

RESEARCH & DEVELOPMENT

Performance of Cracking Mitigation Strategies on Cracked Flexible Pavements

Y. Richard Kim, Ph.D., P.E., F. ASCE Mohammad A. Gabr, Ph.D., P.E. Andrew Wargo, Ph.D. Shuvo Islam Dept. of Civil, Construction, & Environmental Engineering North Carolina State University

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Y. Richard Kim, Ph.D., P.E., F.ASCE Jimmy D. Clark Distinguished University Professor Campus Box 7908 Department of Civil, Construction, & Environmental Engineering North Carolina State University Raleigh, NC 27695-7908 Tel: 919-515-7758, Fax: 919-515-7908 E-mail: kim@ncsu.edu

Mohammad A. Gabr, Ph.D., P.E. Alumni Association Distinguished Undergraduate Professor

> Andrew Wargo, Ph.D. Postdoctoral Research Associate

Shuvo Islam Former Graduate Research Assistant

Department of Civil, Construction, & Environmental Engineering North Carolina State University Raleigh, NC

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project, an extensive literature review w selection, construction, and performanc have the ability to delay reflective crack a reduction in subgrade moisture and in however, is that they tend to increase pr during construction, and can lead to inc During the course of this research, a sur Division personnel to quantify their exp acquired survey information, several in evaluate the various systems' construct for these trials, and five interlayer resea (tack coat only) segment. During the co conclusions about the performance of the the literature review and the field const engineers in understanding the factors a	vas first performed. This literature review pr e of interlayer systems. In general, the litera- sing from three to five years and can help we aproved performance over the pavement life roject costs slightly, may introduce new sour- reased or new distresses in overlays in certa revey was performed that asked North Carolin periences with reflective cracking mitigation terlayer systems were selected to be used in too and performance. A flexible pavement s rch segments were placed that incorporated urse of the field construction, several issues nese materials over time may be problematic ruction, construction guidelines were develo- ind challenges that are present when utilizin	ovided insights into the availability, design, ture review found that interlayer systems do ater-proof the pavement, potentially leading to e. The downside of interlayer treatments, reces of variability into the paving project in situations. na Department of Transportation (NCDOT) interlayers. Taking into consideration this a field trial project in North Carolina to ection of US 1 in Moore County was selected three geosynthetics, a chip seal, and a control were encountered, which imply that forming c. Based on information obtained from both ped in order to aid construction personnel and g interlayer systems in a construction project.
Additionally, these materials also were performance and ultimately to develop test methods were used for this analysis beam fatigue (NBF) test on layered asp	subjected to laboratory testing in order to qu a method that can rank their suitability as re a scaled-down wheel load test known as the halt samples, and a direct shear test.	antify critical factors that affect the materials' flective crack-mitigating interlayers. Three he reflective cracking test (RCT), a notched
The RCTs demonstrated the importance pavement layers, and that pavements w condition that should be avoided in the cracking, how vertical crack growth is a interfacial bond strength and the potent the mechanisms present in these sample criteria that could be used to rank samp shear tests helped to confirm that interla field variability during construction can be used as a screening test to ensure that	e of having high interfacial bond strength be ith high relative displacements across existin field. The NBF tests provided additional inf affected by the presence of an interlayer, and ial for horizontal cracking. Ultimately, these es and provided some indication of material les or predict pavement life in the field requ ayer systems do reduce the shear strength be affect the bond quality of pavement layers. t interlayer systems meet minimum shear st	tween the interlayer system and the other ng cracks or joints represent an extreme formation about the mechanisms of reflective d further demonstrated the importance of two tests provided useful information about suitability; however, development of objective ire further testing and analysis. The direct tween pavement layers and demonstrated that The direct shear test showed the potential to rength requirements for a given application.
Due to the limited ability of the laborato performed. However, a simplified fram effectiveness of interlayer systems in th	bry tests to quantify pavement service life, n ework is presented so that engineers can obt e field. Finally, the project selection guideli	o extensive life cycle cost analysis could be ain a ballpark estimate as to the cost- ne was developed based on the information

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EXECUTIVE SUMMARY

Problems with reflective cracking in asphalt concrete (AC) overlays on cracked flexible pavements have been observed for many years in North Carolina. These cracks can severely degrade the service life of asphalt pavements by allowing water into the underlying pavement layers, weakening the pavement and accelerating deterioration of the entire pavement structure. For this reason it is desirable to investigate methods that reduce or retard reflective cracking in AC overlays.

Numerous studies have shown the advantages of overlaying flexible pavements with a system that consists of AC and an intermediate layer. This fact has led to the development of various interlayer products. However, in North Carolina, division managers have tried out these products without a systematic evaluation methodology. As a result, these investigations have not allowed reliable comparisons of the different products. Moreover, the cost effectiveness of these treatments still remains unclear. The goal of this project was to investigate the use of interlayer systems for reflective crack mitigation in North Carolina.

In order to obtain the necessary background information for this project, an extensive literature review was performed. This literature review provided insights into the availability, design, selection, construction, and performance of interlayer systems. In general, the literature review found that interlayer systems do have the ability to delay reflective cracking from three to five years and can help water-proof the pavement, potentially leading to a reduction in subgrade moisture and improved performance over the pavement life. The downside of interlayer treatments, however, is that they tend to increase project costs slightly, may introduce new sources of variability into the paving project during construction, and can lead to increased or new distresses in overlays in certain situations.

In general, flexible pavements with good support and drainage conditions, narrow existing cracks, and whose primary distresses do not involve thermal cracking, appear to be the best candidates for the use of reflective cracking mitigation interlayers and thin asphalt overlays. The ideal conditions for many interlayer systems involve the use of a leveling course and thick overlays (or multiple layers). These requirements may not be compatible with the typical treatments currently used on low to medium volume roadways in North Carolina.

During the course of this research, a survey was performed that asked North Carolina Department of Transportation (NCDOT) Division personnel to quantify their experiences with reflective cracking mitigation interlayers. Taking into consideration this acquired survey information, several interlayer systems were selected to be used in a field trial project in North Carolina to evaluate the various systems' construction and performance. A flexible pavement section of US 1 in Moore County was selected for these trials, and five interlayer research segments were placed that incorporated three geosynthetics, a chip seal, and a control (tack coat only) segment. During the course of the field construction, several issues were encountered, which imply that forming conclusions about the performance of these materials over time may be problematic. Based on information obtained from both the literature review and the field construction, construction guidelines were developed in order to aid construction personnel and engineers in understanding the factors and challenges that are present when utilizing interlayer systems in a construction project.

Additionally, these materials also were subjected to laboratory testing in order to quantify critical factors that affect the materials' performance and ultimately to develop a method that can rank their suitability as reflective crack-mitigating interlayers. Three test methods were used for this analysis: a scaled-down wheel load test known as the reflective cracking test (RCT), a notched beam fatigue (NBF) test on layered asphalt samples, and a direct shear test.

The RCTs demonstrated the importance of having high interfacial bond strength between the interlayer system and the other pavement layers, and that pavements with high relative displacements across existing cracks or joints represent an extreme condition that should be avoided in the field. The NBF tests provided additional information about the mechanisms of reflective cracking, how vertical crack growth is affected by the presence of an interlayer, and further demonstrated the importance of interfacial bond strength and the potential for horizontal cracking. Ultimately, these two tests provided useful information about the mechanisms present in these samples and provided some indication of material suitability; however, development of objective criteria that could be used to rank samples or predict pavement life in the field require further testing and analysis.

The direct shear tests helped to confirm that interlayer systems do reduce the shear strength between pavement layers and demonstrated that field variability during construction can affect the bond quality of pavement layers. These tests show the potential to be used as a screening test to ensure that interlayer systems meet minimum shear strength requirements for a given application.

Due to the limited ability of the laboratory tests to quantify pavement service life, no extensive life cycle cost analysis could be performed. However, a simplified framework is presented so that engineers can obtain a ballpark estimate as to the cost-effectiveness of interlayer systems in the field. Finally, the project selection guideline was developed based on the information obtained from the literature.

REPORT ORGANIZATION

This report is organized with the intention of reducing the length of the main body as much as possible while still covering all of the major research efforts that were conducted as a part of this project. Section 1 begins with an introduction that describes the motivation for reflective cracking research in North Carolina. Section 2 presents a brief summary of the major findings of the literature review performed at the beginning of this project. Section 3 briefly summarizes the results of the survey sent to the NCDOT Divisions. Section 4 discusses key results from the field trial segments. Section 5 summarizes the laboratory testing of the interlayer systems utilized during this research and discusses important findings. Section 6 presents a simplified cost benefit analysis framework. Section 7 briefly summarizes the efforts related to the field construction guidelines and recommendations for project selection criteria. Practical implications of the

results from this study are discussed in Section 8. Section 9 presents conclusions and recommendations.

In order to keep the main body of the report as short as possible and to also include all information that may be of interest to DOT personnel or researchers in the future, several supplementary appendices are included at the end of the report. Appendix I provides the full literature review, as performed at the beginning of this research. Appendix II presents a detailed summary of the responses to the NCDOT survey as well as all individual response forms. Appendix III provides details about the test section layout to aid in locating these segments in the future by NCDOT personnel. Appendix IV presents additional information about the field trial segments that should be considered before making determinations about the performance of any specific interlayer system in the project. Appendix V presents the distress surveys as collected by the North Carolina State University researchers; this information includes initial crack maps and post construction distress monitoring. Appendix VI describes the details of the laboratory testing, including materials, the development of the test methods, sample fabrication, and a description of the data acquisition methods and investigations into their validity. Appendix VII presents the full version of the construction guidelines. Appendix VIII presents important tables from the literature regarding project selection criteria for interlayer systems. Appendix IX describes the investigation of failure criteria within the NBF test protocol. Lastly, Appendix X describes the potential use of the shear test to develop fundamentally sound requirements for determining the suitability of different interlayer products to meet minimum shear strength requirements, depending on their intended application.

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1. INTRODUCTION

The North Carolina Department of Transportation (NCDOT) has reported many instances of reflective cracking of asphalt concrete (AC) overlays on cracked flexible pavements. Such cracks generally appear soon after the placement of the AC overlay and, if untreated, compromise ride quality and reduce the service life of the pavement. One solution to this problem is the complete repair of the cracks in the existing pavement; however, this option is costly and time intensive. Usually, overlays are placed on pavements that have existing cracks, and the project budget may not allow for repair of all the cracked areas. It is desirable, then, for the NCDOT to find or develop a product or system that can reduce reflective cracking without having to rehabilitate the entire pavement structure.

The severity and rate of reflective cracking is highly dependent on many factors, including: the tensile strength and fracture toughness of the AC overlay, the severity of existing cracks, degradation of the base or subgrade materials, and traffic loading (Zhou et al. 2005, Cleveland et al. 2002). Under particular conditions, each of these factors contributes differently to the reflective cracking process.

Methods that are commonly used to retard reflective cracking attempt either to decrease or redistribute the stress at the bottom of the overlay or increase the tensile strength of the AC (Aldea and Darling 2004). Increasing the strength of the AC layer can be accomplished in a number of ways; however, care must be taken not to compromise the pavement's fatigue resistance. Reducing the stress can be accomplished by increasing the layer thickness. However, in many situations this option is not viable due to thickness constraints and costs. Many different methods have been developed to redistribute the tensile stress, and each method has been applied with varying degrees of success (Aldea and Darling 2004, Al-Qadi et al. 2003, Steen 2004, Francken 2004, Shuler and Harmelink 2004, Davis 2004). Lack of success with any chosen method is due primarily to the lack of a basic understanding of the performance aspects of the whole system, including the supporting sublayers, and incorrect application procedures.

Methods that reduce the stress at the bottom of the AC layer generally are preferred because typically they are less costly than methods that increase the strength of the AC layer. With this observation in mind, other researchers have suggested the use of a well-defined system for overlay projects (Francken 2004), the most important component of which is an interlayer subsystem. Two distinct strategies for the construction of this interlayer are available. The first is to install a soft stress-absorbing layer. When this layer consists of a specific composition of asphalt rubber and aggregate chips, it is referred to as a *stress-absorbing membrane interlayer* (SAMI); however, other such layers have incorporated a highly fatigue-resistant sand asphalt course for the same purpose. The second strategy is aimed at adding tensile reinforcement to the AC using either steel netting or a geosynthetic mesh (Aldea and Darling 2004).

This project examines the performance of these techniques to mitigate reflective cracking in cracked flexible pavements under North Carolina conditions. The chip seal surface treatment and use of geosynthetic fabric were of specific interest. The information gathered in this research was

used to formulate construction guidelines for the NCDOT that can be used by maintenance crews across the state for AC overlay work projects.

The primary objectives of the research were:

- to identify or develop a technique or system that reduces reflective cracking in AC overlays on cracked flexible pavements without having to remove the entire pavement structure, and
- to develop construction guidelines for the interlayer system using the laboratory findings and experience gathered from the field study.

2. LITERATURE REVIEW

As pavement systems age, distresses such as cracking, rutting, raveling, and polishing of the surface aggregate impede the pavement's ability to carry the daily traffic demand safely, comfortably, and effectively. Each year, highway agencies spend billions of dollars on pavement repair and rehabilitation in order to keep these roadways in an acceptable condition for the traveling public. A commonly cited statistic is that 94 percent of the over 2.27 million miles of roads throughout the United States are surfaced with AC. Therefore, the maintenance and rehabilitation of these roads consume a significant portion of highway agencies' transportation budgets.

One common form of rehabilitation for these roadways is the use of thin AC overlays. These overlays usually consist of one or two courses placed on an existing pavement that may or may not have been cold-milled prior to the placement of the overlay. These treatments are widely used because of their ability to provide a new wearing surface while still taking advantage of the remaining fatigue life and load-carrying capacity of the existing pavement. However, in many cases, stress concentrations due to cracks in the existing pavement cause the formation and rapid propagation of cracks in the overlay. Because these cracks exhibit the same pattern as those in the underlying pavement, this phenomenon is known as *reflective cracking*.

The major problem with reflective cracks is that they allow water to enter the pavement structure. Their prevalence in the overlay thus can be a significant contributing factor to the further deterioration of the overall pavement structure. For this reason, the topic of reflective cracking mitigation in AC overlays has been researched extensively over the last several decades. Many studies and construction projects have been undertaken that show varying degrees of success. A comprehensive literature review has been performed as part of this research project in order to obtain insights into the causes of reflective cracking, potential strategies for its mitigation, and details of the design and placement of crack mitigation systems. A brief summary of this literature review is presented here; the full literature review can be found in Appendix I.

2.1 Reflective Cracking Causes and Contributing Factors

Because traffic loading is of primary importance to the adequate design of pavement systems, it is no surprise that it is also a major contributor to the phenomenon of reflective cracking. Lytton (1989) described three stress concentration pulses that occur at the crack tip as a crack

propagates through the overlay. Figure 2-1 illustrates that as a wheel moves across a crack location, the overlay experiences two maximum shear stress pulses at Points A and C and one maximum bending stress pulse at Point B. The intensities of these stress concentrations are affected by the material properties of the pavement layers, the maximum deflection experienced at the crack location, and the load transfer across the crack.



POSITION OF WHEEL LOAD

Figure 2-1. Shear and bending stresses induced at a crack caused by a moving wheel load (Lytton 1989).

Heavy loads and less resilient pavement structures can increase the amount of strain at the crack location, which in turn causes a corresponding increase in the rate of the crack propagation. Various studies and reports show that high load transfer efficiency across an existing joint or crack reduces reflective cracking (Hughes 1975, Mascunana 1981, McGhee 1983, Maurer 1989, Barksdale 1991, Mukhtar 1996, Maxim 1997, de Bondt 1999, Carmichael 1999, Bischoff 2007). For flexible pavement systems, this load transfer is achieved by the interlocking of the aggregate across the crack. At wide crack locations, however, no aggregate interlock is possible and no load transfer can occur. De Bondt (1999) demonstrated that new cracks in asphalt pavements have the ability to transfer load, but over time, this capacity diminishes.

Another major factor that affects the performance of pavements is the environmental conditions under which the pavement is placed. Temperature, annual rainfall, subgrade modulus, and drainage are all important considerations when determining the long-term performance of a pavement structure. Although subgrade and drainage effects can be minimized by proper design of the pavement structure, the effects of temperature and annual rainfall are less controllable. In general, colder climates that see large thermal effects tend to experience more reflective cracking and have less success with the placement of reflective crack-mitigating interlayers than warmer climates (Ahlrich 1986, Lytton 1989, Barksdale 1991, Epps 1994, Amini 2005, and Shatnawi 2008).

2.2 Methods of Crack Mitigation

Given the complex nature of the reflective cracking phenomenon, several different approaches to the problem have been attempted over the years. Because the intent of this study is to concentrate on reflective cracking mitigation for overlays on low to medium volume flexible pavements, only those treatments that are applicable for such applications are discussed in this report. In this regard, the placement of a thick overlay, milling and filling, and the use of interlayers are the most popular methods to reduce reflective cracking.

2.2.1 Conventional Asphalt Paving Practices

Thick overlays: Increasing the thickness of the overlay can decrease the growth rate of reflective cracks that reach the surface in two ways. First, because reflective cracks typically are assumed to propagate approximately 1 inch to 1.5 inches per year, it is reasonable to expect that it will take longer for cracks to penetrate the full depth of thick pavements than thin pavements (Gulen 2000, Makowski 2005). Furthermore, thick overlays can help reduce the overall stress that is experienced in the overlay and can slow crack initiation and propagation. However, thick overlays are not always an option due to cost considerations or concerns about significantly altering the elevation of the roadway.

Milling and filling: Cold milling prior to the placement of the overlay may eliminate some severe surface cracks. In addition, cold milling can be used to allow the placement of thick layers of new AC without drastically altering the roadway profile. However, if cracks in the existing pavement are full-depth cracks, cold milling may do little to address the problem of reflective cracking.

2.2.2 Interlayers

In addition to increasing project costs, the use of thick overlays and cold milling does nothing to address the stress concentrations that exist at the bottom of the asphalt overlay near the cracks in the existing pavement. The desire to construct a layer specifically designed to handle such stress has therefore been a topic of interest for engineers for decades. As early as the 1920s, experiments with cotton fabric interlayers were performed in South Carolina (Beckham 1935). Since then, numerous studies and paving projects have been undertaken to investigate the use of interlayers, with varying degrees of success. The reason for the continued interest in interlayer systems is that, in terms of the mechanics of the materials, interlayers theoretically can reduce the occurrence of reflective cracking in AC overlays. Reflective crack mitigation is achieved either by reinforcing the pavement to migrate into the new AC overlay (Lytton 1989). An additional benefit of many interlayer systems is their ability to reduce the infiltration of water into the pavement structure (Mascunana 1981, Smith 1984, Lytton 1989, Barnhart 1989, Buttlar 1999, Missouri DOT 2001, Cleveland 2002, Blankenship 2004).

Because many different products have been used as interlayers, general categories have been developed to describe them. Of these, geosynthetics and stress-absorbing interlayers are by far the most commonly used ones with thin asphalt overlays.

Geosynthetics: One of the most popular types of products used in the mitigation of reflective cracking of asphalt overlays is geosynthetics. These products are manufactured planar materials (frequently made of polymer or fiberglass) and typically are shipped to the job site in rolls. Typical types of geosynthetics used for reflective crack mitigation are geogrids, geotextiles (including paving fabrics and paving mats), and geocomposites. These products can provide combinations of reinforcement, stress relief, and waterproofing to pavement systems. These benefits and their effectiveness are highly dependent on the materials and construction of each individual product.

Geogrids: These products are composed of stiff materials (typically polymer or fiberglass, although steel grids can be considered part of this category as well) with an open structure (at least half-inch apertures) whose primary function is reinforcement. Because these products must be placed flat and taut, these treatments frequently require the use of a leveling course prior to their placement (Shatnawi 2008).

Geotextiles: This category of products is diverse and includes both stiff products (paving mats) used to provide some level of reinforcement and waterproofing ability and flexible products (paving fabrics) that absorb large quantities of asphalt binder in order to create a pliant stress-absorbing layer. The need to place a leveling course with these materials depends highly on the particular product being used, but generally will improve performance.

Geocomposites: Some interlayer products consist of different types of geosynthetics that are bonded together to provide multiple functions, for example, the attachment of a grid to a paving fabric. These products may come in large rolls that are intended to cover the entire pavement or they may be strip treatments intended to be placed only over well-defined joints and cracks. These products may be more expensive than geotextiles or geogrids alone (Dondi 2000). Like geogrids, geocomposites may require the use of a leveling course in order to function properly (Chen 1982, Sarsam 1982).

Stress-absorbing layers: Another means of mitigating reflective cracks is the use of stressabsorbing interlayers. Unlike geosynthetics that are manufactured and brought to the project site in rolls, stress-absorbing layers typically are constructed in place. These layers include bituminous surface treatments (BSTs), such as chip seals and asphalt rubber seals, and finegraded, high binder content AC layers.

Bituminous surface treatments: The potential benefit of using BSTs as interlayers has been recognized for some time. Due to their high asphalt content, treatments such as chip seals and asphalt rubber seals help to absorb the stress that drives reflective cracking and have the ability to reduce the amount of water that infiltrates through the pavement structure (Bandaru 2010). Proprietary systems, such as fiberglass-reinforced stress-absorbing layers, are also included in this group. The placement of a leveling course prior to the application of a BST interlayer can

help increase the effectiveness of these treatments by preventing the loss of asphalt binder into the existing cracks (Epps 1994).

Fine-graded AC layers: These stress-absorbing layers consist of fine-graded AC with high asphalt binder content. The high binder content of these layers helps them to resist fracturing due to reflective cracking and may provide some level of resistance to water infiltration (Sefass 2000).

2.3 Performance of Interlayers

The extensive use of geosynthetics has resulted in a large number of performance case studies. Although each case is different when considering existing pavement build-up, the distress level of the existing pavement prior to treatment, traffic levels, environmental conditions, pavement treatment used, subgrade modulus, site drainage, and specific materials used, some general observations regarding performance nonetheless can be made. Given favorable conditions and adequate construction practices, geosynthetic interlayers have the ability to delay reflective cracking for two to five years, but usually cannot prevent it altogether (Hughes 1977, McGhee 1983, Barnhart 1984, Ahlrich 1986, Lorenz 1987, Button 1989, Barksdale 1991, Buttlar 1999, Vespa 2005, Bush 2007). Also, some reports suggest that geosynthetic interlayers can reduce the severity of reflective cracks when such cracks eventually propagate to the surface (McGhee 1983, Bush 2007). Although fabrics comprise a considerable portion of the available research focus, geogrids also have been shown to improve resistance to reflective cracking in laboratory tests (Khodaii 2009).

The performance of the various types of stress-absorbing layers is very similar to the performance of geosynthetics. In general, these systems show an ability to help reduce reflective cracking for three to five years (Vallergra 1980, Ahlrich 1986, Peters 1987, Zaniewski 1988, Estakhri 1990, Barksdale 1991, Epps 1994, Estakhri 1994, Missouri DOT 2001, Makowski 2005, Bischoff 2007, Chowdhury 2007, Bennert 2009, Elseifi 2011). These treatments may also reduce crack severity as well as help to waterproof the pavement, even when reflective cracking cannot be prevented (Chen 1982, Ahlrich 1986, Epps 1994).

2.4 Potential Problems with Interlayer Use

Despite their benefits, the use of interlayers introduces several challenges to paving projects. First and foremost is ensuring adequate interfacial strength between the interlayers and the other pavement layers. Proper bonding of the layers is essential for preventing failure of the pavement system due to slippage or delamination. As such, tack coat application rates must be high enough for the interlayer system to function adequately, but not too high, as excessive amounts of asphalt binder in interlayers may lead to bleeding, slipping, or rutting of the overlay (Dykes 1980, Sarsam 1982, Barksdale 1991, Epps 1994, Vespa 2005, Fyfe 2010, Roque 2012, Solaimanian 2013). In the case of fine graded AC layers, rutting of the fine graded layer itself may become a problem. Even in cases where interlayers are placed properly, properties such as the shear strength of the layers may be reduced (Smith 1984, Brown 2001, Vismara 2012). As such, the use of various types of interlayers may be undesirable in areas where high shear stress levels are present (such as slopes, sharp curves, or intersections). The constructability of interlayers also can be a major concern for agencies that have not used them in the past. In areas where interlayer placement is not routine, the additional steps involved for the placement of the

interlayer may complicate construction coordination and contractual arrangements and may increase construction time. Furthermore, having construction personnel who are unfamiliar with the placement of interlayers may lead to incorrect placement of the interlayer or damage to the interlayer during other construction steps, which would result in decreased performance (Barksdale 1991). Lastly, the increase in cost due to the placement of an interlayer should be considered and weighed against any enhanced performance that is needed to justify that cost.

2.5 Conclusions from the Literature

The stated goal of this research is to investigate the use of interlayers to mitigate reflective cracking in thin, single-course AC overlays on low to medium volume flexible pavements throughout North Carolina. From the literature review, the following conclusions can be drawn:

- 1) The reduction of reflective cracking in asphalt overlays is a complex and difficult challenge that has been investigated for decades. Although many reports show that various interlayer types can reduce the rate and severity of reflective cracking, often these treatments are unable to prevent it altogether.
- 2) Flexible pavements with good support and drainage conditions, narrow existing cracks, and whose primary distresses do not involve thermal cracking appear to be the best candidates for the use of reflective crack-mitigating interlayers and thin asphalt overlays.
- 3) The ideal conditions for many interlayer systems involve the use of a leveling course and thick overlays (or multiple layers). These requirements may not be compatible with the typical treatments used on low to medium volume roadways in North Carolina.
- 4) Proper selection and placement of any interlayer system are critical factors in ensuring good performance of the overlay. Care must be taken to ensure that the candidate pavements are suitable for the use of particular interlayer systems or products and that good construction practices are followed when placing these systems.

3. SURVEY

In addition to the literature review, a survey questionnaire was developed and sent to the NCDOT Divisions in order to gain insight into the Divisions' experiences with reflective cracking mitigation on single-course overlays (1.25 in. -1.5 in.) for medium volume two-lane flexible pavements. The survey was sent out in October 2011 and the Divisions were given several months to reply. In total, eight Divisions submitted responses to the survey.

The Divisions reported that cracking (including fatigue, longitudinal, reflective, thermal, and edge cracking) was the most frequent type of distress encountered for pavements that had received thin single-course overlays. After the placement of a single-course overlay, the Divisions reported that reflective cracks usually appeared within the first two years. The most common types of reflective crack-mitigating interlayers used for these types of projects are:

- BSTs: Generally viewed as moderately effective (7 responses)
- Crack seals: Generally viewed as moderately to slightly effective (5 responses)
- Geosynthetics: Generally viewed as slightly effective (3 responses)
- Fine-graded AC layers: Highly effective (1 response)

Some reported notable distresses that were likely due to the use of reflective crack-mitigating treatments included:

- Bumps that formed in the overlay and were caused by paving over a crack-sealed pavement too early
- Delamination caused by an interlayer
- Rutting of the overlay placed over an interlayer
- Blistering of the joints when placing a joint sealer (presumably due to excessive moisture)

Most Divisions reported that interlayer systems are cost-effective in terms of delaying reflective cracking and listed the BST as the best interlayer system. One Division recommended limited use of BSTs, however, and reported that milling and filling was often the best option. One Division reported that crack sealing was the most effective method, as long as the seal was placed at least a year prior to placing the overlay.

The survey, a summary of all the responses for each question, and the individual responses submitted are included in Appendix II of this report.

4. FIELD RESEARCH

4.1 Introduction

One major component of this research project was the construction of trial field segments of various interlayer systems. Aside from correlating the field results to the laboratory results, the main purpose of the field construction was to obtain firsthand experience with the placement of various interlayer systems and to monitor the performance of these sections over time to provide a quantifiable demonstration of their behavior. This information helped in the development of construction guidelines and is expected to continue to provide useful information to the NCDOT in the future through continued distress monitoring.

4.2 Selection of Test Sections

For the field trials of the interlayer systems, the original plan was to have three projects with average daily traffic (ADT) counts ranging from 6,000 to 25,000. Other variables, such as pavement structure and cracking severity, would be incidental to the roadways selected and would not be intentionally varied for the purposes of the overall research effort. With these factors in mind, research segment selection criteria were developed to aid NCDOT personnel in identifying suitable candidate sections and to narrow down the list of potential roadway segments for consideration in this research. These criteria are as follows:

- The segment should exhibit fairly uniform distresses throughout its length.
- The underlying pavement and subgrade should be uniform; that is, they should not alternate significantly between cut and fill.
- Segments with obvious structural deficiencies (severe edge cracking, settlement, etc.) should be avoided.

- The segment should be reasonably straight.
- The traffic throughout the segment should be approximately uniform; that is, the segment should not include intersections or major entrances/exits.
- Each treatment segment should be 600 feet long (with a 100-ft. sampling section contained within). With four treatment segments per project, the total length needed for the experimental section per project should be at least 2,400 feet.

Also, Dr. Corley-Lay at the NCDOT ensured that the pavement type, treatment type, distress type, and ADT range would meet the NCDOT's expectations for typical reflective cracking mitigation projects.

Using these criteria, the NCDOT suggested routes in Moore, Montgomery, and Vance Counties. These routes were then reviewed with site visits to identify candidate test sections within these routes. However, during a subsequent meeting with the NCDOT, it was found that due to contractual, monetary, and constructability concerns, only one field construction project was feasible for this investigation. Ultimately, a section of US 1 in Moore County was selected as the trial project location.

4.3 Field Construction Details

4.3.1 Background

The research segment for this project is a section of US 1 in Moore County between Pine Bluff and Aberdeen, North Carolina. This section is a divided four-lane roadway with ADT of approximately 10,000. The northbound lanes of this section are composed of flexible pavement, and the southbound lanes are composed of composite pavement. At-grade intersections and driveway accesses are present throughout the length of the section. Prior to construction of the test segments, the existing pavement exhibited frequent moderate-to-severe longitudinal and transverse cracking (block cracking) throughout. The NCDOT's selected treatment for the project was to mill the outside lane (driving lane) 1.5 inches and place a 1.5-inch lift of RS9.5C hot mix asphalt (HMA). Next, a 1.5-inch lift of S9.5C HMA was to be placed over the entire roadway (both the driving and passing lanes). An emulsified tack coat was to be placed between all AC layers. The particular AC mixture used for this project consisted of granite aggregate, 4.2 percent virgin PG 64-22 binder, a total binder content of 5.5 percent, and 21 percent reclaimed asphalt pavement (RAP).

The only deviation from the originally selected pavement treatment for the research segments was the placement of various interlayer products between the first and second asphalt courses in the northbound driving lane. The interlayer systems selected were two paving mats (Paving Mat #1 and Paving Mat #2), a paving fabric (Paving Fabric), tack coat only (control), and a chip seal (6M aggregate with CRS-2 emulsion).

Five 800-foot long research segments were constructed for this project, each containing a single interlayer type. Each 800-foot research segment included 200 feet at the end where cores were extracted for laboratory testing. These segments began just north of a culvert 0.18 mile south of Windy Hill Road and extended 0.06 mile south of Rosy Road. An additional short control

segment (300 ft. total) extended beyond the test sections to provide an extra distress monitoring area. More information about the location of these sections, including field notes and GPS coordinates, can be found in Appendix III.

4.3.2 Construction

The construction of this project took place over three days during the week of October 7, 2013. Milling and filling of the outside lanes took place on the first two days of construction. It should be noted that in all the sections, large cracks that were apparent on the surface also were seen in the milled surface below, confirming that most of the cracks were more than surface cracks and were likely full-depth cracks.

On the third day of construction, all the interlayers were placed and the surface course was placed over them in the driving lane. Four different types of interlayers plus a control (tack coat only) section were constructed throughout the site. These four interlayer types included three geosynthetics and a chip seal. All of the geosynthetic sections used PG 64-22 hot asphalt binder as a tack coat applied to the underlying asphalt layer. A tractor with a special broom was then used to apply the geosynthetic to the fresh binder. Next, a pneumatic tire roller was used to seat the geosynthetic into the asphalt binder. The construction of the chip seal section was slightly different and consisted of CRS-2 emulsion applied to the surface followed by a layer of 6M aggregate. Then, a steel wheel roller was used to seat the aggregate into the emulsion. The control segments consisted only of a CRS-2 tack coat between the AC layers.

During the construction process, independent measurements of the tack coat application rates were attempted by placing 12-inch x 12-inch steel plates of known weight on the surface and removing them prior to the placement of the interlayer. These plates were then reweighed, and the difference allowed the tack coat application rate to be computed. In total, six measurements were taken in each section and averaged. With this information, it was found that the target rates and the measured rates were not consistent. It is unknown if the use of the steel plates instead of a more standard absorbent material would allow runoff of the tack coat and cause incorrectly low readings, or if the tack coat rates were actually significantly below their targets. This outcome should be kept in mind when considering the future performance of these sections. Table 4-1 presents these results.

Section Name	Paving Mat #2	Paving Mat #1	Paving Fabric	Control	Chip seal
Target Application Rate (gal/yd ²)	0.20	0.17	0.25	0.04	0.40
Measured Application Rate (gal/yd ²)	0.075	0.078	0.0813	0.036	0.22*

Table 4-1. Tack Coat Application Rate

*From ignition oven testing.

No major problems occurred to prevent the interlayers from being placed, but some potential sources of variability were identified within the trial sections. These variability factors came from several key sources related to natural variations, paving practices, and problems associated

with the interlayers themselves. These factors are discussed in detail in Appendix IV of this report.

4.4 Distress Surveys

In order to assess the performance of the various interlayer systems, information about the pavement distresses was needed prior to construction and shortly after construction. Additionally, continued monitoring was needed for the duration of this research project and will be required in the future in order to obtain the maximum benefit from the trial project.

4.4.1 Preconstruction Distress Survey

Initially, automated distress data were collected from the NCDOT for the distress survey. However, because these data were originally collected in 2012, the results obtained from them were considered to be inadequate. Therefore, visual crack mapping and distress surveys were performed prior to construction. These results are presented in Appendix V of this report. Table 4-2, Figure 4-1, and Figure 4-2 summarize these results.

Crack Type	Severity	Paving Mat #2	Paving Mat #1	Paving Fabric	Control	Chip seal			
	Light	17 ft	9 ft	11 ft	15 ft	20 ft			
Longitudinal	Moderate	227 ft	332 ft	361 ft	250 ft	450 ft			
(ft)	Severe	139 ft	92 ft	45 ft	147 ft	31 ft			
	Light	35	27	41	23	24			
Transverse	Moderate	69	74	56	84	71			
(each)	Severe	18	16	23	39	14			

Table 4-2. Longitudinal and Transverse Cracks in Each Section



Figure 4-1. Summary of longitudinal cracking.



Figure 4-2. Summary of transverse cracking.

4.4.2 Post Construction Distress Survey #1

In January 2014, a field visit was made to the construction site in order to perform a visual distress survey to document the post construction distresses. The main distresses present were located within the Paving Mat #2 section. So-called 'check cracking', longitudinal cracking, and transverse cracking were observed at various locations within this section. Because the Paving Mat #2 section was located on a fairly significant grade, it is believed that the severity of these distresses stemmed primarily from the placement of the material on the slope and were not necessarily due to the type of interlayer used. It is expected that these distresses will become a problem for future pavement performance.

Check cracking also was observed in the Paving Fabric section. However, unlike the Paving Mat #2 section, this segment was not on a slope. Areas of rough surface texture also were observed, most notably in the Paving Mat #1 section at a location where it was noted that the paver stopped for a considerable amount of time to wait for dump trucks. The resulting thermal segregation may have led to the low density values in this area. It should be noted that no distresses were seen in the tack coat only (control) sections or the chip seal section. Figure 4-3 summarizes these results.



Figure 4-3. Paving-related cracking (check cracking and longitudinal cracking due to construction).

4.4.3 Post Construction Distress Survey #2

Another visual distress survey was completed in August 2014. In addition to the distresses recorded from the previous survey, another transverse crack was seen in the Paving Mat #2 section in addition to the previously mentioned cracks, and two transverse cracks were apparent in the Paving Fabric section.

4.4.4 Post Construction Distress Survey #3

In February 2015, a third distress survey was completed. This survey came immediately after a week of cold weather and it was believed that if reflective cracking at this site was related to thermal cracking, cracks may be evident in the surface layers. Several notable observations were made during this survey. First, in the southbound direction of the paving project where concrete pavement was present below the asphalt surface layers, a large number of cracks had reflected through the passing lane, with none reflected through the driving lane. Because the pavement treatment in the southbound direction was exactly the same as in the northbound direction, this observation confirms findings in the literature regarding the difficulty of controlling reflective cracking on composite pavements and demonstrates that a two-course overlay can delay reflective cracking in and of itself.

For the northbound direction, multiple cracks were seen in the passing lane adjacent to all of the test sections. These cracks tended to be short (2 ft. long or less), and located at the inside edge of the pavement. In the driving lane, several cracks were seen in the geosynthetic test sections. Specifically, four more transverse cracks of various lengths were seen in the Paving Mat #2 section and five short transverse cracks were seen in the Paving Mat #1 section, located mainly in the right wheel-path or pavement edge (outside of the milled and filled area). No changes were seen in the distresses of the Paving Fabric, chip seal, and control segments.

Although the original construction-related distresses stayed the same, the transverse cracks changed with time. The number, length, and location (in terms of inside or outside of the overlay section) were different for each section. In order to quantify this outcome, the total lengths of the reflective cracks seen in each section were plotted with time. Figure 4.4 shows that the geosynthetic sections experienced more transverse cracking over time than the control or chip seal sections. For reference, estimates of PCR (pavement condition rating) values were obtained and, although no sections were found to be deficient, the difference in the distress levels among the sections can be seen (Figure 4-5).



Figure 4-4. Length of transverse cracking.



Figure 4-5. Estimated PCR values as of February 2015.

Further details concerning all of the distress surveys can be found in Appendix II.

4.5 Falling Weight Deflectometer Testing

The best tool available to evaluate the strength of pavements in the field was the falling weight deflectometer (FWD). This device was used both before and after construction to assess the differences among the various research segments within the field construction project.

4.5.1 Preconstruction Falling Weight Deflectometer Testing

Due to the variability of the roadway throughout the project site, information was needed about the stiffness of the existing pavement in the test sections to ensure that the various interlayers could be compared fairly. In order to obtain this information, FWD measurements were taken every 100 feet in each research section prior to construction. A stiffness parameter (*S*) (Equation (5-1)) was calculated for each FWD measurement and plotted to obtain an idea of the overall support condition of each section.

$$S = \frac{load(lbs)}{deflection(mils)}$$
(5-1)

These FWD results indicate that the Paving Mat #1 section had the best overall support condition, followed by the Paving Mat #2 section, the chip seal section, and the paving fabric sections, and the control segment had the worst overall support condition. It was expected that some of the differences in performance of these sections would be related to these relative support conditions, with sections that have higher stiffness values experiencing better performance.

4.5.2 Post Construction Falling Weight Deflectometer Testing

In spring 2014, post construction FWD tests and coring were performed for the project. By comparing the preconstruction and post construction FWD values, Figure 4-6 shows that all sections saw an increase in stiffness after construction.



Figure 4-6. FWD stiffness values: preconstruction and post construction.

By dividing the stiffness value after construction by the stiffness value prior to construction, an estimate of the amount of improvement provided by the pavement treatment could be determined. Figure 4-7 shows wide scatter in the improvement ratio within a given research section, and no statistical difference in improvement ratio is evident between one type of interlayer and another. It is clear from these results that, for the application of thin overlays on relatively thick flexible pavement, none of the products tested provided any clear increase in structural capacity (beyond that provided by the new AC layers themselves). It is expected that, over time, the waterproofing ability of these layers may have a more significant impact on their ability to improve the stiffness of the pavement than any structural differences between the interlayer treatments. However, one problem when testing this hypothesis in the future will be that the materials were placed only in a single lane of the project, so water infiltration from cracks in the adjacent lane may reduce or eliminate any benefits seen from this waterproofing ability.



Figure 4-7. Stiffness improvement ratio.

5. LABORATORY TESTING OF REFLECTIVE CRACK MITIGATION SYSTEMS

5.1 Introduction

Field results can be somewhat limited for the evaluation of different interlayer treatments. Problems related to the inherent variability of the construction site, the unpredictability of loading and weather conditions, and the need for continuous monitoring of the sections make the results difficult to interpret and time consuming to compile. As such, another important component of this research is laboratory testing. Laboratory testing allows the same materials used in the field to be evaluated under more controlled, repeatable, and varied conditions in a shorter amount of time, all while obtaining various types of measurements for evaluation purposes. Therefore, laboratory tests are a valuable link between fundamental material properties and the behavior of samples in the field.

