.25 kg/m^2 to 2.0 kg/m^2, with each specimen prepared in duplicate. Control specimens containing HMA base layer, tack coat, and HMA wearing surface were also tested using the same fixture. As stated above, a plot of shear stress vs. number of cycles was generated from each tested specimen. The results of this data analysis are summarized below.

As stated in Chapter Three, 12 flexible pavement specimens were prepared and tested. Similar to the concrete bridge deck specimens, the data was collected for 1-sec cycles, with 50 data points recorded during each cycle. The maximum shear stress during each cycle was then plotted on a semi-logarithmic scale versus the number of loading cycles. A typical plot of the shear stress versus the number of loading cycles is shown in Figure 4.3. The remainder of the plots for the flexible pavement specimens is shown in Appendix D.

Figure 4.3 depicts data that is typical for most of the flexible pavement specimens tested. Each tack coat rate was tested in duplicate to ensure repeatability and accuracy among specimens with the same tack coat. However, during testing on some of the specimens, power failures caused a premature failure or some data to be lost. Due to this inconsistency, these specimens were disregarded for the determination of the optimum tack coat application rate. Table 4.2 shows the flexible pavement specimens that were deemed satisfactory to determine the optimum tack coat application rate. Number of cycles to failure represents the average of two tests, when applicable.

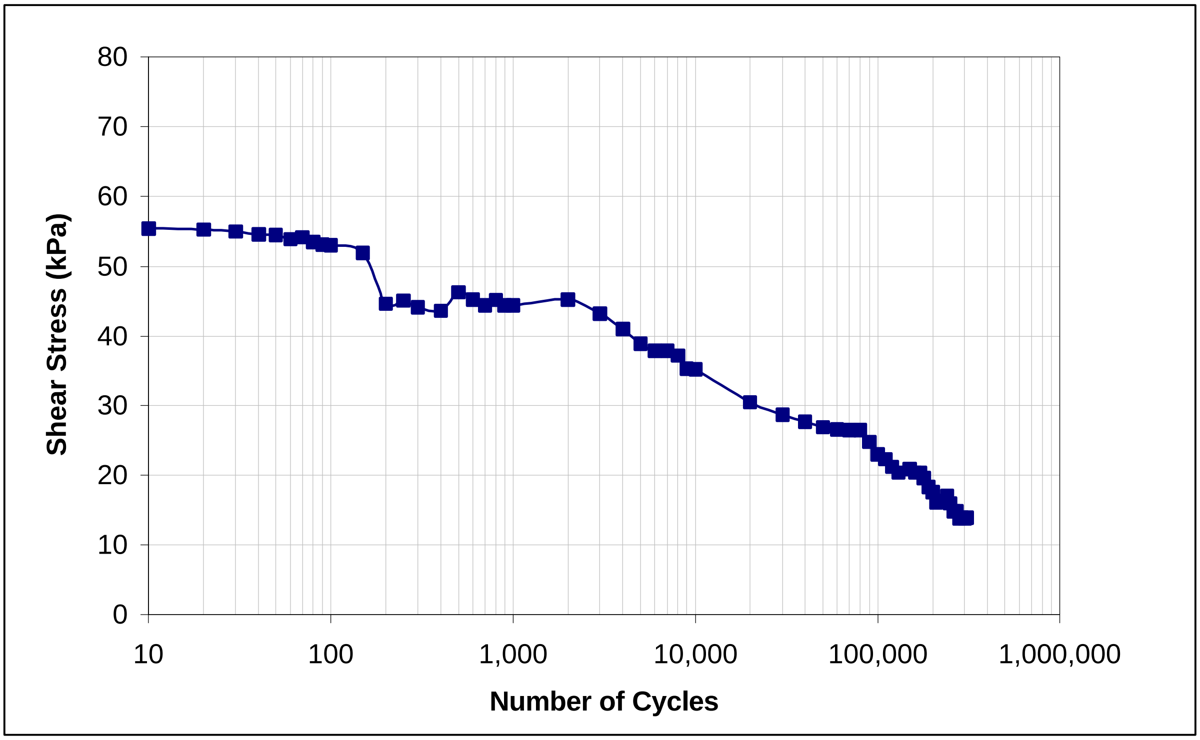

Figure 4.3. Typical Data for a Simulated Flexible Pavement Specimen.

Table 4.2. Flexible Pavement Specimen Test Results.

Sample	Amount of Tack Coat (kg/m^2)	Type of Asphalt Tack Coat	Average Number of Cycles to Failure
R1_25	1.25	PG 64-22	260000
R1_5	1.50	PG 64-22	370000
R1_75	1.75	PG 64-22	150000
R2_0	2.00	PG 64-22	180000
RC0_0	0.00	PG 64-22	250000
RC0_45	0.45	PG 64-22	130000

From the average number of cycles to failure in the above table, a plot was generated to determine the optimum tack coat application rate (Figure 4.4). A third order polynomial was fit to the data points, concluding that the optimum tack coat application rate for flexible pavement was 1.40 kg/m^2. Using a PG 64-22 asphalt binder at a tack coat application rate of 1.40 kg/m^2 will provide a sufficient bond between the geocomposite membrane and the HMA layers. It is the author's opinion that when the geocomposite membrane is used on a newly paved layer in the same day, a tack coat application rate of 1.25 kg/m^2 may be sufficient. However, if the geocomposite membrane is placed after a day or more, a tack coat application rate of 1.40 kg/m^2 is recommended. This rate may be increased for rehabilitation as needed to rejuvenate the existing layer. However, the upper geocomposite surface may receive the rate application of 1.5 kg/m^2 if a surface mix is used, and 1.40 kg/m^2 if a base mix is used.

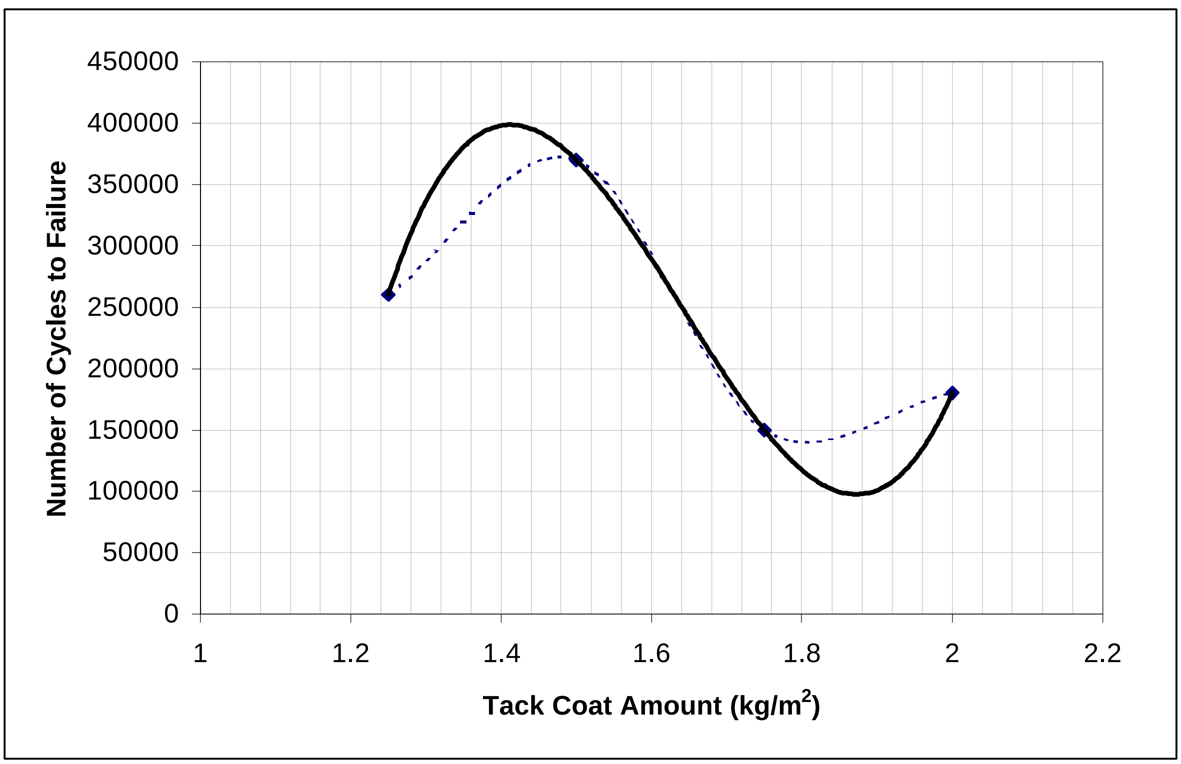

Figure 4.4. Optimum Tack Coat Determination for Flexible Pavement Specimens.

It was observed that control specimens with zero tack coat actually performed better than the other control specimens (0.45 kg/m^2 tack coat), producing a higher number of loading cycles to failure (Table 4.2). From this, it is obvious that there is no justification for the additional cost of a tack coat between HMA layers built in the same day; however, a minimum tack coat rate may be needed if construction of the next layer takes place after a day or more. For overlays, a tack coat is necessary as the binder of the existing layer is usually aged. The tack coat rate is dependent on the type and condition of the existing layer.

CHAPTER FIVE

FINDINGS AND CONCLUSIONS

In order to quantitatively determine the optimum tack coat for use with the geocomposite membrane, a testing approach and program, including the design and construction of a testing fixture and the preparation and testing of laboratory prepared specimens simulating road and bridge cores, was developed at Virginia Tech. Laboratory pavement specimens consisted of the geocomposite membrane sandwiched between a base HMA layer (SuperPave^{TM} mix design BM-25.0) and an HMA wearing surface (SuperPave^{TM} mix design SM-9.5A; A indicates a PG 64-22 asphalt binder). Simulated bridge deck specimens consisted of a geocomposite membrane sandwiched between a concrete cylinder cored from a concrete slab similar to that of a bridge deck in Virginia and an SM-9.5A HMA wearing surface.

The parameter tested was the amount of tack coat rate applied on both sides of the geocomposite membrane, which was varied. Control specimens, those with no geocomposite membrane, were also included. All of these specimens were tested in fatigue using the specifically designed fixture under a specific pattern of loading to simulate field vehicular loading. The optimum tack coat was determined as the amount of tack coat needed to provide the maximum number of cycles to induce failure.

5.1. FINDINGS

Throughout the course of this research, the following findings were noted:

- Analysis of the simulated bridge deck specimens with geocomposite membrane and the control samples, containing no membrane, shows distinct evidence that the membrane acts as a stress-absorbing material.
- The use of a polymerized asphalt binder as a tack coat provides a slight increase in the number of cycles to failure; however, this increase in service life may not be justified by the significant increase in cost associated with the polymerized binder.
- Failure in bridge deck simulated specimens, with geocomposite membrane, was always at the geocomposite membrane-HMA interface.
- A distinct failure was not evident in the simulated flexible pavement samples; this may be attributed to the viscous nature of HMA.
- The optimized tack coat application rates are a function of the structural material type and the tack coat material type (binder performance grade).

5.2. CONCLUSIONS

Based on this research, the following conclusions can be made:

- The fixture, designed and constructed at Virginia Tech specifically for this laboratory testing, proved to be efficient in applying shear stress to a geocomposite membrane interface when used in a simulated bridge deck with overlay or in HMA pavements. Thus, the fixture may simulate stresses resulting from vehicular loading applied to pavements and bridges during their service life.
- From the testing of simulated bridge deck samples containing a concrete core, tack coat, geocomposite membrane and an HMA wearing surface, it was determined that

a tack coat application rate of 1.75 kg/m^2 will provide the optimum performance for the geocomposite membrane in contact with the concrete bridge deck. For the upper surface in contact with a wearing surface mix, a tack coat application rate of 1.5 kg/m^2 may be used.

- From the testing of simulated flexible pavement specimens containing an HMA base layer, tack coat, the geocomposite membrane, and an HMA wearing surface, the optimum tack coat application rate was determined to be 1.40 kg/m^2. If the geocomposite is placed within one day of paving, a tack coat application rate of 1.25 kg/m^2 may be used.
- It is recommended to use 1.40 kg/m^2 when the geocomposite surface is in contact with an HMA base mix, 1.5 kg/m^2 when it is in contact with an HMA wearing surface mix, and 1.75 kg/m^2 when it is in contact with concrete surfaces.

CHAPTER SIX

RECOMMENDATIONS

The following recommendations may be considered for improving the quality of this testing program and to allow for further knowledge on the use of geocomposite membrane in pavement and bridge construction and rehabilitation:

- The testing program can be expanded to include other types of materials including other geocomposite membranes, other flexible pavement layers (i.e. other types of HMA, drainage layers, etc.), and different concrete mixes.
- The permeability of the membrane should be determined in order to prove the assumption that it is an impermeable layer and would indeed prevent further corrosion of reinforcing steel in concrete bridge decks.
- Field installation procedures need to be established.
- A study on geocomposite membrane field performance in bridge decks and roads needs to be conducted.

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APPENDIX A

PHYSICAL PROPERTIES OF MATERIALS USED IN TESTING

Table A.1. Concrete mix design.

Material	Amount	Source	Plant/Quarry Location
Cement (Type I/II)	144.3 kg (318 lb)	Capitol Cement	Martinsburg, WV
Pozzolans (Blast-furnace Slag)	143.8 kg (317 lb)	Blue Circle	Sparrow's Point, MD
Sand	676.3 kg (1491 lb)	Wythe Stone	Wytheville, VA
No.7 Stone	644.1 kg (1420 lb)	Acco Stone	Blacksburg, VA
Water	129.8 L (34.3 gal)	Municipal Water	Town of Blacksburg, VA
Air Entrainment	0.24-0.33 L (8-11 oz)	W.R. Grace	Cambridge, MA

Table A.2. Properties of concrete materials.

Pozzolans (Blast-furnace Slag)	
Specific Gravity	2.94
Sand	
Absorption	0.3
Fineness Modulus	2.8
Specific Gravity	2.64
Coarse Aggregate	
Absorption	0.4
Specific Gravity	2.8
Unit Weight	95.6

Table A.3. Quality control test results.

