# A RATIONAL APPROACH TO THE PREDICTION OF REFLECTIVE CRACKING IN BITUMINOUS OVERLAYS FOR CONCRETE PAVEMENTS

by

THOMAS A. BENNERT

A Dissertation submitted to the

Graduate School-New Brunswick

Rutgers, The State University of New Jersey

in partial fulfillment of the requirements

for the degree of

Doctor of Philosophy

Graduate Program in Civil and Environmental Engineering

written under the direction of

Ali Maher, Ph.D.

and approved by

____________________________

____________________________

____________________________

____________________________

New Brunswick, New Jersey

October, 2009

ABSTRACT OF THE DISSERTATION

A Rational Approach to the Prediction of Reflective Cracking in Bituminous Overlays for Concrete Pavements

By Thomas A. Bennert

Dissertation Director:
Ali Maher, Ph.D.

Hot mix asphalt (HMA) is used as the primary overlying material of concrete pavements during rehabilitation because of its inexpensive nature when compared to most Portland cement concrete (PCC) rehabilitation/reconstruction alternatives. However, due to the majority of the PCC pavements being in average to poor condition, many HMA overlays are exposed to extreme movements (both vertical and horizontal). The combination of associated load and environmentally induced movements creates complex stresses and strains in the vicinity of expansion joints and cracks in the PCC, thus dramatically reducing the life of the HMA overlay, typically in the form of reflective cracking. Reflective cracking is a fatigue cracking distress, which is initiated at the bottom of the HMA overlay and propagates to the surface. When the crack reaches the HMA overlay surface, not only does it affect the ride quality and overall integrity of the pavement surface, but it also creates a path for which water can migrate down into and below the

PCC layer. This can ultimately reduce the overall structural support of the composite (HMA and PCC) pavement and result in a complete pavement failure. Medium to high severity reflective cracking results in poor surface conditions that could lead to poor driving conditions and higher accident rates. Therefore, this research is timely in that it not only addresses the structural integrity of the pavement system, but also the safety of the driving public, which is one of the main objectives of the administration at state agencies.

To better understand the mechanisms associated with the development of reflective cracking, an extensive literature review was conducted. Analysis of the literature review indicated significant gaps in the current state of the practice in using bituminous overlays on PCC pavements. To fill in these gaps, a survey was developed, distributed to the state transportation agencies of all fifty states, and compiled to better define the scope of the research. The survey clearly identified that a major gap in the current state of the practice is linking the field conditions (climate, deflections, traffic levels) to appropriate laboratory testing protocols. Therefore, field test sections were selected with appropriate field forensic testing and traffic collection. During construction of the bituminous overlays, loose mix was collected and brought back to the laboratory for material characterization testing that would simulate the loading conditions associated with the respective test section.

The research conducted during the development of this thesis has led to a rational approach in the prediction of reflective cracking potential in HMA overlays placed on PCC pavements. This methodology utilizes field forensic information that would normally be collected during the evaluation of the PCC/composite pavement prior to rehabilitation and laboratory fatigue and stiffness characterization of the HMA mixture(s), to predict the potential for reflective cracking in the bituminous overlay mixture(s). The extensive laboratory testing and field calibration/verification information utilized in the research has also led to “decision tree” methodology that would allow state agencies to properly select asphalt mixtures for overlaying PCC pavements.

Acknowledgement and/or Dedication

I would like to express my sincere appreciation to Dr. Ali Maher for allowing me to work with him and the staff at the Center for Advanced Infrastructure and Transportation (CAIT) over the past ten years. His guidance and support throughout the course of my research and studies is very much appreciated.

I would like to thank my committee members of Dr. Nenad Gucunski, Dr. Perumalsamy Balaguru, and Dr. Andrew Foden for their advice and encouragement during this research project (as well as other projects) and for providing many engineering tools for which I had gathered from their classes and discussions.

I would to thank the New Jersey Department of Transportation (NJDOT) for providing funding to complete this work. In particular, Dr. Nicholas Vitillo (former Manager of the Research Bureau) for always setting aside time to discuss pavement-related issues, as well as Mr. Robert Blight, Mr. Robert Sauber, and Ms. Eileen Sheehy, for trusting me enough to allow me to "play" with their pavements and materials.

I would like to thank the staff of the Rutgers Asphalt Pavement Laboratory for assisting in some of the material testing. The assistance of Ed Wass Jr., Brett DiFrancesco and John Hencken is much appreciated. I would also like to thank Ed Wass Sr., laboratory technician for the Department of Civil Engineering. His time and effort in helping maintaining and fixing equipment, as well as his general encouragement, is greatly appreciated.

I am tremendously grateful for the love and support of my parents during this whole process. They were very supportive from the start of this whole process, even when I gave them reasons not to be.

Most importantly, I am so grateful and fortunate to be married to the most patient, supportive and loving person that I know. I would like to especially thank my wife Lynn for putting up with me throughout this entire experience.

I would like to dedicate this work to my children, Madison and Jake, who keep me grounded and provide me with a smile no matter how bad my day is. This work is also dedicated to my grandfather, Lawrence Bennert, who passed away before I was able to graduate from my undergraduate studies. His love, support, and lessons of life are memories that will never be forgotten and hopefully passed on to my children.

Table of Contents

Lists of tables

List of illustrations

# CHAPTER 1 – INTRODUCTION

As the nation's infrastructure continues to age, many existing Portland cement concrete (PCC) pavements are in need of rehabilitation. However, due to the expensive nature of many PCC rehabilitation/reconstruction methods, state agencies have resolved themselves in simply overlaying the deteriorating PCC pavements with hot mix asphalt (HMA). Unfortunately, many of these HMA overlays begin to develop transverse cracking over the existing PCC joint/cracks within one to two years of trafficking and environmental loading (Figure 1.1). This transverse cracking is called "reflective cracking". Reflective cracking is one of the major problems witnessed on composite pavements (HMA overlaid on top of a PCC pavement) and is developed due to excessive tensile stress/strain at the bottom of the HMA overlay in the immediate vicinity of the PCC joint/crack. Although the occurrence and existence of reflective cracking has been around for years, there exists a lack of understanding pertaining to the mitigation of reflective cracking and procedures of identifying existing pavements and HMA mixtures prone to reflective cracking.

Some researchers have begun to utilize finite element modeling (FEM) procedures to investigate the factors affecting reflective cracking, many of these are discussed later in this thesis. However, many of these procedures have limited value to actual pavement performance and their true value is simply used for parametric studies. Also, the model development for these applications requires extensive training and time, something that many state agencies are not able to invest in. Therefore, a rational approach to the evaluation, material selection, and modeling of reflective cracking for composite/PCC pavements would be extremely beneficial to the pavement industry.

a) b)

Figure 1.1 – Reflective Cracking on Existing Composite Pavement, a) Early Reflective Cracking and b) Reflective Cracking After 8 Years of Service

1.1 Problem Statement

With the considerable investment state agencies make overlaying PCC pavements with hot mix asphalt (HMA), little understanding exists regarding an appropriate procedure for evaluating the existing pavement structure and identifying appropriate materials to mitigate reflective cracking, especially a procedure that can readily be utilized by state agency engineers.

Laboratory modeling and sensitivity analysis conducted in previously reported studies lack the connection between field conditions (pavement structure, climate, pavement deformations, etc.) and material testing conditions/performance limits. Without the ability to screen various HMA mixtures under identical field conditions, accurate predictions of pavement performance would not be possible. Also, if a procedure could be developed that incorporated pavement responses and field conditions that are measured using conventional pavement testing procedures, a Decision Tree system could be established that would allow state agencies to properly specify appropriate HMA overlay systems.

## 1.2 Thesis Objectives

The goal of this study is to identify and evaluate the critical factors affecting reflective cracking potential of hot mix asphalt (HMA) overlays on Portland cement concrete pavements. Once identified, it is proposed that a rational field and laboratory evaluation procedure be developed to be used in conjunction with a Decision Tree system for HMA overlay design and selection. The specific objectives of this study are:

1. Evaluate the major factors affecting the reflective cracking potential of HMA overlays;
2. Develop a methodology for; a) evaluating the current condition of composite/PCC pavements; b) utilizing measured field movements/conditions in laboratory screening tests; and c) predicting the expected performance life of the selected HMA overlay system.
3. Utilizing field test sections, compare, and if needed, calibrate the above mentioned evaluation procedure; and

4. Establishing a Decision Tree system that state agencies can use when utilizing HMA overlays on composite/PCC pavements.

## 1.3 Thesis Organization

CHAPTER ONE: Problem description and statement of thesis objectives.

CHAPTER TWO: Background and Literature Review to identify known critical factors affecting the reflective cracking potential of HMA overlay systems

CHAPTER THREE: Investigation of the Current State of Practice of Bituminous Overlays Design for PCC Pavements using a national survey submitted to all fifty state agencies to help fill in the gaps of knowledge from the Literature Review

CHAPTER FOUR: Description of Research Methodology

CHAPTER FIVE: Field Evaluation and Testing used to establish laboratory testing procedures for HMA mixture evaluation and the proposed Decision Tree system

CHAPTER SIX: Laboratory Evaluation and Testing used to evaluate the reflective cracking potential of HMA overlay mixtures, as well as used in the Decision Tree system

CHAPTER SEVEN: Analysis of Results generated during the field and laboratory evaluation, program calibration of the prediction methodology, and development of the Decision Tree System

CHAPTER EIGHT: Development of Decision Tree Analysis Procedure

# CHAPTER 2 – BACKGROUND AND LITERATURE REVIEW

A number of studies have been conducted in an effort to minimize or delay the occurrence of reflective cracking. Various techniques that have been used and evaluated vary from simply increasing the HMA overlay thickness to crack arresting interlayers to a three-ply composite that is placed only over the joint/crack area. Although some of these techniques have been successful for mitigating reflective cracking in certain applications, many have performed poorly, particularly in colder climates. Other approaches are centered on fracturing the PCC slab (crack and seat, break and seat, and rubblization). In the case of rubblization, the possibility of reflective cracking is eliminated by fracturing the slab until it resembles a coarse aggregate layer. However, in many cases, the fracturing of the PCC is not cost effective due to the specialized equipment needed to fracture the slab, as well as the need for a thicker overlay to regain the necessary pavement structural capacity. Therefore, most state agencies are relegated to applying a bituminous overlay on the PCC pavement, without a design/prediction methodology that would allow for an estimate of service life.

## 2.1 Mechanisms of Reflective Cracking

Any type of movement taking place within the vicinity of the joint/crack will produce stress and strains in the overlay that can cause it to physically tear. This is purely based on the magnitude of the applied stress being higher than the overlay's resistance to fracture. There are generally three common modes of failure movements associated with reflective cracking (Figure 1.1):

- Horizontal Movement of Slab – Usually temperature associated and causes tensile and bending stresses to develop in the overlay.

- Vertical Movement at the Joint/Crack Area – Primarily load induced and creates shear and tensile stresses within the overlay.
- Parallel Movement of the Slab – Not common, however, the parallel movement may occur if the slab is structurally unstable with minimal frictional resistance.

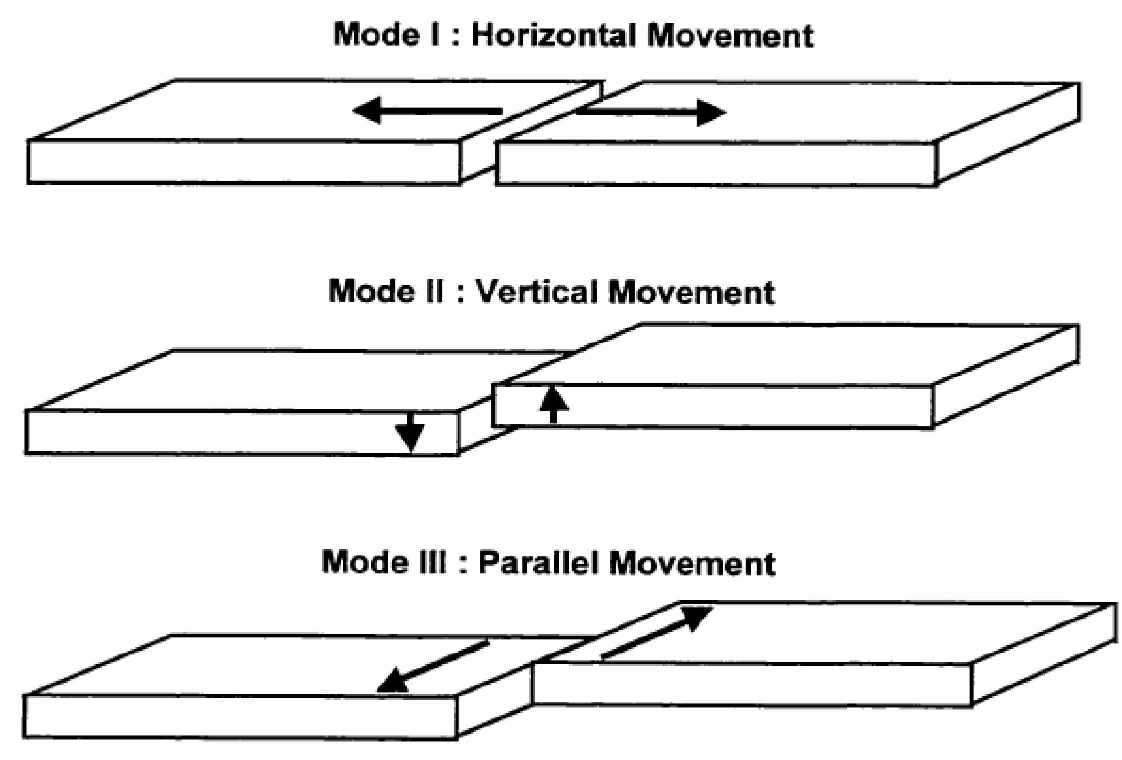



Figure 2.1 – Movements in Underlying Pavement Layers that Contribute to Reflective Cracking (After Mukhtar and Dempsey, 1996)

2.1.1 Temperature Effects

The magnitude of stresses developed in the HMA overlay is not generally associated with the seasonal temperature changes (slow changes in temperature) due to the ability of the HMA to relax under slow moving conditions (Mukhtar and Dempsey, 1996). It is the daily temperature change that has the greatest influence on the performance of the HMA overlay. When the existing pavement contracts during a cooling cycle, the movement creates tensile stresses in the overlay right above the joint/crack. This movement in the PCC depends on the slab length, temperature change, the coefficient of thermal expansion of the PCC slab, and the sliding/frictional characteristics of the PCC slab interface.