The two main goals of the laboratory evaluation are to:

- 1. Identify the important mechanisms of reflective cracking and reflective crack mitigation and the factors that could affect their prevalence in any interlayer system.
- 2. Quantify the failure of the samples such that relative rankings can be established to gauge the effectiveness of the interlayer system and extrapolate these findings for their applicability to field performance.

Three types of laboratory tests were utilized during this research. The first test was a small-scale accelerated pavement test to simulate reflective cracking in layered asphalt pavements under wheel loading. This test is known as the reflective cracking test (RCT). These tests involved

compacting large (6 ft. x 2 ft.) asphalt slabs and required a considerable amount of effort to produce and test each sample.

A second test that utilized a modified version of the ASTM D 7460 standard four-point bending beam fatigue test was selected to enable the construction of multiple samples from a single slab. These samples were notched, and thus, these tests are known as notched beam fatigue (NBF) tests. Due to their easier construction, more NBF test samples could be tested under a wider range of conditions than RCT samples.

Due to space limitations for this report, full details of the developmental history, final test procedures for these two laboratory tests, and the material properties and sample fabrication procedures are presented in Appendix VI. A brief summary of this component of the investigation is presented in Section 5.2.

A third test, an unconfined, displacement-controlled, monotonic shear test, was conducted also in order to evaluate the interlayers as constructed in the field and to compare these results to the laboratory test results. The development of this test is described in the final report for the HWY-2013-04 project *Surface Layer Bond Stress and Strength* also conducted under the auspices of the NCDOT.

5.2 Development of Laboratory Tests to Investigate Reflective Cracking

In order to investigate the crack propagation in layered asphalt samples in the laboratory for this research, two tests were developed to be used in conjunction with digital image correlation (DIC). Further details about the DIC method are presented in Appendix VI.

5.2.1 Notched Beam Fatigue Test

The simplest test developed for this research is a modified version of the ASTM (D 7460) beam fatigue test, which consists of a layered asphalt beam with a notch at the mid-span of the specimen. This test utilizes DIC to track full-field strains and displacements on the surface of the sample in order to determine the mechanisms that are important as cracks propagate through the specimen. Several trial tests were carried out using various interlayer types to study the feasibility of using this test method and to determine the optimal specimen fabrication procedure and test protocol. Once these trials had been completed, all further NBF tests were conducted in an identical manner.

Final configuration of the NBF test: The final procedure for the NBF test that was carried out for all the samples after the trial tests is as follows. First, using a slab compactor, a 1.97-inch x 12.0-inch x 15.75-inch slab of AC was compacted. The selected interlayer treatment was then applied to the surface of this layer. Next, a second AC layer was compacted to bring the height of the total sample to 3.94 inches. Three beams were then extracted (sawn) from the center of the slab in order to reduce the effects of high air void contents near the edges of the mold. These beams were trimmed to their final dimensions (2.13 in. x 2.52 in. x 15.75 in.) and a 0.2-inch x 0.1-inch notch was cut in the center. These beams were then painted and speckled for easy viewing by the DIC system. Once constructed, all the samples were placed in a four-point bending beam fatigue device inside an environmental chamber. The beams were then temperature-conditioned at the testing temperature for two hours. Once all the data acquisition

equipment was ready, displacement-controlled haversine loading was applied using a servohydraulic loading machine until a crack was seen to penetrate the full depth of the specimen. The displacement amplitudes were selected to produce 900 με at the bottom of the beam during maximum displacement and were applied at a frequency of 5 Hz. Three different test temperatures were used: 15°C, 20°C, and 25°C. Load and displacement data were tracked using a 2.5-kip load cell and linear variable displacement transducers (LVDTs) attached to the servohydraulic testing machine. Full-field strains, displacements, and crack propagation in the samples were monitored using the DIC technique.

5.2.2 Reflective Cracking Test

The other type of test developed for this research, the RCT, utilized a small-scale accelerated pavement test device for reflective cracking. The basic concept behind this test method is to construct layered asphalt systems on top of steel plates and subject them to wheel loads using a one-third scale wheel load device, known as the third-scale model mobile load simulator, or MMLS3. Considerable time and effort were needed in order to develop this test to its final configuration and mainly involved developing adequate support conditions and selecting pavement layer thicknesses.

Support conditions: Most of the time required to develop the RCT was dedicated to determining the proper support conditions needed for the tests. The RCT device needed to be rigid enough to prevent cracking during compaction and flexible enough to achieve sufficient deflection amplitude under wheel loading to induce failure in a reasonable amount of time while not deforming excessively due to the self-weight of the slab or wheel loading. A detailed description of this development process can be found in Appendix VI. In the end, a cantilever design was selected as the final design configuration (Figure 5-1).



Figure 5-1. Schematic of the final configuration of the RCT set-up.
Layer thickness: Simultaneously with the modifications to the support conditions, various iterations of layer thicknesses were tried. Although the wheel load of the MMLS3 is one-third scale, the decision was made early on not to attempt to scale the asphalt pavement layers on top of the steel plates, but rather to target a deflection amplitude of 0.030 inch. This approach was taken for two main reasons. First and foremost was the inability to scale the various interlayer products to be used for this investigation. Second was concern over the fracture properties of scaled-down asphalt layers. Because of this, the first tests with the cantilever set-up had a total of three inches of asphalt layers (correlating to the lift thicknesses in the field trial project). However, in order to limit excessive mean deflections, the eventual thickness of the total compacted asphalt layers was reduced to 2.33 inches.

It should be noted that the layers compacted in the RCT device did show air voids higher than those experienced in the field (approximately 12% and 10% for the first and second layers respectively), and imply an inability of the roller to apply proper compactive effort to the slabs. The lower density of the first asphalt layer can be explained by both a high aggregate size to thickness ratio and more rapid cooling caused by the reduced thickness, as well as the high thermal conductivity of the steel plates onto which it was compacted. However, this decreased density of the supporting layer was deemed acceptable, as this layer was only used to support the interlayer, and did not experience cracking itself due to the full depth saw cut at the center.

Final configuration of the RCT device: The final configuration of the RCT device was the best that could be achieved within the time-frame of this research. First, steel tubing and short I-beams were attached to the ground of the testing area using clamps. Next, two 36-inch x 24-inch x 0.5-inch A514 steel plates were placed on top of the tubing and the I-beams. The steel plates were then bolted to each of the pieces of tubing. Next, the I-beams were bolted to the tester plates using clamps. This set-up effectively created two cantilevered plates with a 5/8-inch gap between them. By loosening the clamps and adjusting the position of the I-beams, the length of the cantilever on each side of the joint could be controlled, thus controlling the deflection amplitude of the system under loading.

Next, additional supports were provided beneath the steel plates, a small piece of key stock was placed in the gap between the steel plates (to prevent material loss), and a compaction frame was placed around the reflective cracking tester. The compaction frame was adjusted to the correct height, and a 0.83-inch layer of HMA was compacted using a vibratory steel wheel roller. Once the first layer had cooled, a full-depth saw cut was made at the center of the slab at the gap between the two steel plates to simulate a crack in the existing pavement. The tester was loosened, the two slabs were pushed together slightly to reduce the width of the crack to 1/8 inch, and the tester was retightened. Next, the desired interlayer was constructed, and the compacted pavement layers to 2.33 inches (in the case of the chip seal slab, the chip seal itself made up approximately 0.25 in. of this thickness, in the paving mat case, the interlayer thickness was negligible). Once the second layer had cooled adequately, the vertical edge of the sample was smoothed to provide a flat surface perpendicular to the DIC camera. This surface was painted white and then speckled with black spray paint in order to be viewed by the DIC camera. The end result of this procedure was a layered slab specimen of approximately 72.6 inches x 23.0

inches x 2.33 inches with a transverse joint at the middle of the bottom layer, supported by two cantilevered steel plates.

Prior to testing, the removable compaction supports were taken out from the reflective cracking tester and in their place four spring LVDTs were placed near the joint between the steel plates (two at the center of the steel plates and two near the edge). These LVDTs allowed the deflections of the steel plates to be monitored during loading to provide independent validation of the DIC measurements. Next, an environmental chamber was placed around the tester in order to maintain the temperature of the slab at $20^{\circ}C \pm 1^{\circ}C$. This chamber had a window to allow the placement of both the DIC camera and lights outside of the chamber.

Once all the instrumentation was in place, the MMLS3 was placed inside the chamber on top of the RCT slab, and wheel loads were run over the pavement at a rate of 1,500 applications per hour – one load every 2.4 seconds, with a 1.9-second rest period where no loading on the pavement occurred. Loading was continued until a crack was seen to propagate all the way to the surface.

5.2.3 Monotonic Shear Test

Through the course of this research it was determined that shear testing is also a vital component for evaluating the behavior of interlayer systems. In order to investigate the systems tested in this research, the Modified Advanced Shear Test (MAST), originally used for the NCDOT study of the bond strength between asphalt pavement layers, was used to perform monotonic shear tests on these systems. The test conditions for the shear tests performed on the interlayer systems were selected to approximate those of European standard Leutner tests (Vaitkus 2011), i.e., an unconfined shear test with a displacement rate of two inches per minute and a test temperature of 20°C. The main differences between the MAST tests and the Leutner tests are: 1) the MAST device is significantly more rigid than the Leutner device), 2) the gap between the two sides of the MAST device is 0.31 inch rather than 0.2 inch used in the Leutner device, and 3) the MAST device uses 6-inch diameter cylindrical specimens. The experimental plan for shear testing involved testing both field cores and laboratory samples.

The field cores were obtained by drilling 6-inch diameter cores vertically from the project site and then trimming them in the lab to a final specimen geometry of 4 inches x 4 inches x \sim 3 inches (L x W x H). It should be noted that the overall height of the samples varied somewhat based on the actual thickness of the surface layer as placed in the field, which varied between 1.4 inches and 1.8 inches (Figure 5-2). The samples were then speckled for viewing by the DIC camera, glued into the loading shoes, and then the loading shoes were placed into an environmental chamber to temperature condition for at least two hours. After conditioning, the loading shoes were placed in the MAST device, the set-up was temperature-conditioned for 30 minutes to return to thermal equilibrium with the test temperature, and the test was performed. DIC images and load and displacement data were collected throughout the short test. The shear strength of the material could be determined from the peak load and the area of the sample. Two replicates were performed, and in cases with high variability, an additional replicate was performed.

PM#2-	PM#2-	PM#2-	PM#1-	PM#1-	PF-C1	PF-C2	PF-C3	PF-C4	Ctl-C1	Ctl-C2 MS-C6
C1	C2	C3	C1	C4						
									25' 25' 25' 25' 25' 15' 15' 	
New New	v Surface	Mix	E:	xisting S	urface N	/lix	— C	hip Seal I	Layer	<u>+</u> 6884
Exis Grae	ting Fine ded Mix	2	🗱 E G	xisting C Traded G)pen ravel Mi	ix	Ex	isting Fir	ne Grade	d Mix (assumed)

Figure 5-2. Pavement thickness measured from the field cores.

The laboratory samples were constructed by fabricating slabs in an identical manner as those used for beam testing, the main difference being that two control slabs were made with tack coat application rates of 0.04 gal/yd² and 0.07 gal/yd² (0.03 gal/yd² and 0.05 gal/yd² residuals, respectively). This protocol allowed the shear strength of the laboratory-fabricated tack coat only slabs (with application rate of 0.07 gal/yd²) used for NBF testing to be determined, as well as the comparison of the shear strength of the laboratory-fabricated specimens to the shear strength of the field cores using the same target application rate (0.04 gal/yd²). Once constructed, 4-inch x 4-inch shear samples were sawn from these slabs. These samples were tested in an identical manner to the field core samples.

It should also be noted that no shear testing was performed on the chip seal samples for either the lab or the field, as the gap between the shear test plates was smaller than the aggregate size of the chip seals, and it was determined that the shear strength values obtained from these tests would be questionable.

5.3 Laboratory Test Results

Due to space limitations, only a brief summary of the laboratory test results is presented in this section of the report. For detailed descriptions of the laboratory test results, see Appendix IX.

5.3.1 Notched Beam Fatigue Test Results

All of the interlayer systems used in the field research project were evaluated for this NBF test investigation. Additionally, a limited number of grid-reinforced samples were tested as part of a separate project, and these tests are included in the analysis results. Two main types of information were gathered: DIC information as well as load and displacement data obtained from the servo-hydraulic test machine. Both types of information provided insights into the behavior of the various interlayer systems subjected to this type of loading. Table 5-1 shows the number of NBF tests completed.

Sample	Tack Coat	15°C	20°C	25°C
Tack Coat	0.05 gal/yd ² CRS-2	1	4	1
Paving Mat #1	0.17 gal/yd ² PG 64-22	2	2	2
Paving Mat #2	0.20 gal/yd ² PG 64-22	2	2	2
Paving Fabric	0.22 gal/yd ² PG 64-22	2	2	2
Chip Seal	0.40 gal/yd ² CRS-2 & 0.05 gal/yd ² CRS-2	2	2	2
Grid	None	-	3	-
Grid	0.046 gal/yd ² PG 64-22	-	6	-
Grid	0.15 gal/yd ² highly polymer-modified binder	-	2	-

T-1-1- 5 1	C 1 - + 1	N-4-1-1 D	E-time	T + -
1 able 5-1.	Completed	Notched Beam	Fatigue	Tests

DIC information: The DIC measurements were the most informative data gathered from the NBF tests. From these results, information about crack propagation in the beams and the interfacial behavior of the samples was obtained. Strain contour plots were used to identify areas of cracking and damage as well as to help determine the primary mechanisms involved in causing the motion and deformation of the specimens

As an example, Figure 5-3 shows von Mises strains (which serve as estimates of the total strain) for samples of different interlayers tested at 20°C. By obtaining similar results for all samples and conditions tested, general descriptions of damage evolution within the samples can be developed. For the tack coat only samples, cracking proceeded through the bottom AC layer with minimal interfacial movement. The crack spent little to no time 'trapped' at the layer interface and rapidly propagated through the top layer. For the geosynthetic samples, cracks began to propagate in the bottom AC layer, and interfacial movement started to occur. Once the vertical cracks reached the interlayer, interfacial movement increased significantly. This interfacial movement helped to stall the crack at the interface for some time before cracks (both top-down and bottom-up) in the top layer caused a full-depth crack to develop. For the chip seal samples, crack propagation in the bottom AC layer occurred at the same time as significant interfacial movement. Also, stress concentrations due to the chip seal aggregate helped to initiate bottom-up cracks in the top AC layer early on. Next, additional interfacial movement occurred once the crack penetrated all the way to the interface. Then, one or more top-down cracks formed in the top layer. Lastly, one pair of top-down and bottom-up cracks eventually joined, creating a fulldepth crack.



Figure 5-3. Crack propagation in NBF tests.

Although von Mises strains can be used to track crack propagation and damage of a sample, the component strain fields (horizontal, vertical, and shear strains) can be used to better understand the mechanisms that cause these areas of high strain. Figure 5-4 shows an example of these results. By obtaining similar results for all of the samples and conditions tested, it was found that that both separation and sliding occurred at the interface for all the samples except for the grid samples with the highly polymer-modified (HPM) tack coat (which experienced little interfacial movement). Both the magnitude and the timing of these types of interfacial movement depended on the type of interlayer used as well as the test temperature, with high temperatures and tests with interlayer products exhibiting greater interfacial movement earlier in the test than with low temperatures and tests without interlayer systems. The differences in behavior in the different samples are mainly explained by the relative bond strength between the layers. The samples that exhibited high interfacial movement were those that were expected to have low interfacial bond strength (either due to the interlayer type or reduced strength at higher test temperatures).

One major drawback of using DIC contour plots in making judgments about the relative interfacial movements of various NBF samples tested is the qualitative nature of both constructing and interpreting these plots. Specifically, changing the number and range of the contour intervals can have a large effect on the resulting plot, and can make meaningful comparisons of the images somewhat subjective. In order to eliminate this subjectivity, four virtual gauges were placed in the DIC program, as shown in Figure 5-5, to allow the movement of these four points to be calculated. By calculating the vertical and horizontal differential movements of these points, estimations of the separation and sliding movements across the interface could be obtained. These measurements helped to confirm the qualitative observations of interfacial movement and crack locations obtained from the DIC contour plots (Figure 5-6 and Figure 5-7). These figures confirm that both the amplitude of the layer separation per cycle and the overall magnitude of the accumulated layer separation decrease with increasing layer bond strength. Additionally, these results helped to confirm that interfacial movements tended to increase significantly once the vertical crack reached the interlayer.



Figure 5-4. Example of NBF test component strain fields (Paving Mat #1).



Figure 5-5. NBF test gauge locations.



Figure 5-7. Layer separation (mils) of grid interlayers at 20°C (left to right: no tack coat, PG 64-22 binder, and highly polymer-modified binder).

Load and displacement information: An independent set of data obtained during each test was the load and displacement information obtained by the servo-hydraulic test machine. Using these data, several parameters could be determined and compared with the DIC results. First and foremost was stiffness. By taking the peak stress experienced by the sample for each cycle and dividing it by the peak strain amplitude of the beam, the dynamic stiffness of the sample could be obtained. This stiffness value then was used for several purposes.

First, the stiffness value was simply plotted versus the number of cycles. Because the NBF tests were performed with constant displacement amplitude loading, the initial stiffness value was high and dropped continuously throughout the test. Changes in the slope of this line tended to correspond to physical changes that occurred within the sample, as seen in the DIC results. Figure 5-8 presents a representative curve, with the major regions and points identified. It should be noted that not all of the samples for all of the conditions tested exhibited all of these characteristics, but rather, this figure is meant as a summary of all the potential mechanisms that were seen.

The first region presented in Figure 5-8, denoted as 1, is associated with interfacial movement and cracking in the bottom AC layer. In some cases, both of these phenomena occurred simultaneously at the beginning of the test. During this phase, as the crack approached the interlayer, a significant drop in stiffness occurred. Often this drop corresponded to a significant increase in interfacial movement, as seen in the DIC images. Due to the fact that the DIC device measures only surface behavior, and stiffness data represent the structural integrity of the entire width of the beam, a vertical crack was likely to reach the interlayer at different times throughout the width of the beam. Thus, point A is considered the point at which the crack has reached the interlayer across the full width of the beam.

The second region, denoted as 2, is typically the longest and is characterized by a slow, steady decline in the stiffness curve. This stiffness decrease is due to damage initiation, crack propagation, and viscoelastic/viscoplastic effects within the top AC layer. The end of this region (point B) is often characterized by the initiation of one or more top-down cracks in the beam.

The last region, denoted as 3, sees a nonlinear decrease in stiffness and is associated with the formation of 'dominant' cracks within the top AC layer and their propagation toward one another. The end of this region is marked by point C, where the cracks have reached the full depth of the sample. It should be noted, however, that point C is not well defined, likely due to the similar problems associated with point A.



Figure 5-8. General behavior of stiffness drop for NBF test samples.

Although it may seem from this discussion that the failure of the sample can be characterized sufficiently by the stiffness curve, stiffness curves that are derived from actual test data are generally harder to interpret, with regions that are less distinct. Often, it is the DIC information that allows the researcher to determine the points and regions of the graphs more accurately, rather than simply using the stiffness curve itself to find these points. Moreover, even in cases where clearly defined regions are present, the accuracy of this type of analysis is dependent on the scale on which the graph is plotted and is therefore subjective.

Because of these limitations, other failure criteria were investigated using the load and deflection information to provide a more objective ranking of the types of specimen behavior. These criteria include stiffness-based methods, a method similar to that found in ASTM D 7460 (reduced energy ratio criterion), phase angle drop, and several energy-based methods. These methods were then checked for correlation with DIC criteria, such as vertical crack length, horizontal crack length, number of cycles for the crack to reach the interlayer, and the amount of layer separation and sliding. Many of the load-based criteria, in particular the energy-based criteria, were found to correlate well with the vertical crack propagation rate within the sample. However, it was found that the vertical crack propagation rate is affected by the amount of interfacial debonding. Primarily, extreme interfacial debonding resulted in the cracks 'stalling' at the interface for some time while new cracks initiated in the top AC layer. This situation may be problematic, because extreme cases of horizontal debonding may themselves be an indication of failure. This would mean that the sample failed long before the vertical crack propagated the full depth of the sample. As such, no objective ranking could be determined to capture both horizontal and vertical crack growth rates from NBF tests alone. Table 5-2 shows that the choice

of failure criterion can significantly affect the ranking of the samples (grid samples not included).

In the end, the reduced energy ratio criterion was selected as the criterion that was able to quantify the vertical cracking of the sample during the NBF test in the simplest manner. Figure 5-9 shows the number of cycles needed to reach vertical failure using this criterion for all the samples tested at 20°C. It is very important to remember that this criterion can only be used to give an indication as to whether the interlayer system delays reflective cracking compared to the control sample; it cannot be used to provide an objective ranking of the material's suitability for actual pavement applications due to its inability to capture horizontal cracking.

Criteria	Reduced energy ratio, phase angle, DIC, and 20% of Cycle #200		Cycles to 50% of Cycle #50		Layer separation at cycle #30,000			No. of cycles to reach layer separation of 0.2 mils				
Temp.	15°C	20°C	25°C	15°C	20°C	25°C	15°C	20°C	25°C	15°C	20°C	25°C
Paving Mat #2	1	1	1	3	4	4	4	4	4	4	5	5
Paving Mat #1	3	2	3	1	1	1	3	3	2	2	2	3
Paving Fabric	2	4	2	2	2	2	2	2	3	1	1	1
Chip seal	4	3	4	5	5	5	5	5	5	3	4	4
Control	5	5	5	4	3	3	1	1	1	-	3	2

Table 5-2. Failure Criteria Rankings



Figure 5-9. Cycles to reach vertical crack failure (peak reduced energy ratio); horizontal failure not considered.

5.3.1 Shear Test Results

Because the NBF test results were unable to quantify horizontal cracking and interlayer strength objectively, shear testing was performed. As mentioned previously, the shear testing performed as part of this research involved both field- and laboratory-fabricated samples (only the samples that were part of the NCDOT field project were tested; grid samples were not tested as part of this research). Figure 5-10 shows the shear test results from the field core tests and indicates large variation in the shear strength values for the different interlayer treatments tested. All the geosynthetics had reduced shear strength compared to the control (due to less aggregate interlock at the interface), with Paving Fabric and Paving Mat #2 experiencing a significant decrease in shear strength. All of these values were compared against the minimum required shear strength values found in literature as reported by Collop (2009). Though he found the recommended shear strength values varied from source to source, all were found to be dependent on the depth of the placement of the geosynthetic. This phenomenon is due to the fact that geosynthetics that are placed lower down in the pavement structure tend to experience less shear stress and therefore require less shear strength to resist damage. Of the materials tested in this investigation, Paving Mat #1 is the only geosynthetic with field core samples that met the lowest minimum recommended shear strength value for the surface/binder interface (Collop 2009). Also, it should be noted that the lower values for the shear strength thresholds correspond to recommendations found in German paving standards (Vaitkus 2011).

With regards to the specific thresholds, it should be noted that the terms "surface", "binder", and "base" as used by Collop (2009) in his comparison of thresholds recommended in literature to measured strength values referred to the first, second, and third asphalt layers respectively, and are not meant to imply that these limits are only valid for specific mixture types. While it is true that differences in thicknesses and material types would affect the interfacial strengths and stresses present, the literature relating to the European standards did not go into this level of detail when determining threshold values for a screening test. Ideally a pavement model which accounts for these effects would be the best way to specify the required shear strength values, however, this is not the current state of the practice. Rather, empirical threshold values depending on the general location of the material are used.

Figure 5-11 presents the results from the laboratory tests and indicates that all the geosynthetic materials performed better than the field specimens. In fact, all of the laboratory specimens met the lowest minimum recommended shear strength value for the surface/binder interface (except for one outlier believed to be an experimental error), although none approached the higher two recommendations. The only interlayers that consistently exceeded the higher value of the minimum recommended shear strength value were the tack coat only cases. This result is in spite of the significant decrease in shear strength that was seen when the tack coat application rate was increased from 0.04 gal/yd² (0.03 gal/yd² residual) to 0.07 gal/yd² (0.05 gal/yd² residual). This result confirms previous research that found that applying more tack coat is not always better for performance, and that an optimal rate must be used to achieve the maximum interfacial strength (Al-Qadi 2008).





Figure 5-11. Shear strength of laboratory samples.

One important observation made by the researcher who ran the shear tests is that after failure, an examination of the broken samples seemed to indicate that the field samples, particularly the Paving Fabric samples, had much less tack coat than the laboratory samples. This observation, combined with the significantly lower shear strength of the field samples (as previously mentioned), led to the hypothesis that the previously measured low field tack coat application rates were indeed accurate. If true, this outcome could create a serious problem for the field trial segments, since all of the geosynthetic sections would have received binder application rates significantly lower than the application rate (i.e., the asphalt retention rate) that is required to saturate them. This rate itself is lower than the actual target application rates that are intended to allow full saturation of the geosynthetic as well as provide additional binder to help bond the layers together.

In an attempt to quantify this phenomenon, two factors were calculated and plotted against one another to demonstrate the correlation. The first factor is referred to as *strength deficiency* and is defined as follows:

Strength Deficiency =
$$\left(\frac{Lab-Field}{Lab} - Corr\right) x \ 100\%$$
 (6-1)

where

Lab = average shear strength of the interlayer system measured from laboratory samples (psi),

Field = average shear strength of the interlayer system measured from field samples (psi), Corr = correction factor to account for differences in the shear strength of the control samples due to variability between the laboratory and field samples and not associated with the field application rates given by Equation (6-2):

$$Corr = \frac{Lab_{Control} - Field_{Control}}{Lab_{Control}}$$
(6-2)

The second factor is *asphalt binder saturation deficiency* and is defined as follows:

Asphalt Binder Saturation Deficiency =
$$\left(\frac{ret - r_{field}}{ret}\right) x100\%$$
 (6-3)

where

ret = asphalt retention rate for the geosynthetic (gal/yd²), and r_{field} = field application rate (gal/yd²).

By cross-plotting these values (Figure 5-12), some correlation was found between the amount of tack coat deficiency and shear strength deficiency. This correlation is expected to be stronger for a range of deficiencies within a specific geosynthetic than it is for different deficiencies with different materials. Rather than giving a definite number as a predictive relationship, this correlation simply implies that the poorer field performance of the interlayer systems in the project is not necessarily due to a deficiency of the material itself, but rather to construction variability. Also, this correlation helps to explain the reason that the two sections (Paving Mat #2 and Paving Fabric) with the lowest shear strength values saw the highest level of construction

distress. Low shear strength allowed more slippage of the mixture during rolling that, in turn, resulted in more compaction-related cracking (mostly check cracking), which was not readily observed for any of the other three sections in the project (Figure 4-3). The case of Paving Mat #2 is made even more severe by the presence of the grade, which caused the occurrence of slipping under the right wheel of the paver. In future, care should be taken to note differences in the types of distresses seen in individual sections, with particular attention paid to distresses that are related to slipping or debonding of the layers. It is expected that the Paving Mat #2 and Paving Fabric sections will be more prone to these types of distresses.



Figure 5-12. Strength deficiency vs. asphalt binder saturation deficiency.

5.3.2 Reflective Cracking Test Results

The last type of laboratory test performed in this investigation is the RCT. The original intent of this test was to serve as an intermediate condition between the smaller scale laboratory tests and the field conditions. Aside from the trial tests used for the RCT development, three RCTs were performed using the final RCT configuration. The first test performed contained a chip seal interlayer and was constructed as follows. A 0.83-inch layer of S9.5B mix was compacted, and a saw cut was introduced at the joint. Next, 0.4 gal/yd² (0.3 gal/yd² residual) of CRS-2 emulsion was placed on the surface, and 20 lb/yd² of 78M aggregate was placed on the surface and rolled. After curing, the surface was swept and 0.08 gal/yd² (0.06 gal/yd² residual) of CRS-2 emulsion was applied as a tack coat. This tack coat was allowed to cure completely before the placement of the second AC layer. The surface was cut and then painted to allow for DIC analysis, the compaction supports were removed, the pavement was temperature-conditioned, and the set-up was subjected to MMLS3 wheel loads until failure. The same DIC analysis techniques utilized for the RCTs.

Figure 5-13 (b) shows that debonding of the surface occurred very rapidly. Interface separation was seen within 700 cycles, and full debonding was seen before 10,000 cycles. As the test progressed, the opening that was present at the interface increased. At around 16,060 cycles, a

crack began to propagate vertically, approximately 2.36 inches to the left of the joint. This crack grew until around 44,000 cycles when a second crack began to propagate vertically around the interface, 4.89 inches to the left of the joint. It was this second crack that eventually reached the surface of the overlay, just outside of the viewing area of the DIC camera. It is believed that this type of cracking was caused by the rebounding of the right slab while the left slab was still under wheel loading (Figure 5-14). The total life of this system was around 55,000 cycles.

After the chip seal RCT platform had been demolished, a tack coat only test set-up was constructed. The tack coat only test was performed to provide a basis of comparison for the chip seal test. This control test consisted of a 0.83-inch S9.5B support layer, 0.05 gal/yd² (residual) CRS-2 tack coat, and a 1.5-inch S9.5B overlay. This test set-up performed fairly well and did not result in the large-scale debonding that was evident in the chip seal tests.

Figure 5-13 (a) shows the results of the control tests. It can be seen that the initial strain along the interface, particularly in the left slab, is significant. Next, the crack grew horizontally along the interface on the right slab. After growing for approximately 1.2 inches, the crack stopped, and a second crack formed on the left slab and propagated about 1.7 inches. At this point, both cracks on the right and left slabs began to propagate upwards, and a third crack began to propagate directly above the joint. Over time, it was this center crack that was the first to reach the surface of the sample. The total number of cycles to failure was approximately 84,000 cycles.

The last RCT sample consisted of a 0.83-inch S9.5B support layer, 0.17 gal/yd² PG 64-22 binder, Paving Mat #1, and a 1.5-inch S9.5B overlay. Early on in this test, strains developed along the interlayer; however, these strains did not increase much over time (Figure 5-14 (c)). At around 51,400 cycles, the test had to be stopped due to an equipment malfunction. Thus, the sample could not be tested to failure. Even so, information up to this point could be used for direct comparison with other test results.

Although the types of behavior observed from the RCTs showed differences, these different types of behavior do indeed make sense. All of the test samples experienced sliding along the interface between the pavement layers. The control and chip seal samples saw sliding movement of a similar magnitude: ~0.0075 inch. Also, both samples saw debonding at the interface. However, due to poorer adhesion between the layers in the chip seal sample than the control sample, the magnitude of the separation in the chip seal sample was over ten times that of the debonding seen in the control sample (0.06 in. vs. 0.005 in.). The Paving Mat #1 sample behaved somewhat differently. Paving Mat #1 is a stiff material with a fairly low binder absorption capacity and was bonded to the pavement with a fairly high application rate using PG 64-22 asphalt binder. These factors helped to ensure that a strong bond between the layers was achieved. The combination of the reinforcement provided by the paving mat and the good bond between the layers helped to reduce the deflection amplitude of the slab. These factors decreased the deflection amplitude that corresponded to reduced interfacial movement, with separation and sliding of only 0.0009 inch and 0.0005 inch, respectively. Furthermore, this decreased deflection amplitude meant that no cracks were observed up to 51,400 cycles, at which point the test had to be stopped due to the aforementioned equipment malfunction. Had the test continued, it is expected that the Paving Mat #1 sample would have significantly outperformed both of the other (chip seal and control) RCT samples.





Figure 5-14. Mechanism of debonding in RCTs.

These results fit with previous research results that indicate that the amount of debonding is related to the support condition of the pavement and affects the overall number of cycles to failure (Zhou 2000, Dondi 2000). It is believed that the tendency for the control and chip seal pavement systems to debond during the RCT was due to the boundary conditions of the test setup, and that pavements in the field may not exhibit such an extreme tendency. These factors explain the reason that the chip seal sample performed worse than the tack coat only sample during the RCT, yet the chip seal has been reported to be more effective in the field. Also, these two RCT results fit well with previous research results that suggest the possibility of double or even triple cracking in pavements where large relative deflections across the joint in the existing pavement are seen (McCullagh 1985, Judycki 1996, Scarpas 1996, Vanelstraete 1996, Zhou 2000, Zhou 2005, Zheng 2012). It is believed that the stiffness of Paving Mat #1 helped to reduce the overall deflection amplitude of the slab and that this reduced movement combined with good layer adhesion helped to delay cracking of the sample. However, like the previous RCTs, the boundary conditions and the scaled-down nature of the test set-up suggest that any benefits seen in this test may not be directly proportional to actual field performance, especially considering that no apparent increase in structural stiffness was seen utilizing Paving Mat #1 in the field.

5.3.3 Comparison Between Laboratory Test Methods

The most straightforward tests run during this research were the shear strength tests. These tests are related to a fundamental material property and resulted in repeatable values from test to test. However, the shear test as performed in this research does not have a current standard in the United States and consequently, equipment for such testing must be custom made by each individual laboratory. From a practical standpoint, rather than add new shear test methods to the existing pavement testing standards, it would be desirable to use existing beam fatigue equipment to discern responses in the NBF test that relate to shear strength. However, through the course of this research, it was found that the only parameters that produced a clear ranking of the materials in the NBF tests that correlated to the rankings obtained from the unconfined shear

tests was the amount of layer separation. Even then, due to large scatter in the NBF test values, averages of the replicates must be used (Figure 5-15).



Figure 5-15. Shear strength vs. layer separation.

Although the layer separation results obtained from the NBF tests could give an indication of the relative shear strength between the layers, no objective relationship could be developed that would be expected to hold for a wide variety of conditions (such as mixture type, test temperature, interlayer application rates, etc.). Lacking a sophisticated finite element model that can capture all of the mechanisms present in layered NBF test samples, the best outcomes that can be achieved at this time are empirical relationships between vertical cracking and horizontal cracking (layer separation) and between horizontal cracking (layer separation) and shear strength. The general concept is to develop a chart that would consist of a cross-plot of the number of cycles to reach vertical failure against the number of cycles to reach horizontal failure, and through the use of threshold values, define regions where materials would either pass or fail. Figure 5-16 shows the general concept behind such a chart. In this graph, Region 1 indicates a material that passes both vertical and horizontal thresholds, Region 2 indicates a material that passes the vertical cracking threshold but does not pass the horizontal cracking threshold, Region 3 indicates a material that fails both the vertical and horizontal cracking thresholds, and Region 4 indicates a material that passes the horizontal cracking threshold but does not meet the vertical cracking threshold.



of cycles to reach horizontal failure

Figure 5-16. Concept of an empirical relationship between vertical and horizontal cracking for material acceptance.

Empirical relationships for shear strength, vertical cracking, and layer separation: In order to develop these relationships, the first task was to select a shear strength threshold based solely on experimental data. First, it was noted that all the geosynthetics likely experienced serious tack coat application rate deficiencies in the field. As such, it was assumed that a shear strength threshold would reject all of these materials. A shear strength value (under the standard test conditions) of 145 psi would achieve this result. This threshold would not exclude the laboratory samples that experienced significant interfacial debonding in the NBF tests (Paving Mat #2). If these samples were to be excluded, a threshold value of approximately 160 psi would be needed. Although not determined from numerical modeling, these values are in line with the threshold values reported in the literature, with 163 psi being the threshold value from literature closest to the observed results.

Figure 5-17 shows these three possible shear strength threshold values compared to the number of cycles needed to reach 1.18 mils of layer separation. Next, using the regression equation from Figure 5-17, the number of cycles to reach the 1.18 mils of layer separation threshold could be calculated for each threshold value selected. These values were then used as thresholds in the cross-plot of vertical cracking and number of cycles to reach 1.18 mils of layer separation (Figure 5-18). To help confirm the selection of these threshold values, the results from the grid samples also are included in Figure 5-18, because these samples were expected to have a wide range of interfacial strength values, ranging from poor to good to excellent, depending on the tack coat used (none, PG 64-22 tack coat, and highly polymer-modified, or HPM, tack coat, respectively). It should be noted that the grid HPM tack samples and the control samples never reached the condition where the layer separation threshold was exceeded and, thus, do not show up on the graph.

A vertical cracking threshold also is included in Figure 5-18 and can be seen as a horizontal dotted line. This threshold is simply the average number of cycles to peak reduced energy ratio

for the control samples, and, in order to show improvement, the interlayer samples should be above this average. Thus, although not an objective ranking, the vertical cracking criteria can indicate if a material performs 'better' than the control case.

Figure 5-18 also shows that all the interlayer samples passed the vertical cracking threshold (including the grid HPM tack samples) and, depending on which shear strength threshold was selected, the Paving Mat #2, Grid No Tack Coat, and Chip seal samples could be either accepted or rejected. The limitation of these threshold values is that no shear strength data were available for the grid samples or the chip seal samples, so a firm judgment cannot be made on the reasonableness of their acceptance or rejection. Although it does make sense that the grid no tack coat samples should be rejected (because they represent failure to comply with the construction requirements), and the grid with PG 64-22 tack coat and the grid with the HPM binder tack coat should be accepted (because they represent compliance with the construction requirements), these results could not be confirmed.

Some words of caution about this methodology are that the tests in this research represent a limited number of materials tested under a limited number of conditions. Since this methodology is an empirical approach, additional tests are needed under a wide variety of test conditions and material types before such a concept could be considered useful for implementation. Additionally, independent research at NCSU funded by St. Gobain ADFORS in its investigation of grid interlayer systems, has called into question the validity of using NBF tests at high strain levels (such as 900 $\mu\epsilon$), and suggests that NBF tests at lower levels of strain might represent the field condition more accurately. As such, care must be taken not to put too much weight behind the empirical relationships developed during the course of this investigation, as they may represent an extreme test condition.



Figure 5-17. Relationship between layer separation and shear strength with threshold values.



Figure 5-18. Relationship between layer separation and vertical cracking.

5.4 Conclusions from Laboratory Test Results

Overall, all the laboratory tests performed in this research indicate that the assumption of complete interfacial bonding, which is an assumption that commonly is made in the modeling of pavements, is an unrealistic estimate of behavior, even for the tack coat only case. Furthermore, because interfacial phenomena help to dissipate energy that would otherwise be used to drive crack propagation, they can significantly delay reflective cracking in a two-layered system under certain loading conditions (such as those present in the NBF test, which is a displacementcontrolled test). However, since this dissipation of energy may come in the form of interfacial damage, accelerated failure of the structure could occur under more complex loading conditions (such as those present in the RCTs or potentially in field applications). This research clearly indicates that in most cases, interlayer samples tend to increase the amount of interfacial movement that occurs during fatigue tests, which is due to the reduction in interfacial strength that is due to the presence of the interlayers. For this reason, interlayers should not be placed in areas where layer separation is likely to occur, in particular, in cases of overlays of rigid or composite pavement where high differential movement is seen across cracks and joints. These observations fit well with previous research results that generally report poor performance of interlayers in these situations and discourage their use (Dykes 1980, Button 1989, Barksdale 1991). Literature suggests a minimum load transfer efficiency of 80% (Button 2007). Similarly, the reduction in shear strength means that interlayers should not be placed in areas that experience high shear forces. Therefore, the use of interlayers in areas such as intersections, steep grades, and sharp curves should be avoided. If interlayers are to be used in areas of high shear stress, they should be placed deeper in the pavement structure, and an analysis of the shear stress in the pavement should be performed and compared to the shear strength of the materials within the pavement layers to confirm that these materials will be subjected to loads well below their shear strength.

The results of this study indicate that although the RCT and NBF test results do show the effects of interlayers on the vertical crack propagation rate and magnitude of interfacial movements, they can provide only a general ranking of these parameters, and it is not known how well these rankings correlate to actual field behavior. As such, the results of this study indicate that the only test method that can objectively capture a fundamental material property that is known to correlate directly to field behavior is the shear test. For this reason, specific shear tests in combination with layered pavement simulations may be needed in order to adequately quantify the ability of the interlayer products to resist shear failure (such as slipping) in the field. By far the most rigorous method would be to develop shear strength mastercurves, as outlined in the NCDOT project, *Surface Layer Bond Stress and Strength*, for each material under consideration, and using this information as an input to advanced numerical models. Although this other research effort is not discussed in detail because it is out outside of the scope of this investigation, a brief discussion of this shear strength mastercurve concept is presented in Appendix X in order to ensure that this report is a stand-alone document.