Slump (cm)	16.5
Air Content (%)	5.0
Unit Weight (kg/m^3)	2220
Compressive Strength (MPa)	
Day 7	14.0
Day 28	28.7

Table A.4. SM-9.5A mix design.

Material	Weight (%)	Source	Location
#8 Quartzite	50	Salem Stone Co.	Sylvatus, VA
#10 Quartzite	30	Salem Stone Co.	Sylvatus, VA
Concrete Sand	10	Wythe Stone Co.	Wytheville, VA
Processed RAP	10	Adams Construction Co.	Blacksburg, VA
PG64-22 Asphalt Binder	5.6	Associated Asphalt Inc.	Roanoke, VA

Table A.5. SM-9.5A gradation.

		SPECIFICATIONS	
Sieve Size (mm)	Percent Finer (%)	Lower Limit	Upper Limit
12.5	100	100	100
9.5	93	90	100
4.75	53	53	90
2.36	37	32	67
0.075	3.8	2	10

Table A.6. BM-25.0 mix design.

Material	Weight (%)	Source	Location
#357 Limestone	20	Acco Stone Co.	Blacksburg, VA
#68 Limestone	30	Acco Stone Co.	Blacksburg, VA
#10 Limestone	30	Acco Stone Co.	Blacksburg, VA
Concrete Sand	10	Wythe Stone Co.	Wytheville, VA
Processed RAP	10	Adams Construction Co.	Blacksburg, VA
PG64-22 Asphalt Binder	4.6	Associated Asphalt Inc.	Roanoke, VA

Table A.7. BM-25.0 gradation.

		SPECIFICATIONS	
Sieve Size (mm)	Percent Finer (%)	Lower Limit	Upper Limit
37.5	100	100	100
25	90	90	100
19	83	80	90
2.36	30	19	45
0.075	4.5	1	7

Table A.8. Properties of geocomposite membrane (PVC geomembrane with polyester geotextile backing reinforcement).

Property	Test Method	Value
Thickness (Geomembrane)	ASTM D 1593	2 mm +/-10%
Specific Gravity	ASTM D 792	>=1.2
Areic Mass (Geotextile)	ASTM D 5261	>=150 g/m2
Tensile Strength		
Geotextile Break Tension	ASTM D 882	>=20 kN/m
Geotextile Break Elongation		>=35%
Geomembrane Tension		>=20 kN/m
Geomembrane Break Elongation		>=250%
Tear Resistance	ASTM D 1004	>=80 N
Puncture Resistance	ASTM D 4833	>=310 N
Hydrostatic Resistance	ASTM D 3786	>=1.5 MPa
Low Temperature/Brittleness	ASTM D 1790	no failure at -18 deg C
Volatile Loss	ASTM D 1203	<= -0.3%
Water Extraction	ASTM D 3083	<= -0.2%
Dimensional Stability	ASTM D 1204	+/- 2%
UV Resistance	ASTM G 53	<=15%


Figure A.1. Placing Concrete in Wood Forms.

Figure A.2. Measuring the Slump of the Concrete.

Figure A.3. Determining the Air Content of the Concrete Mix.

Figure A.4. Filling Cylinders for Compressive Tests at 7 and 28 days.

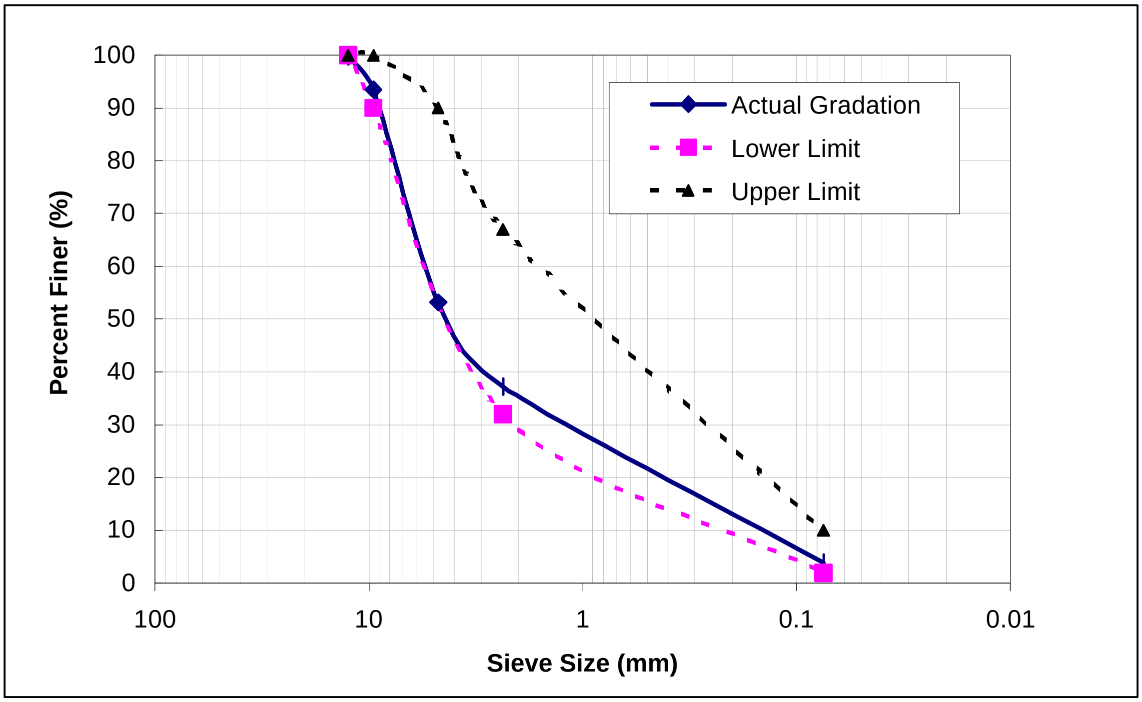

Figure A.5. Graphical Representation of SM-9.5A Gradation with Specification Limits.

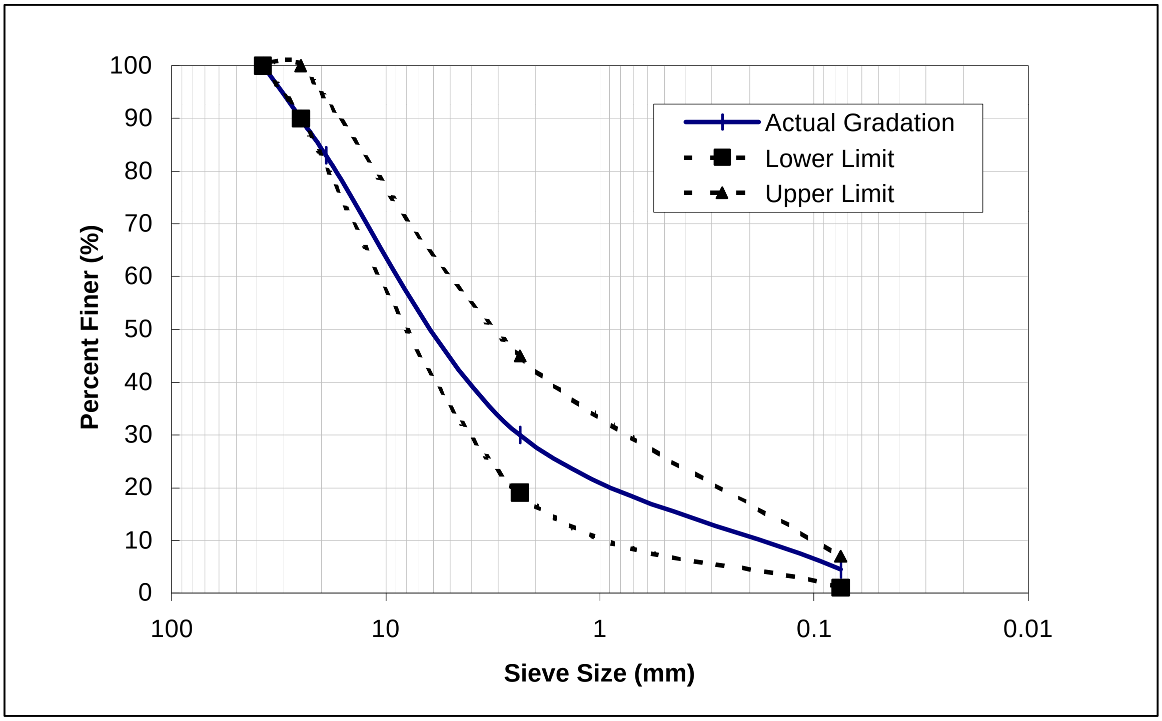

Figure A.6. Graphical Representation of BM-25.0 Gradation with Specification Limits.

APPENDIX B

DETAILS OF TESTING FIXTURE

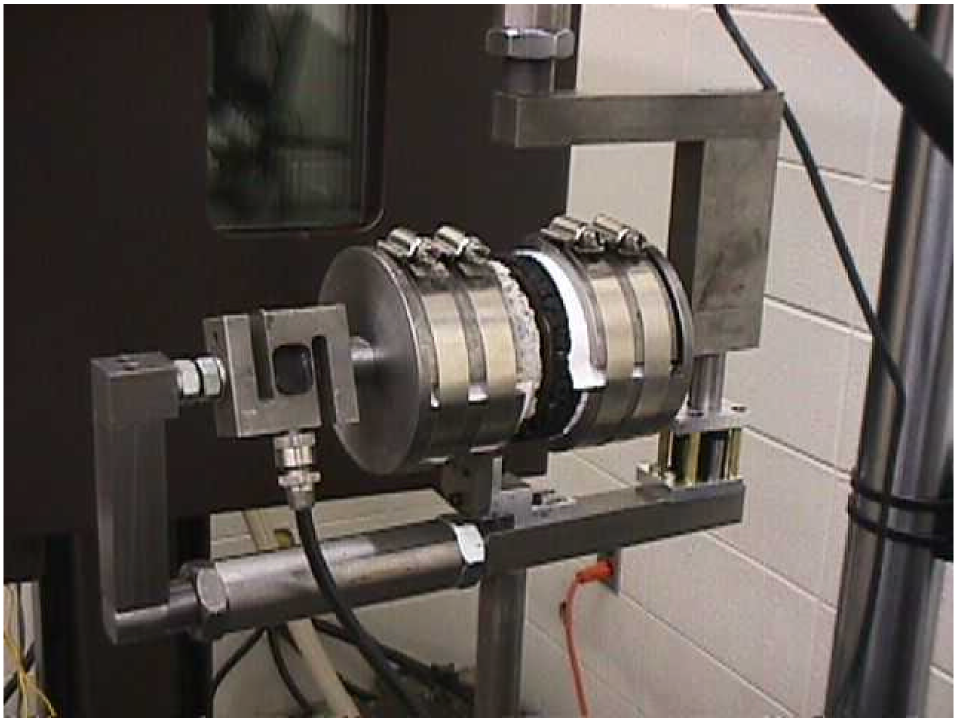
Figure B.1. Side View of the Fixture.

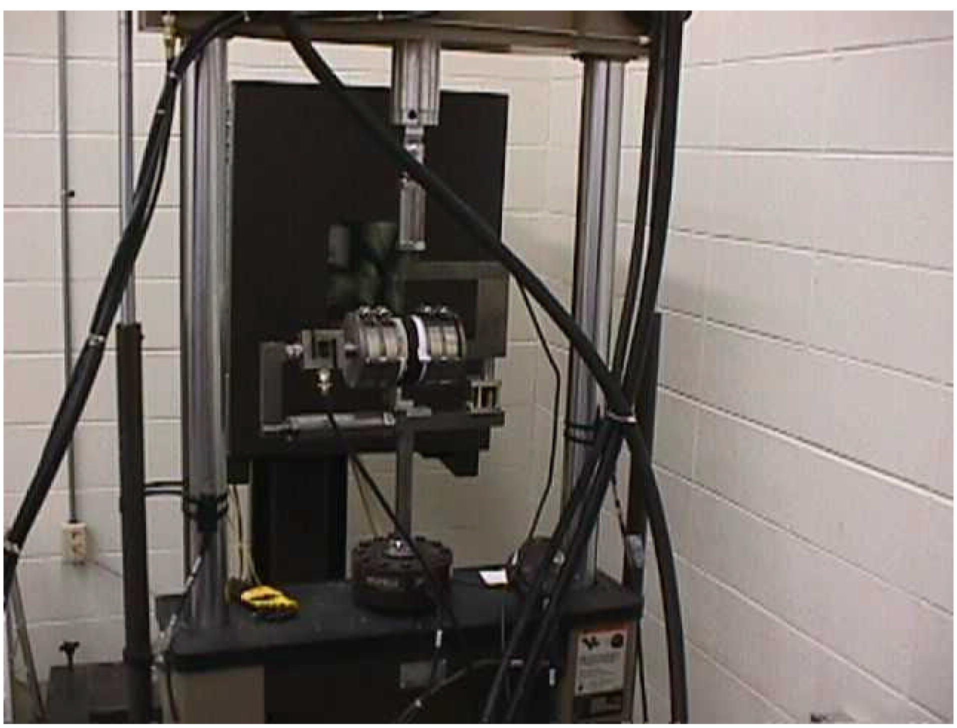
Figure B.2. Fixture with Surrounding Loading Frame.