A large daily cooling rate combined with a very low temperature at the end of the cooling cycle represents the most critical condition with respect to the development of reflective cracking due to horizontal slab movement (Bozkurt and Buttlar, 2002). This is due to the potential for large horizontal movement of the slab due to change in temperature, as well as low temperatures causing the HMA overlay to stiffen (the stiffer the HMA, the less likely it will relax under straining, resulting in the development of a crack). This "critical condition" was also verified by Bozkurt (2002) using 3-D finite element model simulations.

The unrestrained change in length (horizontal movement) produced by a given change in temperature can be calculated as:

$$L = \alpha_{PCC}\ T\ L \qquad (2.1)$$

where,

L = change in unit length of PCC due to a temperature change of T.
$\alpha_{PCC}$ = coefficient of linear expansion of PCC, strain per °F.
T = temperature change (T2 - T1), °F.
L = length of specimen (i.e., joint spacing)

However, it should be noted that there is typically a reduction in L, which is based on the frictional properties of the PCC slab interface. Test results from the Rantoul General Aviation Airport in Illinois, where the FAA has instrumented a number of the PCC slabs, have shown that a 25 to 45% reduction in horizontal movement can be expected due to the frictional properties (Bozkurt, 2002). Measurements of the $\alpha_{PCC}$ from a wide range of

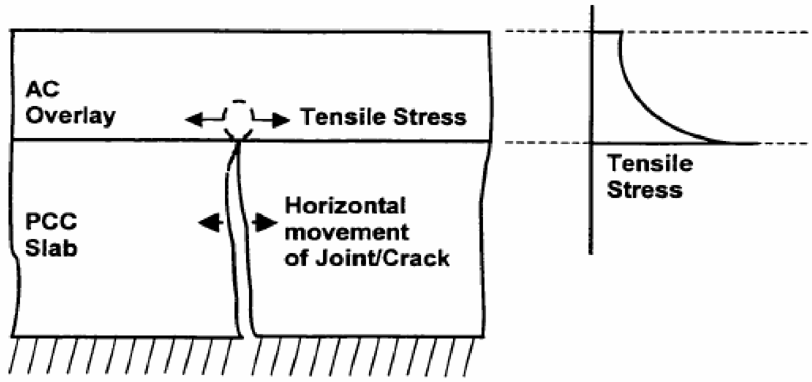
PCC mixes have shown that it ranges generally between 3 and $8*10^{-6}$/°F. This is a very wide range for an important parameter in M-E design for all types of concrete pavements because it affects both critical slab stresses and also joint and crack openings. Therefore, it is highly recommended that the coefficient of thermal expansion (CTE) of the PCC be measured (ERES, 2004).

The daily temperature change will also have an impact on the development of thermal stresses at the surface of the HMA overlay in the form of thermal cracking. Although the low temperature asphalt binder grade has the largest impact on thermal cracking, the daily temperature change, especially when a very low temperature at the end of the cooling cycle occurs, will create additional tensile stresses within the HMA overlay that can assist in the development of the reflective crack.

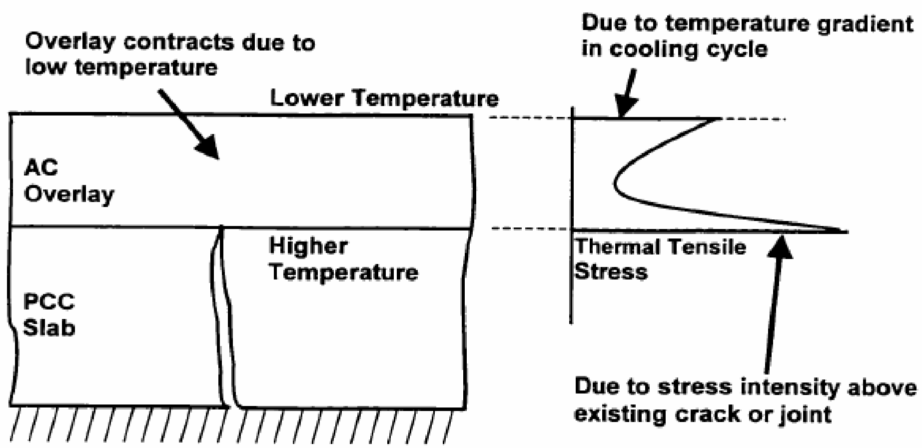
Not only do the daily temperature changes create horizontal movement in the PCC slab, but it also causes curling and warping in the PCC slab that creates both shear and tensile stresses at the bottom of the HMA overlay above the joint/crack. Therefore, simply due to daily temperature cycles at critical temperature conditions (a large daily cooling rate with a very low temperature at the end of the cooling cycle), the following occurs (Figure 2.2):

1. The temperature differential from the daily cooling cause the PCC slab to contract and move horizontally, creating tensile stress at the bottom of the HMA overlay at the joint/crack.
2. Temperature gradients in the PCC slab create a warping of the slab in the evening (at the end of the cooling cycle) which results in an upward curl of the slab at the joint/crack. This creates both shear and tensile stress at the bottom of the HMA

overlay at the joint/crack.




**(a) Contraction Effect of Existing Pavement**




**(b) Contraction Effect of AC Overlay**

Figure 2.2 – Stresses Developed in HMA Overlay Due to Daily Temperature Changes (After Muktar and Dempsey, 1996)

3. The cooling cycle, ending with a very low temperature, creates tensile stresses in the HMA overlay due to thermal contraction of the HMA (thermal cracking). Although the low temperature asphalt binder grade can aid in minimizing the potential for a crack to initiate, tensile stresses are present, with the greatest magnitude being at the surface. However, because the tensile stresses at the bottom of the HMA overlay are greater than at the surface, it is generally assumed that the reflective cracking initiates at the bottom of the HMA overlay.

If the curling forces are large enough, the upward movement can initiate cracking from both the top and bottom of the HMA overlay, creating a "hump" in the HMA overlay at the joint/crack, as shown in Figures 2.3 and 2.4. The photo in Figure 2.4 is taken from NJ Route 34 (southbound) where a sawing and sealing program was implemented, in conjunction with a 3-inch mill and replace maintenance project. The "hump" formed within one month of the overlay being placed. It is interesting to note that the HMA overlay still "humped" due to excessive curling forces even when the HMA overlay was sawed and sealed. However, further visual analysis shows that only the southbound lane was sawed and sealed, while the northbound side remained intact (Figure 2.5). Perhaps if the northbound lane was also sawed and sealed, the HMA overlay may have had a better chance to withstand the curling forces.

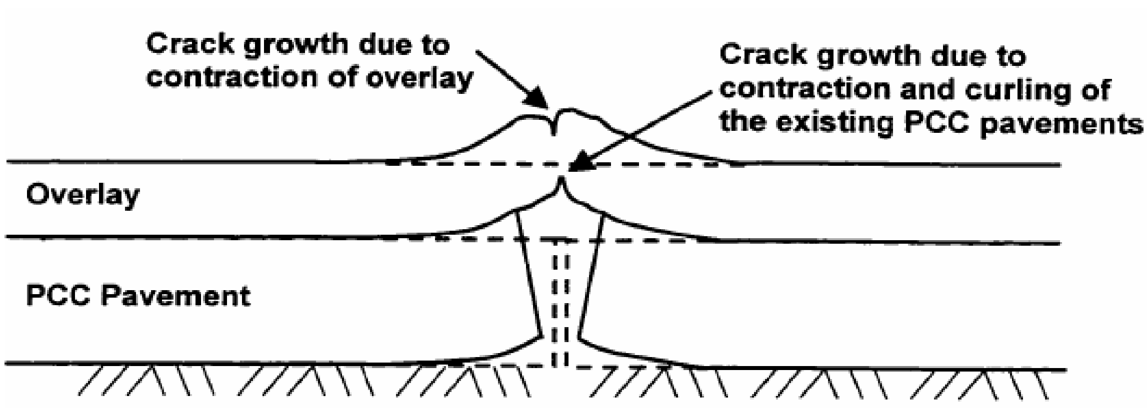



Figure 2.3 – Crack Growth in HMA Overlay Due to Excessive Curling Forces (After Kohale and Lytton, 2000)

2.1.2 - Effects of Traffic Loading

Moving traffic loads create vertical movements in the PCC slabs across the joint/crack. This movement can be caused or amplified by:

Figure 2.4 – HMA Overlay Failure at Joint Area Due to Excessive Curling Forces (Route 34 MP 11.5 Southbound)

Figure 2.5 – HMA Overlay on Route 34 (Picture taken from Northbound Side showing the joint was only sawed and sealed on the Southbound Side)

- Poor PCC slab support or voids located under the joint/crack which is commonly a result of excessive water infiltrating the open joint/crack and pumping out fines; and
- Poor load transfer (poor load transfer efficiency – LTE) causing one slab to vertical move more than the other.

These vertical movements create bending and/or shear stresses in the HMA overlay which is concentrated in the joint/crack zone, eventually leading to reflective cracking. The voids under the joints/cracks cause relatively high bending in this area, which can even lead to failure of the PCC slab in the form of D-cracking (Durability Cracking) and/or Blow-ups (Figure 2.6a and b) under the HMA overlay.

(a)

(b)

Figure 2.6 – (a) Durability Cracking (D-Cracking) of PCC Pavement; (b) Blow-up of PCC Pavement

As the wheel load moves across the PCC joint/crack, three high stress pulses are developed in the HMA overlay, shown in Figure 2.7 (After Lytton, 2000). The stress pulse at points A and C occur as shear stress, with point B occurring as a bending stress. The shear stress at point C can potentially be greater than point A if a void exists under

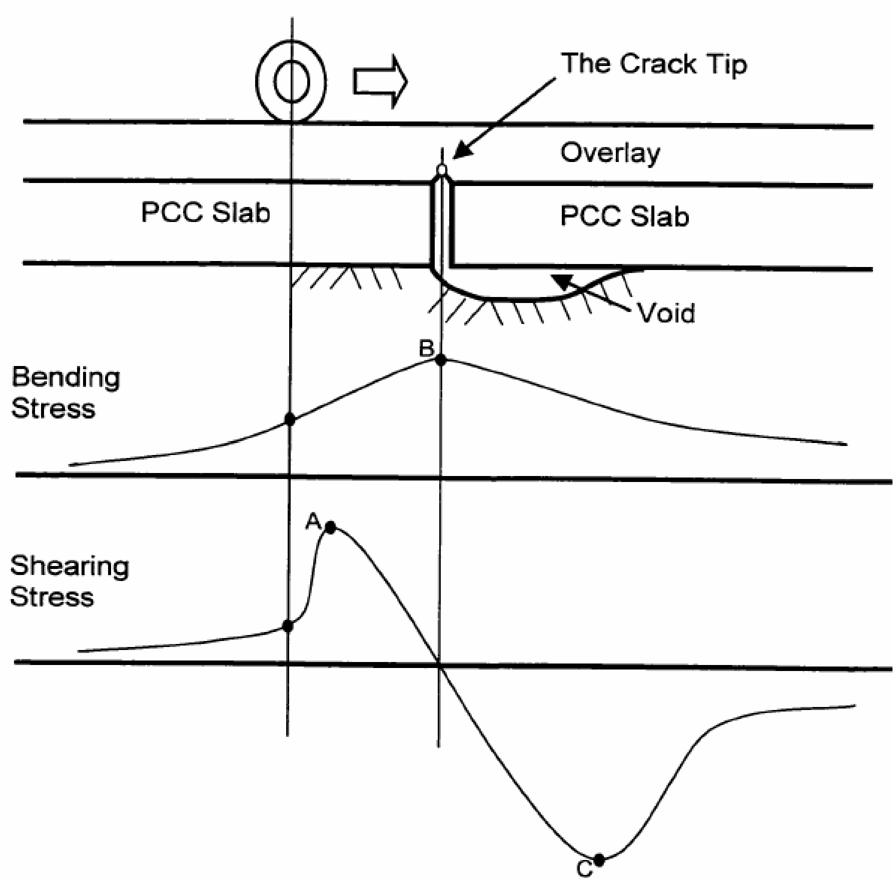



Figure 2.7 – Stresses Developed in the Joint/Crack Area Due to Traffic Loading (After Kohale and Lytton, 2000)

the PCC joint/crack or it is poorly supported. Therefore, both bending and shear stresses are developed due to traffic loading and must somehow be evaluated during the initial pavement evaluation and also in the rehabilitation selection.

Zhou and Sun (2000a) utilized a fracture mechanics-based 3-D FEM analysis to show that traffic loading induced reflective cracking is mainly caused by the deflection on the loaded side of the joint/crack. The deflection creates the effects of both bending stress (due to vertical movement) and shear stress (due to difference in the movements on both sides of the joint/crack). Zhou and Sun (2000a) further showed that the initial crack at the beginning of the upward movement of the crack is controlled by bending, while the further upward movement is due to shear. Therefore, both vertical deflection at the joint and also the relative deflection (also known as Load Transfer Efficiency – LTE) should be evaluated.