Also, it should be noted that the test methods used in this research project do not test explicitly for rutting resistance of the layers. The literature indicates that the high bitumen content of some interlayers can cause increased potential for rutting and bleeding (Sarsam 1982, Barskdale 1991, Epps 1994, Al-Qadi 2003, Roque 2012, Vespa 2005, Fyfe 2010, Solaimanian 2013). The RCTs did not produce significant rutting, as the number of cycles to failure for the layered systems was low. As such, separate characterizations of the potential increase of rutting due to the presence of interlayers might be needed in further research. Originally, it was expected that some quantification of rutting potential could be obtained by monitoring the field sections used in this research; however, uncertainties about the tack coat application rates may make any observations of rut depths in the field trial segments questionable.

5.4.1 Recommendations of Test Methods

Due to its complicated nature, high variability, difficulty in construction, expensive equipment, and unrealistic boundary conditions, the RCT was not considered to be a viable testing method to be applied to new and innovative interlayer materials on a routine basis. The NBF test is considered a better candidate because it is much simpler to perform than the RCT. However, while the NBF test can give an indication if the interlayer systems perform better than the control at mitigating vertical cracking, it cannot adequately capture the detrimental effects of interlayer debonding from load and displacement information alone. Though an empirical relation was developed correlating layer separation and shear strength, enough data was not present to provide confidence that this relationship would remain valid under conditions outside of those tested in this research.

As such, one alternative to these challenges is to use the NBF and a simple shear screening test as part of a two test method acceptance plan. First, using the NBF test, interlayer systems that experience more cycles to failure than the control are considered to 'pass' the vertical cracking requirement. Next, shear screening tests are used to check that the samples that pass the vertical cracking criterion do not experience weak interfacial bonding. A simple shear test method in this regard that resembles the European standard Leutner tests can be employed. Such a standard consists of an unconfined monotonic test and contains a minimum value of shear strength that must be achieved in order to ensure adequate bonding between layers. Although the recommended shear strength to use as a threshold varies from author to author in the literature, the results of this research indicate that 160 psi is a reasonable value. It should be noted, however, that this threshold is an empirical value using a monotonic strength to assess long term durability under traffic and environmental loading conditions and is therefore somewhat speculative at this time. Additional research would be needed to confirm this value. This further study would ideally involve testing interlayer systems at various tack coat application rates in order to obtain shear strength mastercurves and coupling these results with pavement simulations which takes into account material properties and layer thicknesses. The results could then be used to create a "standard" shear strength threshold could be obtained. An even better method would be to base the temperature, loading rate, and confining pressure of the screening test on those parameters likely to be experienced in the field, rather than simply selecting a threshold value at specified test conditions. This would mean material requirements would vary depending on the application and region, much like the performance grading system employed for asphalt binder grading today.

6. COST BENEFIT ANALYSIS

Because detailed information about the performance of these interlayers could not be developed from the laboratory testing or the field research, only a simple cost benefit analysis could be performed. Two simplified approaches are proposed here for this analysis.

First, the additional cost of placing interlayers must be determined as shown in Table 6-1. Next, this additional cost is divided by the typical cost of placing an overlay without any interlayer. This ratio is known as the '% cost increase' that is due to interlayer use. In order to determine if interlayer treatments are cost-effective for any given project, the engineer should first estimate the expected life of the overlay without an interlayer. By multiplying this life by the % cost increase, the engineer can arrive at a simplified life extension (in years) that would be required to justify the additional cost.

Interlayer Type	Tack Coat Only (Control)	Chip Seal	Paving Fabric	Paving Mat #1 and Paving Mat #2
Cost per Square Yard	(Included)	\$2.25	\$2.50	\$4.00
Total Cost w/ Overlay	\$23.98	\$26.23	\$26.48	\$27.98
% Cost Increase	0%	10%	11%	17%

Table 6-1. Treatment Cost for Each Research Segment in Trial Project

However, care should be taken with this method, as it does not take into account the economics of present and future values (discount rates). Because of this limitation, a more rigorous approach is preferred. If a life extension of the interlayer can be estimated (typically between two and five years), and a discount rate is known, Equation (7-1) can be used to determine if a particular interlayer treatment is cost-effective. If the inequality is true, the interlayer can be considered to be cost-effective.

% cost increase
$$\leq \left[100 - \frac{100}{\left(1 + \frac{i}{100}\right)^m}\right]$$
 (7-1)

where

% cost increase =
$$\frac{additional \ cost \ due \ to \ interlayer \ use}{cost \ of \ the \ overlay \ treatment \ without \ the \ interlayer} x100$$
 (7-2)

where

i = yearly discount rate (%),

m = expected life extension provided by the interlayer (years).

The major limitation of this second approach is that it does not take into account any differences in the salvage value of the pavement. Although all the interlayers used in the research project are claimed to be recyclable, little is actually known about their true effect on the quality of the RAP obtained from milling these materials. Other materials, such as steel mesh interlayers, may need to be removed prior to milling (depending on the milling depth). Thus, changes in the salvage value of the RAP obtained from the project or additional costs incurred to remove the interlayers from the pavement are not considered in Equation (7-2), but these factors could significantly affect the life cycle cost of the pavement.

Ideally, further research is needed to consider all of these factors in addition to maintenance costs, user costs, and the effects of multiple cycles of interlayer use.

7. FIELD CONSTRUCTION GUIDELINES AND PROJECT SELECTION CRITERIA

One major goal of this research is the development of project selection criteria and field construction guidelines. These components are critical to engineers for both the design and construction of paving projects that take advantage of crack-mitigating systems.

7.1 Construction Guidelines

The construction guidelines developed during the course of this research are based on information obtained from the literature review, NCDOT surveys, field construction, and conversations with project participants (e.g., contractors, NCDOT personnel, and interlayer manufacturers). The final version of these guidelines can be found in Appendix VII of this report.

7.2 Project Selection Criteria

The inability of the laboratory tests to simulate field loading conditions accurately means that only general behavior of these materials was observed. As such, no firm recommendations or predictive models were developed for use by field engineers to assist in selecting candidate projects for interlayer usage in North Carolina.

The best resource for guidelines that incorporate the selection and use of interlayer treatments by highway agencies for actual field projects is information found in the literature. Perhaps the most complete and easily understood publication in this regard is California Transit Authority's *Maintenance Technical Advisory Guide* (MTAG), Volume 1 (Shatnawi 2008). Chapter 12 of the MTAG specifically addresses the selection and use of interlayers. In addition to interlayers, this guide provides information about various types of surface treatments that can be used with

interlayers, including chip seal placed over a paving fabric, microsurfacing, slurry seals, etc. As such, the MTAG goes above and beyond the materials tested in the NCDOT study. Also, because climate, specifications, traffic, and maintenance practices are different in North Carolina than in California, care must be taken when referencing the MTAG for North Carolina (or other state) projects. Even so, the MTAG is still a good starting point for engineers who are considering an interlayer treatment for any paving project.

Although sufficient space is not available in this summary to present all of the information contained within the MTAG that is relevant to this report, general observations can be made regarding the recommendations presented in the MTAG tables. The following points are paraphrased from the MTAG.

- Most interlayer types require a minimum overlay of 1.5 inches, with certain products (particularly geocomposites) requiring an overlay of 2.0 inches.
- None of the interlayer types is recommended for fatigue-related alligator cracking that is due to a weak pavement structure.
- Most interlayer types are recommended for low severity oxidation-related alligator cracking, except geocomposite strips due to their high cost and impracticality of placing them over large areas.
- Leveling courses generally are needed for all applications of grid or other reinforcing layers, mainly because such layers need to be free from sags and wrinkles that a rough surface texture would cause.
- If oxidation-related alligator cracking is excessive, leveling courses are required for all interlayer types to prevent excessive loss of the tack coat into existing cracks.
- For block cracking, longitudinal cracking, and non-thermal transverse cracking, all interlayer types are acceptable if the cracks are less than 0.5 inch wide. For wider cracks (between 0.5 in. and 1.0 in.), crack filling is needed prior to the placement of the overlay.
- Thermal Cracking:
 - For low thermal cracking (widths less than 0.25 in.), all interlayer types are acceptable.
 - For thermal cracking with crack widths between 0.25 inch and 0.5 inch, geosynthetic interlayer types are acceptable; however, treatments with grids are generally more effective.
 - For thermal cracking of high severity (widths greater than 0.5 in.), only grids and composites are recommended; the use of other interlayers is discouraged.
- All interlayer types are recommended for pavements that have a problem with moisture intrusion, provided that the interlayers have binder application rates that are high enough to act as a waterproofing barrier.
- In general, all geosynthetic interlayers are found to be acceptable for all traffic levels in all climate conditions present in California, and BST interlayer treatments are acceptable for all but high traffic conditions (30M AADT).

The MTAG also presents cost information for the various interlayer types that can be used to obtain a rough idea of their relative cost for various sizes of projects. However, their cost in North Carolina might be significantly different than in California where interlayer use is more common.

Several important tables from the MTAG interlayer chapter are presented in Appendix VIII of this report, and the full MTAG can be found on the CalTrans website (Shatnawi 2008).

8. PRACTICAL IMPLICATIONS OF THE RESULTS

Several important facts may affect the way the results of this study correlate with engineering practice. The first is the tendency of interlayer systems to experience high levels of interfacial movement and cracking in laboratory tests. Because large-scale debonding (especially near the surface) is a worse distress than a single reflective crack, concerns have been raised that the use of interlayer systems could increase the risk of other types of pavement failure in the field. However, it should be noted that the strain levels used in the laboratory tests in this project may be unrealistically high; therefore, although extensive distributed interfacial damage has occurred during the laboratory tests, it is unclear as to how closely such an occurrence correlates to field behavior.

The second influential fact with regard to the implications of the study results concerns the limitations associated with placing interlayer materials under extreme conditions. It is recommended to avoid areas with steep grades, intersections where vehicles would brake often and possibly harshly, sharp curves, and pavements with poor load transfer across existing joints or cracks. Such conditions may restrict the potential use of these interlayer products when taking into consideration the thin overlays in North Carolina.

Finally, the results of this study, as well as the literature, suggest that greater benefits may be derived from placing interlayer materials deeper into the pavement structure, for the following reasons:

- Interlayer systems placed deeper in the pavement structure are subjected to less shear stress than those placed shallower in the structure. Less shear stress is desirable, as all interlayer systems tend to experience reduced shear strength between the pavement layers. Although the literature shows that interlayer systems meet acceptable shear strength values, these values empirically relate the monotonic test results to the long-term durability of the pavements under repeated traffic and environmental loads. For this reason, placing interlayer systems deep into the pavement structure may reduce the risk of large-scale interfacial damage.
- 2. Placing interlayer systems deep within the structure allows multiple subsequent pavement treatments to be constructed without removing the interlayer system, which has two major benefits:
 - First, many interlayer systems have the ability to reduce the permeability of cracks even after the cracks have propagated to the pavement surface. A decrease in

permeability can contribute to a desired decrease in the moisture content of the subgrade and increased pavement strength. If multiple surface treatments are constructed over the life of a pavement, leaving the interlayers in place may provide a much higher return on investment than simply removing the interlayers with each treatment (although removing the interlayers may indeed be necessary if the interlayers were placed under the surface course).

• Second, the unknown effects of interlayer materials in RAP, which would occur when the interlayer system is milled together with the surface course, can be avoided. Although the product brochures and installation guidelines for the materials used in this study all state that these materials are both millable and recyclable, no references were found in these documents that cite scientific studies that have investigated the mechanical properties of the RAP obtained from pavements with and without interlayer materials. Also, not all manufacturers make the claim that their materials are both millable and recyclable, and so, such products must be evaluated on a case by case basis. For example, it is known that steel mesh interlayers must be removed completely before milling (Veys 1996).

All of these facts together suggest that the placement of interlayers under thin overlays may not be the most effective use of these products. However, this assessment may change if there are plans to continue to increase roadway thicknesses with subsequent projects (i.e., another overlay or surface treatment without milling) sometime in the future. In any case, care must be taken when evaluating the shear stress that an interlayer system will experience over its lifetime in order to ensure that the product is not placed under conditions where the potential for failure is likely.

9. CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are presented based on the reflective cracking mitigation study results and findings:

Field Investigation and Literature Review Conclusions:

- Further monitoring of the pavement test sections is essential in order to compare the effectiveness of the interlayer systems. However, due to the likelihood that the tack coat application rates were lower than optimal rates, the performance of the interlayer sections should not be considered to be indicative of the behavior of properly constructed interlayer systems.
- The placement of any interlayer system on a slope should be considered carefully. In the trial project, the placement of Paving Mat #2 on a steep grade resulted in serious pavement distresses due to slippage during compaction of the overlay. It is believed that the slope of the roadway was a more important contributing factor to the cause of these distresses than the particular interlayer type placed in this location.

- The placement of interlayer systems does not result in significant construction delays if hot asphalt binder is used as a tack coat for the interlayer system. If emulsion is used, curing is the most important factor that will affect the paving schedule.
- The selection of an experienced contractor to place the interlayers is critical, as defects associated with poor interlayer placement will degrade the overall effectiveness of the interlayers and may result in premature failure of the pavement.
- Following best paving practices is also essential to reduce variability at a project site and achieve the maximum benefit from the interlayer system. Areas of poor compaction, surface contamination, or segregation will reduce the benefits afforded by the placement of interlayers.
- Interlayers should be placed over the full roadway width in order to achieve maximum benefits from their waterproofing capability. Because the field trial segments were placed only in a single lane, any benefits in this regard may not be realized, thereby masking the true potential of the interlayer system to improve pavement performance.

Laboratory Investigation Conclusions:

- Two test methods, the NBF test and the RCT, were developed to evaluate layered asphalt systems in the laboratory using DIC.
- Both test methods demonstrated that interfacial movement was a significant part of the behavior of the layer systems evaluated in the laboratory.
- The NBF test results indicate that the interlayer systems tended to increase the pavement's resistance to vertical cracking, but at the expense of increased horizontal cracking.
- While severe interfacial debonding in the field would likely represent a worse situation for long term pavement durability than reflective cracking in the field, it is not known how widespread these types of distresses are in field pavements as compared to the laboratory tests.
- Several criteria were identified that correlated to the number of cycles to reach full-depth cracking in the NBF test; however, the number of cycles to failure could not be used to rank the materials objectively due to the inability of the NBF test to capture both horizontal and vertical cracking adequately without the use of non-standard measurement techniques, such as DIC.
- The RCT results were inconclusive due to the scaled-down nature of the test and the extreme differential deflections experienced during these tests, although the results did illustrate the potential negative effects of weakly bonded pavements in situations with high differential deflections.
- Due to the complex nature of the RCT, this test is not recommended for use as a routine test method.

- The difference in the boundary conditions and strain levels between field pavements and laboratory test conditions makes direct comparisons of pavement performance difficult without advanced numerical modeling.
- The shear test results indicate a noticeable reduction in shear strength between the layers for all interlayer types tested.
- The shear test results indicate that the shear strength values of the field geosynthetic specimens were consistently lower than those of the same geosynthetic systems fabricated in the laboratory, possibly indicating the effect of an inadequate tack coat application rate for the field samples.
- The field sections with the lowest shear strength values also saw the highest level of post construction distresses, demonstrating the importance of shear strength between asphalt layers.
- The reduced shear strength of the field interlayer samples along with the increase in distresses suggests that accelerated failure of the research segments is expected to occur in the future.
- Although an empirical relationship was developed to correlate shear strength and beam fatigue behavior (considering both vertical and horizontal cracking), not enough data were present to ascertain if this relationship would hold for a wide variety of materials and test conditions, or to determine if the thresholds used in developing this criterion were adequate at describing field behavior. Additional tests and research are needed to develop a more reliable relationship.
- The recommended procedure for evaluating interlayer products is a two-fold approach. First, by running NBF tests, the reduced energy ratio can be used to quantify the number of cycles to vertical cracking failure and determine if it is greater than that expected from control specimens. Next, shear tests can be employed to screen interlayers by use of a shear strength threshold, with materials failing to achieve this strength threshold being rejected.
- Ideally, shear testing can be improved in the future by developing and incorporating numerical modeling and shear strength mastercurves for different interlayer types and tack coat conditions, thereby allowing better correlations among simulations, test results, and field behavior.

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APPENDIX I: FULL LITERATURE REVIEW

Introduction

As pavement systems age, distresses such as cracking, rutting, raveling, and polishing of the surface aggregate impede the pavement's ability to carry the daily traffic demand safely, comfortably, and effectively. Each year highway agencies spend billions of dollars on pavement repair and rehabilitation in order to keep these roadways in an acceptable condition for the traveling public. A commonly cited statistic is that 94 percent of the over 2.27 million miles of roads throughout the United States are surfaced with asphalt concrete (AC). Therefore, the maintenance and rehabilitation of these roads consume a significant portion of highway agencies' transportation budgets.

One common form of rehabilitation for these roadways is the use of thin AC overlays. These overlays usually consist of one or two courses placed on an existing pavement that may or may not have been cold-milled prior to the placement of the overlay. These treatments are widely used because of their ability to provide a new wearing surface while still taking advantage of the remaining fatigue life and load-carrying capacity of the existing pavement. However, in many cases, stress concentrations due to cracks in the existing pavement cause the formation and rapid propagation of cracks in the overlay. Because these cracks exhibit the same pattern as those in the underlying pavement, this phenomenon is known as *reflective cracking*.

The major problem with reflective cracks is that they allow water to enter the pavement structure. Thus, their prevalence in the overlay can be a significant contributing factor to the further deterioration of the overall pavement structure. For this reason, the topic of reflective cracking mitigation in AC overlays has been researched extensively over the last several decades. Many studies and construction projects have been undertaken that show varying degrees of success. A comprehensive literature review has been performed as part of this research project in order to obtain insight into the causes of reflective cracking, potential strategies for its mitigation, and details regarding the design and placement of the crack mitigation systems.

Mechanisms of Reflective Cracking

In order to implement strategies that can mitigate reflective cracking successfully, a clear understanding of the mechanisms involved is needed. On a fundamental level, fracture mechanics can be used to understand the phenomenon of reflective cracking. In any material subjected to strain, the growth of cracks may involve a combination of any of the three modes of crack propagation, as shown in Figure I-1. Modes I and II are of primary significance to the phenomenon of reflective cracking (Elseifi 2003). The relative contribution of each of these cracking modes to the overall crack propagation in an AC overlay is largely dependent on traffic loading and temperature changes within the pavement, and is site-dependent (de Bondt 1999).



Figure I-1. Modes of crack propagation.

Traffic

Because traffic loading is of primary importance to the adequate design of pavement systems, it is no surprise that it is also a major contributor to the phenomenon of reflective cracking. Lytton (1989) described three stress concentration pulses that occur at the crack tip as the crack propagates through the overlay. Figure I-2 illustrates that as a wheel moves across a crack location, the overlay experiences two maximum shear stress pulses (Mode II at Points A and C) and one maximum bending stress pulse (Mode I at Point B). The intensities of these stress concentrations are affected by the material properties of the pavement layers, the maximum deflection experienced at the crack location, and the load transfer across the crack.



POSITION OF WHEEL LOAD

Figure I-2. Shear and bending stresses induced at a crack caused by a moving wheel load. (Lytton 1989).

Heavier loads and less resilient pavement structures can increase the amount of strain experienced at the crack location, which causes a corresponding increase in the rate of the crack propagation. Various studies and reports show that a high load transfer efficiency across an existing joint or crack reduces reflective cracking (Hughes 1975, Mascunana 1981, McGhee 1983, Maurer 1989, Barksdale 1991, Mukhtar 1996, Maxim 1997, de Bondt 1999, Carmichael 1999, Bischoff 2007). For flexible pavement systems, this load transfer is achieved by the interlocking of the aggregate across the crack. At wide crack locations, however, no aggregate interlock is possible, and so, no load transfer occurs. De Bondt (1999) demonstrated that new cracks in asphalt pavements can resist shear loading; however, this load-carrying capacity is reduced with repeated traffic load applications. De Bondt (1999) also found that the amount of load transfer across the crack depends on aggregate angularity and normal pressure.

Environmental Conditions

Another major factor that affects the performance of pavements is the environmental conditions under which the pavement is placed. Temperature, annual rainfall, subgrade modulus, and drainage are all important considerations when determining the long-term performance of a pavement structure. Although subgrade and drainage effects can be minimized by proper design of the pavement structure, the effects of temperature and annual rainfall are less controllable.

Because AC is a temperature-dependent visco-elastic material, a pavement's temperature has a significant impact on its mechanical properties. As can be expected, the phenomenon of reflective cracking also is affected significantly by the service temperature range of the overlay. At high temperatures asphalt binder is able to dissipate applied loading via visco-plastic deformation. This capability has the effect of reducing the propagation of cracks. However, as pavement temperatures decrease, the AC becomes stiffer and is less able to resist crack propagation.

Furthermore, like most materials, AC experiences thermal contraction when it cools. As the rate of contraction exceeds the rate of elongation due to the viscous flow of the asphalt binder, tensile stress develops in the pavement. When the stress in the pavement exceeds the tensile strength of the AC layer, a low temperature crack develops in the pavement. These low temperature cracks tend to be perpendicular to the traffic direction and evenly spaced. De Bondt (1999) demonstrated through numerical modeling that when an overlay is placed over an existing cracked pavement, thermal stresses are concentrated at the locations of the existing cracks. Thus, in cold climates, thermal cracking may be the dominant cause of reflective cracking. Thermal cracking is affected by a wide range of factors, including the material properties of the overlay and underlying pavement, the length of the underlying slab, the interface characteristics between the pavement layers, and the magnitude of temperature differences throughout the pavement (de Bondt 1999). It also has been found that as thermal-induced crack widths increase, crack mitigation techniques generally become less effective (Lytton 1989).

In addition to thermal-induced tensile forces, thermal curling stresses are also present in pavements. During a low temperature event, a thermal gradient forms in the pavement structure, with the surface being the coldest part of the structure and the subsurface being comparatively

warm. Thus, the surface layers experience more thermal contraction than the lower layers. Figure I-3, originally presented by Lytton (1989), illustrates this effect.



Figure I-3. Curling of pavement due to thermal gradient (Lytton 1989).

Methods of Crack Mitigation

Given the complex nature of the reflective cracking phenomenon, numerous approaches to the problem have been attempted over the years. Because the intent of this study is to concentrate on reflective cracking mitigation for overlays on low to medium volume flexible pavements, only those treatments that are applicable for such applications are discussed in this report. In this regard, the placement of a thick overlay and the use of interlayers are the two most popular methods to reduce reflective cracking.

Thick Overlay

One common method of reducing reflective cracking is the placement of a relatively thick overlay (Barksdale 1991). Increasing the thickness of the overlay can decrease the rate of the reflective cracks that reach the surface in two ways. Because reflective cracks typically are assumed to propagate approximately 1 inch to 1.5 inches per year, it is reasonable to expect that it will take longer for cracks to penetrate the full depth of a thicker layer than a thinner layer (Gulen 2000, Makowski 2005). Furthermore, thick overlays can help reduce the overall stress experienced in the overlay and can thus slow crack initiation and propagation.

Although thick overlays may reduce reflective cracking, they can be less desirable than other mitigation strategies for several reasons, the most obvious of which is that thick overlays are more expensive to construct than thin overlays. Doubling the overlay thickness doubles the amount of AC needed and, depending on the lift thickness required for construction, can increase labor costs and the length of time that the road is closed due to construction. Furthermore, although a thin overlay can be placed on an existing surface with minimal increase in the roadway elevation, a thick overlay cannot. Apart from potential overhead clearance issues, a significant increase in the roadway profile can cause problems with the approach slopes for

driveways and intersecting roadways. Also, narrow roadways may experience a noticeable increase in the drop-off height from the edge of the pavement to the ditch.

One method to reduce some of the negative effects of placing a thick AC overlay is cold milling. This practice involves grinding off a certain thickness of the existing pavement surface prior to placing the overlay. This practice allows the thickness of the new AC layer to be increased without significantly increasing the roadway elevation. However, cold milling increases project costs and increases the time of lane closures due to construction. Additionally, if cold milling is performed at a constant cross-slope from the centerline, the thickness removed at the outer edge of the pavement may be considerably more than at the centerline. For thin pavements with highly variable cross-slopes, this process may lead to weakening at the pavement edges and could result in premature failure of the pavement.

With decreasing budgets for highway agencies, increasing traffic volumes, and an increasing number of lane miles of roadway to maintain, more cost-effective solutions to the problem of reflective cracking are sought. One of the most attractive of these possibilities is the use of an interlayer between the existing cracked pavement and the thin AC overlay.

Interlayers

The use of interlayers to reduce reflective cracking is not a new idea. As early as the 1920s, experiments with cotton fabric interlayers were performed in South Carolina (Beckham 1935). Since then, numerous studies and paving projects have been undertaken to investigate the use of interlayers, with varying degrees of success. The reason for the continued interest in interlayer systems is that, in terms of the mechanics of the materials, interlayers theoretically can reduce the occurrence of reflective cracking in AC overlays. The two main modes of crack mitigation that interlayers can provide are stress relief and reinforcement (Lytton 1989). An additional benefit of many interlayer systems is their ability to reduce the infiltration of water into the pavement structure (Mascunana 1981, Smith 1984, Lytton 1989, Barnhart 1989, Buttlar 1999, Missouri DOT 2001, Cleveland 2002, Blankenship 2004).

Modes of Improvement

Stress Relief

The basic principle behind stress-relieving interlayers is the reduction of the stresses that make the cracks propagate. In 1980, Monismith and Coetzee stated that materials that are able to resist high strain at the crack tip may help slow crack growth. Because asphalt binder is such a material, stress-relieving interlayers usually contain high percentages of asphalt binder. The viscous nature of the bitumen allows plastic deformation to occur even at fairly low temperatures. Such deformation reduces the stress intensity at the crack tip and slows the propagation of the crack through the AC layer. Beam fatigue tests have shown that when stressrelieving layers are present, two main forms of cracking occur (Lytton 1989). In the first mode, cracks that propagate from the bottom of the specimen 'stall' at the stress-absorbing layer before continuing to the surface. In the second mode, the crack completely stops in the stress-absorbing layer, and a second crack propagates from the surface to meet the first crack. In order to construct an effective stress-relieving interlayer, careful consideration of the important required properties of the layer is needed. Barksdale (1991) listed constructability, asphalt retention, ductility, durability, and traffic abrasion resistance as important properties for a stress-relieving layer. Although his report specifically focused on fabric interlayers, Barksdale's list of important properties can be applied also to all stress-relieving interlayer systems.

- 1. Constructability: Constructability is always an important factor in any system considered for widespread use. Difficulties during the construction of interlayer systems can affect the overall quality of the construction project and can reduce the performance of the overlay (Barksdale 1991). Also, ease of construction is related directly to costs and economic feasibility.
- 2. Asphalt retention: Because asphalt binder is typically the material used to allow viscoplastic flow in many stress-relieving interlayers, the ability of the interlayer system to retain bitumen is of critical importance. If the asphalt binder in the interlayer is lost to the underlying pavement or to the new overlay, the effectiveness of the stress-relieving layer may also be lost. For this reason, Caltrans (2001) advises against the placement of fabric interlayers in areas with a coarse surface texture, which would include chip seals, milled surfaces, open-graded AC courses, and areas with a substantial amount of distress or patching.
- 3. Ductility: Ductility is very important for any successful stress-relieving layer. Without stress-dissipating deformation, the interlayer cannot act as a stress-relieving layer.
- 4. Durability: Interlayer systems must be able to retain their mechanical properties after being exposed to the temperatures and stresses of the construction process, and they should be able to withstand the demands placed on them during their service life. Also, the ability of an interlayer system to remain intact, even if the overlay itself cracks, can potentially help reduce water infiltration into the pavement structure.
- 5. Traffic abrasion resistance: Similar to durability, resistance to degradation by traffic loading is important if the interlayer system must be subjected to traffic for a short period of time before the overlay is placed on top of the interlayer. Care must be taken to ensure that the interlayer system will not be significantly damaged by the action of the traffic, and best practices indicate that it is preferable to limit the traffic to construction and emergency traffic only.

Reinforcement

The other main mode of crack mitigation that employs interlayers is reinforcement. Reinforcing layers usually consist of high stiffness, high strength materials. The principle behind their use is to reduce stress in the overlay itself by allowing strong materials to carry some of the load that normally would be carried completely by the AC. Lytton (1989) states that for an interlayer to function as reinforcement, it must be stiffer than the AC layers. Common examples of reinforcements include polymer fabrics, polymer or fiberglass grids, and steel reinforcement mesh. Lytton (1989) states that cracks that propagate in the reinforcing mode are different from cracks seen in stress-relieving interlayers. He found that cracks that propagate upward through

the overlay will turn 90 degrees and propagate horizontally between the reinforcing layer and the AC, causing debonding of the interlayer. This debonding occurs over a finite length until the stress intensity is reduced and the crack can no longer propagate. Research has shown that with repeated loading, cracks can move through the reinforcing layer and eventually propagate to the surface (Sprague 1998, Kuo 2003, Khodaii 2009). Because excessive debonding in field pavements can lead to accelerated failure of the pavement, achieving a good bond between the reinforcing layer and the surrounding material is necessary for a reinforcing layer to perform its intended function (de Bondt 1999).

Also, because sags and wrinkles in the layer will significantly reduce the layer's ability to carry loads at low strain levels, pretensioning the reinforcing layer is important. This process can be complicated if the surface of the existing pavement is excessively rough or uneven. For this reason, the use of a leveling course between the existing pavement and the interlayer is usually required (Mascunana 1981, Cleveland 2002). Leveling courses can also help prevent the loss of the tack coat at the crack location and lead to more consistent bonding of the interlayer throughout the project.

Reduction of Water Infiltration

Another major consideration of pavement design is drainage. Adequate removal of water from pavement substructures is critical to the long-term performance of pavement systems. Cracks in the pavement allow surface water to enter the pavement layers, which weakens the pavement structure and decreases the fatigue resistance of the pavement system. Because interlayer systems can help delay reflective cracking through either the stress-relieving mode or the reinforcement mode, they can initially reduce the amount of surface water infiltration into the pavement structure. Furthermore, research suggests that even when interlayers are unable to prevent reflective cracking, many interlayers have the ability to remain intact and reduce the permeability of the cracks, possibly allowing for reduction of the subgrade moisture content and an increase in strength (Mascunana 1981, Smith 1984, Lytton 1989, Barnhart 1989, Buttlar 1999, Missouri DOT 2001, Cleveland 2002, Blankenship 2004, Makowski 2005, Bennert 2009).

Products Used as Interlayers

Although many different products can be used as interlayers, general categories have been developed to describe them. Of these, geosynthetics and stress-absorbing interlayers are by far the most commonly used with thin asphalt overlays.

Geosynthetics

One of the most popular types of products used in the mitigation of reflective cracking of asphalt overlays is geosynthetics. These products are manufactured planar materials (frequently made of polymer or fiberglass), typically shipped to the job site in rolls. Of the various types of geosynthetics available, geotextiles, geogrids, and geocomposites are the three that are used primarily for reflective crack mitigation in asphalt overlays. These products may be used to provide reinforcement, stress relief, and the reduction of water permeability of cracks. However, due to their relatively high costs, potentially challenging construction laydown, and their potential to reduce the shear strength between the overlay and the existing pavement, care must be taken when applying these materials to any project (Brown 2001). Also, because

geosynthetics vary significantly in their construction and function in the pavement system, the details of their use and design can vary widely from product to product.

Geotextiles: Geotextiles are frequently used in paving applications. Koerner (2005) describes geotextiles as textiles that consist of synthetic fibers and that are used for various applications in engineering, including separation, drainage, filtration, reinforcement, and to a limited extent, containment. As pavement interlayers, geotextiles can perform a combination of stress relief, reinforcement, and water infiltration reduction. The two main categories of geotextiles are woven and nonwoven.

Woven geotextiles: Woven geotextiles are made from synthetic fibers that are woven together like a traditional textile. When these textiles are placed on an existing roadway using a tack coat, they may serve as reinforcement and/or as a waterproofing layer. De Bondt (1999) demonstrated that pullout resistance is a critical factor in determining the effectiveness of a geosynthetic as a reinforcing layer. For woven geotextiles, the main pullout resistance is generated through adhesion. For this reason, the tack coat application rate is a critical factor in their performance (Barksdale 1991). However, because the addition of the fabric reduces the interface shear strength between the overlay and the underlying layers, problems with fabrics have been experienced in areas of high shear forces, such as curves and areas where vehicles stop and start (Barksdale 1991).

Nonwoven geotextiles: Nonwoven geotextiles are made from synthetic fibers that are mechanically, thermally, or chemically bonded together in a random manner (Koerner 2005). The random orientation of the fibers in these textiles generally leads to a substantial thickness and high internal void structure. Due to their low modulus values, little reinforcement benefit is seen from these fabrics at low deformations. These fabrics typically serve as a "vehicle to hold bitumen", which aids in the stress relief function (de Bondt 1999). For cracking mitigation, most of the nonwoven geotextiles used are needle-punched (Koerner 2005). Similar to woven fabrics, the adhesion of the fabric to the surrounding pavement layers is of critical importance for effective performance. The tack coat application rate must be optimized for the fabric layer to ensure that enough tack coat is present to bond the fabric to the surrounding pavement layers, but not too much in order to avoid the potential for slipping or bleeding of the overlay (Barksdale 1991, Fyfe 2010). Smith (1984) developed a simplified equation for estimating the amount of tack coat for a fabric (I-1).

where

$$RTC = 0.05(TW)^{0.30}$$
 I-1

RTC = recommended tack coat rate (gal/yd²), T = fabric thickness (mils), and

 $W = \text{fabric weight } (\text{oz/vd}^2).$

This equation is based on laboratory testing and includes an extra 0.05 gal/yd^2 to account for the tack coat absorption of the existing surface. It should be noted that the values obtained from this equation should be rounded up to the nearest 0.05 gal/yd^2 increment. It should also be noted that most geosynthetic manufacturers will be able to provide suggested binder application rates for their specific products, and that it is best to use these values rather than the above equation.

Although emulsified tack coats often are used in paving applications, hot asphalt binder is preferred as a tack coat for geosynthetics (Smith 1984, Barksdale 1991, Cleveland 2002). The main reasons are summarized by Barksdale (1991): First, large quantities of emulsified tack coat would be required to achieve the desired residual asphalt content, which could cause problems with runoff and uniformity during construction. Second, curing times for the emulsion would increase the construction time. Lastly, cutback asphalts are undesirable due to their potential for damaging the geosynthetics.

Paving mats: A different class of geosynthetics, known as paving mats, has become popular in recent years. These materials are stiff and are intended to act as a barrier to water infiltration and to provide some level of reinforcement to the pavement. These materials may be woven, composite, or nonwoven in construction. Examples of these materials include Paving Mat #1 and Paving Mat #2 used in this study.

Geogrids: Another type of commonly used geosynthetic interlayer is geogrids. Geogrids are planar polymer or fiberglass grids originally developed for soil reinforcement applications. Because of their ability to provide considerable strength at fairly low strain levels, they have been used with varying degrees of success as a reinforcing interlayer beneath AC overlays. Due to their open grid structure, however, geogrids cannot act as a barrier to water infiltration without the use of a supplementary material. As is the case with geotextile reinforcement of overlays, the pullout strength and bonding characteristics of geogrids are important for performance (de Bondt 1999). Most geogrids used beneath asphalt overlays utilize some sort of adhesive, either on the geogrid itself or placed separately (Koerner 2005). In a study of various projects throughout Illinois, von Holdt and Scullion (2006) found that the use of a tack coat is critical in ensuring the adequate bonding of a fiberglass grid to the pavement layers. It should be noted that material type plays an important role in the amount of adhesion present when a geogrid is used within an asphalt layer. Yarn-type geogrids exhibit better bonding performance than smooth strap-type geogrids (Koerner 2005). In addition to adhesion, geogrids develop a significant portion of their strength from the mechanical interlock of the surrounding material through the apertures of the grid. Small grids increase pullout resistance, but the aperture size should be sufficient for the aggregate size (de Bondt 1999). Brown et al. (2001) also note the importance of aperture size in developing strength using grid reinforcement in an AC layer.

Geocomposites: Due to the limitations of some of the individual geosynthetic materials, geocomposites may be used in an attempt to produce a superior material. Geocomposites, as the name implies, are comprised of a combination of two or more geosynthetic materials. For reflective crack mitigation applications, these systems typically are designed to perform reinforcement, stress relief, and reduction of water infiltration functions. A common type of geocomposite consists of a geotextile and a geogrid. The geotextile holds the asphalt binder to aid in the stress relief function, to promote adhesion with the underlying pavement layer, and to reduce the permeability of the interlayer system. The geogrid provides reinforcement to the overlay. Another example of a geocomposite is the interlayer stress-absorbing composite (ISAC) system used in Illinois. This three-layer system consists of a high strength geotextile on top, a layer of rubber-modified asphalt in the middle, and a nonwoven geotextile on the bottom (Vespa 2005). The major disadvantage of geocomposite systems is their relatively high cost.

Geosynthetic Design Considerations

Barksdale (1991) noted several issues that are of critical importance to the overall performance of a paving fabric that are applicable to other types of geosynthetics as well:

- 1. *The type, severity, and extent of the distress in the existing pavement.* Highly distressed pavements generally do not show much improvement from the placement of a geosynthetic interlayer and a thin asphalt overlay (Rollins 1991, Barksdale 1991, Button 2006). Moreover, research suggests that geosynthetics are not particularly effective for mitigating thermal cracking (Ahlrich 1986, Barksdale 1991, Amini 2005). Evidence also suggests that transverse cracking is more difficult to mitigate than longitudinal or alligator cracking (Vallergra 1980, Mascunana 1981, Barnhart 1989, Button 2006).
- 2. The amount of repair work done prior to the placement of the overlay. For best results, both structural deficiencies and wide cracks should be repaired prior to placing an overlay with a geosynthetic interlayer (Vallergra 1980, Lorenz 1987, Button 1989, Carmichael 1999, Caltrans 2001, Fyfe 2010). In laboratory tests that simulate reflective cracking, Khodaii (2009) found that increasing the width of the crack in the existing pavement decreases the number of cycles to failure. Barksdale (1991) noted that wide cracks tend to absorb the tack coat, which can leave the geosynthetic unsupported at the crack. Although the definition of 'wide' varies from source to source, the two most common threshold values are approximately 1/4 inch (Carmichael 1999, Caltrans 2001, Fyfe 2010) and 1/8 inch (Dykes 1980, Ahlrich 1986, Button 1989, Barksdale 1991).
- 3. *The underlying pavement structure.* Geosynthetics do not perform well at reducing reflective cracking when large differential deflections are present due to poor load transfer across the crack or joint in the underlying pavement (Hughes 1975, Mascunana 1981, McGhee 1983, Maurer 1989, Barksdale 1991, Mukhtar 1996, Maxim 1997, de Bondt 1999, Carmichael 1999, Bischoff 2007). Therefore, geosynthetics are not very effective at mitigating reflective cracking over rigid or composite pavements that tend to experience significant differential deflections at the evenly spaced transverse joints and mid-slab cracks. Also, because geosynthetics cannot correct for structural deficiencies of the underlying pavement, their use with a thin asphalt overlay will not provide long-term benefits for pavements that have an inadequate structure and cannot carry the expected traffic loads.
- 4. Climatic effects. Climate plays an important role in reflective crack propagation. In a summary of the use of interlayers, Ahlrich (1986) notes that interlayers tend to perform best in warm climates. He created a map that divides the United States into three climatic zones and offers different recommendations accordingly for the use of interlayers (Figure I-4). Ahlrich's observations fit with those of Caltrans (2001) that notes that fabrics are ineffective in reducing thermal cracking, and with those of Bush and Brooks (2007) who note a significant decrease in the performance of geosynthetic interlayer systems after an extreme low temperature event. Lytton (1989) suggests that the magnitude of crack openings due to thermal effects accounts for this fact. He suggests three ranges of thermal

crack opening sizes that relate to the effectiveness of geosynthetic interlayers for mitigating low temperature reflective cracking. For thermal crack openings between 0.00 inch and 0.03 inch, no geosynthetics are needed; for thermal crack openings between 0.03 inch and 0.07 inch, geosynthetics are effective at reducing reflective cracking; and for thermal crack openings wider than 0.07 inch, geosynthetics generally are not effective.