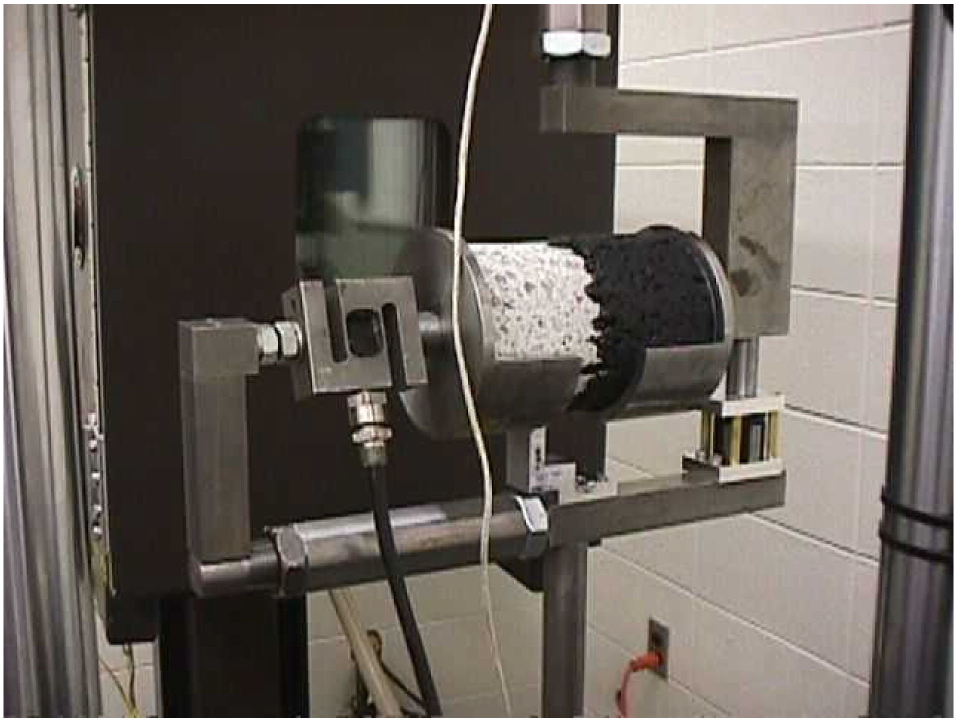
Figure B.3. Loading Simulated Bridge Deck Control Specimen into the Fixture.

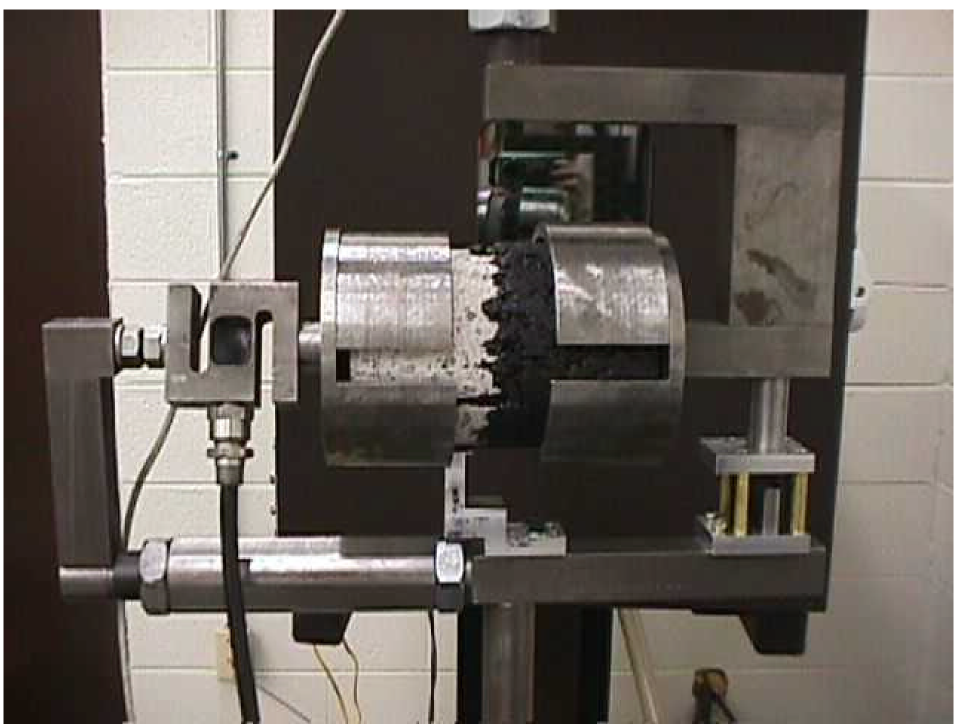
Figure B.4. Front View of the Fixture Containing Bridge Deck Control Specimen.

APPENDIX C

SIMULATED BRIDGE DECK SPECIMEN DATA AND PICTURES

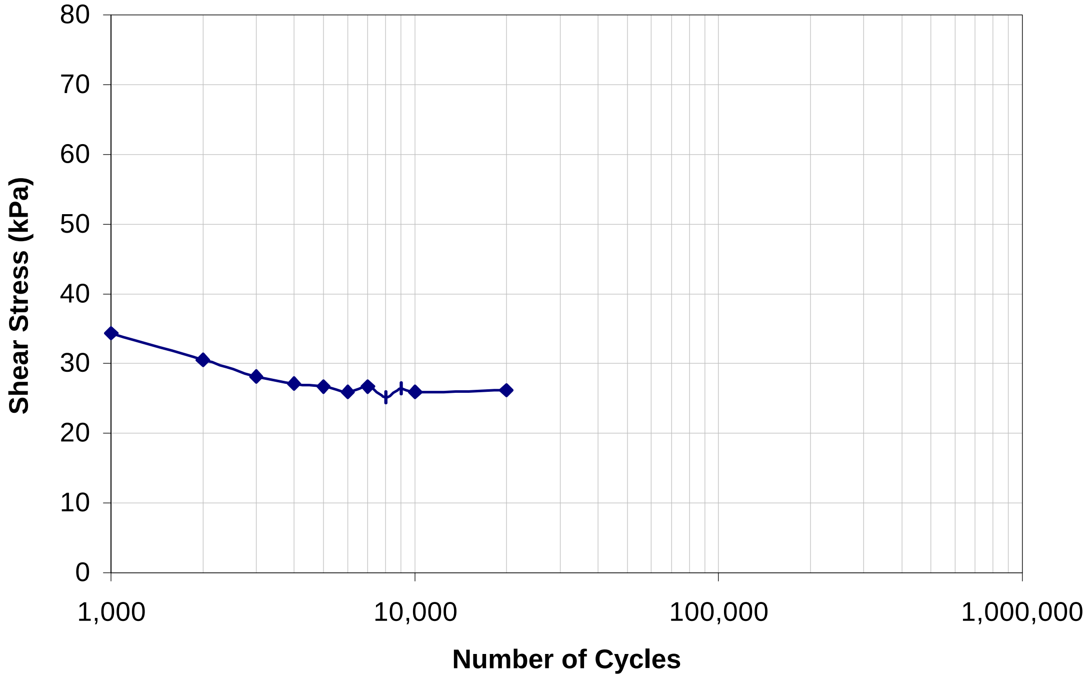

Figure C.1. Shear Stress versus Number of Loading Cycles for Specimen B1_0B.

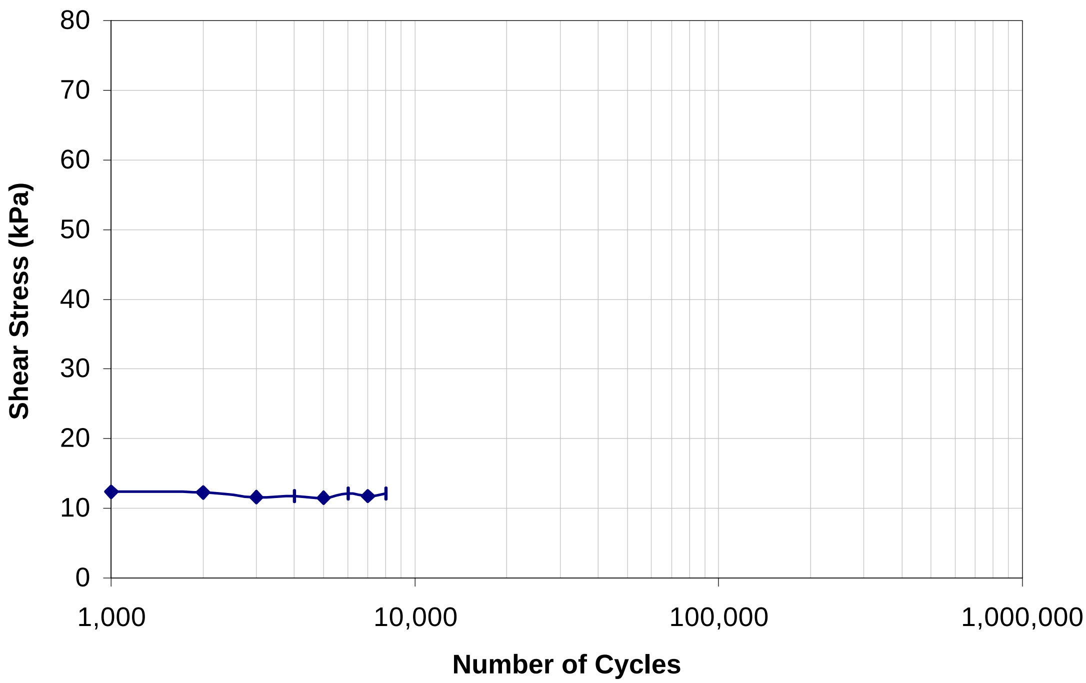

Figure C.2. Shear Stress versus Number of Loading Cycles for Specimen B1_0C.

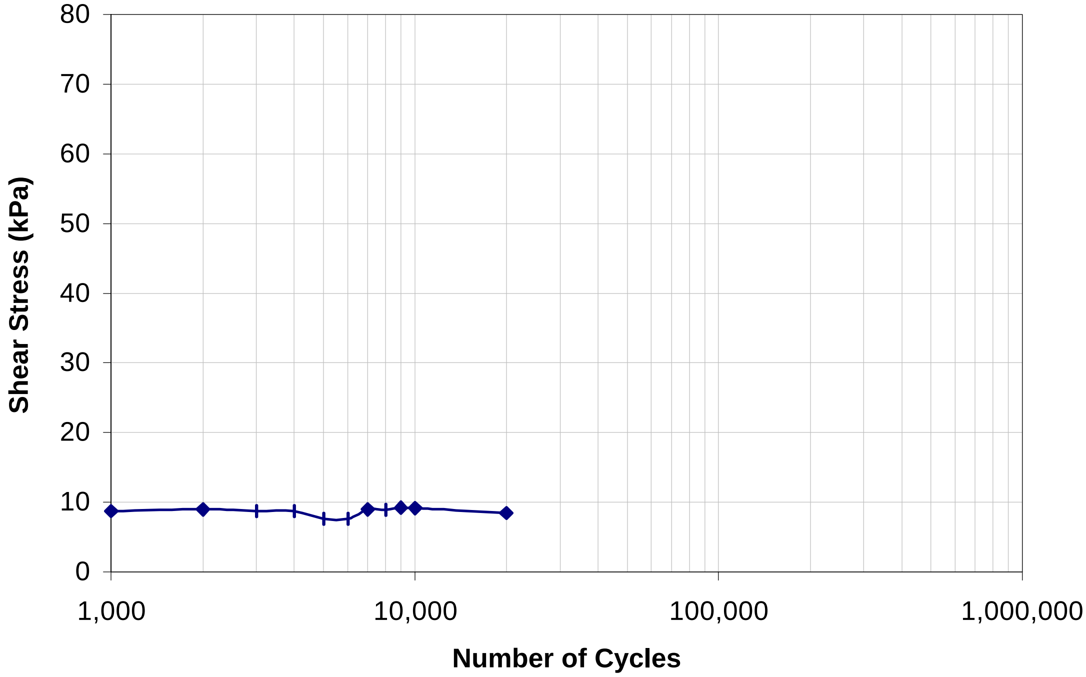

Figure C.3. Shear Stress versus Number of Loading Cycles for Specimen B1_0D.

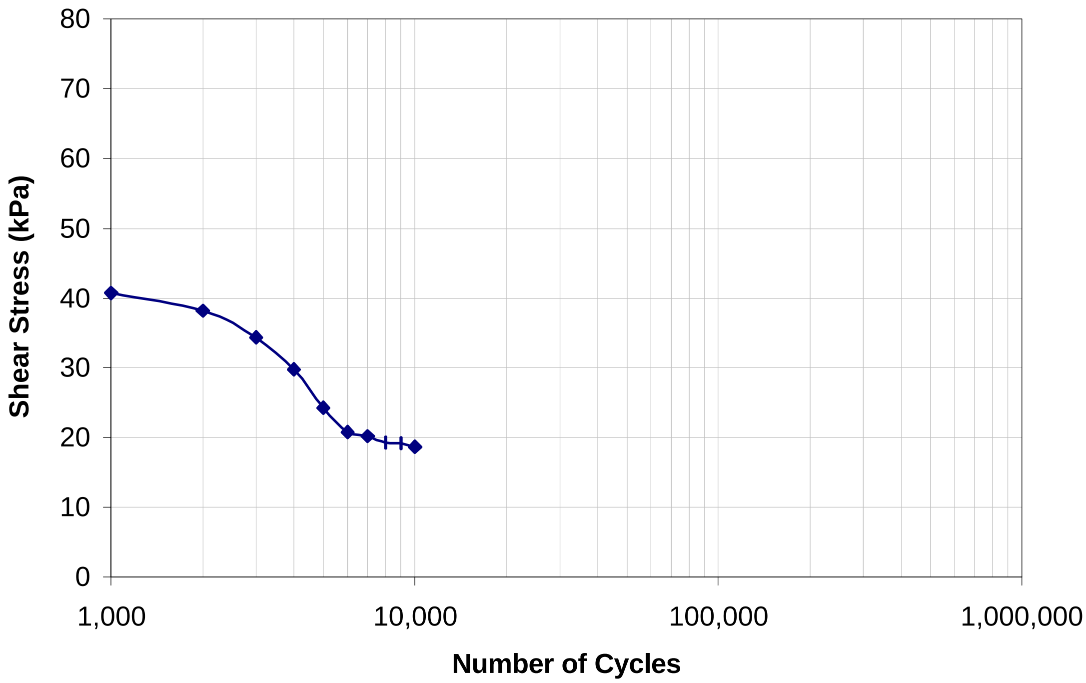

Figure C.4. Shear Stress versus Number of Loading Cycles for Specimen B1_5C.