Sensitivity analysis using 2-D (Pais, 1999) and 3-D FEM (Hammons, 1997; Zhou and Sun, 2000b; Bozkurt, 2002) models indicate that the magnitude of the bending and shear stresses under traffic loading are also dependent on the subgrade modulus (i.e. – higher the subgrade modulus, the lower the magnitude of bending and shear stress). Therefore, the modulus of the subgrade/supporting material should also be evaluated to aid in determining a rigid pavement rehabilitation strategy. In fact, based on their extensive modeling work, Zhou and Sun (2000b) also recommended optimal HMA overlay thicknesses solely based on the modulus of the supporting material, called Modulus of Foundation (Table 2.1).

Table 2.1 – Recommended Optimal HMA Overlay Thickness Based on "Modulus of PCC Foundation"

| Modulus of Foundation (psi) | 14,500 | 29,000 | 40,000 | 43,500 | 58,000 |
|---|---|---|---|---|---|
| Optimal HMA Thickness (in.) | 5.5 | 4.75 | 3.5 | 3.25 | 2.75 |

It should be noted that values in Table 2.1 do not consider stresses developed due to thermal effects.

### 2.1.3 - Summary of Mechanisms Causing Reflective Cracking in HMA Overlays

Reflective cracking is caused by combination of environmental (temperature) and traffic loading. Daily temperature changes result in the expansion and contraction of the PCC causing a horizontal movement. The daily temperature changes also create temperature gradients in the PCC slab which causes curling and warping. A sensitivity analysis centered on the influence of the temperature on development of critical stresses causing reflective cracking showed that a large daily cooling rate combined with a very low temperature at the end of the cooling cycle represents the most critical condition with respect to the development of reflective cracking due to horizontal slab movement and upward curling of the ends of the PCC slabs resulting in bending and shear stresses in the vicinity of the joint/crack (Bozkurt and Buttlar, 2002).

As anticipated, traffic loading creates both shear and bending stresses around the joint/crack area of the HMA overlay. The magnitude of the stress (bending and shear) was found to be significantly influenced by the deflection at the loaded side of the joint/crack, the differential deflection between the loaded and non-loaded side of the joint/crack, and the support of PCC slab (Zhou and Sun, 2000a and 2000b). If a void or poor support is present, severe distress can also occur in the PCC itself.

Therefore, to properly evaluate HMA overlay mixes under conditions that simulate field conditions (i.e. shearing and bending stresses), the HMA mixtures need to be tested in equipment that can apply stresses relating to actual field movements. Test equipment/protocols that have current AASHTO testing specifications that can accomplish this are:

- Dynamic Modulus, E*, (AASHTO TP62-07) is used to determine the HMA mixture stiffness at various loading frequencies (speed) and temperatures. Not only is E* important for the general characterization of how the HMA mixture responds to different temperatures and vehicle loading speeds, but research has also shown a direct link between modulus and fatigue resistance at intermediate/low test temperatures. Research conducted by Tan (2000) concluded that an HMA overlay having a lower modulus can minimize the formation and development of an opening reflective crack in the joint/crack region.

- Flexural Beam Fatigue (AASHTO T321) applies a bending stress on the HMA sample. The test can be conducted in a stress or strain controlled mode and is the recommended test procedure to evaluate the fatigue properties of HMA materials, especially for the evaluation of potential HMA mixtures for overlaying rigid pavements (Blankenship et al., 2002; Sousa et al., 2002, Makowski et al., 2005).
- Overlay Tester (TxDOT Tex-248-F) applies a tensile stress/strain on an HMA sample through a horizontally applied tensile (pulling) movement (Zhou and Scullion, 2004). This mode of loading simulates the expansion and contraction PCC slabs commonly undergo during climatic temperature cycling.

## 2.2 - Evaluating Structural Condition (Deflections and Modulus) of Rigid Pavement

At the moment, the structural condition of the rigid pavement structure can most efficiently and thoroughly be evaluated using the Falling Weight Deflectometer (FWD), although recent work with a Rolling Dynamic Deflectometer (RDD) in Texas shows promise (Lee et al., 2005). In conjunction with coring the PCC pavement to provide samples for coefficient of thermal expansion (CTE) of PCC and visual analysis of the condition of the PCC in the joint/crack area, FWD testing can be conducted to provide the following:

- Modulus of Subgrade (supporting layers) - FWD testing at the midspan of the PCC slab will provide the modulus values of the supporting material. The lower the subgrade or supporting modulus, larger bending and shear stresses can be expected.

- Deflection at the Joint/Crack – FWD testing with different applied loads at the joint/midspan will provide valuable information on the vertical deflection that can be expected due to varying loads. The larger the deflection at the joint/crack, larger bending stresses in the HMA overlay can be expected.
- Load Transfer Efficiency (LTE) of Joint/Crack – FWD testing at the joint/crack area, using the typical LTE test set-up, will provide valuable information on how much of the load applied to one side of the joint/crack is transferred to the other side of the joint/crack. The lower the LTE (a lower efficiency corresponds to the greater difference in deflections), larger shear stresses can be expected. However, sensitivity analyses conducted (Al-Qadi, 2007) has shown that the shear forces associated with poor LTE work more as a crack accelerator, as opposed to a crack initiator like the bending forces associated with vertical joint deflections from traffic loading. Therefore, a crack needs to first be initiated through either vertical or horizontal deformations for the shear component to truly be detrimental to the pavement structure.

The advantage of "screening" the composite/rigid pavement system using the FWD and CTE lab testing is that a "decision tree" system can be developed over time. It is believed that such a "decision tree" will allow the practitioners to choose the most effective methods and overlay materials by inputting pavement and material properties, such as FWD deflection data, etc.

However, it is understood that this type of screening methodology is not cost effective for every time a rigid pavement is overlaid; such as typical maintenance mill and fill work. In these cases, it is more important to select a HMA mixture that will provide both rutting and fatigue resistance, even if the HMA overlay is to be sawed and sealed.

**2.3 - Review of Reflective Crack Mitigation Methods**

To date, many pavement designers have tried various design alternatives to mitigate reflective cracking. Neglecting methods that result in a major modification to the rigid pavement (such as rubblization which may cause vertical profile issues), the following methods have been tried:

- Increasing thickness (Tan, 2000; Zhou and Sun, 2000a);
- Reflective Crack Relief Interlayers (Blankenship et al., 2002; Makowski et al., 2005);
- Geosynthetic/Geogrid Reinforcements (Muktar and Dempsey, 1996; Bozkurt et al, 2000; Bozkurt and Buttlar, 2002);
- Crack Arresting Layer (Hensley, 1980);
- Petromats and fiberglass tapes (Shuler and Hamerlink, 2004).

A majority of the studies have shown that the use of many of these alternatives, especially the geosynthetics and geogrids, are not cost effective when compared to a well-designed HMA overlay with sufficient thickness. Button (1989) concluded that the use of geotextiles on Texas PCC (continuously reinforced) pavements did not provide any additional benefit in minimizing reflective cracking due to the reflective cracking being caused by thermal gradients (horizontal expansion and contraction of PCC slab). A

44-month monitoring project conducted by PennDot showed that the use of fabrics did not reduce life cycle costs when overlaying PCC with asphalt (Maurer and Malasheskie, 1998). Bozurt et al. (2000) found that the use of geotextiles in Illinois only slightly retarded the reflective cracking on the longitudinal joints and were ineffective at retarding the reflective cracking on the transverse joint. Shuler and Harmelink (2004) compared two different Petromats, a geotextile, a reinforced fabric, and a fiberglass tape to two control sections in Colorado. The sections were monitored for 5 years and concluded that the control sections (4 inch and 5.5 inch thick HMA overlay, respectively) provided the most cost effective method.

The use of a Reflective Crack Relief Interlayer (RCI) system has shown some promise in mitigating reflective cracking. The RCI system uses performance-related specifications for the flexural beam fatigue to resist cracking due to movements in the PCC joint/crack. Extensive work by Blankenship (2005) has shown that as long as the RCI mixture can obtain the required laboratory performance criteria, a 50% reduction in the average crack rate can be achieved. In fact, cores taken from a number of sites have shown that even when cracking occurred in the surface layer, the interlayer itself did not crack (Makowski et al., 2005). The intact interlayer, compacted to low air void levels, further protects the pavement from moisture intrusion. A pilot study conducted in New Jersey in 1997, when the proprietary system developed by Koch Materials was fairly new, indicated that a 68% decrease in the average crack growth rate was achieved with the RCI when compared to the control sections. And, very similar to what was witnessed by Makowski et al. (2005), even when the surface layer cracked, the crack did not propagate through the RCI layer.

Another treatment that is showing promise in mitigating reflective cracking is the Interlayer Stress Absorbing Composite (ISAC). ISAC is a three-ply composite interlayer usually placed as a 36-inch wide strip-type treatment over joints and cracks. The bottom non-woven geotextile layer is provided mainly for manufacturing purposes and to facilitate bonding between ISAC and the existing pavement. The viscoelastic membrane layer is designed to provide base isolation benefits due to its low modulus and high ductility even at very low pavement temperatures. This layer consists of a highly-modified, elastomeric binder. The upper woven geotextile layer provides additional protection to the asphalt overlay, serving mainly as a reinforcing layer. The woven polyester used in this layer has a very high pull tensile strength (1000 lbs/in). The open weave of this layer promotes good bonding characteristics with the overlay. Before ISAC is laid down, a tack coat is applied to the surface of the existing pavement. The woven geotextile side of ISAC is covered with plastic to prevent pick-up during construction. The plastic is removed just before paving of the overlay. Field testing and 3-D FEM analysis has shown that the use of the ISAC at airports significantly reduced the shear and bending stress developed in the joint/crack area (Bozkurt and Buttlar, 2002). However, large-scale field implementation in Illinois using this product has just begun and results are still pending.

The success of many of these techniques in mitigating reflective cracking has been found to also be highly site dependent. For example, Buttlar et al. (1999) showed that geotextiles can delay reflective cracking for a few years at airports in warmer climates; however, the same geotextiles cannot delay reflective cracking to the same degree at locations with colder climates. Therefore, the influence of environmental loading should not be overlooked when considering a mitigation method. Similar conclusions were drawn by Button and Lytton (2007).

## 2.4 Evaluation and Modeling of Reflective Cracking

Current design methods utilized by the states are primarily empirical in nature. Even though there is an immediate need to better understand reflective cracking mechanisms, only a limited amount of research has been conducted to understand, model, and predict reflective cracking, with a majority of these modeling procedures being developed recently (since 2000).

One of the first studies to develop a mechanistic-based model to determine reflective cracking was Jayawickrama and Lytton (1987). The model encompassed a crack growth analysis approach by using fracture mechanics principles and beam-on-elastic foundation theory. Their research resulted in a 2-D plane stress/strain finite element code called CRACKTIP and was eventually calibrated using over 40 HMA overlaid flexible pavement sites in dry-freeze climate zones in Texas. However, one drawback of the research of Jayawickrama and Lytton (1987) was that the model only considered cracking caused by bending and shearing stresses applied due to traffic and neglected any affect of expansion and contraction at the PCC joint/crack due to climatic cycling.

Scarpas et al., (1996) developed the CAPA (Computer Aided Pavement Analysis) system using both 2-D and 3-D finite element routines to study the contribution of reinforcement layers to the overlay system, with the 2-D program being capable of simulating crack propagation. This program was eventually upgraded through the implementation of a constitutive model for the material response of viscoplastic materials and elasto-viscoplastic-fracturing model. The new system has since been used to evaluate different reinforcement materials and the affect of bonding of the reinforcement materials.

Owusu-Antwi et al., (1998) developed a mechanistic-based reflective cracking model for HMA/PCC pavements, which was developed to be used by practicing engineers. The procedure used 2-D plain strain finite element modeling for stress intensity computations and 3-D finite element modeling for computing the required mathematical expressions to determine the “J-integral” for temperature and traffic loadings. The authors used 33 LTPP HMA overlaid PCC pavement sections for their analysis and finally derived a mechanistic-empirical model for predicting reflective cracks by using optimization techniques.

Kohale and Lytton (2000) also developed a mechanistic-based reflective cracking model for evaluating different reflective cracking mitigation techniques. The computer program was used to develop design equations for flexible overlays with Stress Absorbing Membrane Interlayers (SAMI’s) and Reinforcing Grids. The equations were then calibrated using in-service data from the Florida Department of Transportation.

In 2000, identifying the lack of research focused on the issue of reflective cracking in bituminous overlays, the RILEM group of Europe sponsored the first international conference solely dedicated to reflective cracking, *Reflective Cracking in Pavements – Research in Practice* (RILEM, 2000). The conference focused on the key components of attempting to understand the reflective cracking mechanism, which include; 1) Design and Analysis of Composite Pavement Systems, 2) Resistance to

Cracking, 3) Crack Prevention – Modified Mixes, 4) Crack Prevention – Stress Relief Layers, 5) Crack Prevention – Pre-Cracking, 6) Crack Prevention – Pavement Reinforcement Materials, and 7) Evaluation of Prevention Techniques. Of significant importance to the further understanding on the mechanisms of reflective cracking and modeling those movements in the laboratory were as follows:

- Sousa, J., J. Pais, and R.N. Stubstad, *Mode of Loading in Reflective and Flexural Fatigue Cracking – Numeric Evaluation*: Utilized controlled stress mode testing in the flexural beam fatigue test for input parameters in a 2-D finite element model. Analysis showed that the initiation of reflective cracking can best be simulated utilizing the flexural beam fatigue test.
- Zhou, F. and L. Sun, *Mechanistic Analysis of Reflective Cracking and Validation of Field Test*: Through a parametric study using Fracture Mechanics theory in a 3-D finite element model showed that the initiation of reflective cracking is a function of the vertical deflection on the loaded side of the PCC joint. The vertical deflection on the loaded side of the PCC joint contains the effects of both bending and shearing since the relative difference is the difference between the deflection on the loaded side and deflection on the other side of the PCC joint. Also, the crack initiation and beginning of the upward movement is controlled by the bending (vertical movement), while the further upward movement is accelerated by the shear movement.
- Pais, J. and P. Pereira, *Evaluation of Reflective Cracking Resistance in Bituminous Mixtures*: The determination of field deformations that are directly related to field loading conditions (traffic) is required to properly assess

bituminous mixtures and their resistance to reflective cracking. In their study, the authors developed a device called a Crack Activity Meter (CAM) to measure the vertical deflections at the PCC joint, while measuring traffic loading through a separate sensor (weigh in motion). The research showed that it is possible to measure field deformations and utilize the measured deformations in laboratory testing.