Overlay thickness is another factor that is cited in the literature and commonly affects the performance of geosynthetic interlayers. In a report of the Geotextile Division of the Industrial Fabrics Association International, Maxim Technologies (1997) states that in cases where fabrics performed poorly, all of the cases involved an overlay that was too thin or an existing pavement that was in poor condition. Barksdale (1991) suggests that two important factors may account for this relatively poor performance of interlayers when used in combination with thin overlays. First, because thin overlays cool more rapidly than thick overlays, they may result in lower densities during construction and, thus, worse overall performance. Second, due to their lower thermal mass compared to thicker overlays, thin overlays are less able to draw the tack coat through the fabric and, thus, may experience bonding problems.

Various sources provide differing recommendations for the overlay thickness to be used with geosynthetic interlayers. As shown in Figure I-4, Ahlrich (1986) recommends a two-inch minimum overlay for pavements in climatic Zone I, as does Barksdale (1991). Caltrans recommends 1.8 inches (2001). Carmichael and Marienfeld (1999) report that case studies in South Africa, Germany, Spain, Belgium, and Austria show little improvement with overlay

thicknesses less than 1.5 inches. This finding agrees with the recommendations of the Industrial Fabrics Association International Nonwoven Paving Fabrics Report (Maxim 1997) and experience reported by Button (1989). Dykes (1980) recommends a minimum overlay thickness of one inch with fabric interlayers.

Stress-Absorbing Layers

Another means of mitigating reflective cracks is a stress-absorbing interlayer. Unlike geosynthetics, which are manufactured and brought to the project site in rolls, stress-absorbing layers typically are constructed in place. These layers include surface treatments (such as chip seals), rubber asphalt layers, and fine-graded AC layers.

Chip seals: Chip seals are popular pavement surface treatments that involve the use of an emulsified asphalt binder covered with aggregate chips. Due to the relatively low cost of chip seals, their ability to improve the surface condition of a paved roadway, and their ability to help seal the existing pavement, highway agencies throughout the United States use chip seal surface treatments to maintain low to medium volume paved roadways. The use of chip seals as an interlayer to help mitigate reflective cracking has been around for some time. It is believed that the chip seal can act as a stress-absorbing layer and provide some level of reduction of water infiltration through cracks in the overlay. Due to their wide usage by highway agencies, chip seals as interlayers are fairly inexpensive, equipment for their placement is widely available, and construction personnel are experienced in their placement. These considerations make chip seals a very attractive option for use as a crack-mitigating interlayer. Several NCDOT Divisions have used chip seal (or chip seal) interlayers and report good performance.

Asphalt rubber stress-absorbing membrane interlayers (SAMIs): Similar in construction to chip seals, asphalt rubber stress-absorbing membrane interlayers (SAMIs) are another method of reflective crack mitigation. These systems resemble conventional chip seals, which consist of a layer of binder and a layer of cover aggregate chips. The main difference between the systems is that the asphalt rubber system uses asphalt rubber instead of an emulsified binder. Asphalt rubber is produced by reacting finely ground rubber (typically from used tires) and asphalt binder (Arizona DOT). Extender oils often are used to control the viscosity of the asphalt rubber (Hicks 2002). Because asphalt rubber utilizes a waste material to modify the properties of the asphalt binder, its usage has been encouraged by policy and legislation for decades. For this reason, an extensive amount of research has been conducted using asphalt rubber. Although its high cost (up to twice as much as conventional binder) has kept the use of this product fairly limited, valuable information about the performance of SAMIs can be gained from these studies (Zaniewski 1988, Allison 1989, Estakhri 1990, Epps 1994, Maupin 1997).

Projects that have utilized asphalt rubber SAMIs can provide considerable insight into the various factors that affect SAMI design and performance. From these reports, it is found that SAMIs are sensitive to many of the same factors that affect geosynthetic interlayers. The existing pavement conditions, the type and amount of repair work undertaken, the underlying pavement structure, and climate all affect SAMI performance.

As with geosynthetics, the type and level of pavement distress are important to the successful use of a SAMI. Research indicates that SAMIs are effective in mitigating alligator cracking but not

effective in mitigating transverse cracking (Estakhri 1990, Anderson 1992, Morris 1993). Also, the amount of repair work undertaken prior to the placement of a SAMI is important. Much like geosynthetics, the sealing of cracks wider than 1/4 inch is recommended (Hicks 2002, Chehab 2007b). If these wide cracks are left untreated, the binder can be absorbed into the crack, eventually leading to an area of distress near the existing crack (Epps 1994). Furthermore, the underlying pavement structure should be considered when attempting to utilize a SAMI. Experience has shown that SAMIs are not effective for overlays on rigid pavement (Epps 1994). Also, research indicates that SAMIs alone will not correct underlying structural deficiencies of the pavement (Ahlrich 1986). Lastly, the climatic conditions of the site where the SAMI is to be placed are also important. Epps (1994) notes that experience shows that SAMIs are more effective in warm climates than cold ones.

The design of a SAMI is controlled primarily by the quantities of its two main components: binder and aggregate. Thick SAMIs are achieved through high binder application rates and allow reduction of the stresses that lead to crack propagation (Jimenez 1985). For crack mitigation alone, Jimenez and Meier (1985) found that a SAMI without aggregate chips provides slightly more resistance to cracking than a SAMI of the same thickness with aggregate chips. These researchers believe that the aggregate particles in the SAMI cause stress concentrations, and by eliminating them, the SAMI with no aggregate will exhibit better performance than the SAMI with aggregate. Although this observation is important to understanding the effects of a SAMI on reflective crack propagation, it does not carry over into a recommendation for actual construction practice. From a practical standpoint, some cover aggregate is needed with any SAMI. Without cover aggregate, paving equipment would be required to drive on a surface of pure binder during construction. Moreover, without adequate chip coverage, bleeding, rutting, or slipping of the overlay may occur (Barksdale 1991, Epps 1994). As noted by Hicks (2002), in order to determine the proper application rates for the binder and the cover aggregate, information about the existing pavement should be considered. Roadways with a rough surface texture or a highly porous existing pavement require a higher binder application rate than those with a smooth texture and less porosity. Also, due to differences in traffic consolidation of the underlying pavement, roadways that experience low surface temperatures or have low traffic volumes require more binder than those with high pavement temperatures or high traffic volumes (Hicks 2002). Also, the size and uniformity of the cover aggregate are important factors. As is the case with traditional chip seals, a uniform layer of coarse aggregate is most desirable (Schnormeir undated). Holtrop (undated) suggests aggregate sizes of 10 mm to 14 mm for SAMI applications. It should be noted that large uniform cover aggregate requires high binder application rates, which can increase the cost of a SAMI (Hicks 2002).

Fiberglass-reinforced SAMIs: Some proprietary systems that are similar in function to chip seals also are available. For such systems, a fiberglass-reinforced SAMI is constructed by placing a layer of emulsion followed by a layer of chopped fiberglass fibers, a second layer of emulsion, and finally a cover layer of aggregate chips. The inclusion of the fibers increases the stiffness of these treatments and may help to bridge cracks in the underlying pavement. Additionally, for interlayer applications, this increased tensile strength is intended to help reduce or delay reflective cracking in AC overlays. As with the other treatments, experience indicates that wide cracks should be filled prior to the placement of a fiberglass-reinforced SAMI interlayer system (Chehab 2007b).

Fine-graded AC layers: Another category of stress-absorbing interlayers is fine-graded AC layers with high asphalt content. Typically, these mixes use at least seven percent polymer-modified asphalt binder (Blankenship 2004). The fine gradation and increased binder content of these layers allow them to absorb some of the stresses that cause reflective cracking (Zhou 2007). Some proprietary systems such as fiberglass-reinforced chip seals are included in this class of SAMIs.

Performance

As mentioned previously, reflective cracking and reflective crack mitigation have been a major topic of study for decades. As such, a wide variety of performance information is available about all types of interlayer treatment systems.

Geosynthetics

The extensive use of geosynthetics has resulted in a large number of performance case studies. Although each case is different when considering existing pavement buildup, the distress level of the existing pavement prior to treatment, traffic levels, environmental conditions, pavement treatment performed, subgrade modulus, site drainage, and specific materials used, some general observations regarding performance nevertheless can be made. Given favorable conditions and adequate construction practices, geosynthetic interlayers have the ability to delay reflective cracking for two to five years, but usually do not prevent it altogether (Hughes 1977, McGhee 1983, Barnhart 1984, Ahlrich 1986, Lorenz 1987, Button 1989, Barksdale 1991, Buttlar 1999, Vespa 2005, Bush 2007). Also, many reports suggest that geosynthetic interlayers can reduce the severity of reflective cracks when such cracks eventually propagate to the surface (McGhee 1983, Bush 2007). Although fabrics comprise a considerable portion of the available research, geogrids also have been shown to improve resistance to reflective cracking in laboratory tests (Khodaii 2009).

One notable exception to the overall favorable reports of geosynthetics is the use of fiberglass reinforcement. Although studies by Darling and Woolstencroft (2000) and Bush and Brooks (2007) show good performance of fiberglass grid products, the results of other studies are not as encouraging. In a statewide review of projects in Louisiana, Elseifi and Bandaru (2011) found that only 38 percent of the fiberglass-reinforced overlays showed an improvement in service life and that 62 percent showed "disimprovement". Texas also has experienced debonding and deterioration problems with fiberglass grids used in various projects throughout the state (von Holdt 2006). A study in Illinois involving overlays of high traffic rigid pavements indicates poor performance despite the fact that all requirements listed by the manufacturer were met (Pfeifer 1995). These differences in performance demonstrate the importance of proper construction of these interlayer products and the risks involved if they are applied improperly or in situations where they will not be effective.

Chip Seals, SAMIs, and Fine-Graded Asphalt Concrete Layers

The performance of the various types of stress-absorbing layers is very similar to the performance of geosynthetics. In general, these systems show an ability to help reduce reflective cracking for three to five years (Barksdale 1991). These treatments can also reduce crack

severity, even though reflective cracking cannot be prevented altogether (Chen 1982, Ahlrich 1986, Epps 1994).

Interlayer treatments that consist of a layer of bitumen covered with aggregate chips largely show good performance. Elseifi and Bandaru (2011) studied pavement projects throughout Louisiana and found that 58 percent of the projects that used chip seal interlayers showed improved performance. Fiberglass-reinforced SAMIs also have been shown to improve resistance to simulated thermal loading in laboratory tests (Chowdhury 2007). Many reports indicate that asphalt rubber SAMIs have the ability to reduce reflective cracking for several years (Vallergra 1980, Ahlrich 1986, Peters 1987, Zaniewski 1988, Epps 1994, Estakhri 1990, Estakhri 1994).

Although possibly outside the range of alternatives to be used for thin asphalt overlays on low volume roadways, fine-graded asphalt layers also have been shown to perform well. Makowski et al. (2005) performed a study involving the placement of a fine-graded, one-inch thick, high asphalt content AC interlayer on four rigid pavements with moderate to high traffic volumes in Wisconsin. The total overlay thickness for these projects, including the one-inch interlayer, was between three and five inches. The interlayer was able to delay cracking in three of these four projects. This result fits with findings by Blankenship (2004) that indicate that a fine-graded, high asphalt content layer may help increase reflective cracking resistance. Bischoff (2007) reported that a specific proprietary fine-graded AC layer can delay reflective cracking in high traffic volume concrete highways in Wisconsin for two to three years, although the report does not recommend the system for widespread use. However, Elseifi and Bandaru (2011) found that only one of three projects that used this same proprietary fine-graded AC layer showed a measurable improvement in Louisiana, although they believed that no firm conclusions could be drawn from this limited data set. Also, as with the other types of interlayers described previously, fine-graded AC layers show the ability to remain intact even when the overlay itself has cracked, possibly reducing water infiltration into the pavement structure (Missouri DOT 2001, Makowski 2005, Bennert 2009).

Potential Problems with Interlayer Use

Although interlayers have shown the ability to reduce reflective cracking in AC overlays, their use is not without compromise. The addition of an interlayer increases costs, construction time, and potentially introduces new problems during construction and the service life of the overlay.

Geosynthetics

The use of geosynthetic interlayers introduces several challenges to paving projects. These challenges relate to the adhesion of the overlay to the underlying pavement (i.e., bonding), the placement of the geosynthetic materials, and possible damage to the geosynthetics as a result of construction.

As mentioned previously, the tack coat application rate is very important to geosynthetic performance. An adequate tack coat is needed to avoid debonding of the overlay from the underlying pavement. However, care must be taken with regard to the application, because an excessive tack coat may result in pick-up of the fabric by traffic or paving equipment during construction and can cause slipping, bleeding, and rutting of the overlay during its service life

(Dykes 1980). Although blotting material can be used to alleviate construction problems caused by high tack coat application rates, care must be taken to ensure that a sufficient bond with the overlay is achieved (Barksdale 1991, VicRoads 2001, Vespa 2005, Davis 2010).

The durability of geosynthetic interlayers during construction is also a consideration. Although early studies indicate that traffic may be allowed on geosynthetics for short periods of time during placement, care must be taken to avoid significant damage due to traffic action (Button 1989). Substantial turning movements by heavy vehicles on geosynthetic interlayers during construction can cause damage and should be avoided (Cleveland 2002). For these reasons, best practices indicate that limiting traffic to construction and emergency traffic only is desirable. Also, certain polymers that are used to make geosynthetics, such as polypropylene, can melt if exposed to high temperatures during paving (Barksdale 1991, Caltrans 2001).

Furthermore, because geosynthetics are planar materials that come in rolls, several special concerns regarding their use in a paving project must be considered. First, wrinkling of the geosynthetic is a challenge faced during placement, especially on curved roadways. To avoid premature cracking of the overlay, large wrinkles should be slit and overlapped (Maurer 1989, Cleveland 2002). Depending on laydown practices and contractor experience, wind can become a problem when placing these types of paving fabrics (Lorenz 1987, Cleveland 2002). Also, care must be taken to ensure that adjacent rolls of geosynthetics have sufficient overlap. Cleveland (2002) recommends a six-inch overlap for transverse joints and a four-inch overlap for longitudinal joints. Brown (2005), on the other hand, recommends a four- to eight-inch overlap. Also, individual product manufacturers may have different recommendations for overlap values.

Chip Seals, SAMIs, and Fine-Graded Asphalt Concrete Layers

The construction problems associated with chip seal interlayers and with many SAMIs that involve bitumen membranes covered in aggregate chips are similar to the problems associated with conventional chip seals. As such, the dust and dirt that are generated during construction and the aggregate loss prior to the placement of the overlay are the major concerns associated with the construction of interlayer systems. Also, bleeding or rutting of the overlay may occur if excessive bitumen from the interlayer migrates into the overlay during its service life.

Other factors, such as availability, constructability, and costs, are major considerations when attempting to use atypical stress-absorbing interlayers. Asphalt rubber systems can cost up to twice as much as those that use conventional bitumen materials (Zaniewski 1988, Allison 1989, Estakhri 1990, Epps 1994, Maupin 1997). Asphalt rubber systems are expensive because specialized equipment is needed to produce the asphalt rubber, and such equipment may not be available in all areas. Furthermore, because clogging of distributor nozzles has been reported with asphalt rubber systems in some cases, modified nozzles may be needed to achieve proper application (Meadors 1986, Maupin 1997). Similarly, selecting proprietary interlayer systems that require specialized equipment for placement, such as fiberglass-reinforced SAMIs, may affect the price and feasibility of using such systems for any given project.

Also, the use of innovative materials for various interlayer systems can introduce additional challenges. Workability can be a problem with asphalt rubber SAMIs due to their high binder application rates and narrow placement temperature range (Hicks 2002). Chehab (2007a) notes

that excessive fiber application rates that may be used when placing fiberglass-reinforced SAMIs can negatively affect the aggregate embedment.

Fine-graded AC layers have shown some problems as well, including the potential for rutting of the overlay (Laurent 1996, Missouri DOT 2001).

Conclusions

The stated goal of this research is to investigate the use of interlayers to mitigate reflective cracking in thin, single-course AC overlays on low to medium volume flexible pavements throughout North Carolina. From the literature review, the following conclusions can be drawn:

- 1) The reduction of reflective cracking in asphalt overlays is a complex and difficult challenge that has been investigated for decades. Although many reports show that various interlayer types can reduce the rate and severity of reflective cracking, these treatments are often unable to prevent it altogether.
- 2) Flexible pavements with good support and drainage conditions, narrow existing crack widths, and whose primary distresses do not involve thermal cracking, appear to be the best candidates for the use of reflective crack-mitigating interlayers and thin asphalt overlays.
- 3) The ideal conditions for many interlayer systems involve the use of a leveling course and thick overlays. These requirements may not be compatible with the typical treatments used on low to medium volume roadways in North Carolina.
- 4) Proper selection and placement of any interlayer system are critical factors in ensuring good performance of the overlay. Care must be taken to ensure that the candidate pavements are suitable for the use of particular interlayers and that good construction practices are followed when placing these systems.

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APPENDIX II: NCDOT REFLECTIVE CRACKING MITIGATION SURVEY

Response Summary

The survey that was sent to North Carolina Department of Transportation (NCDOT) Divisions also included information to inform the participants that the survey is part of a NCDOT research project entitled *Performance of Cracking Mitigation Strategies on Cracked Flexible Pavements* and that the objective of this project is to investigate the effectiveness of several interlayer systems in preventing or delaying the appearance of reflective cracking of thin (1.25"-1.5") asphalt overlays on low-to-medium traffic volume flexible pavements. The participants were asked the following questions and told that more than one answer may apply and that they could attach additional sheets, if necessary. The responses are summarized beneath each question below; the completed questionnaires are provided in Section 11.2.

Question 1: For the type of roadway described above, what pavement distresses are most common prior to the placement of a thin (1.25"-1.5") overlay?

The most frequent distresses that were reported are: fatigue cracking (8 responses), longitudinal cracking (6 responses), reflective cracking (5 responses), thermal cracking (5 responses), edge cracking (3 responses), and rutting (2 responses).

Question 2: For the typical overlay described above, how long until you expect to see reflective cracking?

Most of the responses indicated that reflective cracking could be expected within two years.

Question 3: Please list the interlayer treatments that you have used with thin asphalt overlays and rate their effectiveness at reducing reflective cracking. Include specific product and manufacturer if known.

Many of the Divisions reported using various bituminous surface treatments (BSTs) with varying degrees of success.

- Two Divisions reported using split seals and found them to be highly effective (Division 1) and moderately effective (Division 4).
- Several Divisions reported using mat-and-seal, with performance ranging from highly effective (1 response) to moderately effective (2 responses) to slightly effective (1 response).
- Division 10 reported using a #6 stone mat and found it to be slightly effective.

Several Divisions reported using crack seals and found them to be highly effective (1 response), moderately effective (1 response), and slightly effective (2 responses). Division 9 reported that rubber joint seals are not at all effective.

Division 1 reported using a fine-graded hot mix asphalt (HMA) course (S4.5A) and found it to be moderately effective.

Two Divisions reported using fabrics.

- Division 9 reported using Paving Fabric (a nonwoven geotextile) and found it to be slightly effective.
- Division 10 reported using two geocomposite strip treatments: Mirafi MTK (highly effective) and Paveprep (slightly effective).

Division 5 reported using Fibermat® Type B and found it to be moderately effective.

Division 10 reported using the Strata® reflective crack relief system and found it to be highly effective.

Division 11 reported using microsurfacing and slurry seals for reflective cracking mitigation. Upon following up with this Division, it was found that these treatments were not used as interlayer applications, but rather as surface treatments over existing cracked pavements. The performance listed is an indication of the way the surface treatment itself handled reflective cracking, not an indication of its performance as an interlayer under an HMA surface course.

Questions 4 and 5: Were there any notable construction problems related to the reflective crack mitigation treatments? Have the treated sections experienced any abnormal distresses caused by the reflective cracking mitigation treatment?

- Nearly all Divisions reported that crack seals absorbed in the overlay will cause a bump if the overlay is placed on top of the crack seal too early. Some respondents suggested that warm mix over the crack seal is an option for mitigating this problem.
- Division 6 reported delamination caused by an interlayer system.
- Division 1 reported bleeding after a contractor had placed a mat-and-seal interlayer.
- Division 9 reported blistering of joints when an asphalt rubber joint seal was placed.

Question 6: Do you feel the performance of these treatments justifies their additional cost?

Six Divisions reported that interlayer treatments are cost-effective in reducing reflective cracking.

Question 7: Considering performance, constructability, and cost, which reflection cracking mitigation strategy that you have used is the best? Please specify type and manufacturer.

- Five Divisions selected BSTs as the best performing interlayer system.
- Division 5 indicated that BSTs can work, but not in all cases; the Division 5 respondents believe that mill and fill is the best option for heavily cracked roads.
- Division 10 recommended Mirafi MTK for overlays over concrete.
- Division 6 indicated that joint seals are the best reflective cracking mitigation strategy if they are placed at least a year prior to placing the overlay.

Question 8: Are there any reflective cracking treatment products, system, or strategies that you feel should be evaluated in this research project? Please specify type and manufacturer.

- Fibermat® Type B
- Crack seals
- BSTs
- Mirafi MTK
- Flexible microsurfacing

All Responses

Survey Questionnaire

(Please return survey by e-mail to Dr. Judith Corley-Lay, jlay@ncdot.gov)

District/	District/Division Division One/District Two						
Name	C. W. Bridgers, Jr						
Title	District Engineer						
City Ah	oksie, NC						
Phone N	o. 252-332-4021	Email address cwbridgers@ncdot.gov					

This survey is a part of a NCDOT research project *Performance of Cracking Mitigation Strategies on Cracked Flexible Pavements*. The objective of this project is to investigate the effectiveness of several interlayer systems in preventing or delaying the appearance of reflective cracking of thin (1.25"-1.5") asphalt overlays on low-to-medium traffic volume flexible pavements. <u>Please answer the following questions. Mark answers with a check; more than one may apply. Attach additional sheets, if necessary.</u>

1. For the type of roadway described above, what pavement distresses are most common prior to the placement of a thin (1.25"-1.5") overlay?

[X] Fatigue Cracking	[] Bleeding
[] Thermal Cracking	[] Pumping
[] Rutting	[X] Longitudinal Cracking
[] Stripping of Asphalt	[] Edge Cracking
[] Raveling	[X] Reflective Cracking
[] Shoving	
[] Other	

2. For the typical overlay described above, how long until you expect to see reflective cracking?

- [] 1 year
- [X] 2 years
- [] 3 years
- [] 4 years
- [] 5 years
- [] More than 5 years
- [] More than 10 years
- [] Other _____

3. Please list the interlayer treatments that you have used with thin asphalt overlays and rate their effectiveness at reducing reflective cracking. Include specific product and manufacturer if known.

Treatment Name	Manufacturer	Highly Effective	Moderately Effective	Slightly Effective	Not Effective at All
S4.75A @ 75 PSY	Local hot mix		х		
BST (double seal)	NCDOT Bit Unit	Х			

4. Were there any notable construction problems related to the reflective crack mitigation treatments?

As everybody knows, cracksealing shows immediately behind the roller in many cases.

Even if the cracksealant has cured for a year or more, it is not unusual to see cracking within months

after completion of the paving.

5. Have the treated sections experienced any abnormal distresses caused by the reflective cracking mitigation treatment?

[] Slipping

[X] Bleeding this occurred after a contractor applied mat-and-seal

- [] Delamination
- [] Other ______

6. Do you feel the performance of these treatments justifies their additional cost?

BST treatments have performed best here in delaying appearance of cracks in the new mat. We have experienced excellent results with a mat-and-seal on primary routes ahead of 1.5" of S9.5B.

7. Considering performance, constructability, and cost, which reflection cracking mitigation strategy that you have used is the best? Please specify type and manufacturer.

See questions 6.

8. Are there any reflective cracking treatment products, system, or strategies that you feel should be evaluated in this research project? Please specify type and manufacturer.

See question 6. BST is by far has offered the best B/C ratio here.....

Survey Questionnaire

(Please return survey by e-mail to Dr. Judith Corley-Lay, jlay@ncdot.gov)

District/Division Division Four					
Name Tim Little					
Title Division Operations Engineer					
City Wilson					
Phone No.252-237-6164	Email address timlittle@ncdot.gov				

This survey is a part of a NCDOT research project *Performance of Cracking Mitigation Strategies on Cracked Flexible Pavements*. The objective of this project is to investigate the effectiveness of several interlayer systems in preventing or delaying the appearance of reflective cracking of thin (1.25"-1.5") asphalt overlays on low-to-medium traffic volume flexible pavements. <u>Please answer the following questions. Mark answers with a check; more than one may apply. Attach additional sheets, if necessary.</u>

1. For the type of roadway described above, what pavement distresses are most common prior to the placement of a thin (1.25"-1.5") overlay?

2. For the typical overlay described above, how long until you expect to see reflective cracking?

- [] 1 year
- [X] 2 years
- [] 3 years
- [] 4 years
- [] 5 years
- [] More than 5 years
- [] More than 10 years
- [] Other _____

3. Please list the interlayer treatments that you have used with thin asphalt overlays and rate their effectiveness at reducing reflective cracking. Include specific product and manufacturer if known.

Treatment Name	Manufacturer	Highly Effective	Moderately Effective	Slightly Effective	Not Effective at All
Crack Sealing			х		
Split Seals			Х		

4. Were there any notable construction problems related to the reflective crack mitigation treatments?

Obviously the biggest concern is with the crack sealing material being heated during overlay and coming up through the new mat and thus creating a bump in the pavement. This can be mitigated somewhat by best practices during crack sealing such as using a squeegee.

5. Have the treated sections experienced any abnormal distresses caused by the reflective cracking mitigation treatment?

[] Slipping

[] Bleeding

[] Delamination

 Other None that we have noticed.

6. Do you feel the performance of these treatments justifies their additional cost?

In most cases yes, Obviously the crack sealing and split seal prevent water penetration and I

believe the split seal retards reflective cracking and thus increases the life of the overlay if

used in the right areas.

7. Considering performance, constructability, and cost, which reflection cracking mitigation strategy that you have used is the best? Please specify type and manufacturer.

Split sealing prior to overlay.

8. Are there any reflective cracking treatment products, system, or strategies that you feel should be evaluated in this research project? Please specify type and manufacturer.

None that are as cost effective.

Survey Questionnaire

(Please return survey by e-mail to Dr. Judith Corley-Lay, jlay@ncdot.gov)

District	/Division:	FW	E	
Name_	BRANDON	JONES		
Title	DIVISION	MAINTEN	ANCE EN	161NEER
City	DURHAM			-
Phone N	No. 919-220	1-4600	Email address	thjones Enclot. gol

This survey is a part of a NCDOT research project *Performance of Cracking Mitigation Strategies on Cracked Flexible Pavements*. The objective of this project is to investigate the effectiveness of several interlayer systems in preventing or delaying the appearance of reflective cracking of thin (1.25"-1.5") asphalt overlays on low-to-medium traffic volume flexible pavements. <u>Please answer the following questions</u>. Mark answers with a check; more than one may apply. Attach additional sheets, if <u>necessary</u>.

 For the type of roadway described above, what pavement distresses are most common prior to the placement of a thin (1.25"-1.5") overlay?

[V] Eatigue Cracking	[] Bleeding
Thermal Cracking	[] Pumping
[] Rutting	[]/Longitudinal Cracking
 Stripping of Asphalt 	Edge Cracking
[] Raveling	 [] Reflective Cracking
] Shoving	
[] Other	

2. For the typical overlay described above, how long until you expect to see reflective cracking?

[]] 1 year [] 2 years

[] 3 years

[] 4 years

- [] 5 years
- [] More than 5 years

[] More than 10 years

[] Other _____

Please list the interlayer treatments that you have used with thin asphalt overlays and rate their effectiveness at reducing reflective cracking. Include specific product and manufacturer if known.

Treatment Name	Manufacturer	Highly Effective	Moderately Effective	Slightly Effective	Not Effective at All
FIBERMAT	MIDLAND		1		
BST	NCDOT				
CRACK SEAL				V	

We placed BST on top of Fibermat.

4. Were there any notable construction problems related to the reflective crack mitigation treatments? NONE

Waym mix asphalt is probably

5. Have the treated sections experienced any abnormal distresses caused by the reflective cracking mitigation treatment? ND

- [] Slipping
- [] Bleeding
- [] Delamination
- [] Other _____

6. Do you feel the performance of these treatments justifies their additional cost?

Size the extent am Cracking

7. Considering performance, constructability, and cost, which reflection cracking mitigation strategy that you have used is the best? Please specify type and manufacturer.

Minor cracking can simply be over 0 to 31.0 heright road -0

8. Are there any reflective cracking treatment products, system, or strategies that you feel should be evaluated in this research project? Please specify type and manufacturer.

FiberMat

Survey Questionnaire

(Please return survey by e-mail to Dr. Judith Corley-Lay, <u>ilay@ncdot.gov</u>)

District/Division: Division 6						
Name: <u>R.K. (Ken)</u> Murphy, Jr.						
Title: Division Maintenance Engineer						
City: Fayetteville						
Phone No. <u>910-486-1493</u>	Email address: <u>rkmurphy@ncdot.gov</u>					

This survey is a part of a NCDOT research project *Performance of Cracking Mitigation Strategies on Cracked Flexible Pavements*. The objective of this project is to investigate the effectiveness of several interlayer systems in preventing or delaying the appearance of reflective cracking of thin (1.25"-1.5") asphalt overlays on low-to-medium traffic volume flexible pavements. <u>Please answer the following questions</u>. Mark answers with a check; more than one may apply. Attach additional sheets, if <u>necessary</u>.

 For the type of roadway described above, what pavement distresses are most common prior to the placement of a thin (1.25"-1.5") overlay?

[X]	Fatigue Cracking	[]	Bleeding
[X]	Thermal Cracking	[]	Pumping
[]	Rutting	[X]	Longitudinal Cracking
[]	Stripping of Asphalt	[X]	Edge Cracking
[]	Raveling	[X]	Reflective Cracking
[]	Shoving		
[]	Other		

2. For the typical overlay described above, how long until you expect to see reflective cracking?

[]	l year
[]	2 years
[]	3 years

- [] 4 years
- [X] 5 years
- [] More than 5 years
- [] More than 10 years
- [] Other ____

Please list the interlayer treatments that you have used with thin asphalt overlays and rate their effectiveness at reducing reflective cracking. Include specific product and manufacturer if known.

Treatment Name	Manufacturer	Highly Effective	Moderately Effective	Slightly Effective	Not Effective at All
Chip Seal				x	
Joint Seal				x	
AMZ Joint Seal					
(spray patch)				x	

4. Were there any notable construction problems related to the reflective crack mitigation treatments? Joint Seal – With Superpave mix, we have seen an issue with the joint seal material being pulled up due to the lay down temperature. We are attempting to treat cracks a year or more in advance of plant mix resurfacing with a material that has a higher melting point and placed with a squeegee (no thick overband) to help reduce this issue. The introduction of "warm mix" asphalt will also help to minimize the heat pulling material up and causing a bump in the new pavement.

5. Have the treated sections experienced any abnormal distresses caused by the reflective cracking mitigation treatment?

- [] Slipping
- [] Bleeding
- [X] Delamination
- [] Other _____

6. Do you feel the performance of these treatments justifies their additional cost? Yes – even with the problem we are experiencing, it is still worth it to seal the crack ahead of time.

7. Considering performance, constructability, and cost, which reflection cracking mitigation strategy the you have used is the best? Please specify type and manufacturer. Joint seal works best if you can get it applied at least a year ahead as described in the answer to #4.

8. Are there any reflective cracking treatment products, system, or strategies that you feel should be evaluated in this research project? Please specify type and manufacturer. The crack treatment material that has the lowest melting point or highest softening point. We are specifying (Type 1 - ASTM D6690) or AASHTO M324 (softening point of 176 degrees F). We would like to see a combination of this joint sealing material with warm mix asphalt tested along with the same material in reaction to Superpave temperatures. We feel that the combination of highest softening point and the lay down temperature of warm mix asphalt would give the most desirable results for the climate in Division 6.

Survey Questionnaire

(Please return survey by e-mail to Dr. Judith Corley-Lay, jlay@ncdot.gov)

District/Division Division 9		
Name David Spainhour		
Title Division Maintenance Engineer		
City Winston-Salem		
Phone No. 336-703-6500	Email address <u>dspainhour@ncdot.gov</u>	

This survey is a part of a NCDOT research project *Performance of Cracking Mitigation Strategies on Cracked Flexible Pavements*. The objective of this project is to investigate the effectiveness of several interlayer systems in preventing or delaying the appearance of reflective cracking of thin (1.25"-1.5") asphalt overlays on low-to-medium traffic volume flexible pavements. <u>Please answer the following questions. Mark answers with a check; more than one may apply. Attach additional sheets, if necessary.</u>

1. For the type of roadway described above, what pavement distresses are most common prior to the placement of a thin (1.25"-1.5") overlay?

[X] Fatigue Cracking[] Bleeding[X] Thermal Cracking[] Pumping[] Rutting[] Longitudinal Cracking[] Stripping of Asphalt[] Edge Cracking[] Raveling[] Reflective Cracking[] Shoving[] Shoving

[X] Other Rutting or Deformation due to insufficient Base.

2. For the typical overlay described above, how long until you expect to see reflective cracking?

- [] 1 year
- [X] 2 years
- [] 3 years
- [] 4 years
- [] 5 years
- [] More than 5 years
- [] More than 10 years
- [] Other _____

3. Please list the interlayer treatments that you have used with thin asphalt overlays and rate their effectiveness at reducing reflective cracking. Include specific product and manufacturer if known.

Treatment Name	Manufacturer	Highly Effective	Moderately Effective	Slightly Effective	Not Effective at All
Petro Mat	????			XXX	
BST	NCDOT		XXX		
Rubber Crack Seal	Craftco				XXX

4. Were there any notable construction problems related to the reflective crack mitigation treatments? Petro Mat just added step to paving train. Slowed production a little.

BST - Timing of application followed by overlay, stone selection, coordination between NCDOT

and contractor

Rubber- Has given blistering problems regardless of age.

5. Have the treated sections experienced any abnormal distresses caused by the reflective cracking mitigation treatment?

- [] Slipping
- [] Bleeding
- [] Delamination
- [X] Other <u>Blistering up of joints</u>

6. Do you feel the performance of these treatments justifies their additional cost? Petro Mat unsure of costs?? Placed on Old US 52 in Davidson Co. See Emily McGraw

BST - appears to delay crack return a little

Rubber - not really. We've not had good success with the product. Issues on our end??

7. Considering performance, constructability, and cost, which reflection cracking mitigation strategy that you have used is the best? Please specify type and manufacturer.

BST

8. Are there any reflective cracking treatment products, system, or strategies that you feel should be evaluated in this research project? Please specify type and manufacturer.

BST – Stone selection / size stone versus size of crack.

Petro Mat on Old 52. It also appeared to delay the cracks returning. With time passing,

the cracks appear 2 years younger on the treated side.

Survey Questionnaire

(Please return survey by e-mail to Dr. Judith Corley-Lay, jlay@ncdot.gov)

District/Division: <u>10</u>	
Name : Philip Moxley	
Title: DME	
City: <u>Albemarle</u>	
Phone No. 704.983.4430	Email address: ptmoxley@ncdot.gov

This survey is a part of a NCDOT research project *Performance of Cracking Mitigation Strategies on Cracked Flexible Pavements*. The objective of this project is to investigate the effectiveness of several interlayer systems in preventing or delaying the appearance of reflective cracking of thin (1.25"-1.5") asphalt overlays on low-to-medium traffic volume flexible pavements. <u>Please answer the following questions. Mark answers with a check; more than one may apply. Attach additional sheets, if necessary.</u>

1. For the type of roadway described above, what pavement distresses are most common prior to the placement of a thin (1.25"-1.5") overlay?

[x] Fatigue Cracking	[] Bleeding
[] Thermal Cracking	[] Pumping
[] Rutting	[x] Longitudinal Cracking
[] Stripping of Asphalt	[] Edge Cracking
[] Raveling	[x] Reflective Cracking
[] Shoving	
[] Other	

2. For the typical overlay described above, how long until you expect to see reflective cracking?

- [x] 1 year
- [] 2 years
- [] 3 years
- [] 4 years
- [] 5 years
- [] More than 5 years
- [] More than 10 years
- [] Other _____

3. Please list the interlayer treatments that you have used with thin asphalt overlays and rate their effectiveness at reducing reflective cracking. Include specific product and manufacturer if known.

Treatment Name	Manufacturer	Highly Effective	Moderately Effective	Slightly Effective	Not Effective at All
Strata		х			
Mirafi MTK	Mirafi	х			
#6 Mat Stone				х	
PavePrep	Pavetech International			х	

4. Were there any notable construction problems related to the reflective crack mitigation treatments? Strata- While placing material in would draw moisture out of existing cracks creating bubbles in

the mat. Bubbles had to be punctured by hand to release moisture.

Mirafi MTK- None. Had to be careful paver and asphalt trucks did not ravel membrane as they passed over the fabric.

5. Have the treated sections experienced any abnormal distresses caused by the reflective cracking mitigation treatment?

- [] Slipping
- [] Bleeding
- [] Delamination
- [] Other

None_____

6. Do you feel the performance of these treatments justifies their additional cost? Strata- Although performing well since 2003, unit price exceeded \$5/sq. yd. Would have to be the right

candidate to justify.

Mirafi MTK-Performing well past 3 years on severe reflective cracks from concrete pavement. Price is

approx. \$2/LF. Fabric came in 50 feet rolls, 3 feet wide. Benefit is worth the cost.

#6 Mat Stone- Used occasionally on block cracking. Benefit is worth the cost.

Paveprep-Used as trial sections in '04. Don't recommend this product.

7. Considering performance, constructability, and cost, which reflection cracking mitigation strategy that you have used is the best? Please specify type and manufacturer.

Mirafi MTK on reflective cracks with concrete. We also place an intermediate layer on primary routes to

help retard severe reflective cracks from concrete where appropriate. Still recommend a mat stone for block cracking.

8. Are there any reflective cracking treatment products, system, or strategies that you feel should be evaluated in this research project? Please specify type and manufacturer.

Mirafi MTK

Survey Questionnaire

(Please return survey by e-mail to Dr. Judith Corley-Lay, jlay@ncdot.gov)

District/Division : Division 11	
Name: Charles Reinhardt	
Title: Division Maintenance Engineer	
City: North Wilkesboro	
Phone No.: (336) 903-9121	Email address: <u>creinhardt@ncdot.gov</u>

This survey is a part of a NCDOT research project *Performance of Cracking Mitigation Strategies on Cracked Flexible Pavements*. The objective of this project is to investigate the effectiveness of several interlayer systems in preventing or delaying the appearance of reflective cracking of thin (1.25"-1.5") asphalt overlays on low-to-medium traffic volume flexible pavements. <u>Please answer the following questions. Mark answers with a check; more than one may apply. Attach additional sheets, if necessary.</u>

1. For the type of roadway described above, what pavement distresses are most common prior to the placement of a thin (1.25"-1.5") overlay?

[X] Fatigue Cracking[] Bleeding[] Thermal Cracking[] Pumping[] Rutting[X] Longitudinal Cracking[] Stripping of Asphalt[X] Edge Cracking[] Raveling[] Reflective Cracking[] Shoving[] Other ______

2. For the typical overlay described above, how long until you expect to see reflective cracking?

- [] 1 year
- [] 2 years
- [X] 3 years
- [] 4 years
- [] 5 years
- [] More than 5 years
- [] More than 10 years
- [] Other _____

3. Please list the interlayer treatments that you have used with thin asphalt overlays and rate their effectiveness at reducing reflective cracking. Include specific product and manufacturer if known.

Treatment Name	Manufacturer	Highly Effective	Moderately Effective	Slightly Effective	Not Effective at All
Skin Patch (AST Single Seal)	NCDOT	x			
Crack Sealing	NCDOT	Х			
Microsurfacing	Road Science		Х		
Slurry Seal	Slurry Pavers			Х	
4. Were there any notable construction problems related to the reflective crack mitigation treatments? <u>No</u>

5. Have the treated sect	ions experienced any abnormal distresses caused by the reflective cracking
1 Slipping	
] Suppling	
] Delemination	
] Delamination	
5. Do you feel the perfo	ormance of these treatments justifies their additional cost?
b. Do you feel the perfo des	ormance of these treatments justifies their additional cost?
b. Do you feel the perfo des	ormance of these treatments justifies their additional cost?
5. Do you feel the perfo les	ormance of these treatments justifies their additional cost?

7. Considering performance, constructability, and cost, which reflection cracking mitigation strategy that you have used is the best? Please specify type and manufacturer. Skin Patching with a single seal AST with lightweight aggregate performed by NCDOT. This is

the most effective IF we treat the cracks before they become too large. Crack sealing is also

effective but is more time consuming.