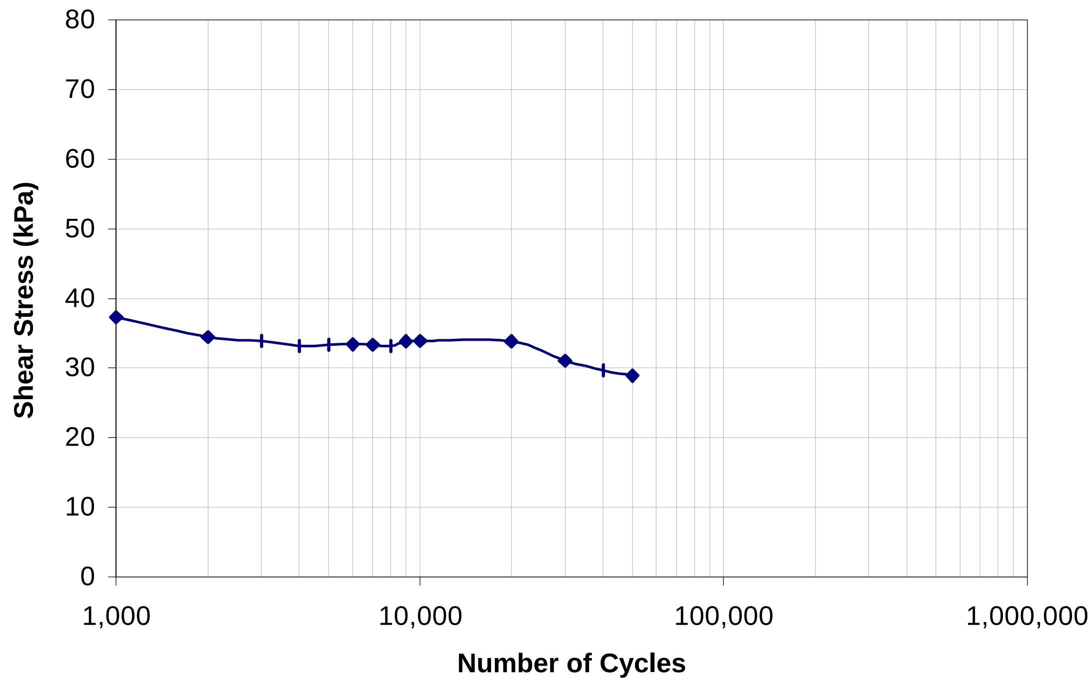

Figure C.5. Shear Stress versus Number of Loading Cycles for Specimen B1_5D.

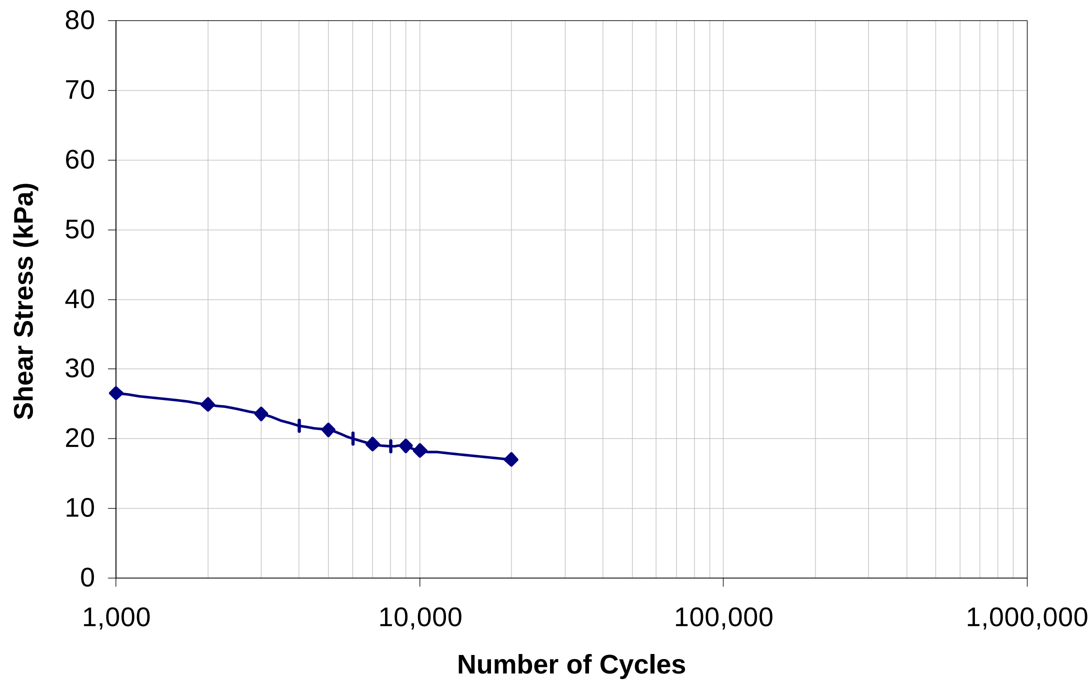

Figure C.6. Shear Stress versus Number of Loading Cycles for Specimen B1_5E.

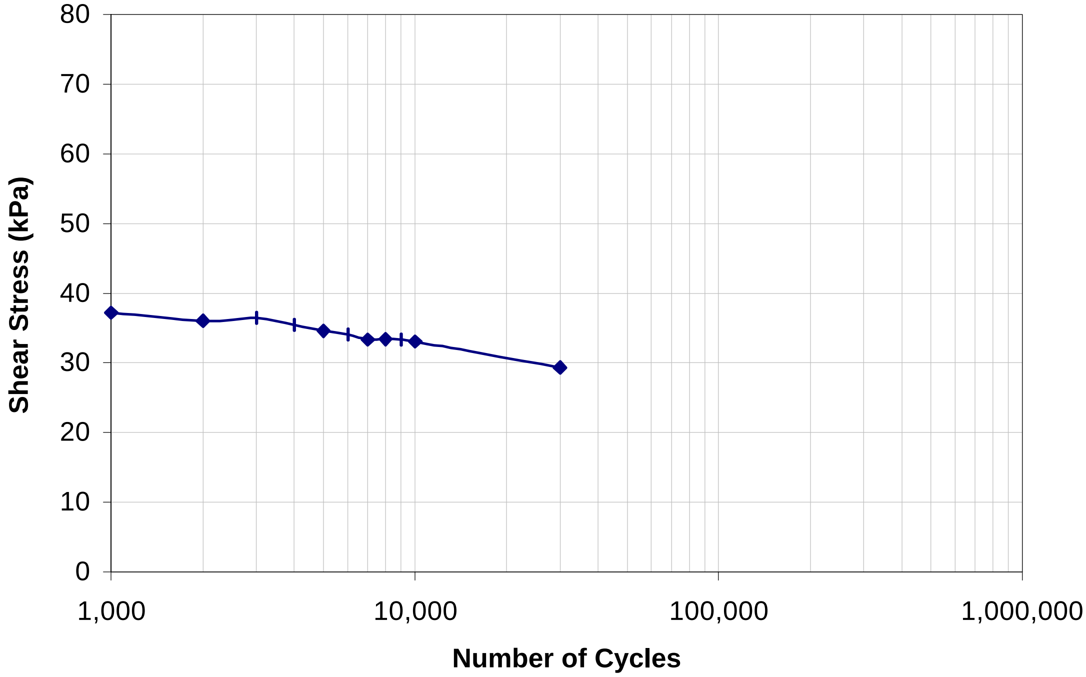

Figure C.7. Shear Stress versus Number of Loading Cycles for Specimen B1_5F.

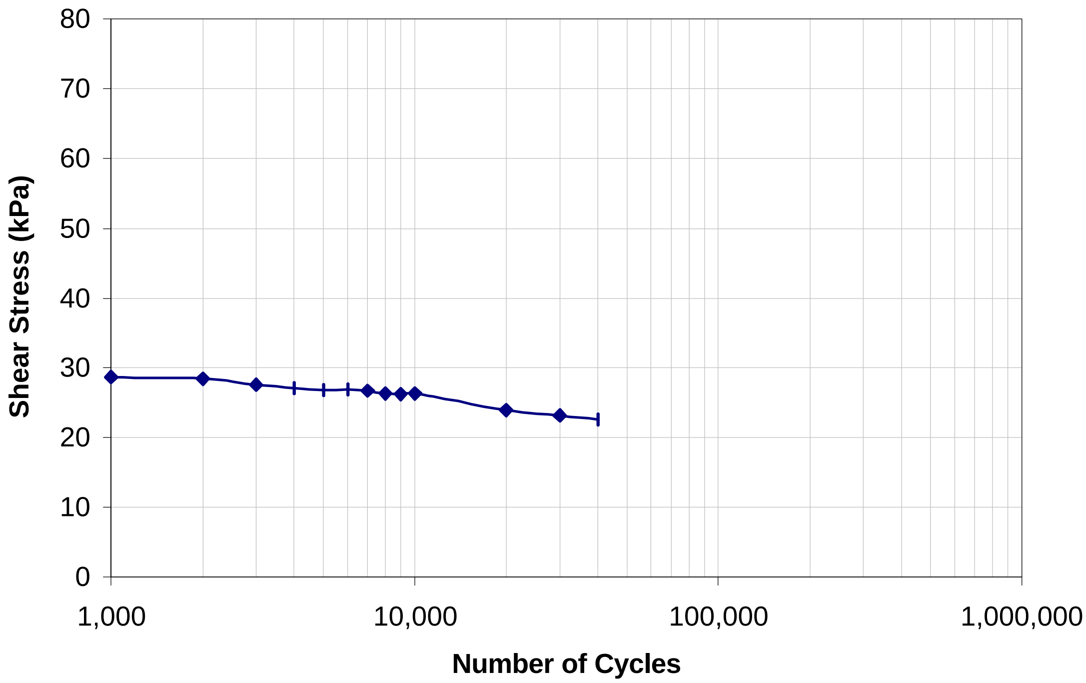

Figure C.8. Shear Stress versus Number of Loading Cycles for Specimen B1_75A2.

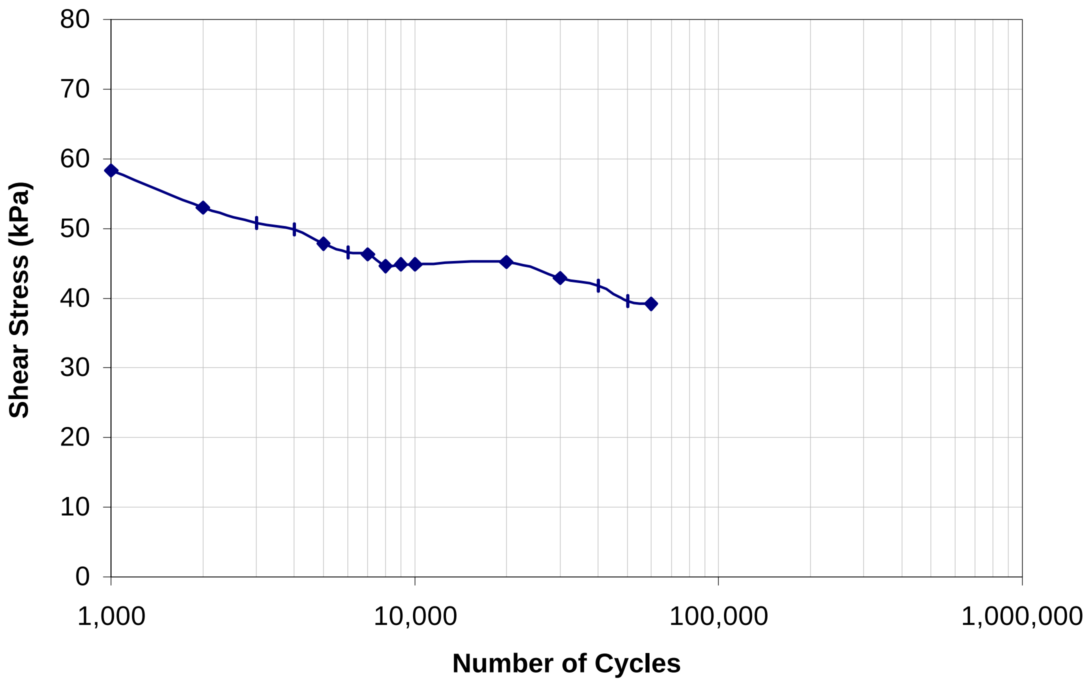

Figure C.9. Shear Stress versus Number of Loading Cycles for Specimen B1_75B.

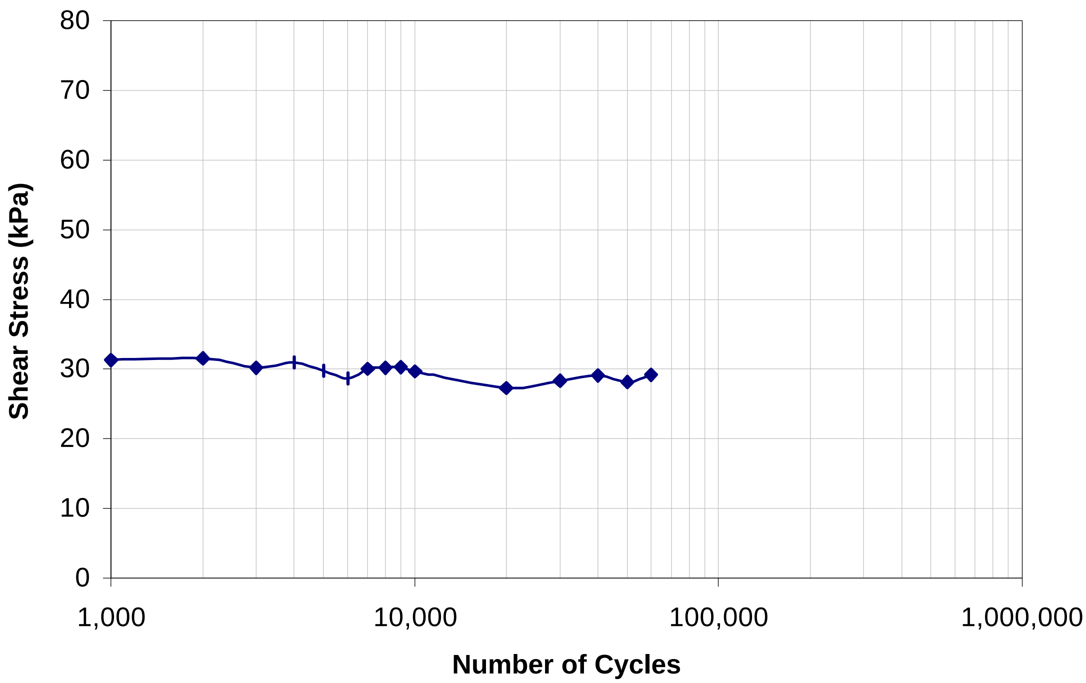

Figure C.10. Shear Stress versus Number of Loading Cycles for Specimen B1_75C.