- Tan, Z., *Mechanistic Analysis for Opening Reflective Cracking in Asphalt Overlays*: The use of an interlayer with lower shear modulus (stiffness) at intermediate and low temperatures can prevent the formation and development of a crack opening. Partial reinforcement near the joint/crack and increasing the interlayer's flexibility were found to be a cost effective way to prevent future reflective cracking.

Sousa et al. (2002) developed a mechanistic-empirical overlay design method for reflective cracking based on predicted field movements from "after overlay" conditions, flexural beam fatigue laboratory tests, and correction factors for temperature and aging. Unfortunately, the methodology only considered the potential reflective cracking from vertically applied loading conditions and did not include any evaluation of the potential for cracking due to expansion/contraction as the PCC joint/crack. However, the methodology did include the generalized movement at the joint/crack and how the material behaves under that movement, something none of the other procedures included at this time.

Bozkurt (2002) utilized 3-D, visco-elastic finite element analysis to evaluate the reflective cracking mitigation potential of a proprietary interlayer system called ISAC.

The analysis, as well as a field study conducted in conjunction with the analytical study, showed that stresses/strains were significantly reduced at the bottom of the HMA overlay. However, residual tensile strains were still significant enough to potential result in reflective cracking in the HMA overlay, as noticed in a field trial. The 3-D visco-elastic modeling also showed that the stresses at the top of the overly can be reduced by using a modified binder or softer binder grade, which would have better relaxation characteristics. These relaxation characteristics were found to be extremely important at critical climatic conditions for a composite pavement where the air and pavement temperature are already cold and the weather under-goes a cooling cycle. This causes an already somewhat brittle HMA material to have to withstand a contraction-type movement due to material contraction, as well as contraction at the PCC joint.

In 2004, the RILEM group once again organized a conference solely related to cracking in pavement systems., *Cracking in Pavements – Mitigation, Risk Assessment, and Prevention* (RILEM, 2004) Although it was not dedicated to solely reflective cracking, two sessions were solely dedicated to reflective cracking related issues. A few of the research papers to note were the following;

- Jun, Y., F. Guanhua, L. Qing, C. Rongsheng, and D., Xuejun, *Deep Analysis on Interlayer Restraining Reflective Cracks in Asphalt Overlay Old Concrete Pavement*: The authors' research showed that reflective cracking in asphalt overlays is mainly caused by the traffic load and temperature change. The traffic load causes the vertical movements (bending and shear), while the temperature changes results in horizontal movements. The authors' research indicated that the temperature changes appear to be more dominant and recommended a minimum asphalt overlay thickness of 8 inches (200 mm) to control cracking caused by temperature changes. Unfortunately, this thickness may not be practical for most applications in the United States.

- Zhou, F., D. Chen, and T. Scullion, Overlay Tester: *A Simple Test to Evaluate the Reflective Cracking Resistance of Asphalt Mixtures*: The authors illustrated the use of a new test device that was developed to model the horizontal deflection movements at the PCC joint, called the Overlay Tester. Preliminary comparisons to field test sections showed that the Overlay Tester compared well to field observations. The preliminary test results showed that asphalt mixtures with stiffer asphalt binders performed poorly in the Overlay Tester and in the field test sections.

Paulino et al., (2006) modified a cohesive zone model, commonly utilized for metals, for use in hot mix asphalt materials. The cohesive zone model utilizes the fracture energy of hot mix asphalt mixtures (Wagoner et al., 2005) as the prime input material parameter for the model. The model shows promise by being able to identify areas of fracture and softening, but calibration is still on-going. However, parametric studies conducted using the model indicated that (Buttlar, 2007);

- Strain-tolerant interlayers can have significant factor of safety against fracture, even under severe thermal and traffic loading;
- Interlayers can significantly lower strain in HMA overlays (relative to control sections), however, traditional HMA overlays may be too brittle to withstand residual strains; and

- The use of improved surface layers, such as SMA, polymer-modified binders, etc., in conjunction with interlayers may provide the best result in the fight against reflective cracking in composite pavements.

Wu et al., (AAPT, 2006) utilized a gradient-enhanced non-local continuum damage model (CDM) in a finite element program to evaluate the HMA degradation under repetitive loading in a composite pavement. Flexural beam fatigue tests were found to be the best suitable test for material characterization and were utilized for the material inputs. The model was eventually verified for two control sections at the Cal-Berkley Heavy Vehicle Simulator (HVS).

## 2.5 Summary of Literature Review

A literature review was conducted to evaluate the mechanisms and factors associated with reflective cracking in composite pavements, as well as to determine applicable laboratory testing and modeling procedures capable of identifying reflective crack-prone materials. Based on the literature review, the following conclusions/observations:

- The major mechanism generating reflective cracking is the tensile stress/strain generated at the bottom of the asphalt overlay. The tensile stress/strain is a coupled resultant of vertical deflection at the PCC joint/crack associated with traffic loading and horizontal deflection at the PCC joint/crack associated with the expansion and contraction from environmental cycling.
- The shearing mechanism at the PCC joint/crack, commonly indexed with the measured Load Transfer Efficiency (LTE), is not a crack initiator but an accelerator. The energy required to initiate cracking is not capable of being

generated from a “confined” shear mode. However, once a crack has initiated from the tensile stress/strain, poor LTE will accelerate the propagation of the crack to the pavement surface.

- A number of mechanical tests exist for the cracking evaluation of asphalt mixtures. However, to properly evaluate the reflective cracking potential of asphalt mixtures, it is important that the mechanical tests are capable of simulating the movements commonly associated in the field. Therefore, the following mechanical tests are recommended for laboratory simulation of field movements associated with reflective cracking of asphalt overlays:
    - Flexural Beam Fatigue (AASHTO T321) – this test provides vertical bending movements associated with the vertical deflection at the PCC joint/crack due to applied traffic loading.
    - Overlay Tester (TxDOT Tex-248-F) - this test device simulates the expansion and contraction movements that occur in the joint/crack vicinity of PCC pavements. Although this test procedure is essentially a fatigue-type test, it currently represents the best method to truly simulate horizontal joint movements of PCC pavements in the laboratory.
- The critical reflective cracking condition in composite/PCC pavements is when the air/pavement temperatures are already cold and the climate is under-going a cooling cycle. This creates an already brittle-like HMA layer that must be able to withstand tensile straining caused by contraction occurring at the PCC joint/crack and material contraction.

- Little information exists regarding which reflective cracking mitigation methods work best. However, research conducted did indicate that geotextiles/geosythetics are not as effective in colder climates. This is primarily due to geotextiles/geosythetics only able to help reinforce against vertical deformations (bending), while not providing resistance to horizontal joint deformation commonly occurring due to temperature cycling conditions.
- Strain-tolerant interlayers can have significant factor of safety against fracture when compared to conventional HMA mixtures, even under severe thermal and traffic loading. These interlayers can significantly lower strain in HMA overlays (relative to control sections), however, traditional HMA overlays may be too brittle to withstand residual strains.
- Research has shown that when field deformations at the PCC joint are accurately measured, these deformations can be utilized in laboratory test devices and provides reasonable estimates on the ability of the HMA mixture to resist the reflective cracking movements.

# CHAPTER 3 – NATIONAL SURVEY ON FLEXIBLE OVERLAYS FOR COMPOSITE/RIGID PAVEMENTS

In an attempt to further understand the factors that affect reflective cracking that were not answered during the Literature Review, as well as both successful and unsuccessful reflective cracking mitigation methods, a survey was sent to the fifty (50) state agencies to ask for their experience. A total of 28 state highway agencies (SHA's) had provided responses to the survey (Figure 3.1). Two (2) of the 28 SHA's responded that PCC pavements are either not built in the state (New Hampshire) and/or PCC pavements are not overlaid with asphalt (New York). Therefore, the analysis of survey responses was based on 26 of the 28 states that reported having both PCC pavements and use hot mix asphalt overlays on their PCC pavements. Of the 26 state highway agencies reporting that they overlay PCC pavements with hot mix asphalt (HMA), 22 of the SHA's (85%) reported that reflective cracking was observed within the first four years after the HMA overlay was placed, while 7 of the SHA's (27%) reported to observe reflective cracking within the first two years (Figure 3.2).

## 3.1 – Pavement Design Features

### *3.1.1 - Base Course Type*

The different base course types found supporting the composite pavements of the responding states were; aggregate, cement-treated, bituminous-treated, lime-stabilized, and none (some reported more than one). The breakdown of the base course types with respect to the time before reflective cracking is observed in the HMA overlay is shown in Figure 3.3. No general trend was found between base course type and reflective cracking. However, the figure does indicate that most states that responded have granular base courses under the composite pavement.

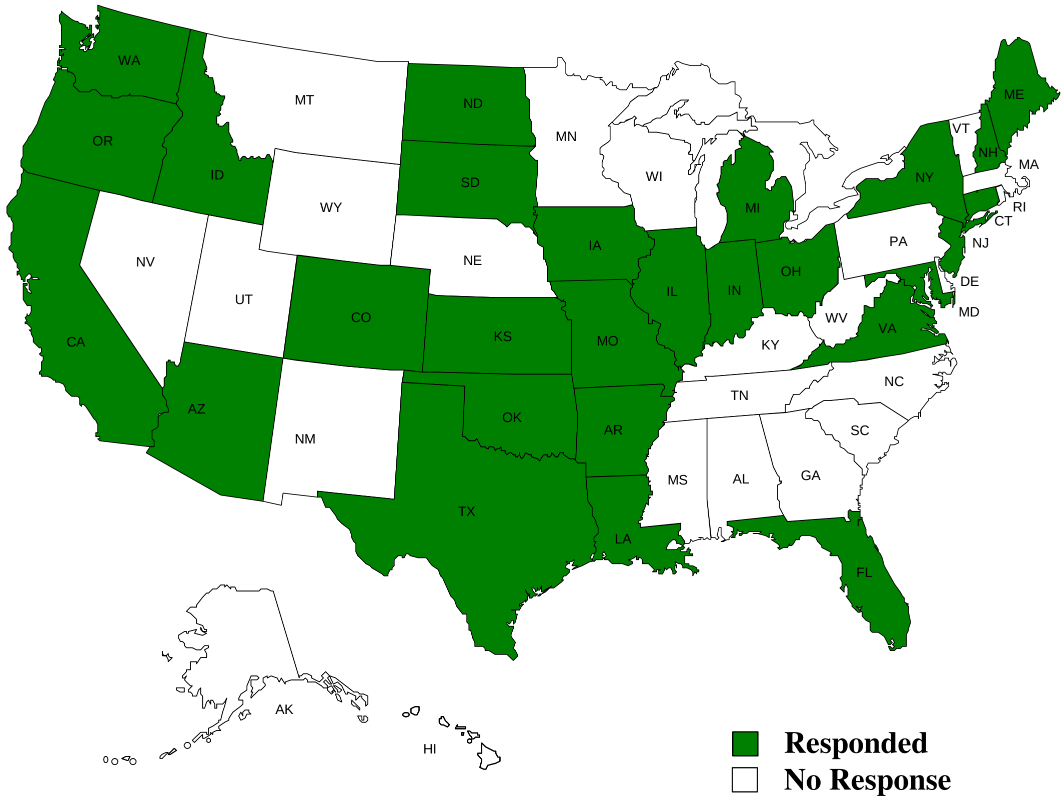



Figure 3.1 – States Responding the NJDOT Technical Survey on the Use of Flexible Overlays on PCC/Composite Pavements

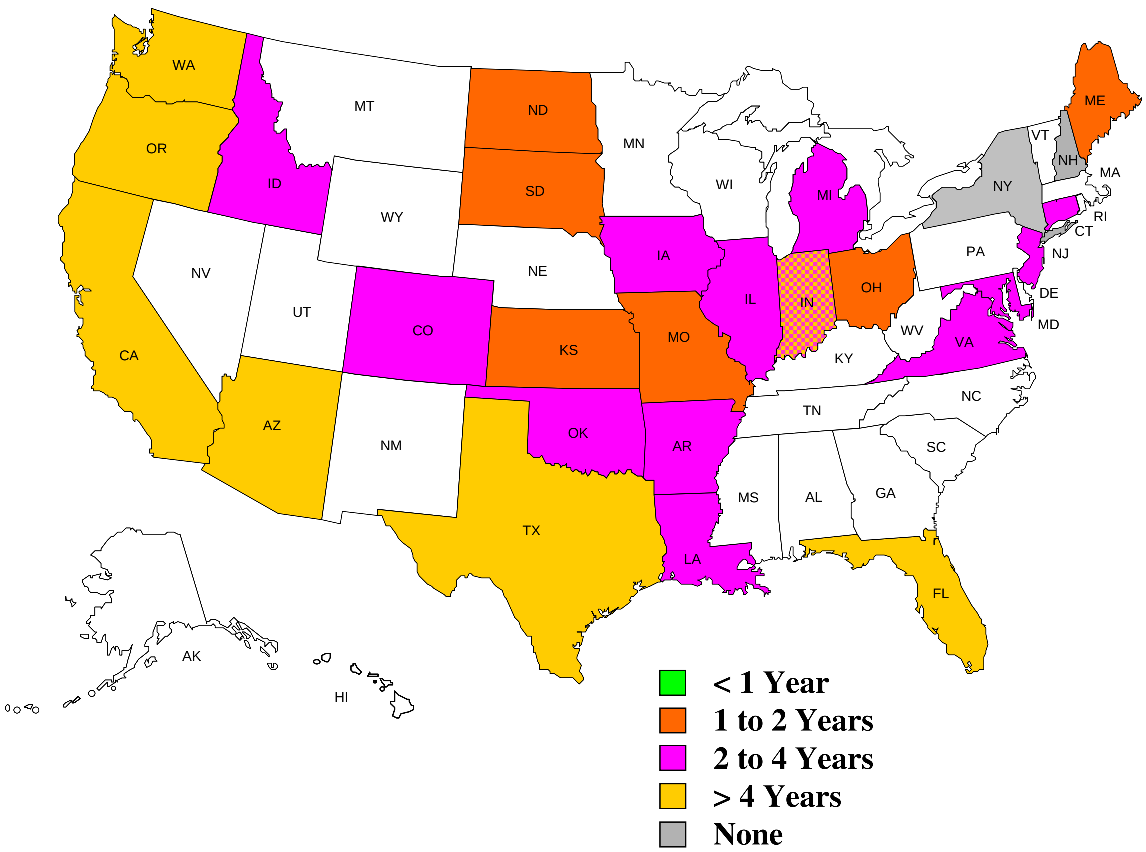



Figure 3.2 – Time Reflective Cracking Observed After Placement of HMA Overlay

### *3.1.2 - Shoulder Type*

The shoulder types of the PCC/composite pavements, which have been overlaid with HMA by the various states, were typically HMA, PCC (tied), and PCC (untied). The HMA and untied PCC shoulders are located in 69% of the states that replied (based on 26 states), while only 31% of the states responding said that their composite pavements have tied PCC shoulders. No trends were found between shoulder type and time before reflective occurs.