8. Are there any reflective cracking treatment products, system, or strategies that you feel should be evaluated in this research project? Please specify type and manufacturer.

Flexible Micro-surfacing - Road Science

APPENDIX III: TEST SECTION LAYOUT DETAILS

Figure III-1 and Figure III-2 present schematic illustrations of the field test sections (listed as Sections 3 through 8 in the field construction notes).



Figure III-1. Research segment layout on US 1.



Figure III-2. Proposed layout of field cores.

Test Section Layout Method

Two hubs were driven into the ground and spaced 10 feet apart (unless otherwise noted). In most cases, the first hub was aligned with an object at the beginning of the test section or at the crack to be tested using the falling weight deflectometer (FWD). Next, a spot was painted on the road, and the distance from each hub to the spot was recorded. Thus, triangulation could be used to locate the exact points after construction. Cases that differed from this procedure are noted in the field notes.

Later, the research team realized that the hubs could possibly be knocked out in the short time between site visits. Thus, short pieces of rebar were pounded into the ground near each hub for additional support. Each rebar was placed such that the hub, the rebar, and the point on the pavement were all in a single line, with the rebar being one foot closer to the spot than the hub (exceptions to this practice were noted). Seven sections were laid out in this manner.

With the help of NCDOT personnel, GPS surveying equipment was used to obtain the coordinates of the hubs so that the test sections could be relocated in the future, even if the short pieces of rebar have been removed or disturbed. Note that the first two sections laid out for this survey were unused during construction. Therefore, all of the research segments were located in sections that are labeled 3 through 7.



Figure III-3. Field layout details.

Test Section Layout Field Notes

Test section layout field notes include the following points:

- Hub A is always aligned with the position of the crack/section line, and Hub B is along the direction of travel (northbound) unless mentioned otherwise.
- The distance between Hubs A and B is always 10 feet unless mentioned otherwise.
- Rebar is located one foot from the hub unless mentioned otherwise.
- The first two sections laid out for this survey were unused during construction; therefore, all of the research segments were located in sections labeled 3 through 7.

Station	Description	Hub A	Hub B			
0+00	Section 3 starts.	7.12'	10.88'			
2+57.6	Severe Transverse Crack	15.65'	18.68'			
4+88.2 Moderate Transverse Crack		15.43'	18.35'			
6+40.2	Moderate Transverse Crack	18.02'	19.42'			
8+00	End of Section 3	15.45'	17.77'			
Section 4 starts 131' after Section 3 ends.						
0+00	Section 4 starts.	16.25'	19.65'			
3+03.5	Severe Transverse Crack	14.25'	16.94'			
<i>B</i> is right next to the white oak near the cemetery.						
5+13.5	Moderate Transverse Crack	15.19'	15.54'			
6+70.6'	Transverse Crack	17.1'	19.91'			
8+00	End of Section 4					
	Section 5 starts 54.4' after Sect	ion 4 ends.				
0+00	Section 5 starts.	8.28'	12.41'			
2+18	Severe Transverse Crack	11.53'	15.83'			
3+46.5	Moderate Transverse Crack	14.87'	17.70'			
5+19	Moderate Transverse Crack	15.35'	17.65'			
The positions of A and B were switched, and the distance between A and B was 9 ft.						
	Rebar B is 2 ft. from hi	ıb.	-			
8+00	End of Section 5	27.7'	50.97'			
A is the south-west corner of the stone light support.						
<i>B</i> is defined as the nail-tack on the electric pole.						
Section 6 starts 56.3' after Section 5 ends.						
0+00	Section 6 starts.	19.95'	40.82'			
A is the north-west corner of the stone light support.						
<i>B</i> is the north-west corner of the southern light support (the previous light support).						
1+85.7	Moderate to Severe Transverse Crack	16.15'	18.76'			
Rebar A is 3 ft. from hub; rebar B is 4 ft. from hub.						
4+48.8	Severe Transverse Crack	13.62'	17.35'			
5+56.3	Severe Transverse Crack	22.90'	17.35'			

Table III-1. Field Layout Notes

A is the nail in the power pole.					
B is the north-west corner of the mobile home park sign.					
8+00	End of Section 6	14.29'	17.80'		
Section 7 starts 50' after Section 6 ends.					
0+00	Section 7 starts.	15.98'	18.84'		
1+67	Moderate to Severe Transverse Crack	10.65'	14.90'		
Rebar A is 2 ft. from hub.					
3+74.2	Very Severe Transverse Crack	19.10'	20.78'		
5+66.9	Moderate to Severe Transverse Crack	15.12'	17.66'		
Rebar B is 2 ft. from hub.					
8+00	End of Section 7	19.15'	18.94'		
A is the nail-tack on the first tall post of the fence.					
<i>B</i> is the electric pole on the back.					

GPS Coordinates of Hub Locations

Note that the first two sections laid out in this survey were unused during construction. Therefore, all of the research sections were located in sections labeled 3 through 7.

Description	North	East
BEGIN SECT 3	497802.524	1861523.154
SECT 3 CRACK 1	497899.216	1861760.312
SECT 3 CRACK 1 HUB 1	497886.587	1861765.626
SECT 3 CRACK 1 REBAR	497890.744	1861774.005
SECT 3 CRACK 2	497985.398	1861972.567
SECT 3 CRACK 2 HUB 1	497972.947	1861977.625
SECT 3 CRACK 2 HUB 2	497976.456	1861987.000
SECT 3 CRACK 3	498042.013	1862112.771
SECT 3 CRACK 3 HUB 1	498026.884	1862118.487
SECT 3 CRACK 3 HUB 2	498031.597	1862127.383
END SECT 3	498100.993	1862259.861
BEGIN SECT 4	498148.167	1862380.765
SECT 4 CRACK 1	498249.110	1862665.831
SECT 4 CRACK 1 HUB 1	498237.318	1862669.075
SECT 4 CRACK 1 HUB 2	498240.554	1862678.348
SECT 4 CRACK 2	498317.876	1862862.397
SECT 4 CRACK 2 HUB 1	498306.682	1862866.908
SECT 4 CRACK 2 REBAR	498309.968	1862875.540
SECT 4 CRACK 3	498369.948	1863010.067
SECT 4 CRACK 3 HUB 1	498355.725	1863015.503
SECT 4 CRACK 3 HUB 2	498359.018	1863025.229
END SECT 4	498412.458	1863131.240
BEGIN SECT 5	498430.299	1863182.908
SECT 5 CRACK 1	498501.714	1863387.384
SECT 5 CRACK 1 REBAR 1	498493.384	1863390.184
SECT 5 CRACK 1 REBAR 2	498495.553	1863399.614
SECT 5 CRACK 2	498544.134	1863508.375
SECT 5 CRACK 2 HUB 1	498532.408	1863512.472
SECT 5 CRACK 2 HUB 2	498535.752	1863521.695
SECT 5 CRACK 3	498600.824	1863670.845
SECT 5 CRACK 3 HUB 1	498588.525	1863673.830
END SECT 5	498692.704	1863934.549
BEGIN SECT 6	498711.532	1863987.255
SECT 6 CRACK 1	498772.283	1864161.748
SECT 6 CRACK 1 HUB 1	498759.658	1864166.212
SECT 6 CRACK 1 HUB 2	498763.170	1864175.536
SECT 6 CRACK 2	498858.882	1864409.340
SECT 6 CRACK 2 HUB 1	498847.588	1864412.760
SECT 6 CRACK 2 HUB 2	498850.088	1864422.506
SECT 6 CRACK 3	498894.191	1864510.303
END SECT 6	498974.160	1864738.937
BEGIN SECT 7	498990.579	1864785.616
SECT 7 CRACK 1	499045.564	1864942.280
SECT 7 CRACK 1 HUB 1	499037.637	1864945.357
SECT 7 CRACK 1 HUB 2	499041.235	1864954.697
SECT 7 CRACK 2	499113.170	1865136.501
SECT 7 CRACK 3	499176.340	1865318.307
SECT 7 CRACK 3 HUB 1	499163.843	1865320.702
SECT 7 CRACK 3 HUB 2	499166.512	1865330.307
END SECT 7	499253.183	1865537.684

APPENDIX IV: FIELD CONSTRUCTION DETAILS

Construction

The construction of this project took place over three days during the week of October 7, 2013. On the first day of construction, the outside lanes were milled and filled until reaching 50 feet from the end of the control section (750 ft. from the start of the control section). Paving was stopped due to bad weather and resumed the next day. It should be noted that for all the sections, large cracks that were apparent on the surface also were seen in the milled surface underneath, confirming that most of the cracks were likely full-depth cracks (Figure IV-1 and Figure IV-2).



Figure IV-1. Pavement condition after milling.



Figure IV-2. Close-up of cracking in milled pavement.

On the third day of construction, all the interlayers were placed and the surface course was placed over them in the driving lane. Four different types of interlayers plus a control layer were placed at the site. These interlayer types included three geosynthetics (two paving mats and a paving fabric) and a chip seal.

All of the geosynthetic sections used PG 64-22 hot asphalt binder as a tack coat applied to the underlying asphalt layer. A tractor with a special broom was used to apply the geosynthetics. Next, a pneumatic tire roller was used to seat the geosynthetics into the asphalt binder. The

construction of the chip seal section was slightly different, consisting of CRS-2 emulsion applied to the surface followed by a layer of aggregate. Then, a steel wheel roller was used to seat the aggregate into the emulsion. The control segments consisted only of a CRS-2 tack coat between the asphalt layers.

No major problems occurred that prevented the interlayers from being placed, but some potential sources of variability were identified within the trial sections. These variability factors came from several key sources related to natural variations, paving practices, and problems associated with the interlayers themselves. These factors are discussed in detail in the following sections.

On-Site Variability of Test Segments

Natural variability: Natural variability came from two main sources. First, the existing condition of each segment differed slightly from segment to segment. These differences included the pavement distress level, roadway width, pavement support condition (assessed using FWD measurements), pavement thickness, roadside drainage, etc. Although these differences could not be controlled, they were documented. The other source of natural variability was weather. During the placement of the first asphalt course and the placement of the geosynthetics, a light mist began to fall on two separate occasions. Although the surfaces did not appear to be dampened significantly by this moisture, it still remains a possibility that the presence of this water could have reduced the bond strength between the pavement layers and, thus, compromised the performance of the pavement.

Paving practices: The paving practices at the site also contributed to a significant amount of variability from section to section. One of the most noticeable of these practices was that the trucks dumped the end of their loads on the pavement ahead of the paver. Some of these amounts of mixture were substantial (see Figure IV-3). A representative of the geosynthetic manufacturer noted that as long as this mix was less than one-third of the overlay thickness it may be considered acceptable, as the hot mix from the paver can reheat the loose mix on the pavement.



Figure IV-3. Loose mix on pavement from end-of-load dumping.

An additional problem associated with the practice of end-of- load dumping was that in some cases it caused sand to be knocked loose from the trucks and onto the pavement, most notably on the Paving Mat #1 section (see Figure IV-4).



Figure IV-4. Sand on pavement due to end-of-load dumping.

Furthermore, other paving practices at the site allowed for the potential of variable density levels from section to section. The practices responsible for this potential variation are listed below.

- *Dumping the paver wings after each truck load.* This practice has long been known as a source of both physical and thermal segregation that can affect mat densities.
- *Pavers stopping frequently to wait for trucks*. In cases where the wait time between trucks was long, variability in the temperature of the mat was observed, possibly resulting in density variation from section to section.
- *Material consistency at the plant*. Recycled asphalt pavement (RAP) balls were observed regularly in the pavement. The paving crew did its best to identify these RAP balls and to remove them by hand and then replace the resultant hole with mix from the paver. However, these areas of handwork, some of them large, may experience segregation (see Figure IV-5).



Figure IV-5. Handwork from removing RAP balls.

Lastly, in all the sections, the pavement was milled only to the white edge line, leaving 10 inches or more of unmilled asphalt at the edge of the pavement. This practice can lead to potential inconsistency in the height between the milled-and-filled portion of the lane and the existing unmilled pavement at the edge. This inconsistency may later lead to longitudinal cracking at the edge of the pavement that was not present in the existing pavement. This problem was most significant in the Paving Mat #1 section, which had the widest shoulder beyond the edge line of all the segments.

Variability and Problems with Interlayer Placement

Four different interlayer types plus a control layer were placed at the project site. Each type has different potential sources of variability. It should be noted that because Sections 1 and 2 were not used for this research project, only data for Sections 3 through 7 are presented.

Section 3, Paving Mat #2: Paving Mat #2 was the first geosynthetic placed at the project site. The target application rate for the tack coat in this section was 0.20 gal/yd^2 of PG 64-22 binder. The major drawback of this section is that it is on a roughly seven percent grade. In retrospect, this location was not an ideal location to place a geosynthetic interlayer research segment. However, due to miscommunication regarding the project start location, this section had to be used in order to fit all the segments within the areas that had been distress-mapped.

The fabric experienced some wrinkling during placement in this section (see Figure IV-6). The product manufacturer stated that the wrinkling was due to the geosynthetic roll being slightly misaligned. Ideally, the geosynthetic roll should be completely perpendicular to the direction of travel of the tractor. As placement continued, the alignment of the roll improved and the wrinkles became significantly less severe. All large wrinkles were slit and laid flat before paving.



Figure IV-6. Wrinkling of Paving Mat #2.

The major problem with this section came during the compaction of the overlay. Due to the steep grade, slipping occurred between the geosynthetic and the overlay, which caused significant compaction cracking in the section (see Figure IV-7).

It is believed that this section will exhibit poorer performance than the other sections and that it should not be compared directly to the other sections due to this inherent problem of the slope. Most importantly, it should be noted that the compaction difficulties associated with this section were likely not a problem with the specific brand of the interlayer product, but rather, are inherent of the risk of placing any geosynthetic interlayer product on a grade.

Also, because the geosynthetic was wider than the pavement, the paver tended to tear the geosynthetic along the edge. As noted by one of the other geosynthetic manufacturers, it would have been better to place a narrower piece of the fabric and 'sandwich' it between the two asphalt layers rather than having excess fabric hanging off the edge of the pavement, because this situation could provide a vector for water infiltration between the pavement layers.



Figure IV-7. Compaction cracking.

Section 4, Paving Mat #1: Paving Mat #1 was the product placed in the second section of the research project. The target application rate of PG 64-22 binder for this section was 0.17 gal/yd². Unlike the previous Paving Mat #2 section, this section was fairly level. This product also saw

wrinkling, similar to the Paving Mat #2 section, although the wrinkles in this section seemed less frequent (Figure IV-8).



Figure IV-8. Wrinkling of Paving Mat #1.

Section 5, Paving Fabric: The third and last geosynthetic placed in the project was Paving Fabric. The target application rate for this section was 0.25 gal/yd^2 of PG 64-22 binder. This product is not as stiff as the other two geosynthetics, and therefore, few wrinkles were seen during its placement. The only problem with the placement of this section was that the roll overlaps seemed to be slightly more prone to pick-up by the trucks than for the other two geosynthetics (see Figure IV-9).

Although it cannot be viewed as a problem with the construction of Paving Fabric, it must be noted that the contractor accidentally laid a total of 1,080 feet of this fabric instead of 800 feet, thereby extending the section by over 200 feet through a non-monitored gap and into Section 4. The most notable problem with this section is that a survey of the site after construction revealed compaction cracks.



Figure IV-9. Overlap between rolls of Paving Fabric.

Section 6, Control: Section 6 was given the standard pavement treatment of a tack coat between two asphalt pavement layers. A target application rate of 0.04 gal/yd^2 of CRS-2 tack coat was applied before placing the overlay. No problems were found with the construction of this segment. Also, it should be noted that this section is slightly shorter than 600 feet due to the previously mentioned over-run of the Paving Fabric from the adjacent section.

Section 7, Chip seal: The last interlayer system placed was chip seal. The aggregate used for the chip seal was 6M, and the target CRS-2 application rate was 0.40 gal/yd². The aggregate application rate was 55 lb/yd². Because the aggregate particles were fairly uniform in size, the result was a surface texture that had significant areas of emulsion present between the large aggregate particles (Figure IV-10 and Figure IV-11). This outcome is ideal for achieving a good bond between the pavement layers.



Figure IV-10. Chip seal section.



Figure IV-11. Chip seal surface texture.

A major problem with the chip seal section was that the distributer experienced failure, which resulted in a large amount of binder on the surface over a limited 20-foot area. This excess binder caused bleeding in the overlay, and the contractor had to remove and replace this short section (see Figure IV-12).

Also, because the paving operation moved fairly quickly near the end of the research segments, it is not known if the chip seal was fully cured before it was paved. If not, such improper curing would be a major cause for concern, as water between the layers could lead to a poor bond and accelerated failure.



Figure IV-12. Bleeding in chip seal section due to over-application of emulsion.

Section 8, Control: Due to the poor drainage and support conditions of the first control section, a second shorter control segment was selected at the end of the chip seal section. It should be noted that a significant amount of aggregate and emulsion was picked up by the trucks from the adjacent chip seal section, and thus, this control surface was not completely clean during paving (Figure IV-13). This scenario should be kept in mind for any future sections where chip seal interlayers are placed adjacent to sections without chip seal interlayers.



Figure IV-13. Tack coat only control section.

APPENDIX V: DISTRESS SURVEYS

Preconstruction Distress Survey

Prior to construction, visual distress mapping was completed for all sections that had been laid out within the paving project. These distress maps were drawn in AutoCAD and are presented in the following pages.



100'



100'

96

90'

82

80'

121

70'

65

60'

55'

50'

45'

40'

35'

30'











Distress Mapping of Section 4 [300'~400]











Distress Mapping of Section 4 [600'~700]











Distress Mapping of Section 5 [100'~200]











Distress Mapping of Section 5 [500'~600]





Distress Mapping of Section 5 [700'~800]









Distress Mapping of Section 6 [500'~600]

















Section 8 [100'~200]

Distress Mapping of



Post Construction Distress Survey #1 (3 Months After Construction)

A quick survey of the distresses at the project site was conducted in January 2014. The distresses observed during this survey are described in the following notes.

US 1, Moore County, Geosynthetic Trial Project January, 2014 Distress Notes

Paving Mat #2 Section (Sect. 3)

- 258': transverse (deflection) crack reflected through
- 309' 334': section of check cracking
- 377': 10' long section of check cracking
- 474': 2' x 5' area of extremely rough surface texture
- 488' 639': severe cracking (mostly longitudinal)
- 670' 711': severe cracking (mostly longitudinal)
- 722': transverse crack
- 800': transverse crack

Paving Mat #1 Section (Sect. 4)

- 289' 303': rough surface texture near cemetery/oak tree
- 430' 516': rough surface texture

Paving Fabric Section (Sect. 5)

- 131': check cracking near edge
- 205': 5' of check cracking near edge
- 291' 296': check cracking
- 576': transverse hump
- 690' 740': check cracking
- 138' 180' (over-run section): check cracking

Chip seal (Sect. 6)

- 620' 640': rough surface texture
- 670' 690': rough surface texture

Distress Survey #2 (10 Months After Construction)

Another visual distress survey was completed in August 2014. In addition to the distresses recorded from the previous survey, the following observations were made.

Paving Mat #2 Section: Another transverse crack was seen in this section in addition to the previously mentioned cracks.

Paving Fabric Section: Two transverse cracks were apparent in this section.

Distress survey #3 (16 Months After Construction)

Another visual distress survey was completed in February 2015. This survey focused only on reflective cracking or other load-related distresses, and did not include any previously noted constructed related distresses (check cracking, segregation, etc.). This distress survey does include all transverse cracks noted in the previous distress surveys, but does not list the other distress types.

Paving Mat #2 Section (Sect. 3)

- 51': 4' transverse crack
- 98': 3' 4' transverse crack
- 203': 3' transverse crack
- 219': 9' transverse crack
- 259': 6' transverse crack
- 709': 2' transverse crack
- 716': 10' transverse crack
- 719': 3' transverse crack
- 769': 3' transverse crack
- 800': 10' transverse crack

Paving Mat #1 Section (Sect. 4)

- 175': 2' transverse crack (outside edge)
- 259': 1' transverse crack (outside edge)
- 443': 3' transverse crack (wheel path)
- 731': 3' transverse crack (wheel path)
- 745': 2' transverse crack (wheel path)

Paving Fabric Section (Sect. 5)

- 365': 10' transverse crack
- 730': 10' transverse crack

Number of cracks seen in the passing lane adjacent to the test sections (where no interlayers were placed)

- Adjacent to Paving Mat #2: 15
- Adjacent to Paving Mat #1: 14
- Adjacent to Paving Fabric: 10
- Adjacent to Control: 17
- Adjacent to Chip seal: 9
- Adjacent to Control: 16

APPENDIX VI: LABORATORY TESTING

This Appendix VI provides an in-depth description of the laboratory testing that was deemed too detailed to include in the initial portion of this document. This information includes background information and verification of the digital image correlation (DIC) technique, the development of the laboratory tests for reflective cracking as well as a description of the materials used, and the sample fabrication procedures developed for this research.

Digital Image Correlation

Background

One problem associated with materials testing is determining the appropriate gauges to use to obtain the needed measurements. For a complex phenomenon such as reflective cracking, simple gauges that read only displacements or strains over a short area may not be sufficient to understand all the mechanisms adequately. Ideally, a method for measuring the three-dimensional displacements and strains of the entire sample is desired; however, such measurements are essentially impossible to take at this time. A reasonable alternative is to obtain the full-field displacements and strains of the surface of a specimen using DIC technology.

The DIC method works by taking a reference image of the sample before loading and subsequently taking multiple images (known as test images) throughout the testing. Next, software is used to compare each of the test images to the reference image, and any differences between the reference image and the test images are explained as deformations or movements of the sample that occurred at the time the test image was taken. In this way, full-field displacements and strains of the sample can be monitored throughout the test procedure. The DIC system allows for two important advantages for studying reflective cracking. First, it allows for easy tracking of the differential movements seen throughout the interlayers of a layered asphalt concrete (AC) sample, which otherwise might be difficult to track using traditional gauges. Second is that strain contour plots allow for easy visualization of the crack location within the sample, as cracks in the surface show up as areas of extremely high strain. Although the North Carolina State University (NCSU) pavement laboratory uses proprietary DIC software, a basic understanding of the DIC method is helpful for understanding its overall usefulness and applicability for engineering applications.

The basic concept of applying DIC algorithms is presented in Figure VI-1. A digital image can be considered as a matrix of values. Each of these values corresponds to the intensity of the image at any given point. Thus, the image (or more commonly, a smaller subset of the image) may be described as an intensity function f(x,y). If the object being photographed is deformed and a second image is taken, the intensity function of this image is $f^*(x^*,y^*)$. The variables x^* and y^* are related to x and y through displacements u and v. These displacements u and v can be described as functions of x and y (Chu 1985, Yates 2010).



Figure VI-1. Concept of digital image correlation.

$$x^* = x + u(x, y)$$
 VI-1

$$y^* = y + v(x, y)$$
 VI-2

In order to facilitate the determination of these functions, *u* and *v* can be approximated using the Taylor series, as presented in Equations VI-3 and VI-4.

$$x^* = x_0 + u_0 + \frac{\partial u}{\partial x}\Delta x + \frac{\partial u}{\partial y}\Delta y + \frac{1}{2}\frac{\partial^2 u}{\partial x^2}\Delta x^2 + \frac{1}{2}\frac{\partial^2 u}{\partial y^2}\Delta y^2 + \frac{\partial^2 u}{\partial x \partial y}\Delta x \Delta y \qquad \text{VI-3}$$

$$y^* = y_0 + v_0 + \frac{\partial v}{\partial x} \Delta x + \frac{\partial v}{\partial y} \Delta y + \frac{1}{2} \frac{\partial^2 v}{\partial x^2} \Delta x^2 + \frac{1}{2} \frac{\partial^2 v}{\partial y^2} \Delta y^2 + \frac{\partial^2 v}{\partial x \partial y} \Delta x \Delta y \qquad \text{VI-4}$$

Next, the DIC algorithm assumes trial functions for u and v and attempts to minimize the error of the correlation coefficient C to find the best trial displacement functions, as presented in Equation VI-5.

$$C = \frac{\sum [f(x, y) - f^*(x^*, y^*)]^2}{\sum f^2(x, y)}$$
 VI-5

Once the displacement field functions are known, differentiation can be used to determine the strain fields within the subset. This process is then repeated for all of the subsets of the image, allowing the construction of contour plots of both displacements and strains.

The fundamental assumption of this simple DIC algorithm is that the gray values of the featured images stay the same. However, due to the discrete nature of pixels in a digital image, this
assumption is rarely the case. In fact, due to the stretching of the features, and the features moving only distances that correspond to fractions of the distance between pixels, changes in the intensities of the features between f(x,y) and $f^*(x^*,y^*)$ are almost always seen. Therefore, more advanced DIC algorithms use interpolation functions to account for these changes in gray values, which greatly increases the accuracy of the DIC algorithms and allows for sub-pixel precision in displacement measurements.

Tracking Crack Propagation using Digital Image Correlation

In early trials with the DIC software, it was noted that strain field contour plots could be used to identify cracking in the sample. In particular, contour plots of von Mises strain (which is an estimate of the total strain given by Equation VI-6) allow the areas of high strain caused by cracking to be tracked easily. As such, von Mises strain was the first strain field viewed in any of the tests performed. From these plots it was noticed that discrete areas of low strain surrounded by areas of higher strain were present in the sample. It was found that these areas of low strain corresponded to the location of the surface aggregate within the specimens (Figure VI-2)



Figure VI-2. Comparison of von Mises strain field and aggregate locations. (Note: areas of low strain are light in color.)

$$\varepsilon^{e} = \frac{\sqrt{2}}{3} \left[\left(\varepsilon_{1} - \varepsilon_{2} \right)^{2} + \left(\varepsilon_{3} - \varepsilon_{2} \right)^{2} + \left(\varepsilon_{3} - \varepsilon_{1} \right)^{2} \right]^{1/2}$$
 VI-6

where

 ε_{xx} , ε_{yy} , ε_{zz} = normal strains in x, y, z directions, respectively, and γ_{xy} , γ_{yz} , γ_{zx} = shear strains.

Although many iterations of the DIC analysis parameters and strain criteria were used to identify cracking in this investigation, all final analyses of the three reflective cracking tests and all notched beam fatigue (NBF) tests were performed with a DIC window size (subset size) of 21 pixels x 21 pixels, and a step size of 3. The von Mises strain threshold of 5,000 was found to identify cracked locations consistently for most DIC analyses performed using these parameters.

Furthermore, increased resolution analysis was carried out with the NBF test samples using a subset size of 19 x 19 pixels and a step size of 1. For this analysis, only the images at peak displacement and zero displacement were analyzed in order to reduce the number of large analysis files produced by the DIC software. This approach reduced the number of images analyzed for each test from 2,000 to around 100. For the increased resolution analysis, vertical cracking was identified by looking at ε_{xx} strain with a cracking threshold of +12,000 µ ϵ . Horizontal cracking was identified by looking at ε_{yy} strain with a threshold of +6,000 µ ϵ .

Accuracy Verification of DIC

The NCSU pavement laboratory has been using DIC equipment and software for years. However, at the beginning of this investigation, it was believed that first-hand experimental validation of the accuracy of the DIC method would help provide confidence in the system. The team had two main questions with regard to the suitability of this method for characterizing the behavior of AC specimens during laboratory testing. First and foremost was the question of accuracy of the calculations for both the displacements and strains present in the samples. The second concern was related to the apparent cracking of asphalt samples under high strain levels. It was feared that cracks in the paint used to speckle the material may be identified by the DIC algorithm, even though these cracks do not correspond to actual cracks in the material.

In order to verify the accuracy of the equipment in the NCSU laboratory, a simple step-wise test was performed on a painted neoprene rubber sample. The objective of this test was to introduce known displacements and strains in the sample. Then, the DIC algorithm would be used to compare the measured results to the expected results. Also, both visual observations and DIC analysis could be used to determine if cracking of the paint on the surface of the specimen at high strain levels could be confused with actual cracks in the sample.

As mentioned previously, the first step in the DIC process is the calculation of displacement fields. If displacement fields are incorrect, then any other variables calculated from this information (such as strains) will also be incorrect. Figure VI-3 shows that the displacements measured by the DIC algorithm in the step-wise test match extremely well with the expected values of the displacement of the sample (3% error). Likewise, Figure VI-4 shows the accuracy of the calculated strain fields from the step-wise test. This figure shows that initially these strains agreed fairly well with the expected strains of the sample. However, as the level of strain increased, a larger difference between the expected strains and the measured strains emerged. One major reason for this occurrence was the relaxation that took place during each rest period of the step-wise test. The figure illustrates that at the beginning of each loading step, the calculated strain values are high and slowly drop with time until the next loading step. The

amount of relaxation during these rest periods accounts for 50 percent of the error between the expected and measured strains in the last loading step (Figure VI-4). After applying a correction factor for this relaxation, the calculated strain of the DIC measurements had an error of approximately 8 percent in this test (Figure VI-5). Even though no corrections or adjustments were made to account for the error potential caused by sample geometry, alignment, and end effects, this test nonetheless demonstrated that the accuracy of the DIC method is well within the range needed to investigate the mechanisms involved in reflective cracking.



Figure VI-3. Step-wise verification test: displacement.



Figure VI-4. Step-wise verification test: strain.



Figure VI-5. Step-wise verification test: corrected strain.

These tests also confirmed that significant cracking of the spray paint on the sample could not be seen even at high strain levels and that the DIC algorithm did not identify any false-positive cracks in the truly uncracked sample. However, one problem related to this sample was noise, which led to the need to average the results over a larger area to obtain the relatively noise-free

figures shown above (i.e. Figure VI-3 through Figure VI-5). As such, further investigation into the noise in the DIC analysis was needed.

Noise in Digital Images: All digital images have some level of noise, which is in large part random. Therefore, if two images of an unloaded, undeformed object are taken one after another, each will have slightly different levels of noise. The amount of noise depends on many factors, including camera quality, camera settings, lighting type, lighting intensity, the object being photographed, and the DIC parameters used, just to name a few (Martinec 2008, Vic-2D Testing Guide 2009). If these images are compared using a DIC algorithm, false displacements and strains of the object will be calculated. In addition to optimizing the DIC set-up and analysis parameters, another important step taken in this research was to reduce the noise in the reference image by using a composite image.

Although having noise-free images throughout the test would have been ideal, this situation was impossible to achieve. A reasonable alternative found in this research was to reduce the noise in the reference image by borrowing a concept from astrophotography (Cambridge In Color 2013). Specifically, when attempting to take images of faint objects far away in the night sky, digital image noise can become a problem. Because many sources of noise are random in nature, if multiple images are acquired and 'averaged' together pixel by pixel, many of these random variations in noise tend to cancel each other out, producing an image that has far less noise than any single image used in the averaging process. If this image is used as a DIC reference image, the reduced noise can help reduce the noise in the DIC measurements for all of the subsequent tests. In this research, this possibility was investigated by taking 100 images for four different speckled samples in a totally stationary, unloaded condition (Figure VI-6). Next, composite reference images were produced by averaging 2, 3, 5, 10, 20, 30, 40, and 50 images together for each sample. Next, for each reference image, DIC analysis was run to correlate the remaining 50 images (those not used in averaging for that sample) and to record the maximum strain noise found in this analysis. Figure VI-7 shows that the more images that are used to create the composite image, the less overall noise will be observed in the DIC analysis, with little gain in resolution after 20 images. Also, samples with smaller speckles and fewer white areas showed a better tolerance to noise and better overall accuracy. For these reasons, averaged reference images and a dense pattern with small speckles were used for the remainder of this research to help maximize accuracy.



Figure VI-6. Speckle patterns used for DIC noise investigation.



Figure VI-7. Results of DIC noise investigation.

Having demonstrated the ability of the DIC program to identify the displacements and strains accurately within the samples, and having selected the best possible set-up to achieve accurate results and reduce noise as much as possible, the next phase in the investigation was to concentrate on the development of laboratory tests to characterize the reflective crack-mitigating systems. This topic is discussed in the following sections.

Materials

Asphalt Concrete

The AC used in this research project was collected in the form of loose mixture. On two separate occasions, dozens of metal buckets were taken to the hot mix asphalt (HMA) plant and several tons of RS9.5B surface course mixes were obtained. The first mix, known as Mix-1, was used for all the reflective cracking tests as well as the trial NBF tests. Mix-2 was used exclusively for the main phase of the NBF testing and for laboratory shear testing. These two mixes contained granite aggregate, PG 58-28 virgin binder of 3.4 percent and 3.3 percent, respectively, with total binder contents of 5.6 percent and 5.7 percent, respectively. Both mixtures contained 40 percent reclaimed asphalt pavement (RAP). Figure VI-8 shows the gradations of these mixes. In order to use these mixtures for the construction of AC slabs, the buckets were heated to 264°F. Multiple pans of these materials were weighed in order to obtain the exact mass needed to produce a slab of a certain density given the known dimensions. Next, these pans were placed into an oven and heated to 300°F, and compaction was completed after the material had reached this temperature.



Figure VI-8. Asphalt concrete gradations used for laboratory testing.

Asphalt Binder and Emulsions

Two asphalt emulsions were used for this research. Nearly all applications of emulsion in this research, including tack coats and chip seals, used CRS-2 emulsion. The only exception is that early trial chip seal NBF test samples were made using CRS-2L emulsion. These emulsions were collected from the emulsion plant in buckets and stored at 140°F in the laboratory to prevent breaking.

A single PG 64-22 binder was used for this research as the tack coat for all of the geosynthetic applications. The binder from one five-gallon bucket was separated into smaller containers at the beginning of the laboratory investigation. Each small container was heated only once during the course of the research to prevent aging during reheating.

A second highly polymer-modified (HPM) binder was used for two grid NBF test samples to serve as an extreme condition of high bond strength.

Geosynthetics

Four types of geosynthetics were used in this research: two paving mats (Paving Mat #1 and Paving Mat #2), a paving fabric (Paving Fabric) and a fiberglass reinforcing grid (Grid). The properties, as found in the manufacturers' literature, are presented in Table VI-1.

Property	Paving	Paving	Paving Fabric	Grid
	Mat #1	Mat #2	U U	
Geosynthetic Type	Paving Mat	Paving Mat	Paving Fabric	Geogrid
	Continuous			Fiberglass grid
	fiberglass fibers	Nonwoven	Nonwoven	w/ a polymer-
Description	coated in an	blend of	polypropylene	modified
	elastomeric	fiberglass and		coating and
	compound	polyester fibers		pressure-
	embedded			sensitive
	between two			adhesive
	polyester			backing 0.5" x
	textiles			0.5" apertures
Tensile Strength	170 lb/in.	40 lb/in.	101 lb/in.	655 lb/in.
	(ASTM D5035)	(ASTM D5035)	(ASTM D4632)	(ASTM D6637)
Tensile Elongation	3.5% (ASTM	<5% (ASTM	5% (ASTM	2.5% (ASTM
	D5035)	D5035)	D4632)	D6637)
Mass/Unit Area	4.0 oz/yd^2	4.1 oz/yd^2	4.1 oz/yd^2	12 oz/yd^2
(ASTM D5261)				
Asphalt Retention	0.10 gal/yd^2	0.18 gal/yd^2	0.20 gal/yd^2	N/A
(ASTM D6140)				
Melting Point	>450°F	>446°F	320°F	>450°F
(ASTM D-276)				(coating)

Table VI-1. Geosynthetic Properties

Chip seal Aggregate

A single granite 78M aggregate was used to construct all the laboratory chip seal samples for this research. For the trial tests, the gradation of this aggregate was simply the natural gradation of the chip seal aggregate obtained from the North Carolina Department of Transportation (NCDOT). However, during the course of the research, this gradation was modified to remove the finer material to help improve layer bonding. This modified gradation was used for all non-trial tests (both reflective cracking and NBF tests). Both gradations can be seen in Figure VI-9.



Figure VI-9. Chip seal gradations.

Reflective Cracking Test and Tester Development

In order to analyze reflective cracking, it is desirable to simulate the complex reflective cracking mechanisms in the laboratory. One major goal of this research is to develop a reflective cracking test (RCT) device to evaluate reflective cracking in an AC overlay system under repeated wheel loading in a controlled laboratory environment. A system to compact and test multilayered asphalt slabs had to be designed from the ground up in order to use North Carolina State University's one-third scale wheel load device, known as the *third-scale model mobile load simulator* (MMLS3).

Development of Compaction Frame for Reflective Cracking Tester

The first task for developing a laboratory test to study reflective cracking was to design an adjustable compaction frame that would allow the compaction of multiple asphalt layers on top of the RCT device. A fixed four-piece frame with eight supports was developed for this purpose. A schematic of this compaction frame can be seen in Figure VI-10. This frame was assembled by first bolting the eight supports to the ground. During this step, small adjustments to the height of the compaction frame could be made by placing shims of the desired thickness between the ground and the support. Next, the sides of the compaction frame were set over the supports using pins and were adjusted to the proper height. Lastly, the ends of the compaction frame were bolted onto the RCT device.

This design proved to be extremely successful, as it was rigid enough to support the weight of the roller during compaction, easy to set up and take apart, and flexible enough to create AC layers of any height desired. As such, the design of the compaction frame did not change throughout the research. Figure VI-11 shows a schematic of the compaction frame in place during compaction of the asphalt mixture. Figure VI-12 shows the frame prior to compaction. Figure VI-13 show the frame during compaction.



Figure VI-10. Schematic of adjustable compaction frame.



Figure VI-11. Schematic of compaction frame during compaction.



Figure VI-12. Reflective cracking tester with compaction frame prior to compaction.



Figure VI-13. Reflective cracking tester with compaction frame partially filled with loose asphalt concrete.



Figure VI-14. Reflective cracking tester with compaction frame during compaction.

Development of Reflective Cracking Test Set-up

The RCT itself was developed simultaneously with the compaction frame. Unlike the compaction frame, multiple iterations and design changes were implemented through the development of this test set-up.

The original prototype of the design of the RCT device can be seen in Figure VI-15. The original concept behind the reflective cracking tester was to compact asphalt slabs on two steel plates with a gap or joint in the middle. These steel plates were to be supported at the ends opposite to the joint, and springs of varying stiffness would be used to control the deflection amplitude at the ends of the steel plate near the joint. However, during the summer of 2011, the design was changed to use movable steel tubing instead of a continuous support and springs (Figure VI-16). This configuration of the RCT device consisted of two pieces of 5-inch x 5-inch x 24-inch steel tubing with a 3/8-inch wall thickness fixed to the ground six feet apart, and two similar pieces of steel tubing positioned between them, which were not fixed to the ground. Two 0.5-inch x 24inch x 36-inch grooved steel plates were used to provide support to the asphalt layers (Figure VI-17). These plates were then bolted to the top of the fixed tubing and placed on top of the movable tubing. A 3/8-inch gap between the plates allowed differential movement of the plates during the application of the MMLS3 wheel loads. During compaction, a 3/8-inch x 1/2-inch x 24inch piece of key stock was placed in this gap between the plates and removed prior to testing. The DIC system was used to monitor the full-field displacements and strains of the side of the sample. The actual deflections of the plates were measured by four linear variable differential transducers (LVDTs) mounted below the steel plates. The positions of these LVDTs are shown in Figure VI-18. The deflection of the plates at the simulated crack was controlled by varying the position of the movable steel tubing.



Figure VI-15. Schematic of original concept of the reflective cracking tester.



Figure VI-16. Schematic of modified version of reflective cracking tester with movable box frames.



Figure VI-17. Reflective cracking tester steel plates.



Figure VI-18. Top view of reflective cracking tester indicating LVDT positions.

Trial 1: The first trial involved running the MMLS3 on the RCT set-up with no asphalt layers (steel plates only) to observe the pattern of deflection under wheel loading. Figure VI-19 shows the deflection values of the steel plates due to MMLS3 loading.

Next, PG 64-22 binder was placed on the steel plates to serve as a tack coat, and a 1.875-inch asphalt slab was compacted on top of the reflective cracking tester. The compaction of this layer was poor, as the slab experienced multiple compaction cracks, including one at the joint between the steel plates. The air void content of this slab was approximately 14 percent. Multiple MMLS3 tests were run with various space intervals of the movable supports to determine which spacing would best simulate field deflection amplitudes.

After selecting a DIC image acquisition interval, the MMLS3 was allowed to run on the AC slab for several hours in order to ensure that a full-depth crack developed. Figure VI-20 shows the peak deflection values during this test. The DIC analysis indicated that the first crack appeared after three hours, and the test was continued for six hours. Over time, the maximum peak deflection of the asphalt slab increased, as was expected. However, at around 60 minutes, the movable tubing moved from its original location, thereby allowing greater deflection until the tubing was adjusted back to its correct position.