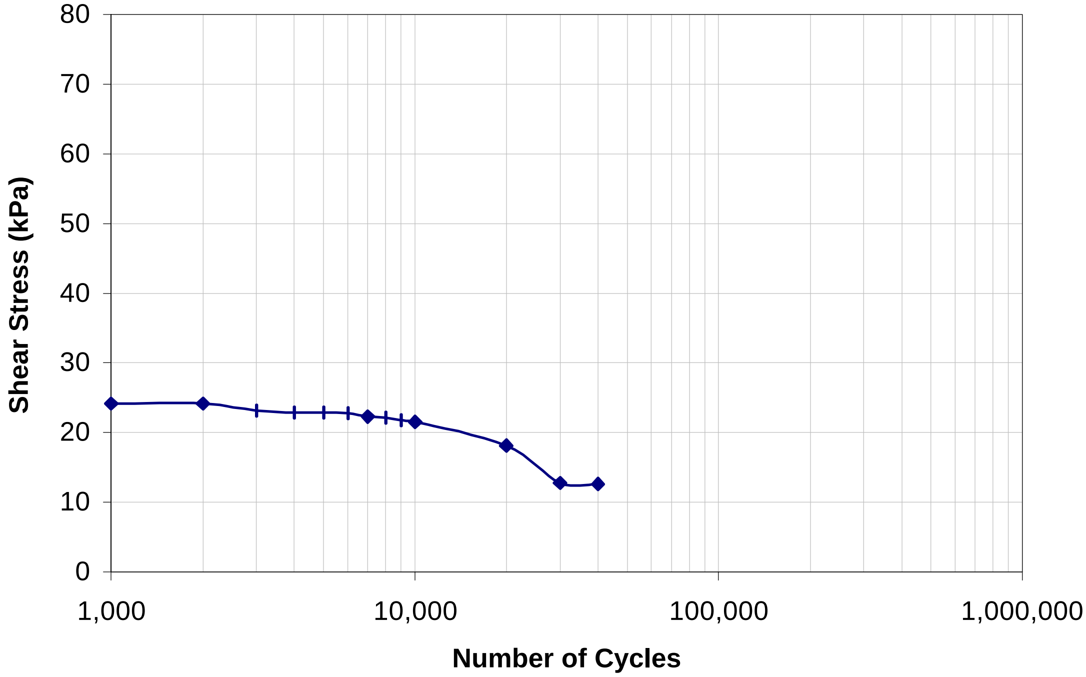

Figure C.11. Shear Stress versus Number of Loading Cycles for Specimen B1_75D.

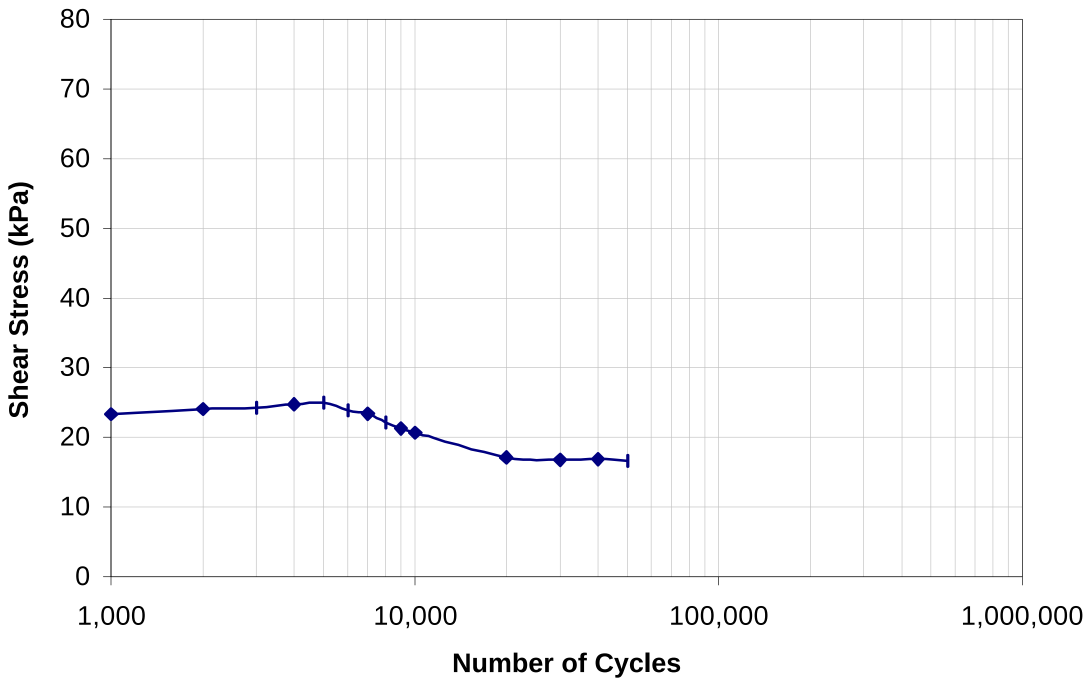

Figure C.12. Shear Stress versus Number of Loading Cycles for Specimen B1_75E.

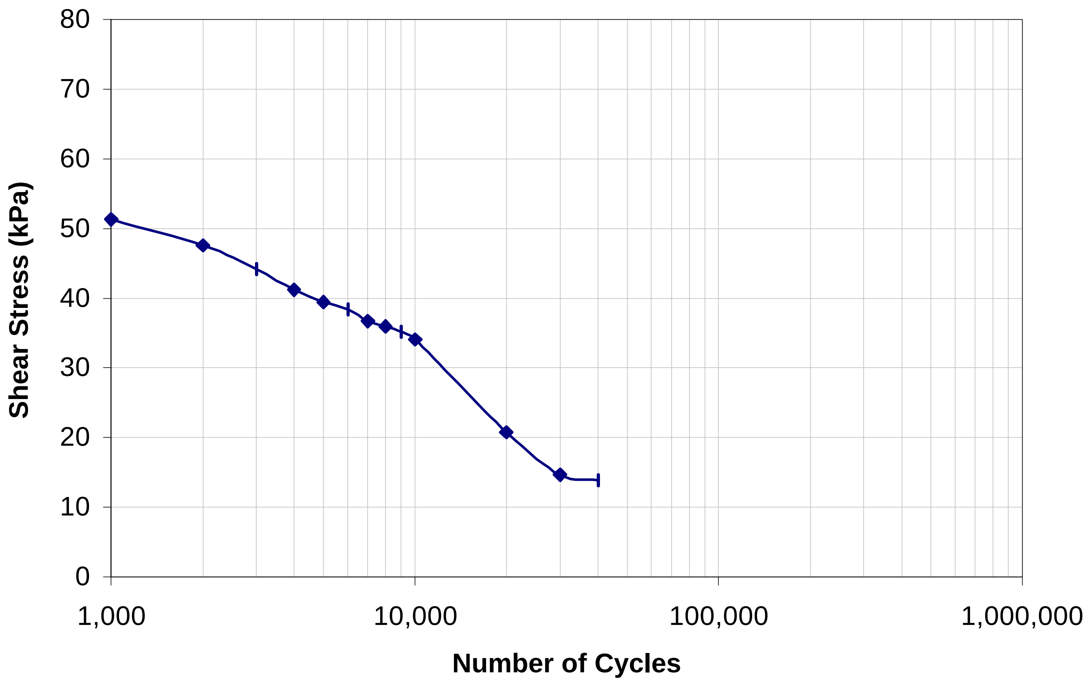

Figure C.13. Shear Stress versus Number of Loading Cycles for Specimen B2_0A.

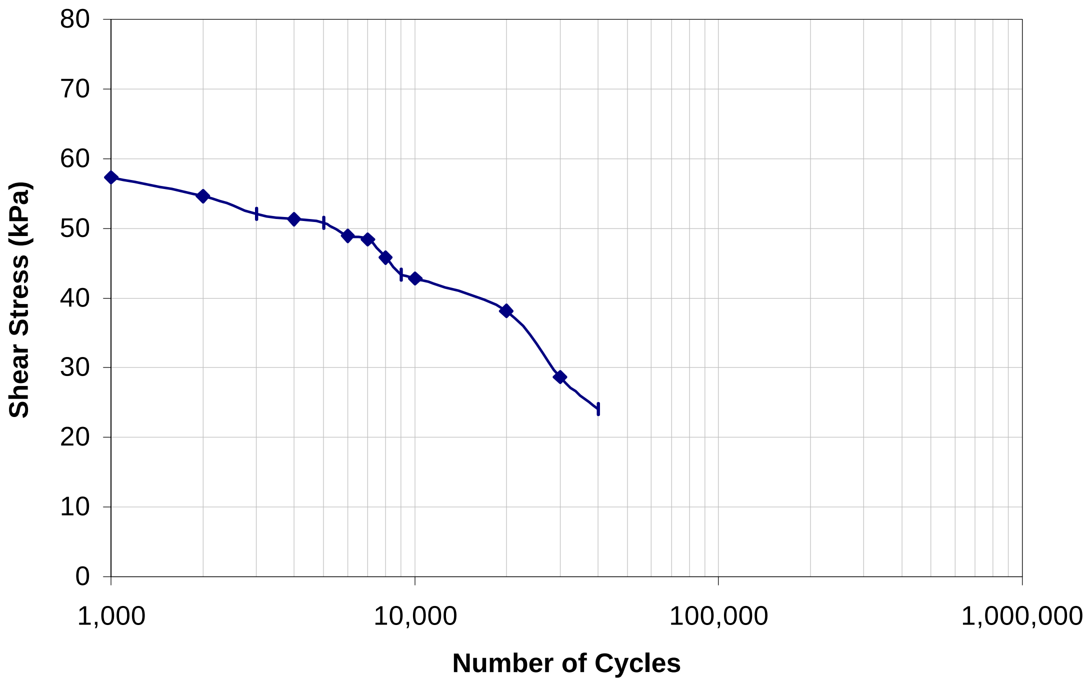

Figure C.14. Shear Stress versus Number of Loading Cycles for Specimen B2_0B.

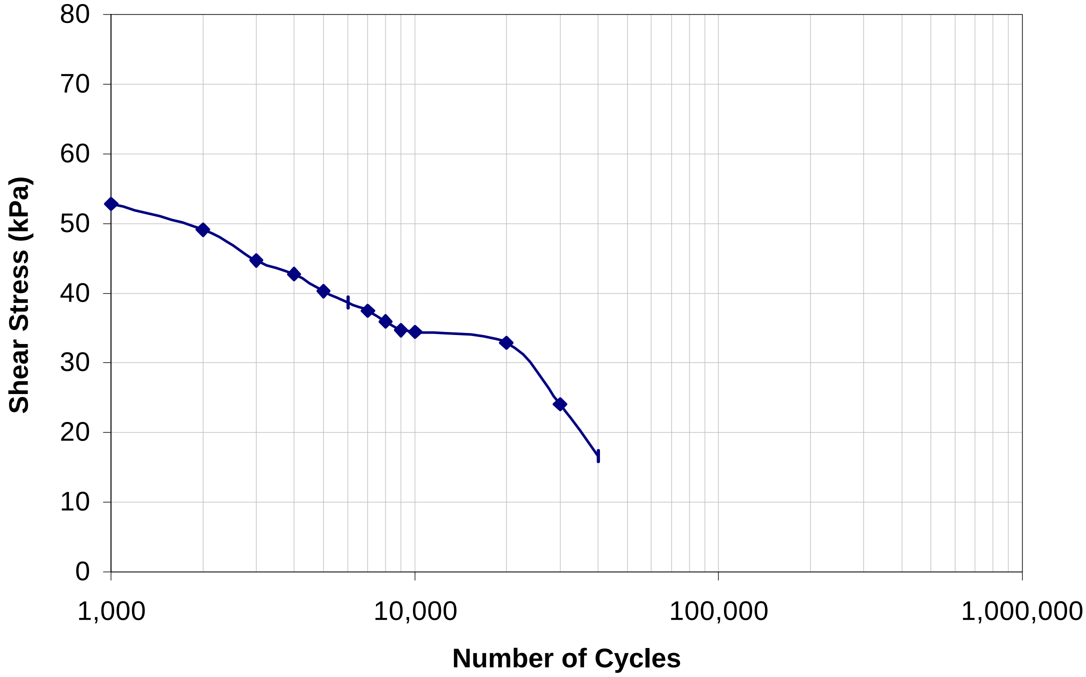

Figure C.15. Shear Stress versus Number of Loading Cycles for Specimen B2_0C.