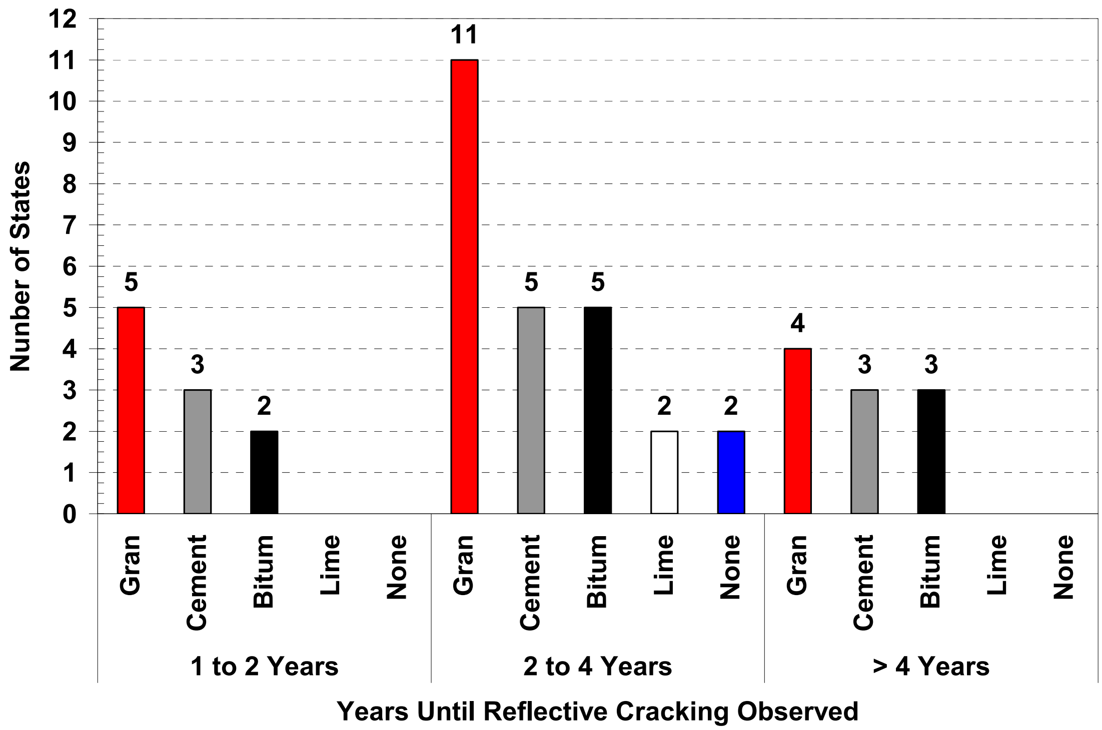



Figure 3.3 – Aggregate Base Courses Utilized Under Composite Pavements (Based on 26 States)

*3.1.3 - Joint Spacing*

Reflective cracking of the HMA overlay is a function of the rigid pavement moving (vertically and horizontally) at the joint/crack area of the PCC/composite pavement. Therefore, the state agencies were asked to provide information regarding typical joint lengths on the in-service jointed composite/PCC pavements where reflective cracking of the HMA overlay is found. Figure 3.4 summarizes the responses of the states compared to the time when reflective cracking is typically observed. The survey results show that the most of the jointed PCC pavements are on the order of 15 ft in length and no general trend existed between joint spacing and reflective cracking. Some states reported that the same PCC pavement and respective slab length resulted in different reflective cracking lives.

3.2 - Sawing and Sealing Joints

In an effort to retard reflective cracking, as well as lessen the extent of damage due to reflective cracking, some states require the sawing and sealing over the top of the transverse and longitudinal joints to "control" where the reflective crack finally arrives at the HMA overlay surface. The responses from the state agencies indicated that only 8 of the 26 states saw and seal the transverse joints on the HMA overlay, while only 3 of the 26 states saw and seal the longitudinal joint. Only 3 of the 26 states utilize the saw and seal method on $2^{nd}$ and $3^{rd}$ generation HMA overlays. State agencies that do not saw and seal $2^{nd}$ and $3^{rd}$ generation overlays responded they were mostly concerned of not properly locating the PCC joints and therefore possibly resulting in both a saw cut and a reflection crack.

3.3 - Hot Mix Asphalt (HMA) Overlay Design Methods

Although reflective cracking significantly shortens the pavement service life of HMA overlays, historically there has been a lack of mechanistic-based design procedures for designing HMA overlays on PCC/composite pavements. Furthermore, neither NCHRP 1-37A (Mechanistic-Empirical Pavement Design Guide) nor NCHRP 9-17 (Superpave Support and Models Management) specially address laboratory tests or mixture design procedures for the evaluation of reflective cracking, although the recently initiated NCHRP 1-41 (Models for Predicting Reflective Cracking of Hot Mix Asphalt Overlays) led by the Texas Transportation Institute (TTI) will try to provide guidance on these issues.

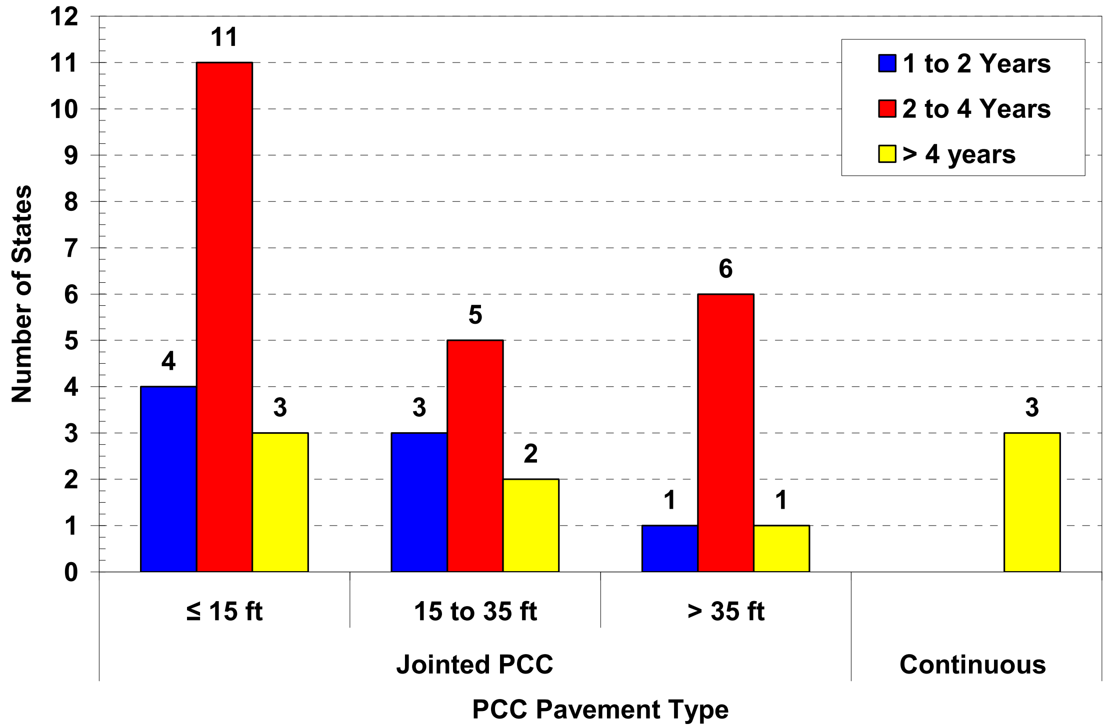



Figure 3.4 – Joint Spacing versus Time Until Reflective Cracking Observed (Based on 26 States)

The state agencies were asked to provide their typical pavement design procedure when designing HMA overlays on PCC/composite pavements. The responses were as follows:

- 1993 AASHTO Design Guide/DARWIN: 20 of the 26 states (77%) responded using the AASHTO Design Guide/DARWIN for selection of an appropriate HMA design thickness. However, a majority of these state agencies also mentioned that they have a minimum thickness requirement that must be met, regardless of the AASHTO Design Guide/DARWIN output. One of the 20 states indicated that Mechanistic-Empirical procedures are used to compare to the 1993 AASHTO/DARWIN thickness recommendations.

- Standard State Policy Overlay Thickness: 6 of the 26 states (23%) responded that their state agency has a standard overlay thickness policy for HMA overlays on composite/PCC pavements. The minimum thickness policies were found to be based on; past history, typical traffic and pavement conditions, geometry of pavement (curb and guard-rail heights), and cost.

3.4 - Hot Mix Asphalt Materials

The state agencies were asked to provide typical HMA overlay material properties that included nominal aggregate size and PG graded asphalt type (i.e. – 12.5mm PG76-22 over a 19mm PG64-22). A majority of the states use either a 9.5mm Superpave mix over a 12.5mm Superpave mix or 12.5mm Superpave mix over a 19mm Superpave mix. However, some states have developed a comfort level with their own unique HMA overlay. Some examples are shown below:

- Arizona – uses 50mm of an asphalt rubber open-graded friction course
- California – standard design requires 30.5mm HMA leveling course, then paving fabric, and then another 75mm of an HMA overlay. This is done in conjunction with crack and seat. CalTrans has reportedly placed 1,000's of miles over the years with good results.
- Oregon – 50mm of a 19mm open-graded friction course over 100mm of a 12.5mm HMA

The PG binder grade used by the different states typically corresponds to that recommended by LTPPBind. However, further analysis resulted in a trend with respect to PG grade binder (Low Temperature Grade) and the time for reflective cracking to be observed by the state agencies. Figure 3.5a shows the LTPPBind recommendations for low temperature PG grade at a 98% reliability and Figure 3.5b shows the typical time until reflective cracking is observed in the HMA overlay based on the survey responses of the SHA's. By comparing the two plots, the states that observe the longest time before the onset of reflective cracking are those states whose low temperature PG Grade is between -16°C and – 10°C. Meanwhile, as the LTPPBind recommended low temperature PG Grade decreases, the time before the reflective cracking of the HMA overlay also decreases.

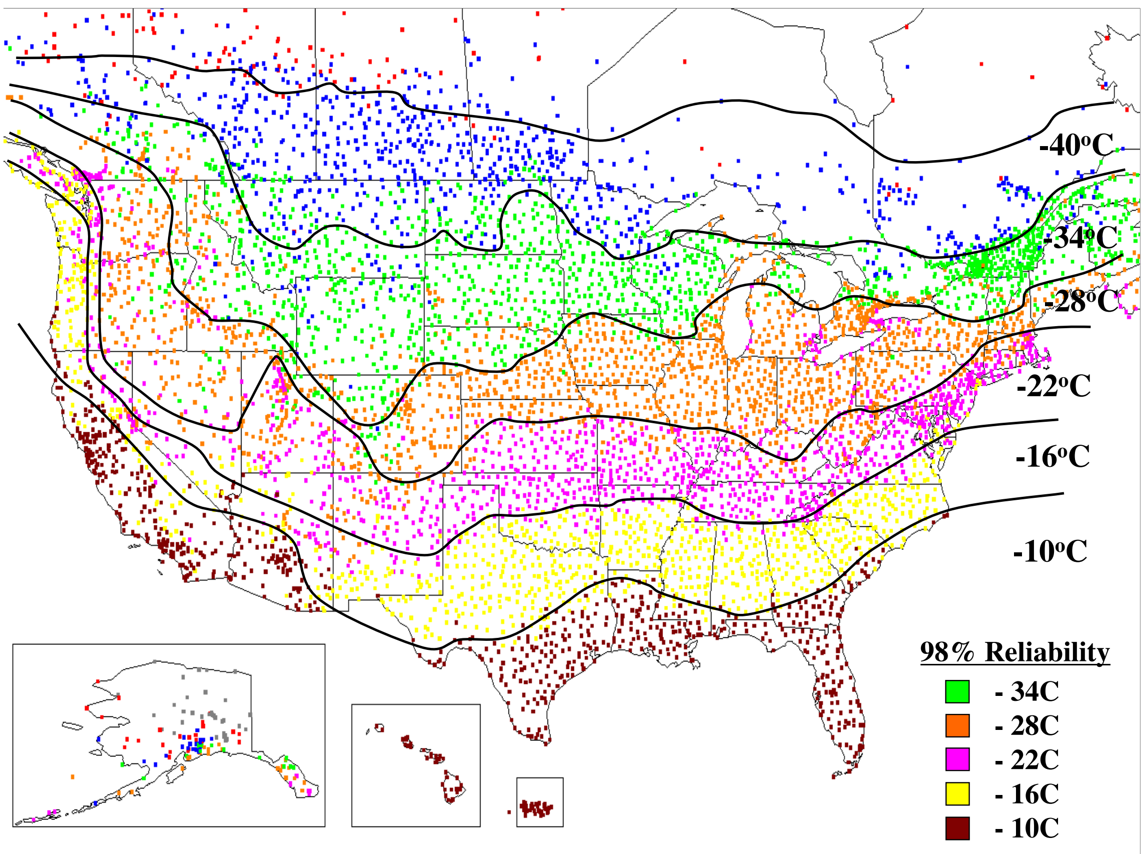



Figure 3.5 - (a) LTPPBind Recommended Low Temperature PG Grade (98% Reliability)