After analyzing all of the data obtained from these trial tests using the reflective cracking tester, a major problem was found. From the plates-only test, it was observed that the free end near the joint experienced negative bending. Specifically, when a load (either a wheel load or a load from the roller during compaction) was positioned between the two supports at point B, upward deflection of the free end of the plate at point A occurred (Figure VI-16). This occurrence caused two major problems. First, during compaction, this upward movement caused cracks to form in the overlay. These cracks caused weak areas and led to rapid failure of the overlay during loading. The second problem associated with this set-up was that during wheel loading, the upward movement caused unrealistic loading conditions compared to conditions in the field.



Figure VI-19. Deflection of the second (rear) steel plate due to MMLS3 loading.



Figure VI-20. Maximum peak deflection vs. time.

In order to eliminate the negative bending, the support system of the reflective cracking tester was modified to increase its rigidity by using a cantilever configuration. In this configuration, each plate was bolted to steel tubing at the end. Next, each plate was set onto three five-inch tall I-beams instead of on the movable steel tubing. The bottom flanges of these beams were then clamped to the ground, and the steel plates were then clamped to the top flanges. This arrangement created a cantilever configuration that could be adjusted to create different cantilever lengths to control the deflection at the joint. The modified set-up is shown in Figure

VI-21. Figure VI-22 shows a close-up view of the supports, and Figure VI-23 shows the test setup and the LVDTs mounted under the steel plates. Also, it was decided to use a mechanical scissor-jack to help support the joint during compaction to prevent downward deflection of the free ends during the next trial.



Figure VI-21. Schematic of final configuration of the reflective cracking tester.



Figure VI-22. Modified set-up of the reflective cracking tester.



Figure VI-23. LVDTs mounted below steel plates.

Trial 2: After the modification of the reflective cracking tester, the MMLS3 was run again on the steel plates and it was observed that negative bending at the free ends was negligible (Figure VI-24). Next, a 1.875-inch slab was compacted on the steel plates. This test also saw the formation of a compaction crack at the joint and had an average air void content of 13 percent.

Two major problems occurred during compaction that may have contributed to the poor compaction of the layers. First, it was noticed that the mechanical jack used to support the joint moved due to vibration of the roller. This occurrence meant that the joint was unsupported during compaction and thus experienced compaction cracking. Second, the MMLS3 testing base was supported by rubber and moved slightly during compaction of the layers. It was decided that the next trial should utilize a hydraulic jack to support the joint and that the rubber under the MMLS3 testing base should be removed and replaced with more rigid support.

Despite the problems with the slab, several MMLS3 tests were performed using multiple cantilever lengths in order to obtain an idea as to the behavior of the reflective cracking tester with varying cantilever lengths. Initially, the cantilever on each side of the joint was 14 inches long (28 in. clear span between supports), and the deflection amplitude of the uncut single layer slab was found to be 27 mils under MMLS3 loading. Figure VI-25 presents these results.



Figure VI-24. MMLS3 test results for plates-only testing using modified set-up.



Figure VI-25. Deflection vs. time for uncut slab (14 in. cantilever).

Because the intent of the experiment was to control the deflections of the cracked 'existing' pavement, a simulated crack (a saw cut) was introduced into the asphalt slab directly above the joint. With the cantilever length of the RCT device remaining the same as for the uncut tests, MMLS3 testing was then performed on the cut AC slab. For this test, the deflection amplitude increased significantly, and the value observed was 95 mils (Figure VI-26). Next, the cantilever length on each side of the joint was changed to 12 inches (24 in. clear span distance between supports), and the maximum deflection observed was 78 mils (Figure VI-27).



Figure VI-27. Deflection vs. time for cut specimen (12 in. cantilever).

After these tests, a 1.5-inch overlay was compacted on top of the AC slab (without any interlayer), and MMLS3 testing was then performed on the overlay. With the 12-inch cantilever length, the deflection amplitude was approximately 7 mils. The slab was then demolished so modifications could be made to the RCT system.



Figure VI-28. Deflection vs. time for 1.5-inch overlay (1.875 in. bottom layer, LVDT measurements).

Trial 3: After implementing the modifications to the MMLS3 testing base and using a hydraulic jack to support the joints, a tack coat was placed on the steel plates, a 1.875-inch asphalt slab was compacted, a saw cut was introduced, the cantilever length was set to 14 inches, a tack coat was placed on the asphalt slab, and a 1.5-inch overlay was compacted. The air void contents of these layers were 14.5 percent and 9.5 percent, respectively.

Next, MMLS3 testing was conducted in a controlled temperature environment at 19°C for several hours. Both the LVDTs and the DIC system were used to monitor the deflection of the slab, and their results matched well. However, due to the increased structural support provided by the overlay, the deflection amplitude was found to be only 4 mils. It was believed that this value would not be sufficient to experience failure in a reasonable amount of testing time. Thus, in order to increase the deflection amplitude after the overlay, the cantilever length of the reflective cracking tester was increased. Unfortunately, the process of changing the cantilever length with a compacted slab on the reflective cracking tester damaged the slab, so long-term failure testing of this slab was not performed. However, the RCT was still used to obtain the deflection amplitude for the 26-inch (25 mils) cantilever set-up used in Trial 4.

Trial 4: In order to increase the deflection amplitude, a new set of slabs was constructed with reduced layer thickness. The thickness of the new bottom AC layer was 1.35 inches, and a saw cut was introduced directly above the joint. The cantilever length was set to 26 inches and an overlay of 1.5 inches was compacted. The air void contents of these two layers were 11.5 percent and 10 percent, respectively. The extended cantilever length between the I-beam supports and the hydraulic jack meant that some upward movement occurred near the joint during compaction, and a crack formed at the joint during compaction of the overlay. This occurrence made the overlay useless for long-term failure testing using the reflective cracking tester. Still, some useful information was obtained by running the MMSL3 on this slab:

- This RCT allowed verification of the deflection amplitude of the RCT device under MMLS3 wheel loads using the current geometry, test temperature, and HMA lift thicknesses. As seen in Figure VI-29, the total deflection amplitude was approximately 29 mils. It was believed that this deflection amplitude would be sufficient to produce observable cracking within a reasonable test time.
- 2. This RCT allowed verification of the DIC set-up and its ability to track movement and determine strains within the HMA layers. In previous MMLS3 testing, DIC images were obtained by setting the camera acquisition speed to its fastest setting and taking pictures continuously during specified periods of time. The main disadvantage of this protocol is that the location of the wheel for any given picture was unknown. Because of this testing constraint, the peak value for each cycle represented in the graph may not correspond to the actual peak value of the deflection of the slab in the reflective cracking tester for that cycle. As seen in Figure VI-30, many cycles would have to be examined in order to determine the true peak of the deflection value. Also, due to the random nature of the pictures, the shape of the deflection versus time curve was impossible to see from the DIC data. In order to synchronize the wheel position and the image acquisition, the delay between DIC images was set to slightly longer than the time for a single wheel to pass. This synchronization produced a more accurate picture of the deflection versus time curve (Figure VI-31). Note that each cycle seen in Figure VI-31 actually consists of data points taken from approximately 30 different cycles. Thus, the time shown is not the actual time of the test, but rather, a reduced time based on the period observed from the LVDT measurements. In comparison to Figure VI-29, Figure VI-31 shows that the overall shape of the deflection versus time curve correlates well with the LVDT measurements. This correlation is important, because the strain field within the HMA layers is related to the deflection of the HMA layers. Thus, the ability of the DIC system to measure the deflection with time accurately implies its ability to capture the development of the strain fields with time. This confidence in the ability of the DIC system became important to this research, as the proper installation of the LVDTs was challenging, and the use of DIC alone to monitor deflections could simplify the set-up procedure of the RCTs.



Figure VI-29. Deflection vs. time for 1.5-inch overlay (1.4 in. bottom layer, LVDT measurements).



Figure VI-30. Deflection vs. time for 1.5-inch overlay (1.9 in. bottom layer, random DIC measurements).



Figure VI-31. Deflection vs. time for 1.5-inch overlay (1.4 in. bottom layer, synchronized DIC measurements).

- 3. The ability of the DIC system to track the development of cracks in the asphalt layers during the RCTs was confirmed. The observed cracks included rapidly appearing bottom-up cracks as well as a top-down crack that developed as the test progressed.
- 4. High strains developed along the interface between the two HMA layers in this test, indicating that complete bonding between pavement layers was not achieved in these samples.

Trial 5: After completing the above RCTs, the steel plates appeared to be slightly bent. It was calculated that the bending stress applied to the steel plates was close to the yield strength of the material. Because of this occurrence, it was decided not to continue testing with these steel plates and to obtain new steel plates that have greater yield strength of 100 ksi (A514). All incremental design improvements to the reflective cracking tester were implemented using these new plates.

Additional improvements included modification of the supports to increase contact with the ground, restraints at the joint between the plates to prevent the plates from moving independently during compaction (Figure VI-32), and guides for repeatable placement of the supporting members. Also, modifications were made to the reflective cracking testing area to create a flat, level, rigid working platform to simplify the leveling of the RCT device during set-up and specimen fabrication.



Figure VI-32. Reflective cracking tester joint support during compaction.

With these improvements in place, a 1.35-inch asphalt layer was compacted. Next a 0.08 gal/yd^2 (0.06 gal/yd^2 residual) tack coat was placed, followed by a 1.5-inch overlay.

Compaction cracking at the joint was still a problem with the first layer. The air void percentages for these slabs were 13 percent and 8 percent for the first and second layers, respectively. Using a 26-inch cantilever length, the MMLS3 was run on the overlay.

Figure VI-33 shows the DIC results from this test. Using these measurements, it was found that the initial deflection amplitude was around 40 mils, and the deflection amplitude increased to around 70 mils by the end of the testing. As in the previous RCTs, significant strains developed almost immediately at the interface between the two layers. Furthermore, it was observed that Mode I crack growth due to bending appeared to be the main driver of the crack propagation during the initial stages of the tests. After that, compression in the top portion of the slab prevented further crack growth until around 112,000 cycles, at which point shear effects due to the differential deflection of the slabs on either side of the joint became the dominant driver of crack formation. Almost 122,000 cycles (approximately 22 hours) were needed to reach complete failure of the AC slab. Figure VI-34 and Figure VI-35 show photographs of the test slab from the side and from the top, respectively, after the test was completed.



Figure VI-33. Crack propagation (von Mises strain contours) for Trial 5.



Figure VI-34. Reflective cracking after testing (side view).



Figure VI-35. Reflective cracking after testing (top view).



Figure VI-36. Stress distribution within overlay (Lytton et al. 2010).

Trial 6: In order to make the base as rigid as possible during compaction, additional mechanical scissor-jacks (with lock nuts) and mid-span supports were placed beneath the slab during compaction along with the hydraulic jack at the joint. Figure VI-37 and Figure VI-38 show these improvements to the RCT system.



Figure VI-37. Modified reflective cracking tester compaction supports.



Figure VI-38. Modified reflective cracking tester supports during compaction (side view).

With these improvements in place, two HMA layers composed of 1.35-inch and 1.5-inch thick AC with a tack coat between them were constructed. No compaction cracks appeared during compaction. The air void content of the first layer was 12.5 percent and 9.5 percent for the overlay.

After compaction of the overlay, the MMLS3 test was conducted in a controlled temperature environment at 19°C for several hours. Both LVDTs and the DIC system were used to monitor the deflections of the slab for this test. The initial deflection amplitude was around 25 mils, and the deflection amplitude increased to around 70 mils by the end of the testing. Almost 57,000 cycles (approximately 10 hours) were needed to reach complete failure of the AC slab. Figure VI-39 shows the growth of the reflective crack in the overlay.



Figure VI-39. Propagation of reflective crack along and through interface with tack coat (von Mises strains).

The DIC results (Figure VI-39) show that the strain at the interface began to develop very early in the tests. Both Mode I separation and Mode II sliding (shear) occurred along the interface between the layers. The tests performed in the reflective cracking tester also show the effect of damage due to the deflection of the sample during the tests. At the beginning of the test, the slabs on either side of the joint acted as a monolithic body and deflected almost simultaneously as the MMLS3 wheels passed over the overlay. However, over time, the slab on each side of the joint started to move independently under the wheel load. Figure VI-40 shows the synchronized DIC deflections, which indicate that the two halves of the slab moved together at the beginning of the test, whereas Figure VI-41 shows that the two halves of the slab deflected separately near the end of the test.



Figure VI-40. Synchronized DIC deflections, indicating two steel plates moving together (Cycle #4,800).



Figure VI-41. Synchronized DIC data, indicating steel plates deflecting separately (Cycle #56,500).

Trial 7: After the previous test, the slab was demolished, and a 1.35-inch layer of HMA was compacted on the reflective cracking tester, followed by the placement of chip seal on top of the compacted HMA. CRS-2 emulsion and a slightly modified 78M aggregate gradation were selected to construct the chip seal interlayer.

After compaction of the overlay on top of the chip seal and the removal of the compaction supports, the RCT specimen was observed to be experiencing debonding at the interface between the chip seal and the overlay due to self-weight. Once the MMLS3 test was started, the initial deflection amplitude was around 35 mils, and due to the separation of the two layers, the deflection amplitude began to increase rapidly. Figure VI-42 shows the DIC results from this test; however, it should be noted that the crack propagated to the surface outside of the viewing area of the DIC camera and thus does not appear to reach the surface in these images.



Figure VI-42. MMLS3-based RCT results for chip seal interlayer.

Within 15,000 cycles, the slab completely failed. This number of cycles to failure is significantly lower than for the previous control test, which took around 57,000 cycles to failure. The main reason for this decrease in the number of cycles to failure is the debonding at the interface due to the creep of the asphalt slab, which was due to both self-weight and wheel loading. This creep created an excessively high mean deflection of the bottom layer during cyclic loading (Figure VI-43). Because the top layer was able to 'bridge' the bottom layer (Figure VI-44), the two layers began to separate along the interface. The more separation that occurred, the higher the deflection amplitude became, which greatly accelerated the failure of the specimen.

Two major changes were implemented to the RCT set-up to mitigate the problem of high mean deflections as much as possible. First, the thickness of the support layer was reduced from 1.35 inches to 0.83 inch. The benefits of this change are two-fold. By reducing the layer thickness, the self-weight of the slab was decreased, and the overall mean deflection of the system due to self-weight was reduced. More importantly, the reduction in pavement thickness decreased the stiffness of the pavement structure under dynamic loading, which allowed the cantilever length

of the reflective cracking tester to be shortened and still produce the same deflection amplitude during loading. This shorter cantilever length helped to reduce the mean deflection due to self-weight and the creep deflection due to wheel loading even more.

Second, three of the four wheels of the MMLS3 were removed. This modification meant that the peak-to-peak time between wheel load applications increased from 0.6 second to 2.4 seconds. This increased time produced a rest period of approximately 1.9 seconds where no load was being applied to the pavement at all. This larger time gap between wheel load applications helped to increase the amount of recovery during the rest period, thereby further reducing the mean deflection.



Number of Cycles

Figure VI-43. Reflective cracking trial test with high mean deflection.



Figure VI-44. 'Bridging' ability of the overlay.
Reflective Cracking Tests 1 and 2: Two RCTs were carried out with these changes in the summer of 2013: one chip seal test and one control test.

The chip seal test was performed first. The support layer was a 0.83-inch S9.5B layer, followed by 0.4 gal/yd² (0.3 gal/yd² residual) CRS-2 emulsion and a 20 lb/yd² (14.5 lb/yd² after sweeping) 78M chip seal (modified gradation), followed by a 0.08 gal/yd² (0.06 gal/yd² residual) CRS-2 tack coat, followed by a 1.25-inch S9.5B overlay. The overall thickness of the chip seal was 1.5 inches.

After the chip seal RCT platform had been tested and demolished, a control specimen was constructed and tested to provide a basis of comparison. This control test consisted of a 0.83-inch S9.5B support layer, 0.07 gal/yd² (0.05 gal/yd² residual) CRS-2 tack coat, and a 1.5-inch S9.5B overlay. This test performed fairly well and did not result in the significant debonding that was evident in the chip seal tests. The number of cycles to failure for the control test was around 70,000.

These two RCTs clearly indicated that all of the changes that were made to the RCT set-up together helped to minimize the mean deflection during the tests to an acceptable level (Figure VI-45). Figure VI-45 illustrates this rebounding effect in the deflection versus time graph. It was decided that this significantly reduced value was the best that was practically achievable for the RCT set-up given the time constraints of the research. For this reason, no further modifications to the RCT device or RCT set-up were made during this research, and the chip seal and control tests were considered actual RCTs rather than trials.



Cycles

Figure VI-45. Final configuration of the RCT set-up showing minimized mean deflection.





Final Configuration of the Reflective Cracking Tester

Although the final configuration of the reflective cracking tester was not perfect, it was believed that it was the best that could be achieved within the timeline of this research. The set-up of the reflective cracking tester is described as follows.

First, steel tubing and short I-beams were attached to the base plate. Next, two 36-inch x 24-inch x 0.5 inch A514 steel plates were laid on top of the tubing and the I-beams. The steel plates were then bolted to each of the pieces of tubing. Next, the I-beams were bolted to both the base plate and the reflective cracking tester plates using clamps. This approach effectively created two cantilevered plates with a 5/8-inch gap between them. By loosening the clamps and adjusting the position of the I-beams, the length of the cantilever on each side of the joint could be controlled, thus controlling the deflection amplitude of the system under loading.

Next, additional supports were provided beneath the steel plates, and a compaction frame was placed around the reflective cracking tester. Then, a CRS-2 emulsified tack coat was placed on the steel plates in order to aid bonding of the asphalt layer to the steel plates. Next, the compaction frame was adjusted to 0.83 inch Next, twelve areas were designated on the tester slab (Figure VI-47), and 24 pans of material were placed in these areas in two lifts. The mix was smoothed using rakes, and then a vibratory steel wheel roller was passed over the pavement a total of 32 times, 16 of which were without vibration. Once the first layer had cooled, a fulldepth saw cut was made at the center of the slab at the gap between the two steel plates to simulate a crack in the existing pavement. The reflective cracking tester was loosened, the two slabs were pushed together slightly to reduce the width of the crack to 1/8 inch, and the tester was retightened. Next, the desired interlayer was constructed, and the compaction frame was used to construct a second AC layer, bringing the total height of the compacted pavement layers to 2.33 inches. Once the second layer had adequately cooled, the vertical edge of the sample was smoothed to provide an adequate flat surface perpendicular to the DIC camera. This surface was painted white and then speckled with black spray paint in order to be viewed by the DIC camera. The end result of this procedure was a layered slab specimen of approximately 72.6 inches x 23

inches x 2.33 inches with a transverse joint in the middle of the bottom layer, supported by two cantilevered steel plates.



Figure VI-47. Designated areas for placement of loose mixture during RCT compaction (plan view).

Prior to testing, the removable compaction supports were taken out from the reflective cracking tester, and in their place four spring LVDTs were placed near the joint between the steel plates (two at the center of the steel plates and two near the edge). These LVDTs allowed the monitoring of deflections of the steel plates during loading to provide independent validation of the DIC measurements (if desired). Next, an environmental chamber was placed around the RCT area in order to maintain the temperature of the slab at $20^{\circ}C \pm 1^{\circ}C$. This chamber had a window so that the DIC camera and lights could be placed outside of the chamber.

Once all the instrumentation was in place, the MMLS3 was placed inside the chamber on top of the RCT slab, and wheel loads were run over the pavement at a rate of 1,500 applications per hour. DIC images were acquired at a rate of one image every 2.41 seconds. Loading was continued until a crack was seen to propagate all the way to the surface.

Development of the Notched Beam Fatigue Tests

Due to the significant problems associated with developing the RCT device, a second test method was developed in parallel with the RCT. Previous research at NCSU included modified versions of standard beam fatigue tests to test interlayer fatigue behavior. Two-layered HMA beams with a fiberglass grid embedded between the two layers were tested. Using DIC software, these test results have provided useful information about the crack propagation behavior in these specimens with varying interface conditions (control, control with tack coat, grid, grid with tack coat, etc.).

Due to the success of these tests, it was decided that similar tests could be performed on the interlayer materials used in this investigation. A major advantage of these tests is that they allow the fabrication and testing of multiple replicates per interlayer treatment much more easily than the RCT system in which each two-layered slab is one specimen. Also, because the parameters of the beam fatigue test can be controlled more closely than for either field samples or RCTs, it was believed that the beam fatigue tests may become important in developing correlations between the laboratory material-level properties of the interlayer systems and field performance. Lastly, constructing the different interlayer treatments on smaller specimens allowed the

procedure for placement of interlayer systems in the laboratory environment to be perfected, without the need to use as much material per trial as is needed for a single RCT sample. Figure VI-48 shows the general geometry of the notched beam fatigue (NBF) test.



Figure VI-48. NBF test configuration.

Specimen dimensions: Standard beam fatigue specimens are typically 14.96 inches long, 1.97 inches thick, and 2.48 inches wide for an AC beam. However, because some interlayer systems, such as chip seal, have a significant thickness, the overall height of the beams for the NBF tests was increased to 2.13 inches to allow more room for crack propagation above the interlayer. Because previous research had shown that reinforcing interlayers are most effective when placed at one-third from the bottom of the specimen (Khodaii 2009), the final beam specimens were sawn such that the interlayer was placed approximately 0.71 inch from the bottom of the sample, leaving 1.42 inches for the top layer.

Inclusion of a notch: Previous research at NCSU had included beam fatigue tests using DIC technology to characterize behavior. However, due to the small viewing window between the loading points of the fatigue test device, cracks had a tendency to form outside of the area monitored by the DIC system. Because of this problem, the first trial test in this research incorporated a 0.1-inch deep by 0.1-inch wide notch across the full width at the midpoint of the beam. In addition to controlling the location of failure, the notch had the benefit of initiating early failure and allowing for a reduction in the needed testing time, particularly at lower strain levels. It is believed that the inclusion of such a notch in the beam is reasonable for this research, because stress concentrations are the mechanisms that drive reflective cracking in the field.

Loading frequency: The standard beam fatigue test utilizes test frequencies between 5 Hz and 10 Hz. Although both frequencies are found in the literature, 5 Hz seems to be fairly common (Dondi 1996, Zhengqi 2000, Vismara 2012). Also, because images are acquired in this type of test, the use of 5 Hz is preferred as it reduces the overall motion of the sample that occurs during the image acquisition period, helping to minimize the effects of image blurring due to motion. Therefore, 5 Hz was selected for use in this research.

Notched Beam Fatigue Trials

Early on, two sets of beams were fabricated using a two-wheeled vibratory compactor (Figure VI-49 and Figure VI-50). The control specimens (tack coat only) constituted the first set. The fabrication of these specimens was as follows. First, a 24-inch x 24-inch x 2-inch layer of HMA

was compacted using a vibratory roller. Next, a 0.08 gal/yd^2 (0.06 gal/yd^2 residual) CRS-2 emulsion tack coat was brushed on the surface of the first layer. Next, a second 24-inch x 24-inch x 2-inch overlay was placed in the same manner as the first. Once cooled, the beam fatigue specimens were cut from this slab by removing 0.55 inch from the top layer and 1.26 inches from the bottom layer. The final specimen had a 2.13-inch thickness with the tack coat at one-third depth from the bottom of the specimen. Figure VI-51 shows one of these control specimens.



Figure VI-49. Compaction of slabs for trial beam fatigue samples.



Figure VI-50. Two-wheeled roller during compaction.



Figure VI-51. Beam fatigue specimen with tack coat.

The second set of beams was constructed in a similar manner to the first, the only difference being that a chip seal was placed between the two layers (Figure VI-52). The chip seal consisted of 0.35 gal/yd² (0.26 gal/yd² residual) of CRS-2L emulsion that was spread on the surface using spatulas. Next, 20 lb/yd² of granite 78M aggregate was spread over the surface. A sheet of rubber was placed on the surface, and the roller was passed over the sample four times in order to seat the aggregate into the emulsion. The entire chip seal placement procedure was completed in less than five minutes. The chip seal was allowed to cure for a day prior to the placement of the second HMA layer. Once the second layer had cooled, the specimens were sawn in a similar manner to the first set of test beams. Figure VI-53 shows one of the chip seal specimens.



Figure VI-52. Laboratory-placed chip seal.



Figure VI-53. Chip seal beam fatigue specimen.

Trial 1 control: A tack coat only control specimen was selected for the first trial test. The specimen was notched, painted, and placed in the beam fatigue test device. Next, a constant displacement cyclic test was performed. The displacement was selected in order to produce the desired strain level due to bending in the center of the specimen. This specimen was tested at room temperature due to the lack of a chamber that would allow the use of the DIC system with the beam fatigue test device.

Two important determinations were made from this first trial:

- 1. Over time, cracks began to propagate in the beam near the notch. Eventually, a single crack dominated this region and propagated to the interface between the two layers. Due to the strain level used in the test, failure did not occur until around 100,000 cycles (after approximately six hours of testing). The test was stopped at this time. Because this beam was a control specimen, and specimens with interlayers were expected to take significantly longer to experience failure, a larger notch (approximately 0.20 in. x 0.1 in.) was to be introduced for the subsequent trial tests in order to reduce the number of cycles to failure.
- 2. The calculated strain field obtained from the DIC analysis showed many large areas of low strain. It was assumed that these areas corresponded to the aggregate particles on the surface of the specimen. After completion of the test, the sample was removed from the device and the paint was removed in order to expose the aggregate particles. The aggregate particles were then highlighted with white paint to facilitate visualization. By comparing the photo of the specimen to the calculated strain field obtained from DIC analysis, it was found that the locations of the low strain were in very good agreement with the locations of the large aggregate particles (Figure VI-2). This finding demonstrated that the DIC method is capable of capturing a realistic state of strain on the surface of the HMA sample, which provided confidence in the results obtained from the DIC analysis.

With this information, seven more trial tests were completed using the beam samples. The results are summarized in Table VI-2.

Sample Name	Notch Length (in.)	Strain Level (με)	Cycles to Reach Interface	Cycles to Reach Top of Beam	Test Temperature (°C)
Trial Control 2	0.195	900	29,800	60,715	24.2
Trial Control 3	0.206	900	25,300	61,500	24.4
Trial Control 4	0.196	900	26,200	60,300	26.7
Trial Control 5	0.196	900	28,017	60,610	24.5
Trial Chip seal 1	0.213	1,000	18,400	99,000	n/a
Trial Chip seal 2	0.226	900	9,000	108,000	n/a
Trial Chip seal 3	0.193	900	22,900	86,250	n/a

Table VI-2. Comparison of Notched Beam Fatigue Test Results

These early NBF trials also revealed a few areas where improvements could be made for specimens to be tested later in this research.

Compaction: The method of using a two-wheeled roller for slab construction was problematic. First and foremost was that compacting the slabs involved considerable effort, requiring the assistance of five people for each layer that was compacted. Second was the chance of high variability from slab to slab due to loose mixture sticking to the drums of the roller or being pushed off the end of the compaction frame. For these reasons, the research team decided to utilize a new roller compactor for all future tests (Figure VI-54). This device would allow the construction of smaller, more consistent slabs and would require less manpower in a shorter amount of time. The only significant change to the specimen geometry between the trial tests and the subsequent tests came about due to this change in compaction equipment. The roller compactor produced specimens 15.75 inches in length. In order to simplify the specimen fabrication procedure and to eliminate any variability due to attempting to saw 0.39 inch from each end of the beam, it was decided that the extra 0.79 inch in length would be left on the specimens.



Figure VI-54. Roller compactor used for fabrication of main phase NBF test samples.

Chip seal placement: The three chip seal specimens experienced severe layer separation during testing. It was not known at the time if this occurrence was due to a poor bond between the asphalt surface course and the chip seal. As such, steps were taken to ensure good bonding between the layers in all the subsequent chip seal tests. First, the gradation of the chip seal aggregate was changed to eliminate all material below the #16 sieve. This modification helped to produce a more uniform gradation and to reduce the fine content. This change also promoted more AC-to-emulsion contact. In addition, a 0.08 gal/yd^2 (0.06 gal/yd^2 residual) tack coat was applied to the surface of the chip seal to help promote better bonding between the layers in all future samples.

Temperature conditioning: Due to the variability of the laboratory temperature, a chamber was developed that could be attached to the front of the existing environmental chamber to allow the DIC system to be used with the beam fatigue test device. This chamber also allowed testing at several temperatures in order to investigate the effect of temperature on interlayer behavior.

The rigidity of the notch-making saw: The tile saw used for making the notches in the beam specimens was determined to be undesirably flexible. Variations in the amount of pressure placed on the sample during sawing caused variations in the depth of the notch produced on the beam (nearly 20 percent variation in some cases). As such, the saw was stiffened in order to produce more repeatable notches in the future (Figure VI-55).



Figure VI-55. Tile saw used to make notches.

Effect of Geosynthetic Development Length and End Constraint on NBF tests

With all of the improvements from the trial tests in place, one important consideration for the NBF tests was the development length of the geosynthetic interlayer systems. In engineering, any material that experiences tension and is embedded within another material must have adequate contact with the surrounding material to ensure that the stress can be transferred from the material that experiences tension to the surrounding material. For applications such as reinforced concrete design, a minimum length of embedment of the reinforcement into the surrounding material is required. If the embedment length is inadequate, pullout failure can occur (Nilson 2004). A similar concept of development length is a common consideration in certain geosynthetic reinforcing applications as well (Koerner 2005). Thus, it is no surprise that the concept of embedment length must be considered for geosynthetic interlayers in asphalt pavements.

A flexible pavement that experiences wheel loading can help exemplify this phenomenon. These wheel loads create bending in the pavement layers. When reinforcing layers are present, this bending occurrence causes tensile force to develop in the interlayer near the site of the load application (Figure VI-56). This tensile force must be carried by the reinforcement and transferred to the surrounding AC layers through the development length of the reinforcement (de Bondt 1991). In the field, it is reasonable to assume that the large dimensions of asphalt

pavements compared to the dimensions of the loads applied to them means that reinforcing interlayers should have sufficient development lengths to realize their full strength. For the NBF tests, it was not clear if sufficient embedment length was present for the reinforcing layers to develop their full strength. Also, it was not clear as to the effect that the free end of the sample that was relatively close to the load application points would have on the overall behavior of the samples tested (Figure VI-57). Therefore, multiple trial samples were constructed to investigate the effect of fixed ends on the NBF test samples. These samples were constructed and tested as part of a separate project through St. Gobain ADFORS in its investigation of grid interlayer systems. Although these samples are not associated directly with the NCDOT field project, they served as a key part in developing the NBF tests used for this research and, thus, must be mentioned.



Figure VI-56. Bending and development length in flexible pavement.



Figure VI-57. Potential difference between free (a) and fixed (b) boundary conditions.

In order to test the effects of the end restraint conditions, slabs of tack coat only, fiberglass grid with PG 64-22 tack coat, fiberglass grid with HPM asphalt binder, and fiberglass grid with no tack coat were constructed. In the grid cases, the slabs were constructed in a manner such that the grid extended beyond the edges of the slab. Next, NBF test samples were sawn from these slabs. Then, beams were randomly selected to be tested with one of three end constraint conditions: 1) no clamps (Figure VI-58), 2) a flat-shaped clamp (Figure VI-59), and 3) L-shaped clamps (Figure VI-60). For the beams with clamps, epoxy was used to glue the clamps to the grid and to the sample end. These clamps served two purposes: 1) to ensure that the grid was not experiencing pull-out failure due to an inadequate embedment length and 2) to ensure that excessive deformations of the beam due to the end boundary conditions were not occurring.



Figure VI-58. Beam sample with the free boundary condition.



Figure VI-59. Beam sample with the flat clamp boundary condition.



Figure VI-60. Beam sample with the L-clamp boundary condition.

Once clamped, the beams were then subjected to displacement control fatigue loading. The numbers of cycles to failure for various failure criteria were examined, as well as the stiffness of the beams at certain cycles. In this early phase of the testing, it was unknown exactly which criteria were the best indicators of performance of the beam, so all of the results were taken together rather than focusing on a single failure criterion. This investigation helped determine that no statistically significant changes were evident in the parameters that were measured for the different end conditions. Thus, it was assumed that both embedment length and end fixtivity were not important factors in terms of affecting the NBF test results. For this reason, and due to the fact that clamping the ends significantly increased the time needed to prepare a sample (including curing of the epoxy), this step was abandoned in all further testing.

Final Notched Beam Fatigue Test Procedure

With the questions of development length and end fixtivity answered, the final iteration of the NBF test protocol used for all of the subsequent NBF tests was as follows. First, using the slab compactor, a 1.97-inch x 12.0-inch x 15.75-inch slab of AC was compacted. Next, the selected interlayer treatment was applied to the surface of this layer. Then, a second AC layer was compacted to bring the height of the total sample to 3.94 inches. Next, three beams were cut from the center of the slab in order to avoid high air void contents near the edges of the mold. These beams were trimmed to the final dimensions (2.13 in. x 2.52 in. x 15.75 in.), and a 0.2-inch x 0.1-inch notch was cut into the center of each beam. These beams were then painted and speckled for easy viewing by the DIC camera. Once constructed, all the samples were placed in a four-point bending beam fatigue test device. The beams were then temperature-conditioned at one of three testing temperatures (15° C, 20° C, or 25° C) for two hours. Once all the data acquisition equipment was ready, displacement-controlled sinusoidal loading was applied using a servo-hydraulic loading machine. Displacement amplitudes were selected to produce 900 µ ϵ at

the bottom of the beam during maximum displacement and were applied at a frequency of 5 Hz. Load and displacement data were tracked using a 2.5-kip load cell and LVDTs. Full-field strains, displacements, and crack propagations were monitored in the samples using the DIC technique. These tests were run until a crack was seen to propagate through the full depth of the specimen.

Interlayer Placement

Although several modifications were made to the RCT and tester and to the NBF test procedures, the procedures for interlayer placement remained relatively unchanged throughout the course of this research. In general, the interlayer placement procedures for the RCT and NBF test slabs were the same, the main differences being the number and size of the areas.

Tack Coat

The placement process for the tack coats used for this research remained the same regardless of whether the tack coat was being placed on the steel plates of the reflective cracking tester, the support AC slab on the reflective cracking tester, or smaller scale specimen slabs. For the RCTs, the reflective cracking tester was divided into 12 areas, as seen in Figure VI-61. Next, a paint brush was used to apply a specific weight of CRS-2 emulsion onto the surface of each of these areas. The weights were recorded for each area. For the NBF trial samples, only six areas were marked out. For the main phase of NBF testing, each slab consisted of a single area.

1	3	5	 7 	9	11
2	4	6	 8 	10	12



Chip seal

The placement of chip seal on asphalt slabs was performed for the RCTs and on smaller slabs used for the NBF tests. The laydown procedure was the same for both types of test, the only difference being the number of areas that needed to be placed. For the NBF trial samples, only two areas were marked out, and the chip seals were placed simultaneously (Figure VI-63). For the main phase NBF testing, however, each slab consisted of a single area. For the RCTs, the slabs were divided into ten separate areas of chip seal (Figure VI-64), which were placed two at a time.



Figure VI-62. Areas of chip seal placement on NBF trial slab (top view).

1	3	5	7	9
2	4	6 	8	10

Figure VI-63. Areas of chip seal placement on reflective cracking tester (top view).

In order to place a chip seal interlayer, first a premeasured volume of emulsion was poured onto the surface of the AC slab and spread using a notched squeegee (Figure VI-64 and Figure VI-65). Next, a premeasured amount of aggregate was spread over the emulsion (Figure VI-66 and Figure VI-67). Then, a sheet of neoprene rubber was placed on top of the aggregate (to prevent crushing of the aggregate), and 16 passes of a steel wheel roller (without vibration) were used to embed the aggregate into the emulsion (Figure VI-68). The whole process was completed within four to five minutes. The chip seal was then left to cure for at least 24 hours. For the NBF test slabs, after this curing, the surface of the sample was swept and vacuumed to remove as much loose aggregate and dust as possible. For the RCT slabs, due to the large number of areas on the tester, the process took somewhat longer. On the first day, Areas 1, 2, 9, and 10 were placed two at a time. On the second day, Areas 3, 4, 7, and 8 were placed two at a time. On the third day, Areas 5 and 6 were placed simultaneously. Next, 24 hours were allowed for the center sections to cure, and the entire slab was swept and vacuumed. Figure VI-69 shows a typical chip seal area after sweeping. Lastly, a CRS-2 tack coat was placed on top of the chip seal in order to help promote bonding with the overlay. The final application rates for the chip seal interlayer were approximately 0.4 gal/yd² (0.3 gal/yd² residual) for the emulsion and approximately 15 lb/yd^2 of aggregate retained after sweeping, with a 0.08 gal/yd² (0.06 gal/yd² residual) tack coat placed on top of the cured chip seal.



Figure VI-64. Pouring CRS-2 emulsion for chip seal placement (NBF test slab).



Figure VI-65. Spreading chip seal emulsion (NBF test slab).



Figure VI-66. Spreading chip seal aggregate on surface (NBF test slab).



Figure VI-67. Spreading chip seal aggregate on surface (NBF test slab).



Figure VI-68. Rolling chip seal with rubber sheet using a steel-wheeled roller (NBF test slab).



Figure VI-69. Chip seal after sweeping (NBF test slab).

Geosynthetics

The remaining interlayer types used in this research were geosynthetics, most of which (excluding the grid) called for the placement of binder on the surface of the slab followed by the immediate placement of the geosynthetic. The placement of the binder was similar for both the NBF test slabs and the RCT slabs, the only differences being the number of areas marked out and the procedure for placing the geosynthetic itself.

For the RCTs, the slab was divided into five areas (Figure VI-70). Then, the required quantity of PG 64-22 binder for each area was weighed and placed in a glass jar with a metal lid containing holes. When the binder was poured from the jar, these holes produced streams of liquid binder that were spread slowly over the surface (Figure VI-71). This distribution produced a randomized grid pattern of binder on the surface of the asphalt slab. Because this grid pattern was not representative of binder placement in the field, a temperature-controlled heat gun and a spatula were used to heat and smooth the binder into a more regular surface texture (Figure VI-72 and Figure VI-73). Because this process takes several minutes to perform on a small slab and several hours to perform on a large slab, the binder had cooled to room temperature long before the geosynthetic could be placed. Because placement of a geosynthetic on cool asphalt binder could lead to poor adhesion that could in turn lead to problems during compaction, the binder needed to be heated to its softening point prior to the placement of the geosynthetic.

To achieve this objective, the NBF test slabs were placed in an oven at 300°F (149°C) for five minutes before pressing the geosynthetic into the soft binder by hand. For the large RCT slabs, heat lamps were used to heat the binder on the surface of the slab for 15 minutes to achieve a temperature between 158°F and 212°F (70°C and 100°C) (Figure VI-74). Once the heat lamps were removed, the geosynthetic was placed quickly on the surface and rolled with a steel roller (no vibration) to improve the seating of the material. Figure VI-75 shows the RCT slab after the placement of a geosynthetic prior to overlay placement.



Figure VI-70. Areas of PG 64-22 binder for geosynthetic placement on reflective cracking tester (top view).



Figure VI-71. Pouring PG 64-22 binder onto surface of RCT slab.



Figure VI-72. Smoothing the binder using a heat gun and spatula (RCT slab).



Figure VI-73. Final surface texture of smoothed binder (RCT slab).



Figure VI-74. Heat lamps used to heat binder for geosynthetic application (RCT slab).



Figure VI-75. Slab after placement and rolling of Paving Mat #1 (RCT slab).



Figure VI-76. Paving Mat #1 near crack in existing support slab (RCT slab).

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APPENDIX VII: CONSTRUCTION GUIDELINES

INTERLAYERS FOR REFLECTIVE CRACKING MITIGATION: CONSTRUCTION GUIDELINES Originally Created: 10-10-2014

Background for reflective crack-mitigating interlayers

Reflective cracking is a major distress that often is experienced when placing thin asphalt concrete (AC) overlays on existing cracked pavements. These reflective cracks generally appear within a couple of years after paving and can lead to accelerated failure of the overlay. As such, many methods have been implemented over the years in an attempt to mitigate reflective cracking. Of these, thin interlayer treatments frequently are used. The most common types of thin interlayers seen are geosynthetics and bituminous surface treatments (BSTs).