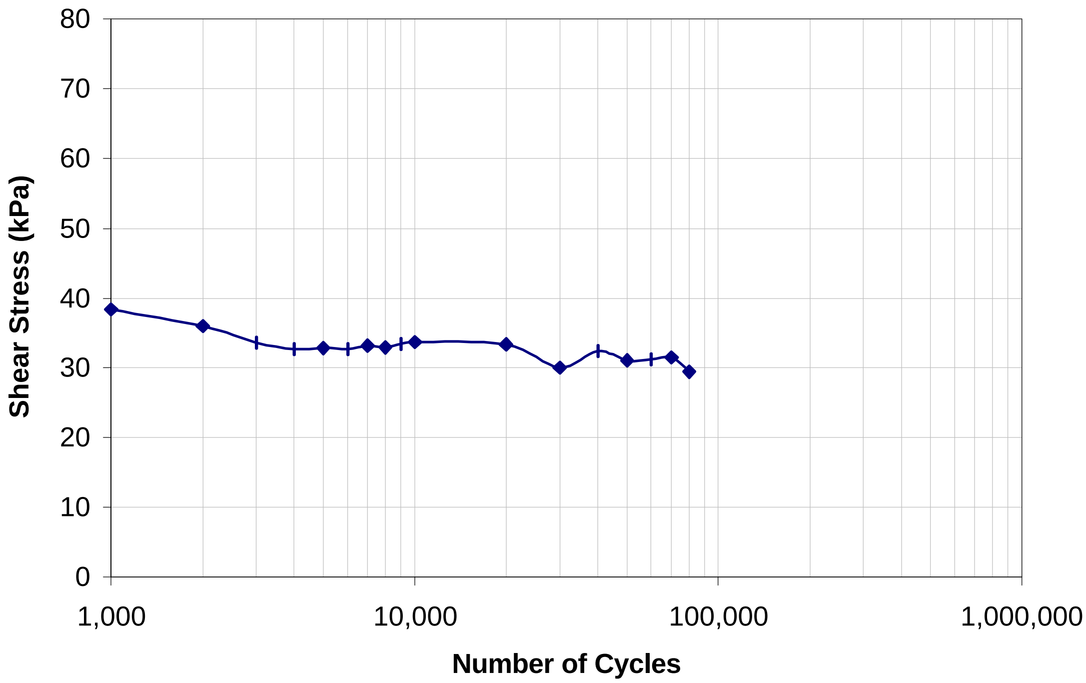

Figure C.16. Shear Stress versus Number of Loading Cycles for Specimen B2_5A.

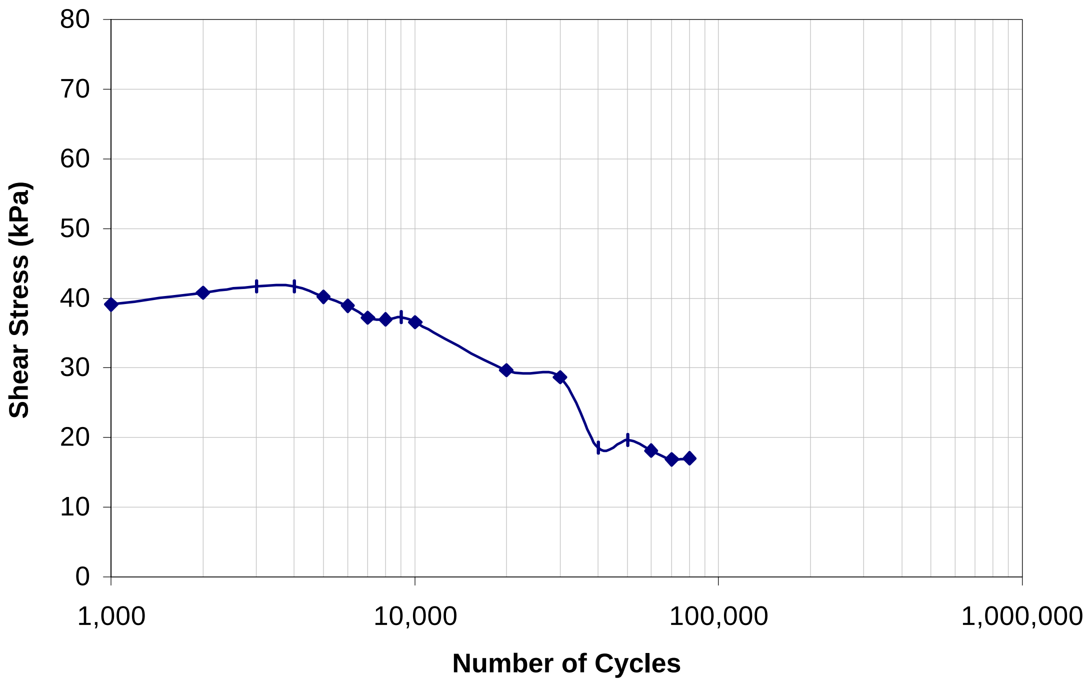

Figure C.17. Shear Stress versus Number of Loading Cycles for Specimen B2_5C.

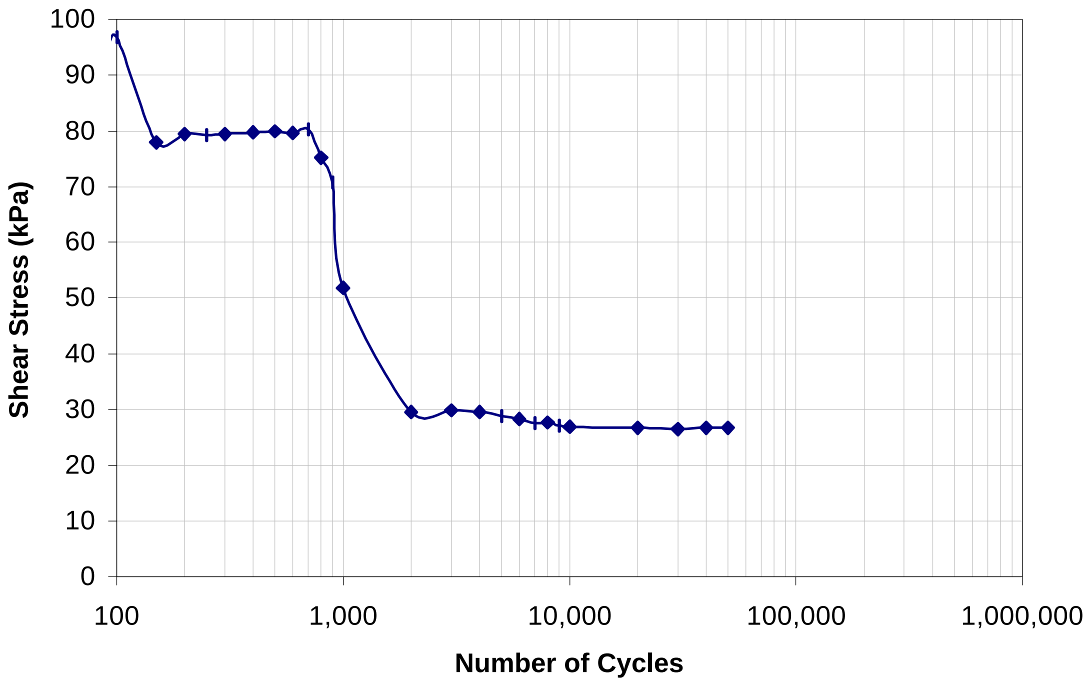

Figure C.18. Shear Stress versus Number of Loading Cycles for Specimen BC_A.

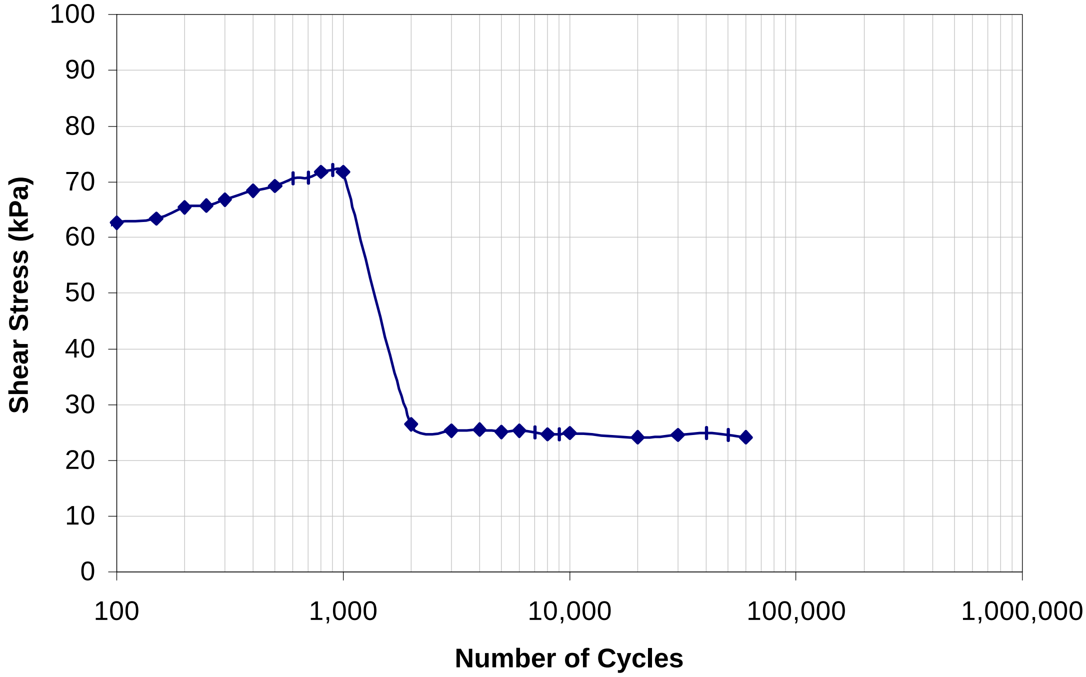

Figure C.19. Shear Stress versus Number of Loading Cycles for Specimen BC_B.

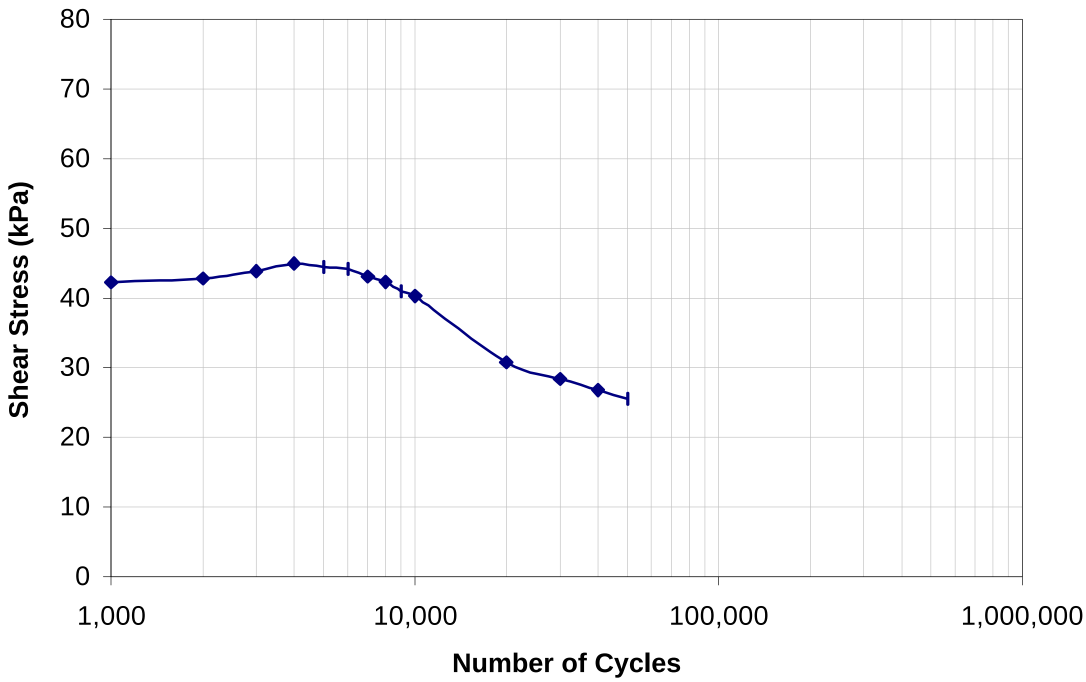

Figure C.20. Shear Stress versus Number of Loading Cycles for Specimen B2_0P1.

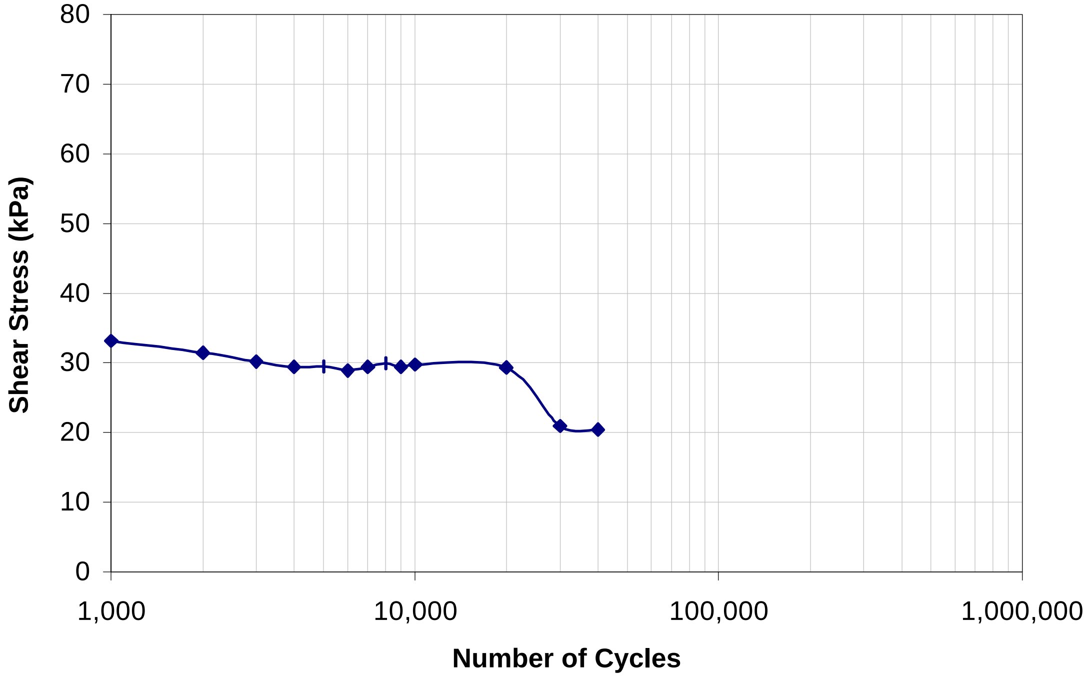

Figure C.21. Shear Stress versus Number of Loading Cycles for Specimen B2_0P2.