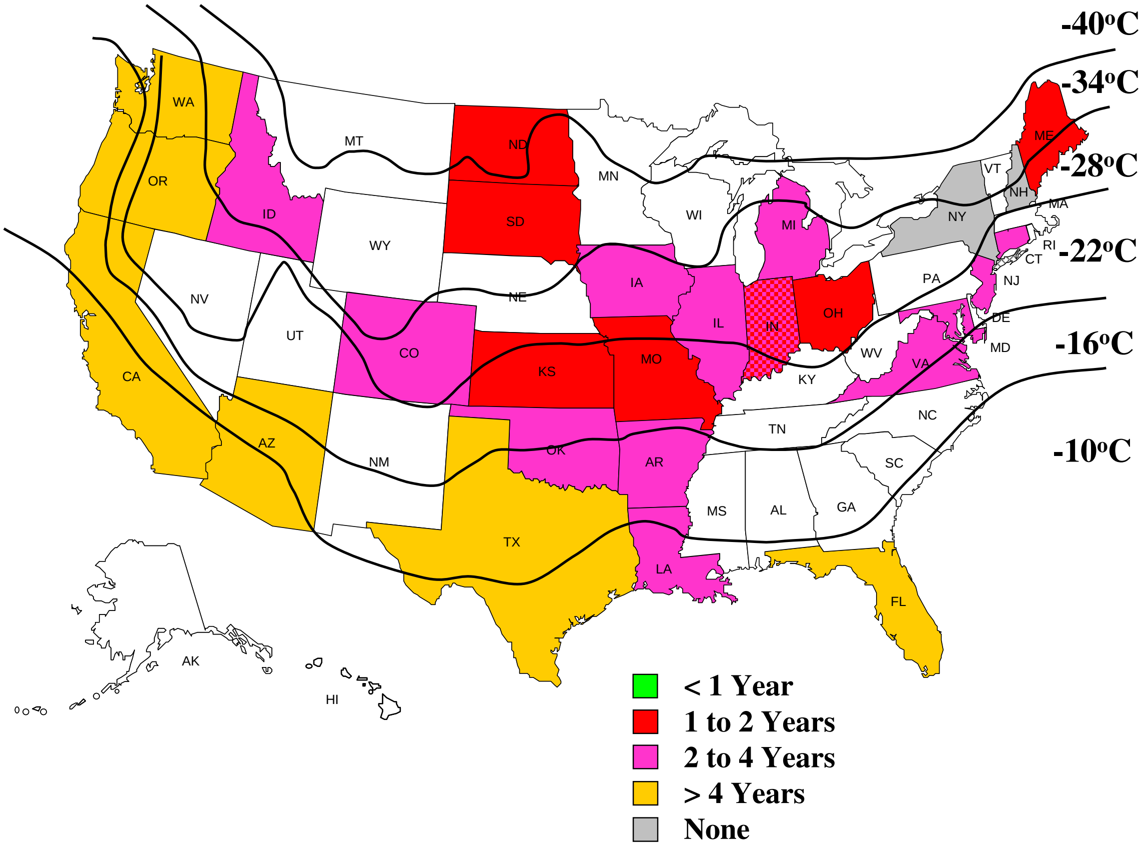



Figure 3.5(b) Time Until Reflective Cracking is Observed in HMA Overlay

Table 3.1 provides further evidence of the low temperature PG Grade versus the time before reflective cracking of HMA overlay relationship. Eight of the responding states are shown in the table, along with their recommended LTPPBind low temperature PG grade, asphalt binder currently used, and the time until reflective cracking typically occurs in the HMA overlay. The table shows that the greater the difference between the in-service low temperature PG grade and the LTPPBind recommended low temperature PG grade, the longer the HMA overlay will last before reflective cracking is observed. For example:

- States that use a low temperature PG grade one to two grades less than that recommended by LTPPBind reported the longest time before observing reflective cracking (> 4 years). This is illustrated by the responses of Florida and Texas. Arizona, who typically uses asphalt rubber, which is known to have excellent low temperature properties, also responded with a "> 4 years" before reflective cracking. However, actual PG grade of asphalt binder was not provided.

- States that use a low temperature PG grade equal to or one grade less than that recommended by LTPPBind reported having reflective cracking lives of 2 to 4 years.
- States that use a low temperature PG grade equal to or greater than that recommended by LTPPBind reported having reflective cracking lives of 1 to 2 years.

Although pavement structure and condition of the pavement heavily influence the reflective cracking life of the HMA overlay, the difference between the LTPPBind recommended low temperature PG grade and the in-service low temperature PG grade asphalt clearly are related.

Table 3.1 – PG Grade of Asphalt Binder and Reflective Cracking Life of Different States

| State | LTPPBind | Binder Used | Reflective Cracking |
|---|---|---|---|
| North Dakota | -34C/-40C | 64-34 over 58-28 | 1 to 2 Years |
| South Dakota | -34C | 64-34 or 64-28 | 1 to 2 Years |
| Kansas | -22C/-28C | 70-22 | 1 to 2 Years |
| Arkansas | -16C/-22C | 76-22 | 2 to 4 Years |
| New Jersey | -22C | 76-22 | 2 to 4 Years |
| Ohio | -28C | 70-22 over 64-28 | 2 to 4 Years |
| Florida | -10C | 76-22 | > 4 Years |
| Texas | -10C/-16C | 76-22 or 64-22 | > 4 Years |

## 3.5 – Site Conditions

With any type of design, having proper knowledge of the structure's in-situ condition provides the designer invaluable information that can be used for a cost-effective design. In the case of designing an HMA overlay on a PCC/composite pavement, this typically means acquiring one or more of the following information: vertical deflections at joints/cracks, Load-Transfer Efficiency (LTE) of joints/cracks, in-situ pavement thickness, modulus/strength of supporting material (base/subbase/subgrade), traffic counts/vehicle classifications, visual distress information and laboratory testing.

### *3.5.1 - Field Forensic Methods*

The utilization of field forensic testing methods provides the designer in-situ information regarding the pavements performance and current in-situ condition. The SHA's were asked to provide information regarding the use of; Falling Weight Deflectometers (FWD), Ground Penetrating Radar (GPR), Dynamic Cone Penetrometer (DCP), Coring and Sampling (C&S), Visual Distress Surveys (VDS), Traffic Counts/Vehicle Classification (TRAF), and Laboratory Testing (LAB). The distribution of the different field forensic testing methods is shown in Figure 3.6.

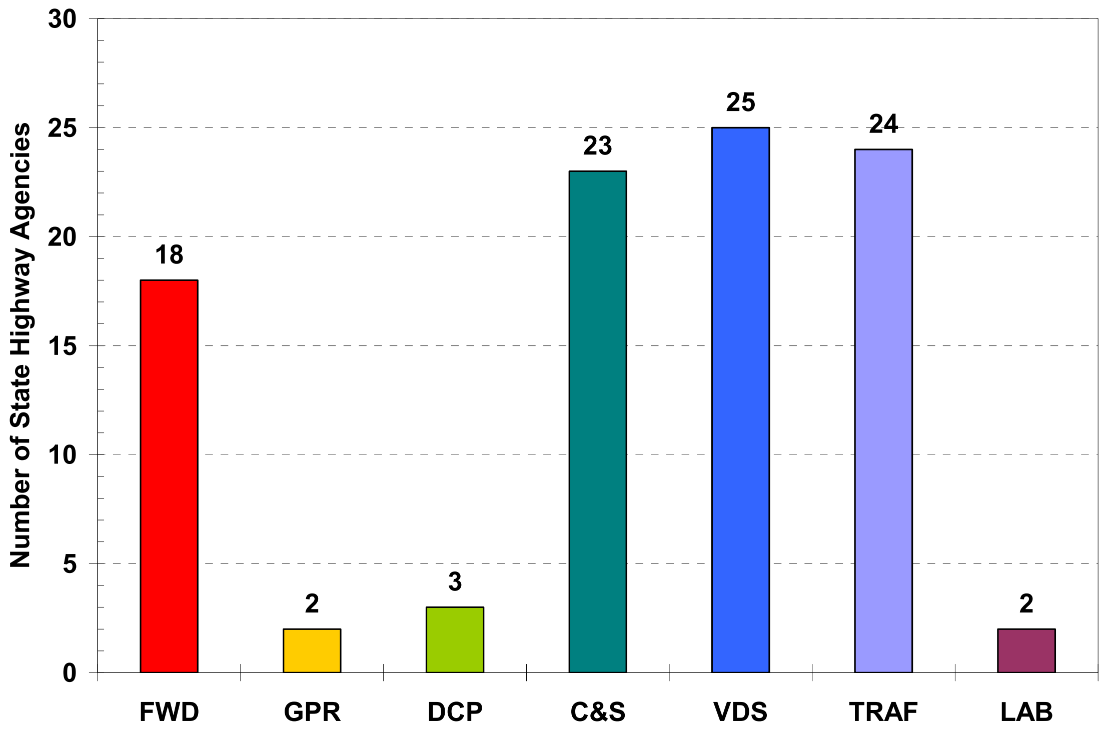



Figure 3.6 – Number of States Using Various Field Forensic Methods (Based on 26 States)

The use of FWD testing for designing HMA overlays for PCC/composite pavements has the advantage of providing not only design information, such as modulus, that can be inputted directly into design software, but also actual field performance, such as joint/crack deflections and load transfer efficiency that can be used as performance criteria to trigger various rehabilitation techniques. Of the 18 state agencies that are using the FWD for field forensic testing:

- 13 of the 18 SHA's use the FWD information directly for pavement design purposes (i.e. – DARWIN)
- 5 of the 18 SHA's use the FWD data in state specific criteria to trigger a rehabilitation strategy (i.e. – underseal, slab stabilization, etc.)

Examples of state specific criteria to trigger rehabilitation strategies are shown below:

- New Jersey
  - Deflection at Joint ($\delta_V$) > 10 mils (9000 lb Load) = Underseal/dowel bar retrofit
  - Load Transfer Efficiency (LTE) < 60% and $\delta_V$ > 7 mils = Joint Rehabilitation
- Texas – uses information from Rolling Wheel Deflectometer (RWD) to trigger rehabilitation type
- Indiana: FWD Undersealing Triggers
  - Interstates: $\delta_V$ > 8 mils
  - US Routes: $\delta_V$ > 10 mils
  - State Routes: $\delta_V$ > 12 miles
- Ohio – uses AASHTO Variable Load Corner Deflection Analysis; also uses LTE as a guide for dowel bar retrofit.

The survey responses with respect to laboratory testing showed that one state is using the resilient modulus test to characterize the subgrade soil (Arkansas) and one state is using volumetric testing of the current, in-place HMA overlay to determine appropriate milling depths prior to the new HMA overlay (Missouri).

The Texas Department of Transportation (TxDOT) reported that they have begun using a performance-based HMA mix design procedure to specify HMA materials that will provide better performance in areas where reflective cracking is an issue. Detailed information regarding the test procedures and criteria can be found in Zhou and Scullion (2004) with a general overview found below:

- After initial HMA mix design has taken place, evaluate mix performance by:
  - Conducting testing in the Texas Transportation Institute (TTI) Overlay Tester to simulate horizontal movement in the PCC pavement
  - Conducting Asphalt Pavement Analyzer (APA) for all HMA materials placed at the bottom of the HMA overlay
  - Conducting Hamburg Wheel Tracking Test (HWTT) for all HMA materials placed at the surface of the HMA overlay

Case histories provided by Zhou and Scullion (2004) indicate that the performance-based HMA mix design procedure has great potential in retarding reflective cracking.

*3.5.2 - PCC Treatments Prior to HMA Overlay*

Depending on the condition of the PCC pavement prior to the HMA overlay, a number of treatments are available to improve the conditions of the PCC and possibly increase the reflective cracking life of the overlay. State agencies were asked to provide typical PCC treatments they have performed in the past prior to overlaying with HMA. Figure 3.7 summarizes the state agencies' survey responses. The most common PCC treatment has been to replace poor joints and slabs with void filling being the least used treatment. Other responses from one state mentioned the use of spall repair and PCC pavement grinding prior to HMA overlay.