Geosynthetics are planar materials that are shipped to the job site in rolls. These products are fabrics, grids, or combinations of both, and frequently are made of polymer and/or fiberglass. In general, most geosynthetic interlayer systems are placed by spraying a heavy application of asphalt binder onto the existing pavement or leveling course and then placing the geosynthetic on top, followed by the overlay. It should be noted, however, that not all geosynthetics are placed in this manner, and thus, the manufacturer's recommendations for the specific product being used for the given project should be verified prior to construction.

BSTs, such as chip seals, consist of a heavy application of asphalt binder covered with aggregate chips. These non-proprietary products are widely used by state departments of transportation, and their construction should be familiar to both construction personnel and engineers. BSTs that require specialized construction equipment are not within the scope of these guidelines.

Both types of interlayers included in these guidelines provide one or more of three basic functions: reinforcement, stress relief, and waterproofing. In general, stiff products, such as grids and paving mats, are intended to provide reinforcement of the overlay and help prevent cracks from reaching the surface of the pavement and widening. Softer products, such as nonwoven paving fabrics and chip seals, contain high amounts of asphalt binder, thus helping to absorb some of the stress that induces cracking. Another benefit of many interlayers is their ability to reduce water infiltration through the pavement structure. Even if the overlay eventually cracks, the high asphalt binder content in these interlayers can act as a water-resistant barrier, thereby helping to prevent water from reaching the subgrade.

Although many factors affect the selection of a reflective cracking mitigation strategy for any particular project, one of the most important factors for the long-term success of the reflective cracking mitigation interlayer is proper placement. Many cases of failure have been documented in the literature as a result of poorly constructed interlayers and/or overlays placed on top of such interlayers. As such, the construction guidelines presented here are intended to inform

construction personnel and engineers who may be unfamiliar with all of the considerations needed for proper placement of these systems in the field.

Construction guidelines for all interlayer types

Existing pavement preparation

- Fill existing pot-holes with acceptable patching material.^{22,35}
- Repair obviously failed areas (severe alligator cracking, large settlements, etc.).
- If no leveling course is to be placed, fill existing cracks larger than 1/8 inch with an acceptable material.^{3,6,14,20} Note: Paving over fresh crack fill material may cause bumps in the overlay.²⁶ The experience of North Carolina Department of Transportation (NCDOT) personnel has shown that it may be advisable to fill cracks at least a year or two prior to paving.
- Ensure that the pavement is dry on the day of paving. Excessive moisture on the surface will compromise interlayer bonding,¹⁴ and moisture in the cracks has been reported to cause blistering of certain products.^{7,35}
- Sweep the surface clean prior to placement of the interlayer system.^{32,35}
- Ensure pavement temperatures are warm enough for paving (pavement temperatures of 40°F and higher, air temperatures of 50°F and higher).^{14,35}
- Be aware that high pavement temperatures (130°F) may cause problems with excessive pick-up on the tires of construction vehicles in the case of geosynthetics or chip seals.^{2,3,14,35}

Use of a leveling course

- Consider the use of a leveling course prior to interlayer construction. Leveling courses may improve the performance of all interlayer types by preventing loss of the tack coat that may penetrate into existing cracks (which would cause the reduction of the bond strength near the cracks).^{13,17,34}
- Consult the manufacturer's recommendations for details. Stiff reinforcing products often require a smooth existing pavement or a leveling course and cannot be placed on rough existing pavements or milled surfaces.^{14,16,17,19,32,34,35,41,43}

Overlay thickness

• Place AC layers that are at least 1.5 inches thick over all types of interlayers. Note: Some product manufacturers may require thicker layers for specific interlayer types.^{6,14,29,35}

Construction guidelines for geosynthetic interlayers

Geosynthetic type

• Use only products that are designed specifically to be used as interlayers for paving applications in the construction of geosynthetic interlayers.

Considerations

- Minimize construction traffic on the geosynthetic; especially avoid abrupt turning or starting/stopping movements that can damage the interlayer.^{14,19,22}
- Do not allow vehicular traffic on geosynthetics to avoid damage to the pavement system and geosynthetic product.^{3,6,30}
- Do not place geosynthetics in areas of high braking or acceleration, such as intersections, to avoid potential slipping of the overlay.^{18,35}
- Take care when placing geosynthetics on grades. The placement of geosynthetics on grades may cause slippage of paving equipment during construction (either the paver or the rollers).^{27,28,31,42} Consult the manufacturer's recommendations, especially when construction is to take place on slopes of seven percent or more (as seen in the trial project associated with these guidelines).
- Place geosynthetics that can act as a waterproofing barrier across the full pavement width, because their benefit is reduced if they are placed in only one lane.²⁵
- Select geosynthetic widths such that the geosynthetic does not overhang the edge of the pavement. 'Sandwiching' the geosynthetic between the pavement layers can help to reduce the possibility of water intrusion during the service life of the pavement and mitigate the possibility of damage during shoulder maintenance or paving.
- Place longitudinal overlaps of adjacent rolls of geosynthetics at the lane lines rather than in the wheel-paths, if possible.³⁵
- If milling is performed, do not create excessive vertical height variations between milled and unmilled areas if both are to be covered with geosynthetic interlayers.³⁵

Tack coat

- Always follow the manufacturer's recommendations. Many geosynthetic interlayers are rolled onto tack coats that are placed on the surface of the underlying pavement, and heat from the overlay can cause this binder to work up through the geosynthetic, thereby bonding the overlay to the geosynthetic.³ For this reason, many commonly used geosynthetics do not require a tack coat to be placed on top of them. Notable exceptions do exist, where the geosynthetic is placed first, followed by the tack coat. Additionally, some geosynthetics may be self-adhesive. Thus, care should be taken to follow the manufacturer's recommendations and apply the tack coat and the geosynthetic in the correct order.
- Ensure that the distributer used for tack coat application is calibrated properly and is in good working order (spray bar height, nozzle type, nozzle angle, cleanliness, etc.).³⁵ Because the proper tack coat application rate is critical for the performance of the interlayers, electronically-controlled distributers are strongly recommended.³⁵
- Use appropriate application rates. Target application rates will vary depending on the surface texture of the pavement on which the geosynthetic is being placed and the type of

geosynthetic being used. Follow the manufacturer's recommendations for selecting the proper application rate.

- Use appropriate adhesive material. Because application rates are relatively high (typically 0.15-0.25 gal/yd²), hot asphalt binder (PG 64-22) is recommended instead of emulsified tack to prevent runoff, pooling, and excessive curing times.³ Note: Using hot asphalt binder as a tack coat for the geosynthetic may be a manufacturer's requirement, depending on the product type.
- If emulsified asphalt binder is used, ensure that it has cured completely prior to placement of the overlay.³⁵ Note: Only certain geosynthetics allow the use of an emulsified tack coat, so check the manufacturer's recommendations.
- Use the appropriate binder application rate for milled or rough surfaces. Increased binder application rates of 0.05-0.1 gal/yd² may be needed on milled surfaces and areas of rough surface texture.^{3,35} Note: Not all geosynthetics can be placed directly on a milled surface, so check the manufacturer's recommendations.
- Apply hot asphalt binder at a temperature of 290°F or higher.³⁵
- Spray the binder two to four inches wider than the width of the geosynthetic itself to ensure adequate overlap.³⁵

Application of the geosynthetic

- Ensure that the geosynthetic is oriented correctly on the pavement. Most geosynthetics have a top and a bottom side and should always be placed in the correct orientation. In many cases, the more textured side is placed facing down in order to promote adhesion of the geosynthetic to the underlying tack coat. Some geosynthetics are self-adhesive on one side to promote bonding to the underlying pavement. For these reasons it is important to follow the manufacturer's guidelines for orientation of the particular geosynthetic being placed.
- Use a specialized tractor equipped with a broom to apply the geosynthetic onto the pavement.³⁵ A contractor that specializes in the placement of geosynthetics should be employed to achieve the best results.
- Place the geosynthetic soon after the binder is sprayed onto the surface to ensure that the binder is warm enough to allow the geosynthetic to adhere to it. Do not allow the distributer to get too far ahead of the geosynthetic application equipment, as the binder may cool too much before the geosynthetic can be applied.³⁵
- Use a pneumatic tire roller to seat the geosynthetic into the binder (two to four passes).⁴⁵ However, discontinue rolling in hot weather if the tires of the roller begin to pick up the fabric, which may damage the geosynthetic.

Overlap

• Overlap adjacent rolls of geosynthetics in both the longitudinal and transverse directions according to the manufacturer's requirements (typically 4-6 in. and 2-4 in. respectively for paving mats and paving fabrics, and 3-6 in. and 1-2 in. respectively for grids).^{4,14,35,41}

- Overlap the rolls in the direction of paving to help prevent pick-up by construction equipment.³⁵
- Place the tack coat at all of the overlaps in accordance with the manufacturer's recommendations.^{32,35}

Wrinkling

- Avoid wrinkling as much as possible. Wrinkling may be a significant problem if geosynthetics are placed on curves in roadways, if the application equipment is not properly aligned with the pavement, or if the material is not being pulled taut by the application equipment.^{26,35}
- Slit and flatten wrinkles that are wider than one inch, as they can cause cracking in the overlay.^{3,14,19,35}

Paving

- Ensure that the mix placement and compaction temperatures are within the range specified by the geosynthetic manufacturer, as excessively hot AC may melt certain products.^{3,8}
- Ensure that the temperature is adequate for paving, because mixtures that are too cool may not have the ability to draw the tack coat up through the geosynthetic (depending on geosynthetic type) to promote adequate bonding to the overlay, which may contribute to the inability to achieve adequate overlay density during compaction.³
- Ensure that the most appropriate paver is used. Track pavers generally are preferred over wheeled pavers, especially when placing geosynthetics on grades, although care must be taken not to back these pavers over placed geosynthetics and cause damage.
- Check for pick-up. If pick-up of the interlayers by the dump trucks or the paver is a problem, small quantities of loose mix or sand can be projected onto the surface of the interlayers in most cases.^{1,3,28,32,35,45}

Construction guidelines for BST interlayers

- Ensure that the distributer and chip spreader are calibrated properly and in working order.³⁵
- Use polymer-modified emulsions when possible, as they are preferred over other emulsion types.³⁵
- Ensure that aggregate chips are as uniform in size as possible for best performance of the interlayer system.
- Avoid excessively dusty aggregate that may cause poor bonding between the chip seal and the overlay.
- Place chip seals according to NCDOT standard protocol.
- Allow chip seals to cure completely prior to paving of the overlay to prevent moisture from being trapped beneath the overlay.
- Sweep off excess aggregate from the surface (if present) before placing the overlay.³⁵

• Place chip seal interlayers across the full pavement width. Placing them in only one lane reduces their benefit as a waterproofing pavement layer.

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APPENDIX VIII: IMPORTANT MTAG TABLES

NOTE: In order to facilitate locating these tables in the MTAG, the table numbers in this section are those from the MTAG.

		НМА С						
Type of Interlayers	Chip Seal	Less than 1.2-inch	Min 1.5-inch	Min 2.0-inch	Slurry or Microsurfacing			
Paving Fabric w/ Overlay			Х					
Paving Fabric w/ Chip Seal	Х	Х			Х			
Paving Mat			Х					
Paving Grid			Х					
Paving Composite Grid				X				
Composite Strip Membranes			Х					
AR Chip Seal		Х			X			
PMA Chip Seal		Х			X			
PMRE Scrub Seal		X			X			
X = Acceptable Surface Treatm	X = Acceptable Surface Treatment for Interlayer							

Table 12-1 Cover requirement minimums for various interlayer materials

	Alligator Cracking			Bl Longi and The Tran Cra	ock, tudinal, Non- ermal sverse cking	Ther	Moisture Intrusion		
Interlayer	Load Related	Age Oxidation (Low to Medium)	Age Oxidation (Medium to High)	Low to Medium (CW <1/2 in)	High (12/ < CW < 1 inc)	Low (CW < ¼ in)	Medium (1/4 < CW < ½ in)	High (1/2 in or grater CW)	
Paving Fabric w/ Overlay	Ν	E	G(1)	F	F(2)	G	F	Ν	E(3)
Paving Fabric with Chip Seal	Ν	E	G(1)	G	G(2)	F	Ν	Ν	E(3)
Paving Mat	Ν	E	E(1)	G	G(2)	E	G	N	E(3)
Paving Grid	Ν	E(1)	E(1)	E(1)	E(1)	E	E(1)	E(1)	N
Paving Composite Grid	Ν	E	E(1)	Е	E(2)	E	E	E(2)	E(3)
Composite (Strip) Membranes	Ν	Ν	Ν	E	E	E	E	G	E
AR Chip Seal	Ν	E	E	E	G(2)	F	N	N	E
PMA Chip Seal	Ν	E	E	E	G(2)	F	N	N	E
PMRE Scrub Seal	Ν	E	E	E	E	G	F	N	E

Table 12-2 Anticipated Effectiveness vs. Types of Cracking and Moisture Intrusion

(1) Interlayer with leveling course first

(2) Interlayer with crack filling first

(3) Interlayer dependent on binder application rate

E = Excellent

G = Good

F = Fair

N = Not Recommended

L = Low Severity M = Medium Severity H = High Severity CW = Crack Width

The values in this table are informational based on available information. This table will be updated as more data becomes available.

	Desert		Mountain			Coastal			Valley			
Interlayer	< 5 M AADT	5 to 30M AADT	> 30M AADT	< 5 M AADT	5 to 30M AADT	> 30M AADT	< 5 M AADT	5 to 30M AADT	> 30M AADT	< 5 M AADT	5 to 30M AADT	> 30M AADT
Paving Fabric w/ Overlay	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
Paving Fabric with Chip Seal	Y	Ν	Ν	Y	Ν	Ν	Ν	Ν	Ν	Y	Ν	Ν
Paving Mat	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
Paving Grid	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
Paving Composite Grid	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
Composite (Strip) Membranes	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
AR Chip Seal	Y	Y	Ν	Y	Y	Ν	Y	Y	Ν	Y	Y	Ν
PMA Chip Seal	Y	Y	Ν	Y	Y	Ν	Y	Y	Ν	Y	Y	Ν
PMRE Scrub Seal	Y	Y	Ν	Y	Y	N	Y	Y	Ν	Y	Y	Ν

Table 12-3 Use vs. Climate and Traffic

Y = Recommended N= Not Recommended

APPENDIX IX: INVESTIGATION OF FAILURE CRITERIA AND THEIR RELATIONSHIP TO OTHER TEST PARAMETERS

One common problem in laboratory asphalt fatigue tests is selecting an appropriate failure criterion for the test. Ideally, such a failure criterion would be based on easy-to-measure quantities, such as load and displacement of the sample, and would not require specialized equipment or analysis techniques such as digital image correlation (DIC) or visual crack monitoring. One important component of this research was to correlate the observations of cracking seen in the DIC results with those obtained from more traditional failure criteria. The goal was to allow researchers to correlate changes seen in the load- and displacement-related parameters with the physical mechanisms that occurred within the beam fatigue samples and thereby potentially eliminate the need to use DIC to obtain information about these mechanisms under specified test conditions. In the end, this ability allowed the selection of one or more failure criteria to rank the vertical crack development for notched beam fatigue samples (NBF). These efforts are presented in the following sections of this appendix.

Fatigue Tests

Because reflective cracking is a phenomenon that is associated with fatigue in the field, the advantages of fatigue tests are that they can quantify some aspects of the fatigue fracture behavior of the materials. Typically, these tests are performed using fairly small, easy-to-fabricate samples and equipment that is readily available to laboratories. These tests tend to simulate only a single mechanism of reflective cracking, however. Like material-level tests, methods that are used to correlate the results from fatigue tests with field behavior depend on the specific test being conducted and the data collected during the experiment. Various methods, including strictly empirical rankings to use as inputs of fracture properties for a finite element model, have been tried. As mentioned in other sections of this report, the fatigue test.

Background of Beam Fatigue Tests

Beam fatigue tests are not new. Because they simulate one of the mechanisms (bending) of pavement cracking, they have long been viewed as a way to characterize the fatigue behavior of an asphalt mixture. Standard beam fatigue tests (ASTM D7460-10) involve constructing single-layer asphalt beams and subjecting them to cyclic loading in a four-point bending arrangement. These tests can be either displacement-controlled or load-controlled.

Regardless of which test mode is run, both displacement and load data can be captured throughout the test and can be used for analysis of the fatigue behavior of the sample. Because of this ability of the test, many criteria have been developed that characterize the failure point of the specimen and relate this failure to both mechanical changes of the specimen itself (such as crack initiation and growth) as well as to field performance. Because of the complexity of the problem of fatigue in a heterogeneous viscoelastic material and because of inherent differences between the modes of loading, different failure criteria have had different levels of success. As such, one major topic of debate for fatigue testing has been the selection of a failure criterion that can predict the failure of specimens consistently for both load-controlled and displacement-controlled tests. Many different methods have been attempted, but the main methods are stiffness-based approaches, energy-based approaches, and a few other methods.

Stiffness-based approaches. The stiffness-based approaches are the simplest failure criteria used. These criteria involve finding the initial stiffness value of the material at a certain cycle and defining failure as the point where the stiffness has degraded to a certain threshold value. For many years, a reduction of stiffness to 50 percent of the initial stiffness was used as the threshold (Tayebali 1992, Tayebali 1993, Abojaradeh 2007, Al-Khateeb 2011, Shen 2011, Abojaradeh 2013). Critics of this method state that 50 percent is an empirical threshold value and does not correspond to any critical material-level event that clearly separates the behavior (such as cracking) of the material before and after this threshold and thus is not reliable (Rowe 2000, Al-Khateeb 2011). These claims, however, are disputed by other researchers, who believe that the 50 percent reduction in stiffness criterion does correspond to the initiation of cracking within the material (Aborjaradeh 2007, Shen 2011).

Kim et al. (1997) described another approach to define a stiffness-based failure criterion. Because asphalt concrete (AC) is a viscoelastic material, they argued that a better failure criterion would be a 50 percent reduction of pseudo stiffness, as this parameter is able to account for the reduction in stiffness that is due to viscoelasticity and, thus, should be independent of the mode of loading and loading rate. Although this approach is theoretically sound, limited work has been undertaken to apply this failure criterion (i.e., 50% pseudo stiffness) to beam fatigue tests due to their complicated state of stress.

Energy-based approaches. Several studies have proposed various energy-based failure criteria. Early work in this area involved an energy ratio that is defined as the dissipated energy during the initial cycle divided by the energy dissipated by the n^{th} cycle (Equation IX-1) (SHRP 1994).

$$energy\ ratio = \frac{n * w_o}{w_n}$$
 IX-1

where

energy ratio= the energy ratio, w_o = initial dissipated energy, and w_n = total dissipated energy.

By plotting this parameter against the number of cycles, failure is indicated by the curve that deviates from a straight line. However, the difficulty in fitting a straight line to the data as well as the difficulty in determining the point at which the curve deviates from this line are the greatest challenges associated with this method (Abojaradeh 2013). Pronk later attempted to adapt this energy ratio for strain-controlled tests by defining it as the total accumulated dissipated energy until cycle *n* divided by the energy dissipated for cycle *n*, referred to as the *dissipated energy ratio*, or DER (Equation IX-2). This parameter was then plotted against the number of load cycles. Again, difficulties with determining the exact point at which the curve deviates from a straight line remained this method's biggest problem (Abojaradeh 2013). As such, attempts to define failure as a certain percentage of deviation from the 'no damage' line were made.

$$DER = \frac{\sum_{i=1}^{n} W_i}{W_n}$$
 IX-2

where

DER = dissipated energy ratio, W_t = total dissipated energy, and W_n = dissipated energy during cycle *n*.

Later, Rowe and Bouldin provided an alternative concept of the energy ratio, known as the *reduced energy ratio*, to use in formulating failure criteria (2000). By multiplying the stiffness value for a given cycle by the cycle number and plotting this parameter (R_n^s) against the number of cycles, a curve with a peak emerged (Figure IX-1). This peak was present in either stress-controlled or strain-controlled mode, and was believed to be related to the onset of cracking in the sample. Abojaradeh et al. (2013) later expounded on this concept, demonstrating that normalizing the energy ratio by the initial stiffness value could provide a criterion that is consistent for both load- and displacement-controlled tests. A normalized version of this parameter is used as a criterion for ASTM four-point bending tests (ASTM 2012). As such, this method has been used by North Carolina State University (NCSU) researchers in past investigations of layered beam fatigue tests.

$$R_n^s = nE_n^*$$
 IX-3

where

 R_n^s = reduced energy ratio, E_n^* = dynamic modulus, and n = number of cycles.



Figure IX-1. Reduced energy ratio failure criterion (Rowe 2000).

A somewhat different energy parameter that involves the change in dissipated energy from one cycle to the next divided by the dissipated energy for that load cycle was developed by Ghuzlan

and Carpenter (2003) and is referred to as the *ratio of dissipated energy change* (RDEC). By plotting this parameter against the number of cycles, a curve is produced that decreases initially, reaches a 'plateau value' (or PV) where it remains for a considerable amount of time, and then begins to increase again rapidly (Figure IX-2). This point is considered the point of failure of the mixture. High scatter in the data is the biggest limitation of this parameter (Chiangmai 2010, Abojaradeh 2013).

$$RDEC_n = \frac{DE_n - DE_{n+1}}{DE_n * (n+1-n)}$$
 IX-4

where

 $RDEC_n$ = the average ratio of dissipated energy change at cycle *n*, compared to the next cycle n+1,

 DE_n and De_{n+1} = dissipated energy produced in load cycles *n* and *n*+1, respectively.



Figure IX-2. Plateau value concept (Ghuzlan 2003).

Another energy-based approach was developed by Al-Khateeb and Shenoy (2004). This approach involves the hysteresis loops of the material and notes changes to the shape that indicate significant damage of the sample. By comparing the expected outputs from an undamaged material to the measured outputs, a failure criterion was developed that correlates these two factors (Figure IX-3). Later refinement of this method includes determining the R^2 value of the difference between the hysteresis loop at cycle *n* versus the initial hysteresis loop (Al-Khateeb 2011). Plotting the R^2 value versus the number of cycles produces a bilinear relationship on a lin-log scale (Figure IX-4). This method is viewed as having the potential to allow continuous monitoring of the changes in the sample until complete failure.



Figure IX-3. Change in hysteresis loops during fatigue testing (Al-Khateeb 2011).



Figure IX-4. Correlation coefficient failure criterion (Al-Khateeb 2011).

Shen (2011) developed another energy-based approach that involves a parameter, ΔA . This approach is based on comparing the area under the dissipated energy curve and the area of a trapezoid that connects the initial dissipated energy to the current dissipated energy (Figure IX-5). It was found that in a strain-controlled test, this parameter reaches a peak value and then decreases (Figure IX-6). It was believed that this peak value corresponds to the beginning of macrocrack propagation in the sample.



Figure IX-5. Trapezoidal area vs. area under the dissipated energy curve concept (Shen 2011).



Figure IX-6. Shen ΔA results (2011).

Other approaches. Another approach to determine failure in an AC sample takes advantage of the fact that during a cyclic test using undamaged visco-elastic material, a certain time lag occurs between the peak stress and peak strain (Figure IX-7). This time lag is related to the phase angle of the material. As damage occurs in the sample, this time lag begins to change (Figure IX-8). When the material is sufficiently damaged, the phase angle begins to decrease (Zhang 2013). This drop in phase angle has been used as a failure criterion to determine the onset of macrocracking for beam fatigue samples (Rowe 2000, Shen 2011).



Time (s) Figure IX-7. Definition of phase angle (δ) .



Figure IX-8. Phase angle drop fatigue criterion (Zhang 2013).

Applicability of Failure Criteria using Layered Asphalt Samples

Several researchers have performed beam fatigue tests on multilayered asphalt samples. In general, these tests have shown that interlayers between the asphalt layers do have a measurable effect on the fatigue life of the samples, and that different mechanisms are at work for samples of different interlayer systems (Jimenez 1985, Lytton 1989, Doligez 1996, de Bondt 1999, Sousa 2000, Brown 2001, Bennert 2009, Vismara 2012). However, no study has been performed that provides a comprehensive evaluation of the various fatigue criteria in combination with DIC or visual crack monitoring using such samples. The major challenge with such a study is that, due to the complicated geometry of the layered specimens, traditional beam theory cannot be used to approximate the distribution of strain with depth. This problem worsens with the presence of a notch cut into the test beam, and later, a propagating crack. Thus, the parameters that rely on stresses and strains cannot be calculated accurately and will only be approximations. Furthermore, the validity of many of these criteria in the presence of localization is questionable. Despite these facts, it is believed that these fatigue criteria may still be able to identify and quantify the differences in fatigue life that are seen in samples that contain different interlayers and to help explain the mechanisms that drive the crack propagation through the sample. The following sections describe the results of this investigation.

Notched Beam Fatigue Test Results

All interlayer systems used in the field research were evaluated for this investigation. Two main types of information were gathered: DIC information and load and displacement data from the servo-hydraulic test machine. Both types of information provided insights into the behavior of the various interlayer systems subjected to this type of loading. Table 5-1 shows the number of tests completed.

Digital Image Correlation Information

DIC measurements were the most informative data gathered from the notched beam fatigue (NBF) tests. From these results, information about crack propagation in the beam and interfacial behavior of the samples was obtained. Strain contour plots were used to identify areas of cracking and damage as well as to help determine the primary mechanisms involved in causing the motions and deformation of the specimens.

Sample	Tack Coat (Residual)	15°C	20°C	25°C
Tack Coat	0.05 gal/yd ² CRS-2	1	4	1
Paving Mat #1	0.17 gal/yd ² PG 64-22	2	2	2
Paving Mat #2	0.20 gal/yd ² PG 64-22	2	2	2
Paving Fabric	0.25 gal/yd ² PG 64-22	2	2	2
Chip seal	0.3 gal/yd ² CRS-2 & 0.05 gal/yd ² CRS-2	2	2	2

Table IX-1. Completed Notched Beam Fatigue Tests

Figure IX-9 shows the von-Mises strains, which are estimates of the total strain (Equation VI-6), for samples of all of the different types of interlayers tested at 20°C. From these results, general descriptions of damage evolution within the samples could be developed. For the tack coat only samples, cracking proceeded through the bottom AC layer, with minimal interfacial movement

occurring. The crack spent little to no time 'trapped' at the interlayer and rapidly propagated through the top layer. For the geosynthetic samples, cracks began to propagate in the bottom asphalt layer, and interfacial movement started to occur. Once the vertical crack reached the interlayer, the interfacial movement increased significantly. This interfacial movement helped to 'stall' the crack at the interface for some time before cracking in the top layer (both top-down and bottom-up cracks) and causing a full-depth crack to develop. Samples with poorer adhesion (bond quality) tended to see a longer delay in cracking of the top layer due to greater interfacial debonding. For the chip seal samples, crack propagation in the bottom AC layer occurred at the same time as high interfacial movement. Also, stress concentrations due to the chip seal aggregate helped to initiate cracks in the top AC layer early on. Increased interfacial movement occurred once the crack penetrated all the way to the interface. Next, one or more top-down cracks formed in the top layer. Lastly, one pair of top-down and bottom-up cracks eventually joined, creating a full-depth crack. From the DIC information, plots of crack length versus cycles were generated and could help to quantify these results (Figure IX-10).



Figure IX-9. Crack propagation in NBF test.



Figure IX-10. Example of vertical and horizontal cracking in NBF test.

Although von-Mises strains can be used to track crack propagation and damage in the sample, the component strain fields (horizontal, vertical, and shear strains) can be used to understand the mechanisms that cause these areas of high strain. Figure IX-11 through Figure IX-15 indicate that both separation and sliding occurred at the interface for all the samples tested. Both the magnitude and the timing of these interfacial movements depended on the type of interlayer used, as well as the test temperature, with higher temperature tests and tests with interlayer products seeing greater interfacial movement earlier in the tests.



Figure IX-11. Tack coat only NBF test component strain fields.



Figure IX-12. Chip seal NBF test component strain fields.



Figure IX-13. Paving Mat #1 NBF test component strain fields.



Figure IX-14. Paving Fabric NBF test component strain fields.



Figure IX-15. Paving Mat #2 NBF test component strain fields.

These results indicate that nearly all of the interlayer types exhibited increased horizontal cracking compared to the tack coat only samples, and that the Paving Mat #2 and chip seal samples experienced more cracking than the control, Paving Fabric, and Paving Mat #1 samples. For the Paving Fabric and Paving Mat #1 samples, horizontal cracking remained quite minimal until the vertical crack in the samples reached the interlayer, at which time horizontal cracking increased significantly. For Paving Mat #2 and, to a greater extent, the chip seal samples, significant horizontal cracking occurred immediately and increased once the vertical crack reached the interlayer. The differences in behavior are explained by the relative bond strength between the layers. The layers that experienced high interfacial movement were those that were expected to have low interfacial bond strength. Shear test results later helped to confirm this hypothesis. This testing is discussed in Section 5.3.3 of this report.

To help quantify the results obtained from the DIC contour plots, which can be subjective based on the scale and number of contour levels used to generate them, four virtual gauges were placed in the DIC program, as shown in Figure IX-16. These gauges allowed the differential movement of these four points to be calculated. By calculating the vertical and horizontal differential movements of these points, estimations of separation and sliding across the interface were obtained. These measurements helped to confirm the qualitative observations of interfacial movement and crack locations obtained from the DIC analysis. Figure IX-17 and Figure IX-18 show an example of these plots for each of the interlayer types at 20°C. Figure IX-19 shows that the magnitude of layer separation tends to increase with temperature. These plots were used to obtain the magnitudes of layer separation at 15,000 cycles and 30,000 cycles, the maximum values of layer separation and sliding seen throughout the tests, and the number of cycles needed to reach various threshold values of layer separation: 0.1 mil, 0.2 mil, 0.55 mil, 0.98 mil, 1.18 mil, 1.97 mil, and 3.15 mil (0.0025 mm, 0.005 mm, 0.014 mm, 0.025 mm, 0.03 mm, 0.05 mm, and 0.08 mm, respectively).

Lastly, to confirm the results from all of the previous DIC analyses, the DIC displacement information from each test was exported, and exaggerated deflected shapes (10x) were plotted using a user-developed computer program. Figure IX-20 shows that these results agree completely with the conclusions drawn from the earlier observations.



Figure IX-16. NBF test gauge locations.



Figure IX-17. Layer separation graphs (mils) at 20°C: a) tack coat only, b) Paving Mat #2, c) Paving Mat #1, d) Chip seal, and e) Paving Fabric.



Figure IX-18. Layer sliding (mils) graphs at 20°C: a) tack coat only, b) Paving Mat #2, c) Paving Mat #1, d) Chip seal, and e) Paving Fabric.



Figure IX-19. Comparison of layer separation (mils).



Figure IX-20. Exaggerated displacements from DIC measurements.

Load and displacement information

An independent set of data obtained during each test includes the load and displacement information obtained by the servo-hydraulic testing machine. Using these data, several parameters could be determined and compared with the DIC results. First and foremost was stiffness (*S*). By taking the peak theoretical stress value for each cycle and dividing it by the peak theoretical strain value of the beam, the dynamic stiffness value of the sample could be obtained. This stiffness value was used for several purposes and is given by Equation IX-5. It should be noted that because the stresses and strains were calculated using beam theory, they represent theoretical values rather than actual values experienced by the sample, as they do not take into account stress concentrations or localizations due to the notched and layered nature of the beam.

$$S = \frac{\sigma_t}{\varepsilon_t}$$
 IX-5

where

 σ_t = maximum theoretical tensile stress in the extreme tension fiber of the beam (using beam theory) (psi), and

 ε_t = maximum tensile theoretical strain in the extreme tension fiber in the beam (using beam theory).

 σ_t and ε_t can be calculated using Equations IX-6 and IX-7.

$$\sigma_t = \frac{2aP}{bh^2}$$
 IX-6

$$\varepsilon_t = \frac{12\delta h}{3L^2 - 4a^2} \qquad \text{IX-7}$$

where

- a = distance between the loading points of the four-point bending arrangement (in.)
- P = applied load (lb),
- b = average width of the beam (in.),
- h = average height of the beam (in.), and
- L =length of the beam between the supports of the four-point bending arrangement (in.)

Once calculated for each data point during the test, the stiffness value was simply plotted against the number of cycles. Because the NBF tests were performed with constant displacement amplitude loading, the initial stiffness value was high and continuously dropped throughout the test. It was found that changes in slope of this line tended to correspond to physical changes that occurred within the sample, as seen also in the DIC results. Figure IX-21 presents a representative curve with the major regions and points identified. It should be noted that not all of the samples for all of the conditions tested exhibited all of these characteristics, but rather, this figure is meant to serve as a summary of all the potential mechanisms observed.

The first region of the graph (1) is associated with interfacial movement and cracking in the bottom AC layer. In some cases, both of these phenomena occurred simultaneously at the beginning of the test. During this phase, as the crack approaches the interlayer, a significant drop in stiffness occurs. Often this drop corresponds to a significant increase in interfacial movement, as seen in the DIC images. Due to the fact that the DIC method measures only surface behavior and the fact that stiffness data represent the structural integrity of the entire width of the beam, the vertical crack is likely to reach the interlayer at different times along the width of the beam. Thus, point A is considered the point at which the crack has reached the interlayer across the full width of the beam.

The second region (2) of the test is typically the longest and is characterized by a slow, steady decline in the stiffness curve. This stiffness decrease is due to damage initiation, crack propagation, and viscoelastic/viscoplastic effects within the top AC layer. The end of this region (point B) often is characterized by the initiation of one or more top-down cracks in the beam.

The last region of the test (3) sees an accelerated decrease in stiffness and is associated with the formation of 'dominant' cracks within the top AC layer and their propagation toward one another. The end of this region is marked by point C, where the cracks have reached the full depth of the sample. It should be noted, however, that point C is not well defined, which is likely due to similar problems as point A.

The two major exceptions to this behavior are found in the tack coat only samples and the chip seal samples. For the chip seal cases, no second drop in stiffness value is seen. This outcome can be explained by the presence of multiple stress concentrations in the top AC layer due to the presence of the chip seal aggregate. Thus, even though debonding occurs, crack initiation and propagation in the top layer start early, eliminating the long period of crack arrest (Region 2). For the tack coat only samples, little interfacial debonding means the crack is not stalled at the interface, and thus, no second period of initiation is needed. Rather, a single decrease in stiffness is seen throughout the test, which corresponds to a single crack propagating through the full depth of the sample.



Figure IX-21. General behavior of stiffness drop for NBF test samples.

Although this discussion seems to suggest that the failure of the sample is characterized well by the stiffness curve, stiffness curves from actual test data are generally harder to interpret, with regions that are less distinct (Figure IX-22). Therefore, often it is the DIC information that allows the researcher to determine the points and regions of the graphs more accurately than simply using the stiffness curve itself to find these points. Additionally, even in cases where clearly defined regions are present, the accuracy of this type of analysis is dependent on the scale on which the graph is plotted and is therefore subjective.

Figure IX-23 illustrates the effect of test temperature on the stiffness curves. This figure shows that as the temperature decreased, the stiffness increased. Although it took longer for the initial stiffness value drop to occur (Region 1), Regions 2 and 3 tend to be noticeably shorter due to more rapid crack propagation. This occurrence led to earlier failure compared to during warmer temperatures. It should be noted that in Figure IX-22 and Figure IX-23, the negative stiffness values relate to displacement of the beam, and the positive stiffness values relate to the load needed to pull the beam back to the zero position. Because both the positive and negative stiffness curves are of similar shape, Figure IX-21 can be applied to both positive and negative values.



Figure IX-22. Stiffness curves for all non-grid interlayer conditions.



Figure IX-23. Stiffness graphs at different temperatures: a) Control, b) Paving Mat #1, c) Paving Mat #1, d) Paving Mat #2, e) Paving Mat #2, f) Paving Fabric, g) Paving Fabric, h) Chip seal, and i) Chip seal.

Much like the graphs for stiffness (i.e., Figure IX-22 and Figure IX-23), similar plots can be produced for other parameters and failure criteria used in this study. Each of these types of graphs is explained similarly to the stiffness graphs, taking care to note the strengths, weaknesses, variability, and usefulness as a means to capture the important milestones seen in crack propagation in layered asphalt beams. However, for the sake of brevity, not all of the graphs are presented for this discussion.

Slope of the stiffness graph: The discussion of the stiffness graphs (i.e., Figure IX-22 and Figure IX-23) and their correlation to the major events of crack propagation in the beams demonstrates that most major cracking events happen near significant drops in stiffness values. As such, the next logical step is to consider using the slope of the stiffness curve (ΔS) in an attempt to better visualize the regions of stiffness change. However, considerable variation in stiffness was evident from peak to peak, and thus, the stiffness slopes calculated from secant lines between each data point were found to be extremely noisy. Because data were collected for only five seconds out of every 60 seconds of testing time, an average stiffness value over 25 cycles could be determined for each data acquisition block, which had the effect of smoothing the data for each 60-second period. Next, the slope of the slope of the stiffness graph. Unfortunately, this process still produced extremely noisy graphs. Rather than looking at adjacent data acquisition blocks, the final smoothing procedure involved obtaining the secant slopes between data acquisition blocks 1,200 cycles ahead and behind the current number of cycles under consideration, as per EquationIX-8.

$$\Delta S = \frac{S_{i+4} - S_{i-4}}{2400}$$
 IX-8

where

 ΔS = slope of the stiffness curve (psi/cycle) S_{i+4} = stiffness value 1,200 cycles ahead of the current data acquisition block (psi), and S_{i-4} = stiffness value 1,200 cycles behind the current data acquisition block (psi).

Figure IX-24 shows the general behavior of the stiffness slope graph. Again, like the stiffness graph, both positive and negative values are seen, thereby correlating to the displacement of the sample and pulling the sample back to zero. In many tests, two minima in change in stiffness are seen. These minima correspond to the local minimum slopes in the stiffness graph. After each minimum, the stiffness slope tends to level off. These points are considered to be the points at which the cracks have propagated the full depth of the layers. Specifically, the point after the first stiffness minimum where the slope begins to level off is when the crack has reached the interlayer, and the point after the second stiffness minimum where the slope begins to level off is when the crack has reached the full depth of the sample. By comparing the results at different temperatures, the minima tend to be shallower as the temperatures increase. However, it must be noted that these regions may not be as well defined in real tests (Figure IX-25). In cases where drops in stiffness value are not significant, it can be difficult to ascertain definite points from the stiffness slope graphs alone, and often DIC information is needed in order to locate the points more accurately. Another consideration is that both stiffness and stiffness slope characterize only the behavior during the peak and zero displacement of the sample. Other criteria, specifically energy-related criteria, can provide more information about the behavior of the entire load and displacement history of the beam.



Figure IX-24. General behavior of slope of the peak stiffness graph for NBF test samples.



Figure IX-25. Slope of the peak stiffness graph for a NBF test Paving Fabric sample at 20°C.

Fitting of the load response (\mathbb{R}^2): Significant problems were encountered when attempting to correlate the coefficient of determination (\mathbb{R}^2) values based on the shape of hysteresis loops, as previously mentioned (Figure IX-3). As such, a simplified method of fitting a sine curve to the load response measured from the servo-hydraulic testing machine for a brief window of time and obtaining the \mathbb{R}^2 value as an indication of the shape of the load response graph was used (Khateeb 2004). Specifically, a computer program was employed to fit the load response data for each five-second block of data acquired using a general sinusoidal function in the form of Equation IX-9 and to report the \mathbb{R}^2 for each data acquisition block (Equation IX-10).

$$F(t) = A\sin(Bt - C)$$
 IX-9

$$R^2 = 1 - \frac{SS_{res}}{SS_{tot}}$$
 IX-10

where

$$SS_{res} = \sum_{i} (y_i - f_i)^2$$
 IX-11

$$SS_{tot} = \sum_{i} (y_i - \mu)^2$$
 IX-12

and,

 R^2 = coefficient of determination, SS_{res} = sum of residual squares, $\begin{array}{ll} \mathrm{SS}_{\mathrm{tot}} &= \mathrm{total \ sum \ of \ squares,} \\ y_i &= \mathrm{observed \ values,} \\ fi &= \mathrm{predicted \ values, \ and} \\ \mu &= \mathrm{mean \ of \ observed \ values.} \end{array}$

This R^2 value was then plotted versus the cycles in an attempt to see if such a graph was able to characterize the behavior of the beam. Figure IX-26 sketches the general behavior seen in these graphs. Importantly, Region 1 in these graphs was very difficult to see. Even when changing the y-axis scale of the graph, lack of a clear trend from test to test made making rules of thumb for identifying this region difficult. Load responses in this region took the form of those seen in Figure IX-27. Region 2 would often appear as a straight line, with little change in the R^2 , which was due to the fact that the load response curve approximated a sinusoidal function fairly well (Figure IX-28). Once dominant cracks formed, a change in the R^2 was present but difficult to discern. The load shape around point B can be seen in Figure IX-29. Lastly, as the dominant crack propagated through the top layer (Region 3), the load shape became severely distorted (Figure IX-30) and a significant drop in the R^2 occurred. However, defining an objective point to classify failure based on this drop was difficult, especially in cases where the drop was relatively gradual. Also, fitting a straight line to this final region of the graph and projecting this line to the x-axis (which is similar to the method suggested in the literature for R^2 plots of hysteresis correlations (Al-Khateeb 2011)) seemed to over-predict the number of cycles to failure greatly for the samples tested. Significant variations of the shape of the curve were not seen with changing temperature, only that the number of cycles to the drop generally increased as the temperature increased.