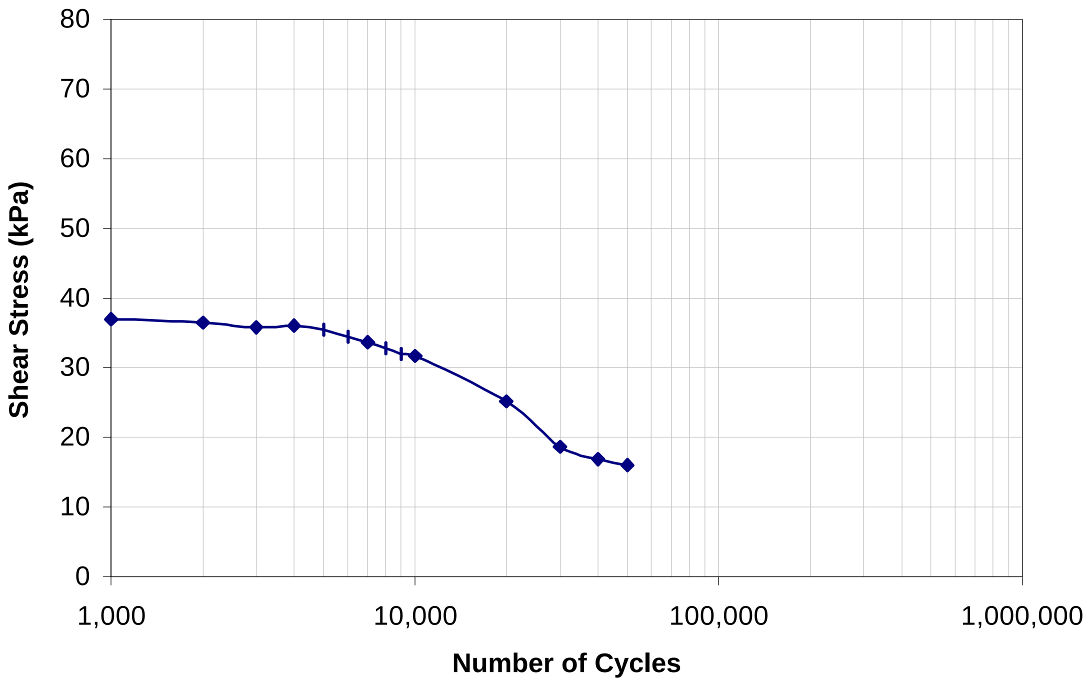

Figure C.22. Shear Stress versus Number of Loading Cycles for Specimen B2_5P1.

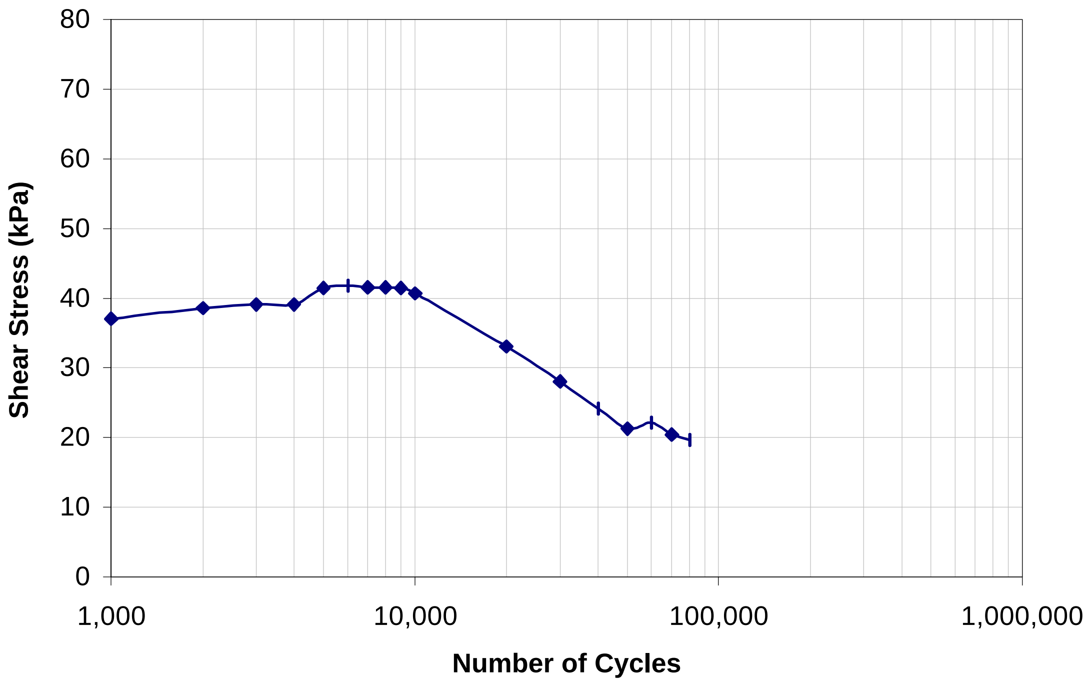

Figure C.23. Shear Stress versus Number of Loading Cycles for Specimen B2_5P2.

Figure C.24. Typical Simulated Bridge Deck Specimen before Failure.

Figure C.25. Typical Simulated Bridge Deck Specimen after Failure.

Figure C.26. Typical Control Bridge Deck Specimen before Failure.

Figure C.27. Typical Bridge Deck Control Specimen after Failure.

APPENDIX D

SIMULATED FLEXIBLE PAVEMENT SPECIMEN DATA AND PICTURES

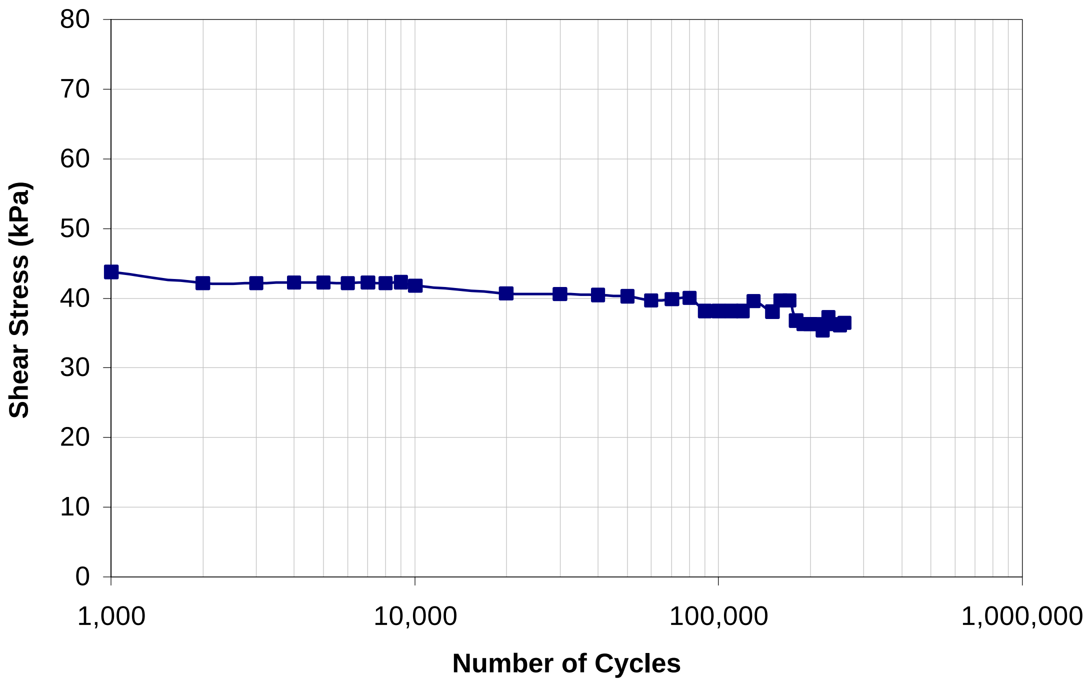

Figure D.1. Shear Stress versus Number of Loading Cycles for Specimen R1_25A.

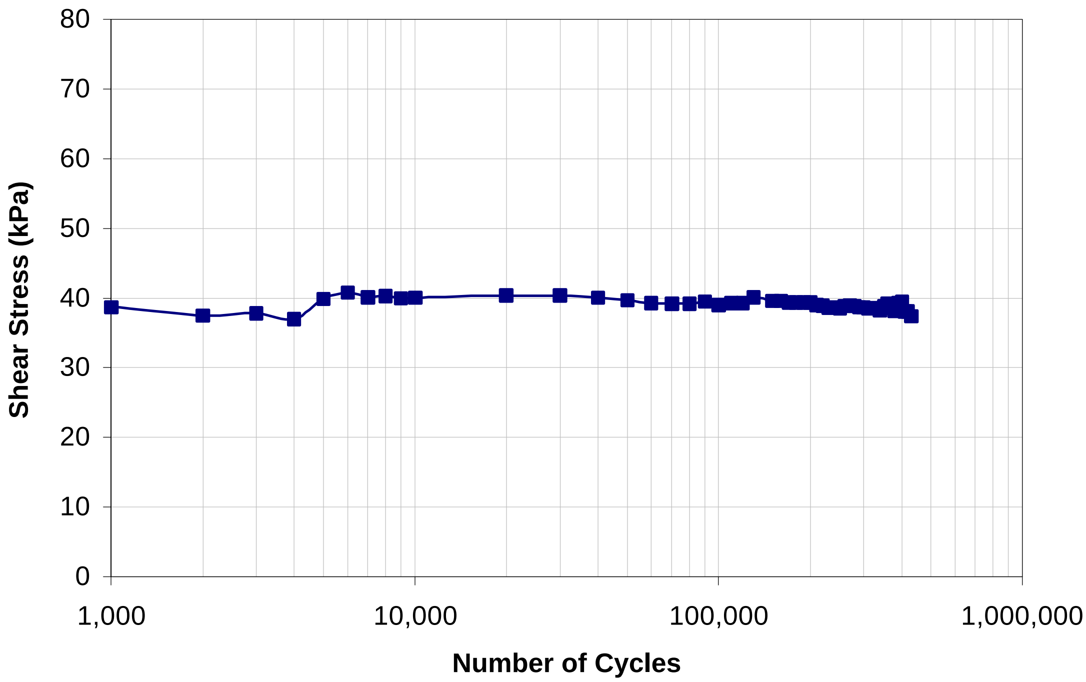

Figure D.2. Shear Stress versus Number of Loading Cycles for Specimen R1_5A.

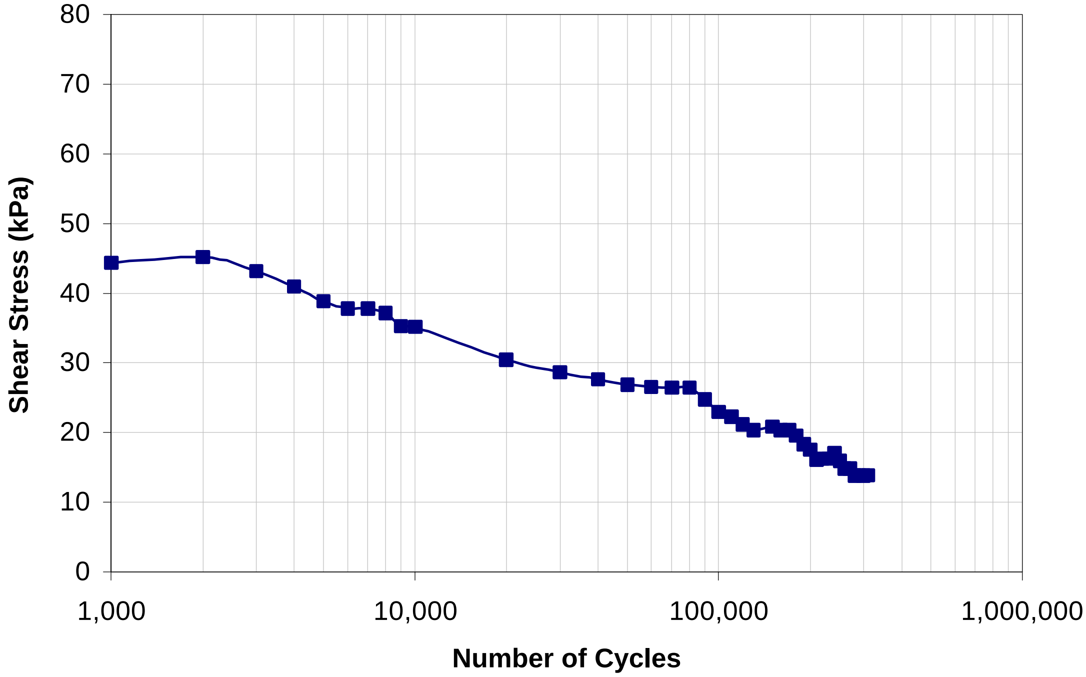

Figure D.3. Shear Stress versus Number of Loading Cycles for Specimen R1_5B.

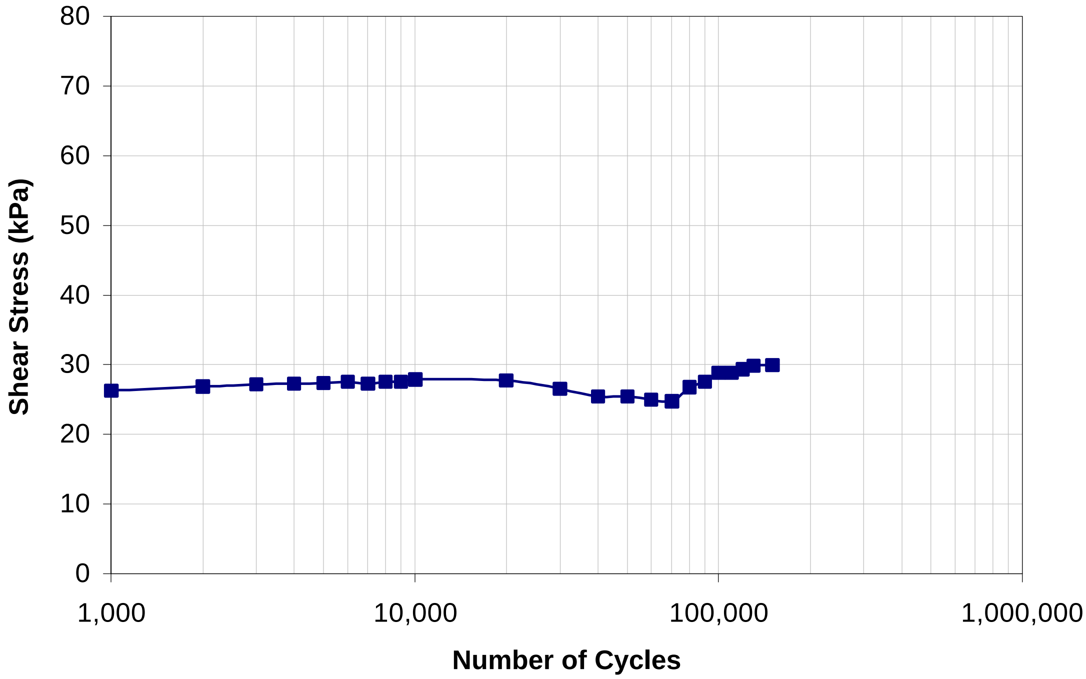

Figure D.4. Shear Stress versus Number of Loading Cycles for Specimen R1_75A.