*3.5.3 - Traffic*

The state agencies were asked to provide typical traffic levels (ESAL's) where reflective cracking has been problematic. The ranges of ESAL's in the survey corresponded to those typically recommended for Superpave HMA mix design [2]: < 3 million ESAL's (Low to Medium Traffic), 3 to 30 million ESAL's (Heavy Traffic), and

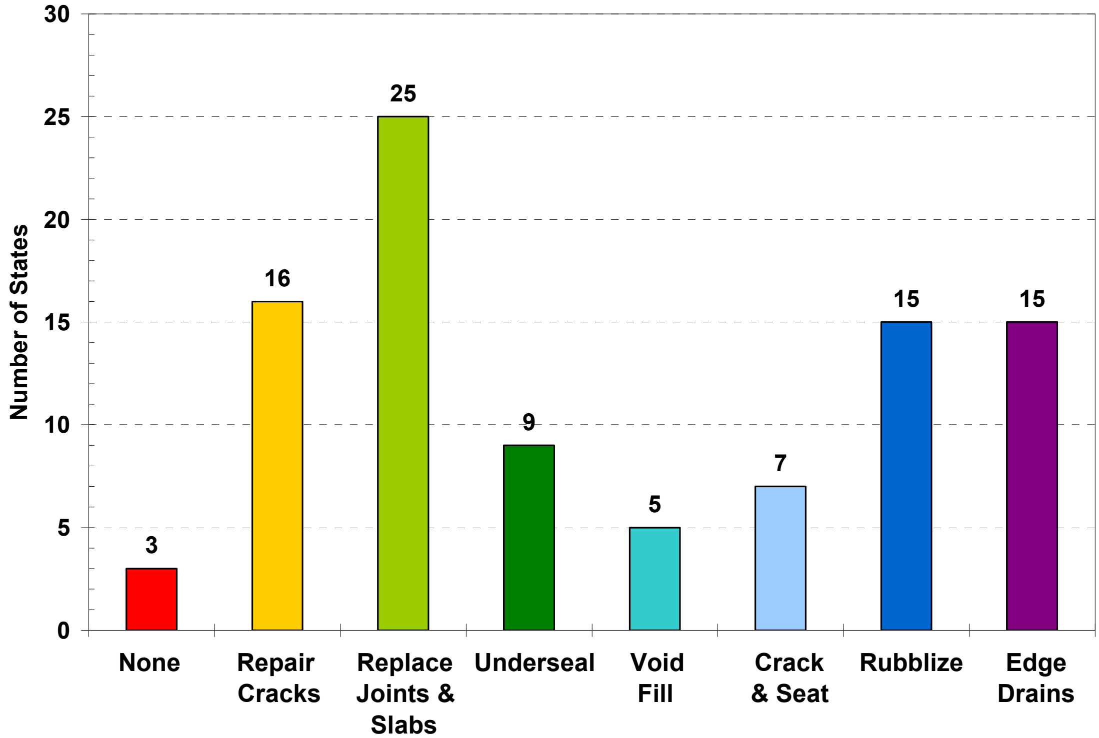



Figure 3.7 – Common PCC Treatments Used by State Agencies Prior to HMA Overlay (Based on 26 States)

> 30 milllion ESAL's (Very Heavy Traffic). The state agencies were asked at what traffic levels are most of their composite pavements located and at what traffic level is the greatest amount of reflective cracking observed. A majority of the SHA's composite pavements have traffic levels of 3 to 30 million ESAL's (58%), while 39% responded that most of their composite pavements have traffic levels greater than 30 million ESAL's, and the remaining 3% (1 state) indicated that most of their composite pavements have traffic levels less than 3 million ESAL's. Meanwhile, an overwhelming majority of the state agencies noted that reflective cracking is observed on composite pavements having traffic levels of 3 to 30 million ESAL's (Figure 3.8). However, this may simply be due to some state agencies noted that reflective cracking occurs at traffic levels of both

3 to 30 million ESAL's and greater than 30 million ESAL's (states responding both traffic levels are marked with checkered pattern in Figure 3.8). None of the state agencies have observed reflective cracking at traffic levels less than 3 million ESAL's. This indicates that reflective cracking is simply not just a function of traffic loading alone, but most likely a combination of vertical deformations created by the traffic loading, and horizontal deformations caused by temperature cycling.

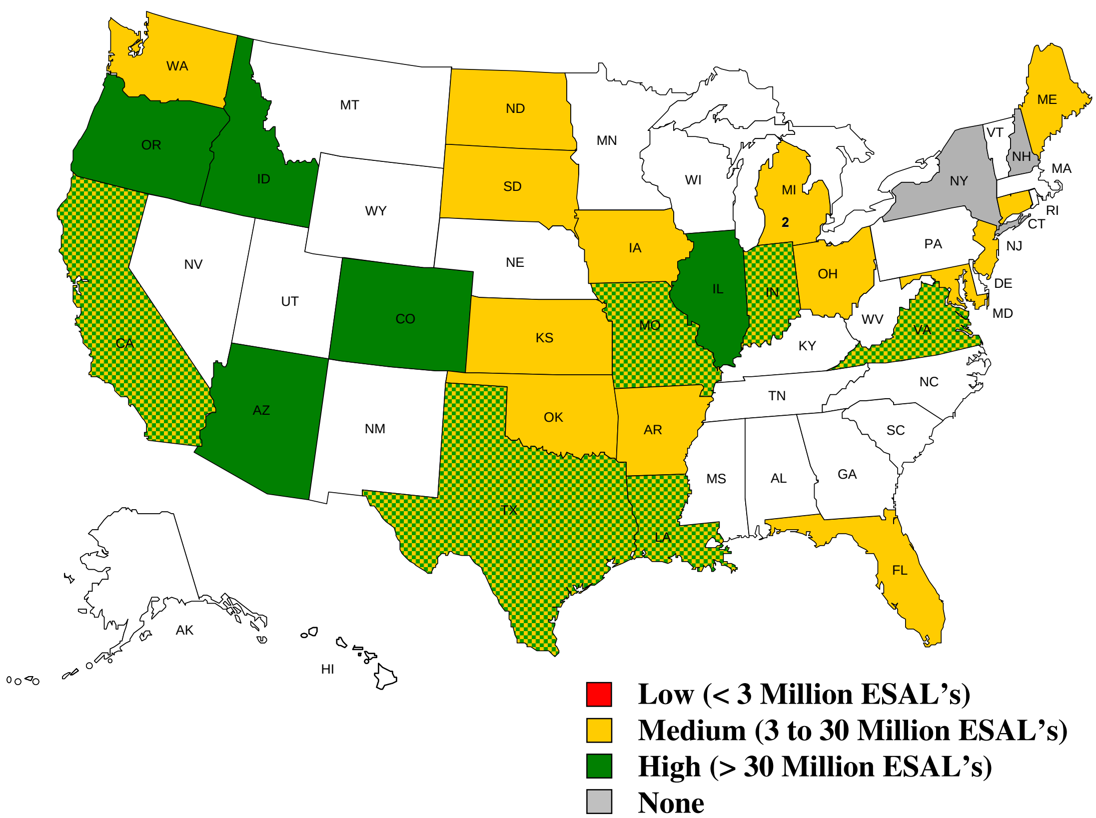



Figure 3.8 – Traffic Levels Where Greatest Occurrence of Reflective Cracking Observed

## 3.6 – Reflective Cracking Mitigation Methods

The use of reflective cracking mitigation techniques have become popular alternatives to conventional HMA overlays due to their promise of retarding reflective cracking. The state highway agencies were asked about their experiences with;

- Paving Fabrics/Geotextiles (PFG)
- Geogrids (GEO)
- Stress-Absorbing Membrane Interlayers (SAMI's)
- Reflective Crack Relief Interlayer Mixes (Strata® type mixes) (RCRI)
- Crack Arresting Layers (CAL)
- Excessive Overlay Thickness (EOT)
- Others

The SHA's were also asked to provide information as to whether or not the mitigation technique was successful. For this study, the definition of a successful reflective cracking mitigation technique is one that provides a minimum of five years of service before reflective cracking is observed. Responses from the SHA's are shown in Figure 3.9. The most popular mitigation technique evaluated has been the use of paving fabric/geotextiles. Unfortunately, the paving fabrics/geotextiles have had the worst performance history with only an 11.5% success rate.

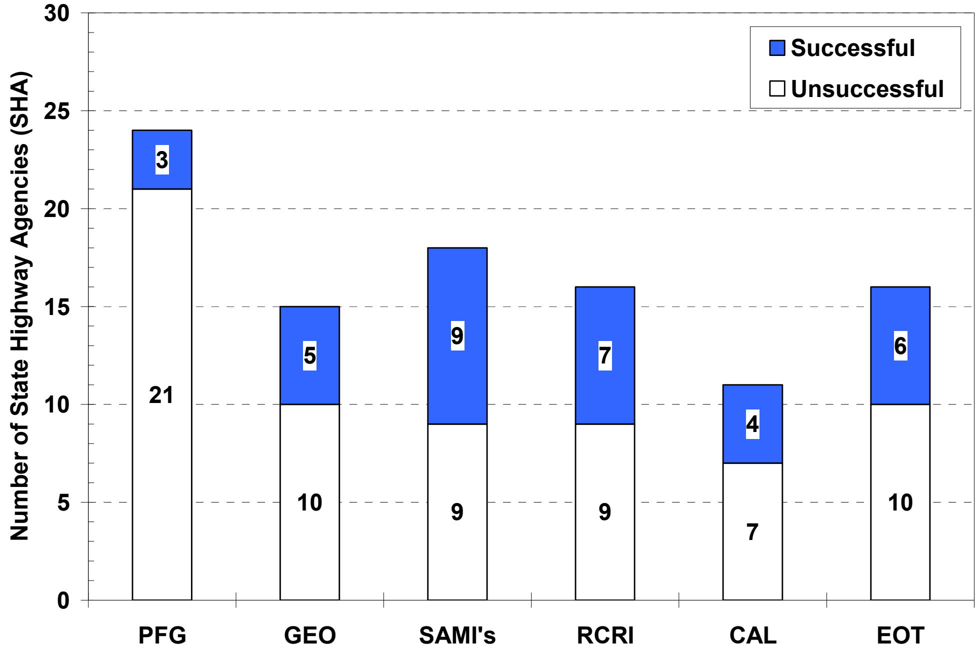



Figure 3.9 – Success History of State Highway Agencies Use of Reflective Cracking Mitigation Methods (Based on 26 States)

State Highway Agencies were also asked to provide any "Other" types of reflective cracking mitigation technique which they have used/evaluated. The following methods were provided:

- Successful use of asphalt rubber overlay (Texas/Arizona)
- Successfully using a Rich Bottom Layer, which is very similar to the Reflective Crack Relief Interlayer mixes, but without the highly polymerized asphalt binder (Texas)
- Limited success with an Interlayer Stress Absorbing Composite, ISAC (Illinois)
- Unsuccessful in using a fiber-rich hot mix asphalt overlay (Indiana)
- Successfully used a granular layer, above PCC and below HMA overlay, as a crack arresting layer (Louisiana/Washington)
- Currently evaluating the use of 5 inch SMA overlay on PCC (Virginia)

## 3.7 – Summary of National Survey

Based on the responses provide by the various state agencies regarding the design and selection of flexible overlays for composite/PCC pavements, the following observations were made:

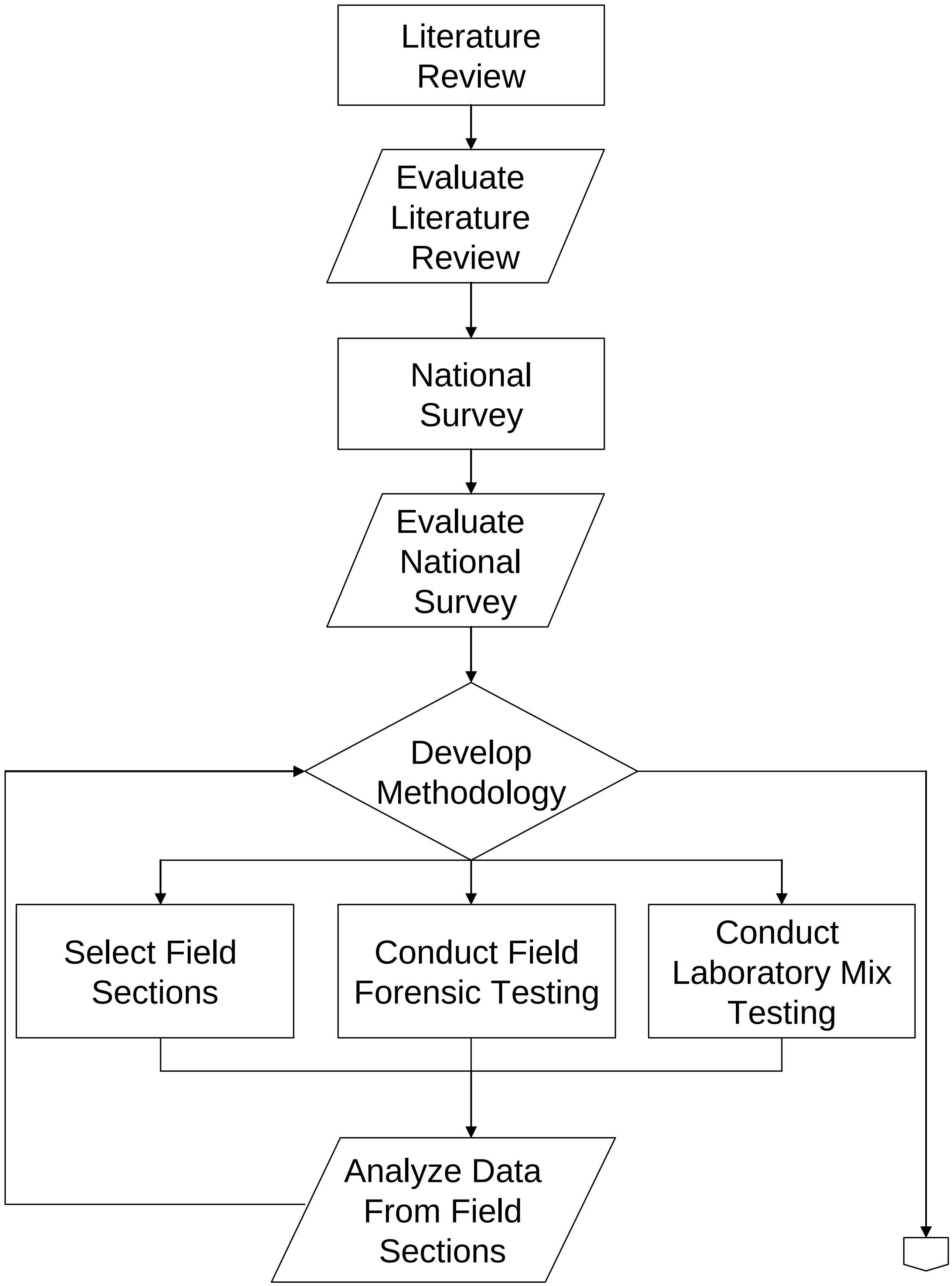
- Reflective cracking was found to occur almost equally in granular, cement-treated, and bituminous treated base courses at the time intervals specified, with composite pavements supported on granular base courses tending to have a lesser reflective cracking life. The results showed that:
    - In states where reflective cracking was observed in 1 to 2 years after the HMA overlay: 25% of SHA's had granular bases, 27% of SHA's had cement-treated bases, and 20% had bituminous treated bases.
    - In states where reflective cracking was observed in 2 to 4 years after the HMA overlay: 55% of SHA's had granular bases, 45% of SHA's had cement-treated bases, and 50% had bituminous treated bases.
    - In states where reflective cracking was observed after 4 years of the HMA overlay: 20% of SHA's had granular bases, 36% of SHA's had cement-treated bases, and 30% had bituminous treated bases.
- A majority of states have either HMA or untied PCC shoulders, although the shoulder type did not show to be correlated to the time of reflective cracking.
- A majority of states have a 15 ft (or less) joint spacing on their composite pavements and, similar to the base course types, no trend was found between the joint spacing and the time when reflective cracking occurred.