Cycles

Figure IX-26. General behavior of the R^2 vs. cycles graph for NBF test samples.



Figure IX-27. Load shape during crack propagation in the bottom layer (initial portion of the test).



Figure IX-28. Load shape during crack initiation in the top layer (middle portion of the test).


Figure IX-29. Load shape during the initiation of dominant cracks in the top layer (final portion of the test).



Figure IX-30. Load shape after full-depth cracking (post failure).

Phase angle: As mentioned previously, the phase angle (δ), which is related to the time lag between the peak load and peak displacement in a cyclic test, has commonly been used as an indicator of failure (IX-13). Due to extreme variability in the phase angle from cycle to cycle, the Solver function in Microsoft Excel was used to fit two sine curves (one for the displacement and one for the load response) over each 25-cycle data acquisition block and to find the time

difference between their peak values to calculate the phase angle. This time difference then could be converted to the phase angle based on the known loading frequency. This fitted phase angle was then plotted throughout the test. For fatigue tests of various types of samples, the phase angle is seen to increase during the test until it reaches a maximum point where a drop in the phase angle occurs. This drop is considered the failure point of the sample (Zhang 2013). In the case of displacement tests, this drop is due to the distortion of the load response and the fitting of a sinusoidal curve to that load shape. For this reason, the phase angle shown in this research is not a true phase angle; even so, this ability of the fitted phase angle to detect changes in load shape means that it can be used to help detect failure of the sample.

$$\delta = \frac{t}{f} \cdot 360^{\circ} \qquad \text{IX-13}$$

where

 δ = phase angle (degrees), t = time (s), and f = frequency (Hz.).

When applied to NBF test samples, two distinct types of behavior were observed. For the tack coat only samples, graphs are of the form seen in Figure IX-31. For other interlayer samples, graphs are in the form of Figure IX-32, which shows that the main difference between these graphs is, in the case of most interlayers, a clear increase in phase angle as the crack approaches the interlayer, and then the change in phase angle per cycle decreases. A comparison with DIC images shows that the crack always reaches the interface before this change in slope takes place. This behavior is in line with the previous discussion about the R^2 , because both the R^2 and phase angle were obtained from the same fitting of the load and displacement data. Distorted load shapes early on in the test due to cracking in the bottom AC laver, interfacial movement, and crack propagation along the interface caused the calculated phase angle to be low initially (Figure IX-33). However, as crack initiation and propagation began in the upper layer of the sample, these mechanisms became far less active, and the change in phase angle began to reflect the changes that are associated primarily with the upper layer (Figure IX-34). Eventually, the phase angle reached a maximum around the time of dominant crack formation in the upper AC layer, and the drop in phase angle occurred (Figure IX-35). Full-depth failure, as seen using DIC, occurs sometime after this drop in phase angle (Figure IX-36).



Figure IX-31. Example of a phase angle graph for a tack coat only (control) NBF test.



Figure IX-32. General behavior of phase angle vs. cycles graph for NBF test samples.

Figure IX-33 through Figure IX-36 demonstrate that fitting the distorted load shape greatly affects the calculated phase angle. The actual peak values of the load versus peak displacement curves are noteworthy because these peak values shift much more than the fitted curve; so, a 'phase angle' based on these values would show a greater drop than the fitted curve. This behavior is expected because cracks, which reflect lack of material, can open and close quite quickly, and thus should be closer to the displacement peaks and minima than the peaks and minima in the overall fitted data. However, because the entire load history is important to

understanding damage, it is believed that the fitted curve provides a better representation of the material's behavior than simply looking at the peak values.



Figure IX-33. Phase angle determination during crack propagation in the bottom layer (initial portion of the test).



Figure IX-34. Phase angle determination during crack initiation in the top layer (middle portion of the test).



Figure IX-35. Phase angle during the initiation of dominant cracks in the top layer (final portion of the test).



Figure IX-36. Phase angle after full-depth cracking (post failure).

Dissipated energy per cycle: Another failure criterion that can help quantify failure of the beam is dissipated energy. Traditional beam fatigue tests calculate dissipated energy based on the peak tensile stress, peak tensile strain, and the phase angle of the material. Because notched layered beams already have stress concentrations at the beginning of the test, and propagating cracks makes this situation worse, distortions of the load response from a sinusoidal function were seen; thus, this simplified method to calculate dissipated energy was not considered reasonable.

Instead, hysteresis loops were used. The use of hysteresis loops has long been understood as a way to determine the dissipated energy for a material under repeated loading (Al-Khateeb 2011). Specifically, by plotting the actuator displacement against the load response, hysteresis loops can be created very simply. A simple coordinate method is used to find the area of these loops (Equation IX-14).

$$A = \frac{(x_1y_2 - x_1y_2) + (x_2y_3 - y_2x_3)... + (x_ny_1 - y_nx_1)}{2}$$
 IX-14

where x_i and y_i correspond to the horizontal and vertical coordinates of each data point used to plot the hysteresis loop (which can be thought of as a polygon).

The only caution with this method is that errors may arise in cases where the polygon selfintersects. In this research, occasionally, due to noise, some data points were found to cause these errors. Spot checks of several of the worst-case scenarios, however, produced errors of less than 0.1 percent in the calculated dissipated energy for that cycle.

Figure IX-37 shows the general behavior of these graphs found from NBF testing. Just as in the cases of the stiffness and stiffness slope graphs, crack propagation in the bottom layer is characterized by Region 1 in the dissipated energy per cycle graph. Depending on the sample tested, this region may be associated with a slight decrease in dissipated energy per cycle, or, in some cases a slight increase in dissipated energy per cycle up to the point where the crack reaches the interlayer. These opposite behaviors seem counter-intuitive looking simply at the peak stiffness of the material in the previous sections (Figure IX-21), which decreases throughout the test. Thus, the dissipated energy per cycle would be expected to decrease continually as well. However, the change in dissipated energy throughout the test also is related to the change in phase angle between the displacement input and the load response. In other words, an increase in phase angle produces a wider hysteresis loop, thereby increasing its overall area and thus increasing the dissipated energy of each cycle (Figure IX-38). If the increase in dissipated energy per cycle due to this change in phase angle is more than the difference due to an overall decrease in peak load values, a slight increase in dissipated energy is evident. If the latter is more significant, then the opposite trend is seen, with a decrease in dissipated energy per cycle during crack propagation through the bottom layer. In either case, once the crack actually reaches the interlayer, a drop in dissipated energy per cycle is frequently seen. The second region of the dissipated energy per cycle graph is characterized by a slow decrease until dominant cracks form in the top layer (point B), where Region 3 begins. The end of this region is characterized by an asymptote, and consequently, the failure point (point C) must be determined subjectively. In this research, changing the temperature did not have much effect on the amount of dissipated energy per cycle, as the results were fairly similar; however, higher temperatures, especially 25°C, tended to cause a larger first drop, which means that these curves tended to be slightly lower than at the other two temperatures.



Figure IX-37. General behavior of the dissipated energy per cycle vs. cycles graph for NBF test samples.



Figure IX-38. Example of the change in the area of hysteresis loops solely due to a change in phase angle.

Change in dissipated energy per cycle: Much like the stiffness slope, the change in dissipated energy per cycle (ΔD) is determined by finding the slope of the dissipated energy per cycle

graph. This determination can be accomplished by averaging the dissipated energy per cycle for the data acquisition blocks and obtaining the secant slope between the data points. Due to high levels of noise in the dissipated energy per cycle graph, smoothing was necessary before a useful plot of slope could be created. For this analysis, the slope of the dissipated energy was obtained as the secant between the data acquisition blocks 1,500 cycles ahead and behind the current cycle under consideration (Equation IX-15).

$$\Delta D = \frac{D_{i+5} - D_{i-5}}{3000}$$
 IX-15

where

- ΔD = average change in dissipated energy (psi/cycle)
- D_{i+5} = dissipated energy per cycle 1,500 cycles ahead of the current data acquisition block, and
- D_{i-5} = dissipated energy per cycle 1,500 cycles behind the current data acquisition block.

Figure IX-39 shows a summary of the behavior associated with the samples tested in this research. This graph is characterized by two local minima (corresponding to rapid drops in dissipated energy per cycle). The points after the minima where the dissipated energy per cycle flattens out are associated with the crack that propagated through the full depth of the layer in question. However, in cases without a significant drop in dissipated energy, especially in cases of Paving Mat #1 and Paving Fabric at a low temperature, the first minimum is very small, or nonexistent, and point A (which separates Regions 1 and 2) may be difficult or impossible to find using this graph alone (Figure IX-40). Also, frequently, the second minimum could be very shallow and broad; this behavior made it difficult to determine objectively where crack localization and propagation began and ended solely by looking at this graph (Figure IX-41). This parameter tended to show a more significant initial drop and a flatter second drop as the temperature increased. It should be noted that this value is related to the RDEC, the only difference being the lack of normalization by the dissipated energy for cycle *n*. In this investigation that used notched layered samples, the RDEC was found to be too noisy to be of use.



Figure IX-39. General behavior of the change in dissipated energy per cycle vs. cycles graph for NBF test samples.



Figure IX-40. An example of a change in dissipated energy per cycle vs. cycle graph without a clear initial peak (Paving Fabric, 15°C).



Figure IX-41. An example of a change in dissipated energy per cycle vs. cycle graph without a definite end to crack propagation (Paving Fabric, 20°C).

Accumulated dissipated energy: From an engineering intuition perspective, the use of accumulated dissipated energy is attractive because it is related directly to the energy consumed by the specimen during the test through a combination of visco-elasto-plastic behavior and damage (including fracture). For the purposes of this analysis, the previously calculated average dissipated energy for each loading block was integrated numerically using the trapezoidal rule to obtain the accumulated dissipated energy. This value was obtained for each data acquisition block throughout the test (Equation IX-16).

$$D_{accum} = 150 \sum_{k=1}^{N-1} (D_{k+1} + D_k)$$
 IX-16

where

 D_{accum} = accumulated dissipated energy (psi),

N = number of data acquisition blocks,

 D_{k+1} = average dissipated energy per cycle for the following data acquisition block (psi), and

 D_k = average dissipated energy per cycle for the current data acquisition block (psi).

A representative graph of this type can be seen in Figure IX-42. The first section of the graph appears fairly linear. In cases where there was a severe drop in dissipated energy per cycle in the initial stages of the test, the accumulated energy graph is nonlinear, especially in the region around point A. However, in many cases, even cases that experienced a noticeable drop in dissipated energy per cycle, this change in slope is difficult to see when viewing the graph at a scale that is on the order of magnitude of the ultimate accumulated dissipated energy (Figure

IX-43). Zooming in to the initial region may or may not be helpful in visualizing this initial change in slope, and can be subjective (Figure IX-44). After this region is a long region (Region 2) of relatively uniform increase in accumulated dissipated energy. As the dominant cracks develop in the top layer, a significant change in behavior and then a slow transition to an asymptote are evident. This asymptote is associated with the complete failure of the sample, although selecting a definite point along this asymptote was not possible. The best that could be achieved was a scale-dependent approximation. Also, it should be noted that samples that showed a high number of cycles to failure tended to show a high ultimate amount of accumulated dissipated energy (Figure IX-45). No significant differences in the shape of these graphs were evident by changing the temperature.



Figure IX-42. General behavior of the change in accumulated dissipated energy vs. cycles graph for NBF test samples.



Figure IX-43. An example of an accumulated dissipated energy vs. cycle graph (Paving Mat #2, 20° C).



Figure IX-44. Zoomed-in view of accumulated dissipated energy vs. cycle graph (Paving Mat #2, 20°C).



Figure IX-45. Accumulated dissipated energy (all tests at 15°C).

 $\Delta A \& \Delta A_w$: As previously mentioned, the ΔA criterion involves the difference between the area under the dissipated energy curve versus the cycles up to cycle *i* and the area under a line drawn through the initial dissipated energy and the dissipated energy at the *i*th value on this same curve (Figure IX-5, Equation IX-17).

$$\Delta A = |D_{accum} - A_{trap}| \qquad \qquad \text{IX-17}$$

where

$$A_{trap} = \frac{(D_{ini} + D_i) \cdot i}{2}$$
 IX-18

where

 $\begin{array}{ll} A_{trap} &= \text{trapezoidal area from the initial stiffness to the current cycle (psi)} \\ i &= \text{current cycle number,} \\ D_{accum} &= \text{accumulated dissipated energy (psi),} \\ D_{ini} &= \text{dissipated energy during cycle #50 (psi), and} \\ D_i &= \text{dissipated energy during cycle #i (psi).} \end{array}$

After calculating this parameter for each cycle, the values were plotted against the cycles. As seen in Figure IX-6, the general form of this curve for a single layered standard AC beam is a single drop in dissipated energy, which produces a nearly S-shaped curve. This curve produces a ΔA curve that shows the local peak behavior, as seen in Figure IX-6. For the tack coat only specimens, this criterion works well, because the shape of the dissipated energy curve is

reasonably close to the assumed behavior of this criterion (Figure IX-46). This phenomenon produces a ΔA curve that matches results reported in literature for peak ΔA that corresponds to failure of the specimen (Figure IX-47). However, in cases of other beams with interlayer systems, the dissipated energy curve tends to see two drops in dissipated energy, and thus, it was expected that these specimens would experience two peaks in the ΔA curve. One problem with this method is that, because the ΔA criterion is defined as the absolute value of the difference between these two areas, many curves were hard to interpret. A significant improvement was realized by removing the absolute value from Equation IX-18 all together and simply looking at the local behavior (minima and maxima) of these graphs. Figure IX-49 shows this improved correlation for a single sample when the absolute value is removed. This value is termed as ΔA_w .



Figure IX-46. Dissipated energy per cycle for tack coat only samples (3 replicates, 20°C).



Figure IX-48. ΔA for a Chip seal sample (20°C).



Figure IX-49. ΔA_w for a Chip seal sample (20°C).

Figure IX-50 describes the general behavior of the ΔA_w graphs for the samples tested during this investigation. Specifically, the initial region of the graph often is characterized by a local peak in ΔA_w , with the second region ending in a local minimum or in some cases a significant change in slope when dominant cracks form in the top layer. The third region sees a local peak (in most cases), which corresponds to the crack reaching the full depth of the sample.



Figure IX-50. General behavior of the ΔA_w vs. cycles graph for NBF test samples.

The advantage of this criterion is its ability to produce local maxima after the points of large changes in dissipated energy. Furthermore, these peaks seem to correlate well with the full-depth cracking of the samples. Lastly, the local minimum, frequently seen around point B, confirms previous work that states that this point (which would correspond to a peak ΔA) is related to the onset of unstable crack growth within the sample (Shen 2011). One disadvantage of this criterion that was found during this research is that these two peaks were not seen for all cases. For materials that do not experience an initial drop in dissipated energy per cycle, the initial peak can be difficult or impossible to see from the ΔA_w graph (Figure IX-51). Also, for materials whose final drop in dissipated energy is slight and gradual, the second peak is significantly reduced, or perhaps eliminated in rare cases (Figure IX-52). Even so, this criterion seems to work well at characterizing the events in most of the cases tested. Most of the cases where the criterion did not work well were due to the lack of the first local peak being clearly identifiable, often at a low temperature (15°C). The two cases where a second peak could not be identified were both at 25°C. One of these tests was a chip seal and the other was a Paving Mat #2 sample that seemed to be an extreme outlier in the testing due to an unusual cracking pattern. One word of caution is that increasing the temperature tended to shift the curves downward, implying that at extremely high temperatures, the second local peak behavior could be eliminated for all types of materials tested. As such, the parameter may not be useful for high temperature testing.



Figure IX-51. Example of a ΔA_w graph without an initial local peak (Paving Mat #1, 15°C).



Figure IX-52. Example of a ΔA_w graph with two local peaks (Chip seal, 25°C).

Energy Ratio and Dissipated Energy Ratio: As previously noted in Equations IX-1 and IX-2, these two energy parameters, the energy ratio and DER, have been described in early work with regard to beam fatigue criteria (SHRP 1994, Abojaradeh 2013). In this study, both of these criteria were found to be roughly equivalent. Although their ordinate values are completely

different, their overall shapes are essentially identical by visual inspection alone (Figure IX-53 and Figure IX-54).

Figure IX-55 describes the general behavior of the samples. It should be noted that Region 1 is very poorly defined for all cases and is nearly impossible to see for almost all samples tested. Only in cases with a significant drop in dissipated energy when the crack reaches the interlayer is a slight change of behavior evident between Region 1 and Region 2. In contrast to this relatively poor indication of the crack propagation in the first layer, once dominant cracks appear in the top layer, a noticeable increase in slope occurs and approaches a second asymptote. Similar to the accumulated dissipated energy, this asymptote is not well defined, and determining a definite failure point was not possible and remains subjective. Fitting straight lines through these data points and selecting offsets to define failure (as reported in the literature) were deemed undesirable approaches due to the arbitrary nature of fitting these lines and selecting offset values. Of these two parameters, the DER parameter showed more noise, particularly near the failure of the sample, due to its reliance on normalization by the energy dissipated during cycle *i*. No significant changes to the shape of the graphs were evident with increasing temperature.



Figure IX-53. Energy ratio vs. cycles (Paving Mat #1, 20°C)



Figure IX-55. General behavior of the DER and energy ratio vs. cycles graph for NBF test samples.

Reduced energy ratio failure criterion: Another energy-related criterion is the previously mentioned reduced energy ratio failure criterion that relies on the peak in the stiffness value

 $(|E^*|)$ multiplied by the number of cycles (n) versus number of cycles (n) graph, as given by Equation IX-3.

Figure IX-56 shows the general behavior of this graph. In most cases, two local peaks are apparent in the graphs, and these peaks are related to the crack reaching the interlayer and the crack propagating through the top layer, respectively; however, these peaks occurred significantly earlier than the cracks detected by DIC. As such, the reduced energy ratio criterion significantly under-predicted the number of cycles for the vertical crack to reach the interlayer and the number of cycles for the crack to reach the full depth of the sample. This result makes sense, because this point tended to lie around point B and the onset of dominant crack growth in the sample, and thereby confirms previous research that indicates that the peak value of this parameter relates to the onset of macro-cracking in the sample (Rowe 2000). For these reasons, this parameter works well at simply ranking the vertical crack propagation through the layered asphalt samples. Increasing the temperature tended to flatten out the curves.



Figure IX-56. General behavior of the stiffness x cycles vs. cycles graph for NBF test samples.

Stiffness criteria: In addition to the previously mentioned stiffness graphs, other stiffness criteria include typical stiffness threshold criteria. These failure criteria involve finding the initial stiffness of the sample (typically the stiffness from cycle #50 or cycle #200) and defining failure as the point at which the stiffness value of the sample has degraded below a certain threshold value. Threshold values of 50 percent and 20 percent of the initial stiffness value are found in the literature. An additional failure criterion is defined as the percentage of the stiffness decrease that occurs at a sharp drop in the initial portion of the test. This parameter is known as the percentage of initial drop, or % *initial drop*. In this research, increasing the temperature had the effect of reducing the initial stiffness value, increasing the % initial drop, and extending the number of cycles to reach the stiffness threshold value.

DIC criteria: Various DIC criteria were utilized for this research, including the number of cycles for cracks to reach the interlayer, the number of cycles for cracks to reach the full depth of the sample, layer separation at cycle #15,000, layer separation at cycle #30,000, maximum layer separation, interfacial sliding, number of cycles to reach various thresholds of layer separation (0.1 mil, 0.2 mil, 0.55 mil, 0.89 mil, 1.18 mil, 1.97 mil, and 3.15 mil), number of cycles to the

initial jump in horizontal cracking, number of cycles to reach two inches of horizontal cracking, number of cycles to reach 2.36 inches of horizontal cracking, and the number of cycles to reach full horizontal cracking in the DIC viewing window. Most of these values could be used only for ranking and comparison with other failure criteria rather than objective indications of failure. Increasing the test temperature tended to cause layer separation to occur earlier in the test and to cause the ultimate values of interfacial movement to be high.

Comparison of Failure Criteria

This research investigated a total of 49 parameters (Table IX-2) that were considered to be simple failure criteria. With all this information, a 49x49 correlation coefficient table was created to help identify the level of correlation between all of these parameters. In this table, values of 1 indicated a perfect direct correlation between the variables, values of -1 indicated a perfect inverse correlation between the variables, and values of 0 indicated no correlation between the variables. Although the font size would be too to fit the table in one page width of this dissertation, the findings are summarized in the following sections of this appendix.

|--|

Parameter Name	Units	Data source	Additional Comments
# Cycle to Reach full depth vertical cracking (DIC)	# of Cycles	DIC Contour Plots	
# Cycles to Reach the Intelayer	# of Cycles	DIC Contour Plots	
# of Cycle to Complete Horizontal Debonding	# of Cycles	DIC Contour Plots	
# of Cycles to Initial Horizontal Crack Jump	# of Cycles	DIC Contour Plots	
# of Cycles to Reach 2 in. of Horizontal Cracking	# of Cycles	DIC Contour Plots	
# of Cycles to Reach 2.36 in. of Horizontal Cracking	# of Cycles	DIC Contour Plots	
Final Horizontal Crack Length	# of Cycles	DIC Contour Plots	Varied based on the exact viewing window of DIC (approximately 63 mm)
# of Cycles to Reach 0.1 mils of Layer Separation	# of Cycles	DIC Gauge Points	
# of Cycles to Reach 0.2 mils of Layer Separation	# of Cycles	DIC Gauge Points	
# of Cycles to Reach 0.55 mils of Layer Separation	# of Cycles	DIC Gauge Points	
# of Cycles to Reach 0.89 mils of Layer Separation	# of Cycles	DIC Gauge Points	
# of Cycles to Reach 1.18 mils of Layer Separation	# of Cycles	DIC Gauge Points	
# of Cycles to Reach 1.97 mils of Layer Separation	# of Cycles	DIC Gauge Points	-
# of Cycles to Reach 3.15 mils of Layer Separation	# of Cycles	DIC Gauge Points	
Layer Separation at 15,000	in.	DIC Gauge Points	
Layer Separation at 30,000	in.	DIC Gauge Points	
Maximum Layer Separation	in.	DIC Gauge Points	Ultimate value of layer separation achieved in the test
iviaximum Layer Silding	in.	DIC Gauge Points	Drop in initial stiffnass in region 1 of
% Initial Stiffness Drop	in.	Stiffness vs. cycles graphs	the stiffness graphs (if present)
20% of # 200 Stiffness	# of Cycles	Stiffness vs. cycles graphs	
20% of # 50 Stiffness	# of Cycles	Stiffness vs. cycles graphs	
50% of # 200 Stiffness	# of Cycles	Stiffness vs. cycles graphs	
S0% 01 # 50 SUITINESS	# of Cycles	Stiffness clope vs. cycles graphs	
First Winima in stiffness slope	# of Cycles	Stiffness slope vs. cycles graphs	
second Minima in surmess slope	# of Cycles	Surmess slope vs. cycles graphs	Deint where accumulated dissinated
Accumulated Dissipated Energy First Slope Change	# of Cycles	Accumulated Dissipated Energy graphs	energy change slope in region 1 (if present)
Accumulated Dissipated Energy Begin	# of Cycles	Accumulated Dissipated Energy graphs	Point where accumulated dissipated energy deviated from the slope of region 2
Accumulated Dissipated Energy End	# of Cycles	Accumulated Dissipated Energy graphs	Point where accumulated dissipated energy reached an asymptote
Accumulated Dissipated Energy Average	# of Cycles	Accumulated Dissipated Energy graphs	Average of begin and end (accumulated dissipated energy)
First local Peak in Reduced Energy Ratio (Sxn vs. n)	# of Cycles	Sxn vs. n (first peak in the reduced energy ratio) graphs	Local Peak in the reduced energy ratio graph (if present)
Peak Reduced Energy ratio (Sxn vs. n)	# of Cycles	Sxn vs. n (peak reduced energy ratio) graphs	Peak in the reduced energy ratio graph
DER Begin	# of Cycles	DER vs. cycles graphs	Point where DER deviated from the slope of region 2
Der End	# of Cycles	DER vs. cycles graphs	Point where DER reached an asymptote
Der Average	# of Cycles	DER vs. cycles graphs	Average of begin and end (DER)
ER Begin	# of Cycles	ER vs. cycles graphs	Point where ER deviated from the slope of region 2
ER End	# of Cycles	ER vs. cycles graphs	Point where ER reached an asymptote
ER Average	# of Cycles	DER vs. cycles graphs	Average of begin and end (DER)
First Local Peak in ΔA_w	# of Cycles	ΔA_w vs. cycles graphs	
Local Minim in ΔA_w	# of Cycles	ΔA _w vs. cycles graphs	
Final Peak in ΔA _w	# of Cycles	ΔA_w vs. cycles graphs	
First Drop in Dissipated Energy	# of Cycles	Dissipated Energy vs. cvcles graphs	
Second Drop in Dissipated Energy	# of Cycles	Dissipated Energy vs. cycles graphs	
First Minima in ΔDissipated energy	# of Cycles	ΔDissipated Energy vs. cycles graphs	
Second Minima in ADissipated energy	# of Cycles	ΔDissipated Energy vs. cycles graphs	
Peak Phase Angle Slope	# of Cycles	Angle vs. cycles graphs	
Phase Angle Drop	# of Cycles	Phase Angle vs. cycles graphs	
R ² Drop	# of Cycles	R ² vs. cycles graphs	
Shear Strength	PSI	Measured from shear test load graphs	

Stiffness and energy-related parameters and full-depth cracking: The parameters that correlated well with the actual number of cycles for the crack to reach full depth (i.e., the DIC full-depth criterion) are reflected by the following failure criteria: reduced energy ratio, phase angle, 20 percent of cycle #200, 20 percent of cycle #50, energy ratio, DER, accumulated dissipated energy, dissipated energy, stiffness peak, ΔA_w , R^2 drop, and dissipated energy per cycle. The reason for these good relationships between ultimate vertical cracking in the sample and these parameters is straightforward. First, the presence of vertical cracks in the sample had a significant effect on the stiffness of the beam. Second, this stiffness directly correlated to the load response of the sample under cyclic loading and, therefore, dissipated energy. These two findings indicate that all the parameters that are related to either the stiffness or the load shape of the graph near the end of the test would correlate highly with the depth of the crack.

Stiffness, energy, cracking in the bottom AC layer, and interfacial cracking: Several of the parameters correlated with one another during the initial portion of the tests. The 50 percent of cycle #50 criterion correlated fairly well with the % initial drop criterion, the first drop in dissipated energy per cycle criterion, the number of cycles for the horizontal crack to reach two inches criterion, and somewhat to the number of cycles to reach 0.55 mil of layer separation criterion. However, distinguishing the effects of these parameters on one another is difficult. Specifically, in cases where extreme drops in stiffness occurred, both horizontal cracking and vertical cracking in the bottom layer occurred simultaneously. For these samples, once the cracks reached the interlayer, horizontal cracking accelerated significantly. Thus, it can be assumed that the weak interfacial conditions helped to accelerate the vertical cracking in the bottom layer and thus increased the stiffness drop, but this analysis cannot determine the percentage of the stiffness drop that was due to interfacial debonding alone and the percentage that was due to vertical cracking in the bottom AC layer.

Looking at the number of cycles needed to reach the interlayer itself indicates that this number correlates to the first slope change in the accumulated dissipated energy, the first drop in dissipated energy, the first minima in Δ dissipated energy, somewhat to the % initial drop, the first local peak in ΔA_w , and the first local peak in the reduced energy ratio criteria.

Interfacial sliding: The magnitude of sliding between layers does not relate significantly to any other parameter.

Layer separation: Some correlations that were expected from engineering judgment were found not to be as significant as hypothesized. Specifically, early research had indicated that layer separation correlates fairly well with the number of cycles to reach full depth. In order to investigate this hypothesis, these parameters were cross-plotted.

Figure IX-57 shows that most samples tested fall along a narrow band, with the chip seal layers falling significantly outside of this area. The reason for this outcome is twofold. First and foremost is that the top AC layers of the chip seal samples contained several areas of stress concentration due to the presence of the chip seal aggregate. Thus, crack initiation in the top layer required fewer numbers of cycles to occur. Secondly, the chip seal aggregate physically took up more space than the thin geosynthetic layers; thus, cracks in the top layer had less distance to travel to reach the surface of the sample. For these reasons, a second correlation

coefficient matrix was created by eliminating the chip seal data and rerunning the analysis. These results indicate that the amount of layer separation does correlate to the number of cycles to full-depth failure for all the control and geosynthetic samples (Figure IX-58). Moreover, removing chip seal samples generally improved the correlations with the parameters that correlated to the full-depth cracking of the sample and generally weakened the correlations with the parameters that correlated with the 50 percent reduction in stiffness criterion.



Figure IX-57. Full-depth vertical cracking (DIC) layer separation @ cycle #30,000 with chip seal.



Figure IX-58. Full-depth vertical cracking (DIC) layer separation @ cycle #30,000 with chip seal removed.

Recommendations for NBF Tests Regarding Failure Criteria

The various failure criteria considered in this study do show some promise in characterizing the behavior in NBF tests, particularly in terms of vertical crack propagation. This information can be in the form of: 1) an empirical ranking of the full-depth cracking in the sample, 2) the ability to identify regions (a range of cycles) associated with the vertical position of the crack within the beam, and 3) finding definite points that correspond well with the ultimate vertical cracking of each of the pavement layers.

Ranking of full-depth cracking: One major advantage of the existing failure criteria is their ability to rank vertical crack growth in the NBF sample without the need to use DIC information. Two criteria in particular proved to be superior for this purpose due to their relative simplicity. First is the 20 percent of the initial stiffness criterion. This parameter proved to correspond fairly accurately to the full-depth cracking observed in DIC analysis (Figure IX-59). The second successful criterion is the reduced energy ratio criterion. This criterion showed better correlation than the 20 percent of the initial stiffness criterion, but it always under-predicted the number of cycles required to reach the full depth of the sample (Figure IX-60). Because both of these criteria require the calculation of stiffness values throughout the test, both should be evaluated together, as little extra effort is required to obtain two independent rankings of the vertical crack growth of the materials. An additional criterion, local peak ΔA_w , was considered a very good criterion for ranking full-depth vertical cracking. In nearly all cases tested, this criterion corresponded well to the final full-depth cracking seen using DIC (Figure IX-61). However, one major downside of this criterion is the need to calculate dissipated energy throughout the test, which is more computationally intensive than simply looking at stiffness-related criteria. This research also suggests that ΔA and, thus, ΔA_w do not work for load control testing (Shen 2011).



Figure IX-59. Correlation of full-depth cracking (DIC) and 20% of cycle #200 criterion.



Figure IX-60. Correlation of full-depth cracking (DIC) and the reduced energy ratio failure criterion.



Figure IX-61. Correlation of full-depth cracking (DIC) and ΔA_w .

Identification of regions associated with cracking: The various parameters calculated in this study were able to provide insights as to the occurrence of major events that took place within the samples. Specifically, by plotting the results, physical changes in the samples became evident as the cracks reached the interlayer. Also, the formation and propagation of dominant cracks in

the top layer of the sample became evident. These phenomena could be investigated simply by viewing the plots of the failure criteria parameters. Of particular interest are the stiffness plots and several of the plots associated with dissipated energy (change in dissipated energy per cycle, dissipated energy per cycle, accumulated dissipated energy, ΔA_w , and phase angle). The advantages and disadvantages of the stiffness plots have been discussed previously. The change in dissipated energy per cycle, dissipated energy per cycle, and accumulated dissipated energy criteria are attractive options because they relate to an easily understandable quantity that correlates directly to the energy that is expended within the beam. Unfortunately, in terms of identifying clear regions of behavior, some samples, particularly those at low temperatures, did not correspond readily to identifiable regions that would allow an accurate determination of the region where the crack approached the interlayer, based on these criteria. As such, these criteria can be used only in some cases and thus are limited for this purpose. Another similar criterion is the ΔA_w parameter. Plots of this parameter can frequently give an indication of the major regions of vertical crack propagation; however, like the energy-related criteria, some test results did not show significantly identifiable features that correspond to the cracks that reach the interlayer.

The phase angle is another interesting parameter that can provide better understanding of these regions. Nearly all cases tested during this research produced identifiable regions that corresponded to the crack propagation through each of the layers in the sample. The only exceptions were the tack coat only samples. However, the major downside of the phase angle drop criterion is that its relationship to the physical behavior of the beam is difficult to comprehend intuitively due to the fact that the phase angle is calculated based on a 'best fit' of the load response. Even so, the phase angle criterion is a good surrogate to predict major events in vertical crack propagation without the aid of DIC.

Taken together, these criteria can give researchers a qualitative idea as to the general number of cycles at which major events in crack propagation occur; however, most cannot provide an exact number of cycles at which the cracking begins and stops, and the selection of these regions can frequently be subjective based on the researcher's judgment and expertise as well as the scale at which the graphs are viewed.

Finding definite points that correspond to the ultimate cracking of each pavement layer: In order to eliminate the subjectivity associated with identifying regions of crack growth in a sample, objective criteria that correspond to definite points in the tests were sought. Two failure criteria showed some ability to perform this task for displacement control testing. First is the ΔA_w criterion. In cases where two local peaks were seen, these peaks tended to correspond to the points when cracks reached the interlayer and when they reached the full depth of the sample (Figure IX-62 and Figure IX-61). Downsides of this ΔA_w criterion are that the first local peak is not always seen and the first peak seems to under-predict the number of cycles for the crack to reach the interlayer. Furthermore, this criterion is limited to displacement control testing, as previously noted (Shen 2011). Lastly, further testing would be needed to verify this criterion's accuracy at higher temperatures. As such, a second criterion, which is related to the phase angle, appears to be better at identifying cracks that propagate through the full depth of the bottom AC layer. Specifically, looking at the peak slope in the phase angle during the initial portion of the test provides an indication of the point at which the crack reaches the interlayer (Figure IX-63). Although the first peak in the ΔA_w and the peak phase angle are not directly equivalent, they are

related to one another (Figure IX-64), with the peak phase angle criterion fitting better with the DIC images of the crack reaching the interlayer than the ΔA_w criterion



Figure IX-62. Cycles to reach interlayer vs. first peak in ΔA_w .



Figure IX-63. Cycles to reach the interlayer vs. peak phase angle slope.



Figure IX-64. Correlation between first peak in ΔA_w and peak phase angle slope.

Horizontal Cracking

Many of the load-based criteria, in particular the energy-based criteria, were found to correlate well with the vertical crack propagation rate within the sample. However, because the vertical crack propagation rate is affected significantly by interfacial debonding (i.e., the crack stalls at the debonded interface), which in extreme cases itself may be an indication of failure, a criterion that correlates with this behavior was sought. Unfortunately, no objective criterion could be determined that could quantify horizontal cracking; as such, no objective ranking could be produced using a combination of the failure criteria used for the NBF tests in order to capture both horizontal and vertical crack growth rates. Lacking such an objective criterion, the hope was that some of these criteria would correlate to the shear strength of the interlayer systems, and that this information could be used to develop a composite failure criterion to determine an overall description of failure within the sample. However, these efforts also were only partially successful (as discussed in the main body of the report). As such, the rankings obtained from the vertical crack growth found in the NBF test samples can be misleading due to their inability to capture interfacial behavior and debonding adequately. At this time, only the behavior of the samples for the specific test conditions of the NBF test can be described using the load and displacement based failure criteria, and thus these criteria are of limited utility beyond purely research related activities.

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APPENDIX X: DISCUSSION OF SHEAR STRENGTH MASTERCURVE

Although it is outside the scope of this research, related research involving shear strength mastercurves has been conducted at North Carolina State University (NCSU) by other researchers on other projects. As such, details of this work are not presented in this report. However, the overall concept of shear strength mastercurves should be mentioned in order to emphasize its potential significance and impact on the improved characterization and acceptance of materials for use as interlayer systems.

Background of Mastercurves

Bituminous materials have long been known to be visco-elastic in nature. That is, at short loading times (high cyclic loading frequency), these materials behave elastically and have a high modulus value, and at long loading times (low cyclic loading frequency) these materials behave like a viscous fluid with very low modulus values. Changes in temperature of the material can be used to achieve the same responses; that is, heating the material causes a decrease in modulus value and more fluid-like behavior, whereas conversely, cooling the material produces an increase in modulus value and more elastic-like behavior. Moreover, bituminous materials are commonly known as thermorheologically simple materials; that is, the changes in their material properties in response to certain changes in temperature can be correlated directly to the changes in properties that are produced by a change in loading rate (and vice versa). Therefore, if the stiffness properties of a material are known for multiple loading rates at multiple temperatures, the behavior of the material at any combination of loading rates and temperature can be calculated.

Pavement engineers have taken advantage of this phenomenon for decades by utilizing a parameter known as the dynamic modulus |E*|. By subjecting a cylindrical sample to uniaxial sinusoidal cyclic loading, two sinusoidal quantities can be obtained: stress (σ) and strain (ϵ). The dynamic modulus value is the ratio of the amplitude of the peak stress value to the peak strain value. By testing a sample at multiple temperatures and multiple frequencies, a curve can be developed for the dynamic modulus value versus loading rate for each temperature. As long as sufficient overlap exists between each of the temperature curves, each set of temperature data can be shifted in the horizontal direction such that it overlaps the adjacent data set to produce a single curve. This shift is accomplished by multiplying the loading frequency of the data to be shifted by a so-called *shift factor*. This shift can be accomplished manually or with the use of automated computer software. Once all the data have been overlapped, a continuous curve can be developed. This curve is known as the mastercurve. It should be noted that due to the shift that occurs, the x-axis is no longer the actual loading frequency, but a reduced frequency. This mastercurve is valid for the temperature of the data that were not shifted, and this temperature is referred to as the *reference temperature*. Lastly, by recording each of the shift factors needed to shift the data for each temperature, a relationship can be obtained between the shift factor and the temperature.

The utility of this arrangement is that, using the shift factor relationship, the mastercurve itself can be shifted to any other temperature by finding the shift factor needed between the desired temperature and the reference temperature. Thus, the stiffness of a material for any combination of loading rate and temperature can be used in numerical simulations.

Applicability to Shear Testing

Work in other areas related to bituminous materials has demonstrated that this time-temperature relationship is valid for other parameters than just stiffness, including the damage of bituminous materials and strength (Chehab 2002). The parameter of most interest for this report is strength. Specifically, separate projects at NCSU carried out by other researchers simultaneously with this research project have investigated the shear strength between asphalt layers. These tests include conditions with and without a tack coat, and conditions with and without grid interlayer systems.

The general concept behind these tests incorporates the use of the Modified Advanced Shear Test (MAST) device to perform monotonic direct shear tests on interface materials at multiple loading rates at multiple temperatures, and to determine the peak strength of the materials. Next, just as with the concept of the dynamic modulus, shear strength mastercurves and shift factor relationships can be generated for these materials. Also, by adding different levels of confining pressure to these tests and creating separation mastercurves for each confining pressure, relationships also can be generated between these mastercurves and confinement pressure values. These relationships then can be used in combination with layered pavement simulations to determine the shear strength properties of pavement layers for any combination of loading conditions and temperatures.

Thus, if shear strength mastercurves are generated for each material that is considered for use in reflective crack-mitigating interlayer systems, layered pavement simulations may be used to verify the materials' suitability to withstand the stress experienced within a particular pavement structure for a given application. Moreover, relating the shear strength required under field loading conditions to a more simplified test condition, such as that used in the European standard Leutner tests, could help to develop a more reasonable shear strength threshold value for these screening tests. Lastly, such mastercurve relationships could help to develop an understanding of the effects of the variability of tack coat placement on interlayer systems and help identify cases of poor interlayer performance that are caused solely by construction variability of this parameter, as seen in the field construction trials associated with this research.

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