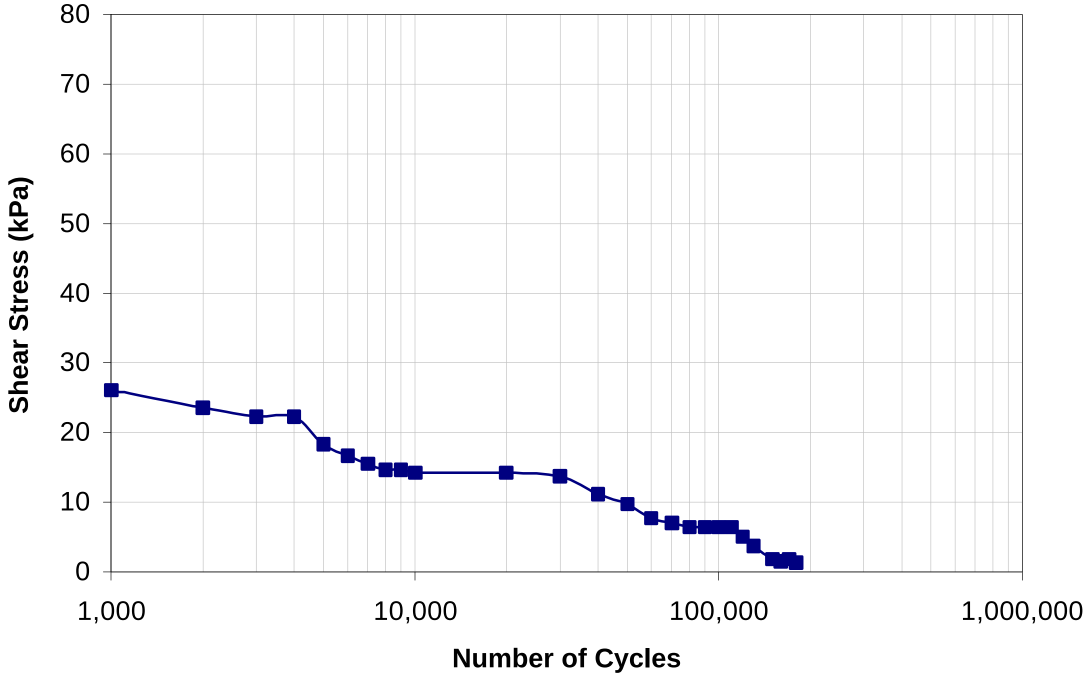

Figure D.5. Shear Stress versus Number of Loading Cycles for Specimen R2_0B.

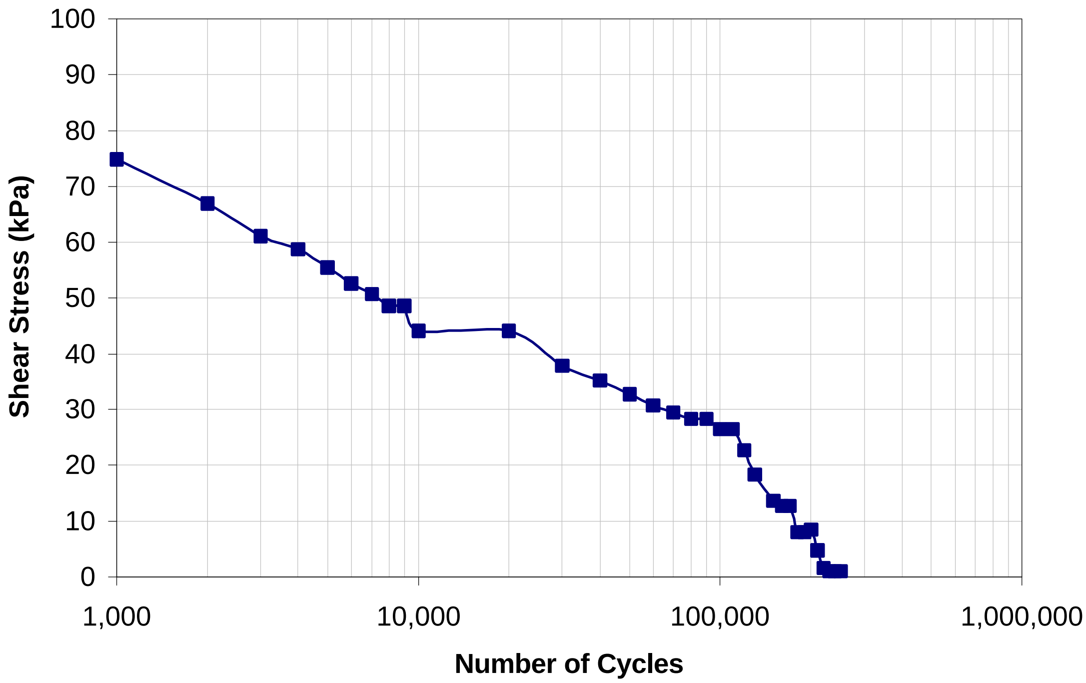

Figure D.6. Shear Stress versus Number of Loading Cycles for Specimen RC0_0A.

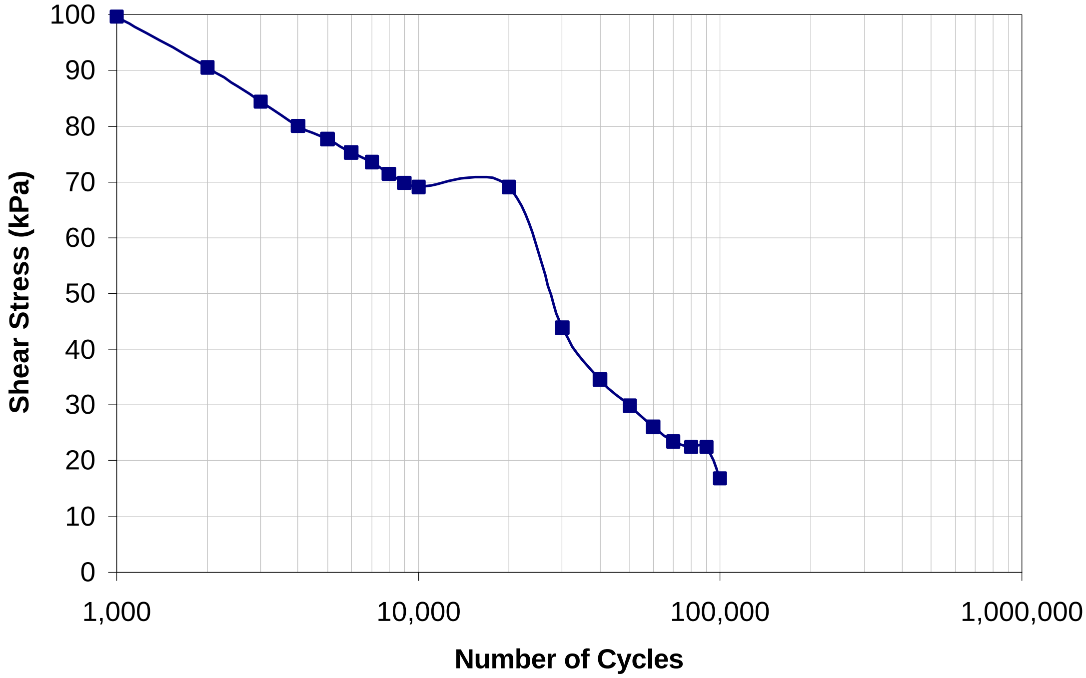

Figure D.7. Shear Stress versus Number of Loading Cycles for Specimen RC0_45A.

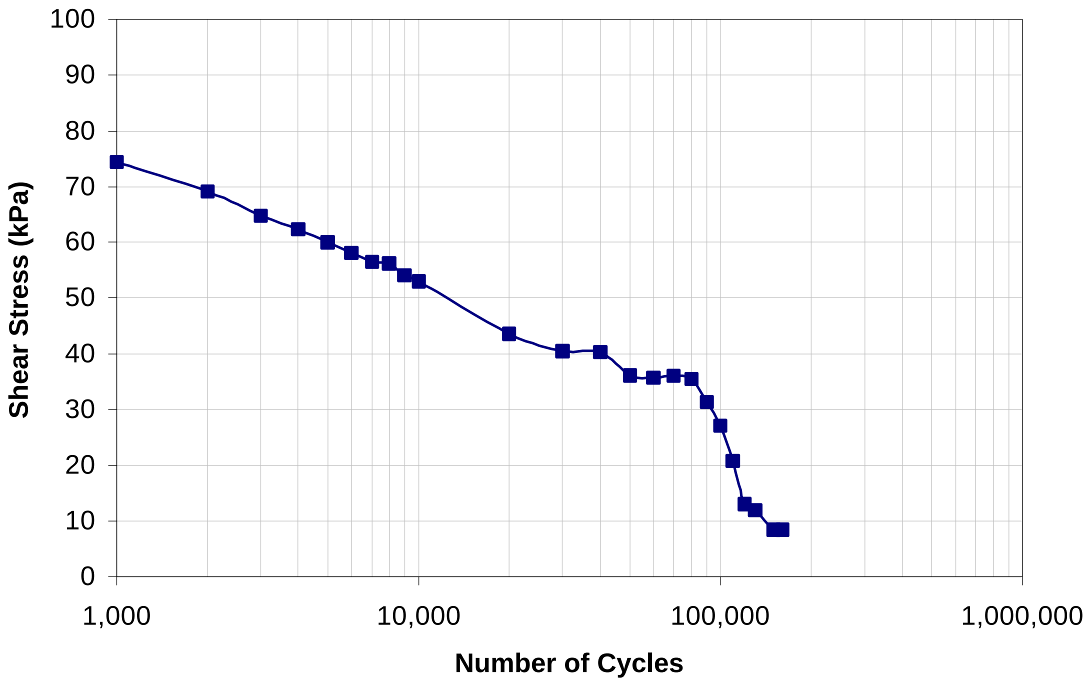

Figure D.8. Shear Stress versus Number of Loading Cycles for Specimen RC0_45B.

Figure D.9. Typical Road Specimen before Testing.

Figure D.10. Typical Road Specimen after Testing.

Figure D.11. Typical Road Control Specimen before Testing.

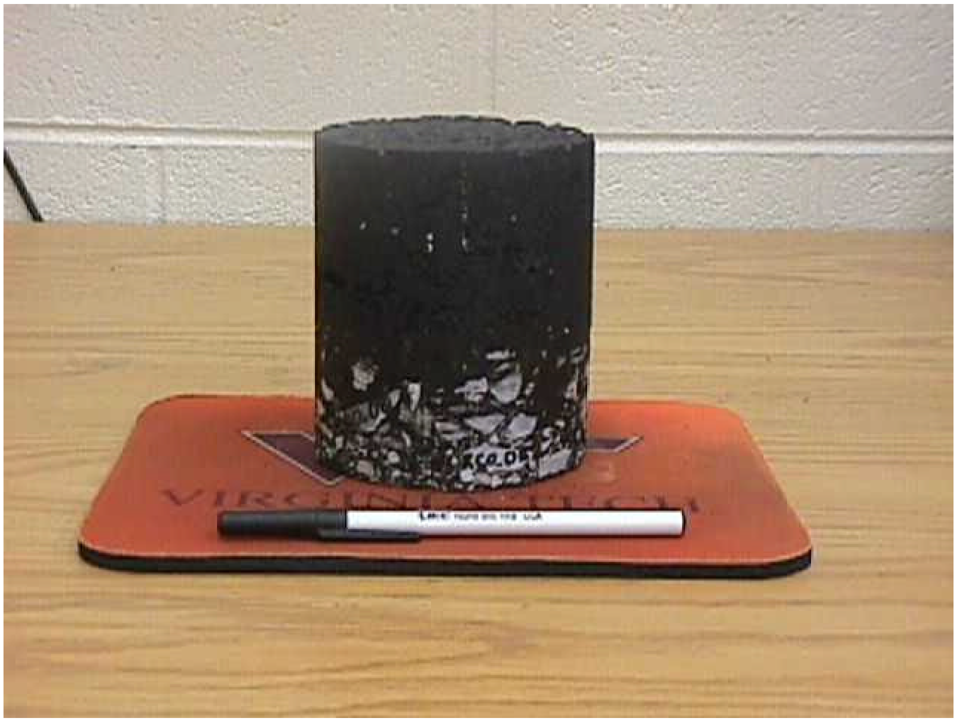

Figure D.9. Vertical View of Typical Road Control Specimen.

VITA

Erin Patricia Donovan was born Erin Patricia Walsh in Staten Island, New York on December 19, 1976. She moved to Manalapan, New Jersey in 1989 and went on to attend Manalapan High School. After graduating high school in June 1994, Erin entered the Catholic University of America in Washington, DC. She received her Bachelor of Civil Engineering degree in May 1998, graduating with the honor of Magna cum Laude.

Upon graduation, Erin went on to pursue a Masters of Science in Civil Engineering at the Virginia Polytechnic Institute and State University. She was honored as a Via Fellow and worked as both a graduate research and teaching assistant during her tenure. She expects her Masters degree in December 1999.

Erin Patricia Donovan