- Sawing and sealing the HMA overlay is not commonly done by the state agencies (only 31% of those responded actually have used it). However, many of those states who responded that have used Saw and Seal, mentioned that they find it to be effective at minimizing the effects of reflective cracking. Only 12% of the states that responded utilize the Saw and Seal method on 2$^{nd}$ and 3$^{rd}$ generation HMA overlays.
- Low temperature asphalt binder grade was found to be related to the time until reflective cracking is observed. The survey results indicated that states that use a low temperature PG grade one to two grades lower than recommended by LTPPBind (at a 98% reliability level) for the HMA mixture immediately overlaying the PCC pavement, have a better chance at retarding reflective cracking longer.
- Crack repair and replacing joints and slabs were the two most common PCC treatments conducted by the states prior to an HMA overlay. Methods such as rubblizing and the installation of edge drains were also popular treatments.
- A number of reflective cracking mitigation methods have been attempted by the state agencies over the years. Statistically, the best performing mitigation methods were found to be the SAMI's and the Reflective Crack Relief Interlayer mixes (Strata®-type mixes). The worst performing mitigation methods were found to be the paving fabrics and geogrids. However, it should be noted that even the best mitigation method only had a 50% success rate, when considering a successful method was defined as one that provided five years before reflective cracking was observed.

# CHAPTER 4 – PROPOSED APPROACH TO REFLECTIVE CRACK ANALYSIS

As shown in the literature review and national survey, there currently exists a large gap in the current practice of evaluating the potential for reflective cracking of asphalt overlays when placed on composite/rigid pavements. However, the literature review did indicate that to be able to "predict" the potential for reflective cracking, two important factors need to be considered; 1) Field movements due to realistic loading conditions (traffic and climatic) and 2) Understanding of material response (in the laboratory) to vertical and horizontal movements commonly associated in the PCC joint/crack vicinity. However, it should be noted that to completely understand the material response, the magnitude (i.e. – physical amount of vertical and horizontal deformation) of the field movements must also be known. The material response of the asphalt materials to loading conditions that do not represent realistic conditions (movement and magnitude) only serve as a means of "indexing" material response and not truly modeling those responses.

This chapter discusses the proposed approach taken during this research to develop a rational approach to determining the reflective crack susceptibility of bituminous overlays for composite/rigid pavements and the ultimate development of a Decision Tree system for designing and selecting bituminous overlays for composite/rigid pavements. The flowchart shown in Figure 4.1 illustrates the overall methodology followed during the development of this thesis.


Literature Review
Evaluate Literature Review
National Survey
Evaluate National Survey
Develop Methodology
Select Field Sections
Conduct Field Forensic Testing
Conduct Laboratory Mix Testing
Analyze Data From Field Sections

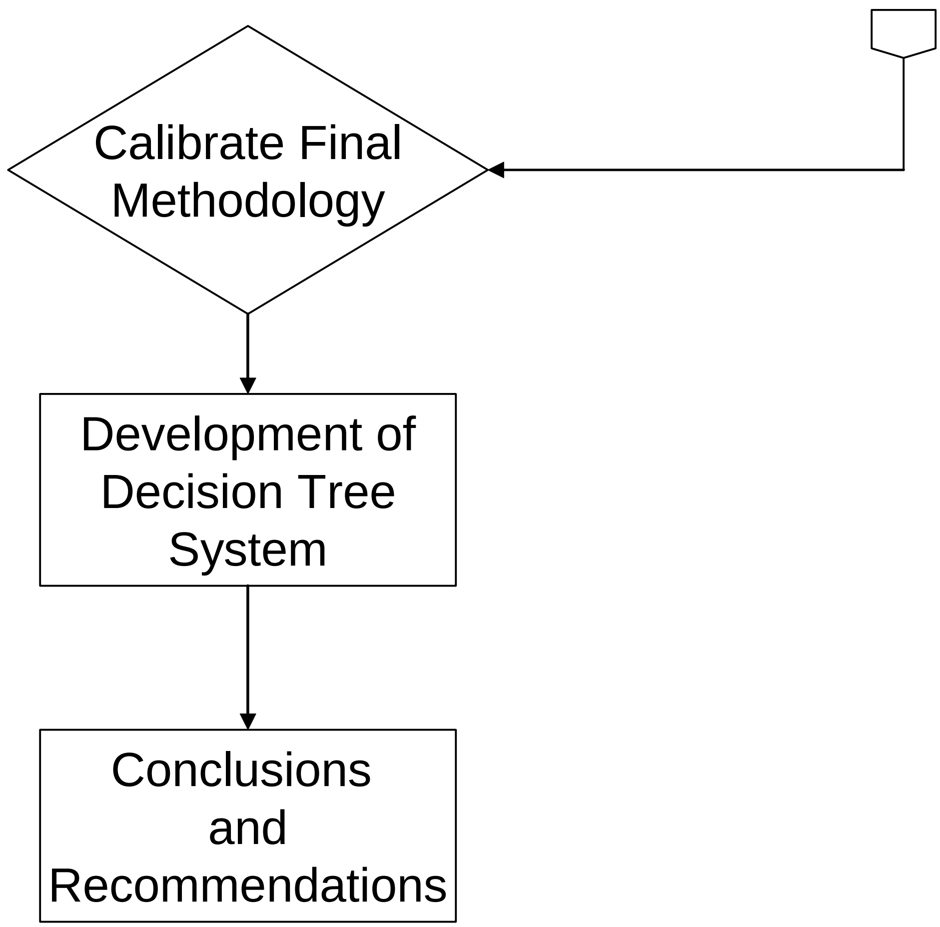



Figure 4.1 – Flowchart Describing Research Approach

### 4.1 Literature Review

A Literature Review was conducted to determine the critical factors affecting the reflective cracking properties of bituminous overlays. The Literature Review was conducted primarily using on-line databases available through the Rutgers University library and other technical resources. Database searches included: Transportation Research Information System (TRIS), National Technical Information Service (NTIS), and Ei COMPENDEX. In addition to the on-line searches for published literature, the materials were also collected the included research performed by state Departments of

Transportation (DOTs), research performed by other Universities (including Master's and Ph.D. theses), and completed/ongoing federal and state research.

### 4.2 National Survey

A National Survey was conducted to help fill in the gaps of knowledge collected during the Literature Review. The National Survey was sent to all 50 state agency departments through the New Jersey Department of Transportation list server system and was addressed to the pavement design engineers from each state agency. A copy of the survey provided to the state agencies is shown in Appendix A of this thesis for review.

### 4.3 Development of a Rational Reflective Cracking Prediction Methodology

Based on the information generated from the Literature Review and National Survey, a methodology was proposed that would enable the evaluation of bituminous mixtures, under expected pavement movements, resulting in a prediction methodology. The prediction methodology relies on; 1) utilizing common field forensic testing procedures to generate expected pavement deflections due to actual traffic loading conditions. Both horizontal and vertical deformations are proposed to be determined, and 2) the resultant deformations are then simulated in the laboratory under laboratory testing procedures commonly used to assess the cracking resistance of bituminous mixtures.

## 4.4 Field Evaluation

Research test sections were selected to evaluate the proposed methodology to evaluate the reflective cracking resistance of bituminous overlays on composite/PCC pavements. Based on the Literature Review and National Survey results, the greatest opportunity for a bituminous overlay to survive in a reflective cracking environment is to incorporate a highly flexible, low modulus bituminous interlayer at the bottom of the bituminous overlay prior to placement of the structural bituminous overlay. This was clearly identified in Literature Review (work conducted by Buttlar, 2007) as well as in the National Survey (Figure 3.9) and was implemented at each of the test sections discussed in this thesis.

The field evaluation was conducted on 2 test sections in New Jersey, 1 test section in Pennsylvania, and 1 test section in Massachusetts. This provided a range of different materials and pavement conditions so as not to bias the prediction methodology. At each test location, the following information/materials were collected; 1) Traffic data using either Weight-in-Motion/Automatic Vehicle Classifiers (Figure 4.2), 2) Falling Weight Deflectometer deflections at the PCC joint/crack (Figure 4.3), 3) PCC cores extracted from pavement (Figure 4.4), and 4) General climate conditions collected from the database of the National Oceanic and Atmospheric Administration (NOAA) website. The climate data was inputted into the Enhanced Integrated Climatic Model (EICM) developed by Larson and Dempsey (2006) to predict temperature profiles through the pavement structure.

Figure 4.2 – Weigh-in-Motion/Automated Vehicle Classifiers (WIM/AVC)

Figure 4.3 – Falling Weight Deflectometer (FWD) Testing at the PCC Joint

Figure 4.4 – Extracted Portland Cement Concrete Core from Pre-Overlaid Pavement

### 4.5 Laboratory Evaluation

During construction of the bituminous overlays, loose mix was collected at the paver for each mixture type utilized. The mixtures, along with the PCC cores extracted from the pavements, were brought back to the laboratory for material characterization. The material characterization testing conducted was based on the information collected during the Literature Review and National Survey as prime material parameters required to properly evaluate bituminous materials under reflective cracking type movements, and therefore, utilized in the reflective cracking prediction methodology. The laboratory testing conducted and utilized in the prediction methodology are; 1) Dynamic Modulus Test, AASHTO TP62-07 (Figure 4.5), 2) Flexural Beam Fatigue, AASHTO T321 (Figure

4.6), 3) Overlay Tester, TxDOT Tex-248-F (Figure 4.7), and 4) Coefficient of Thermal Expansion of PCC, AASHTO TP60-07 (Figure 4.8).


Figure 4.5 – Dynamic Modulus Test Device

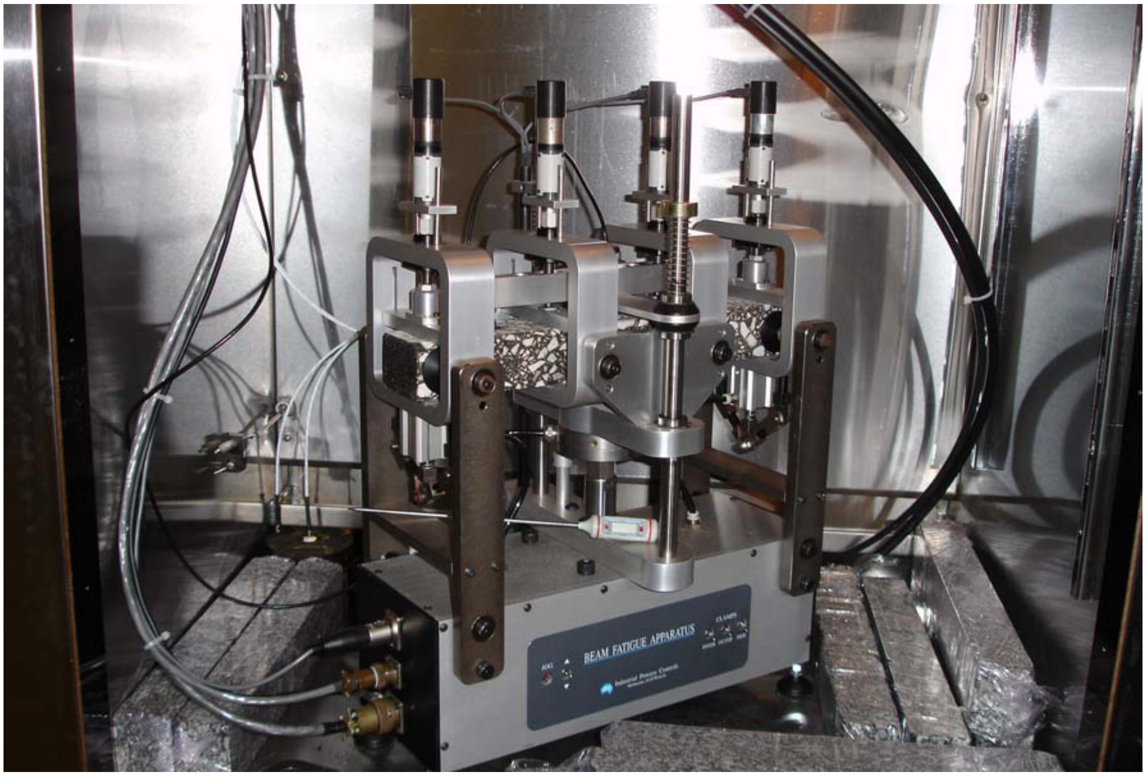



Figure 4.6 – Flexural Beam Fatigue Test Device

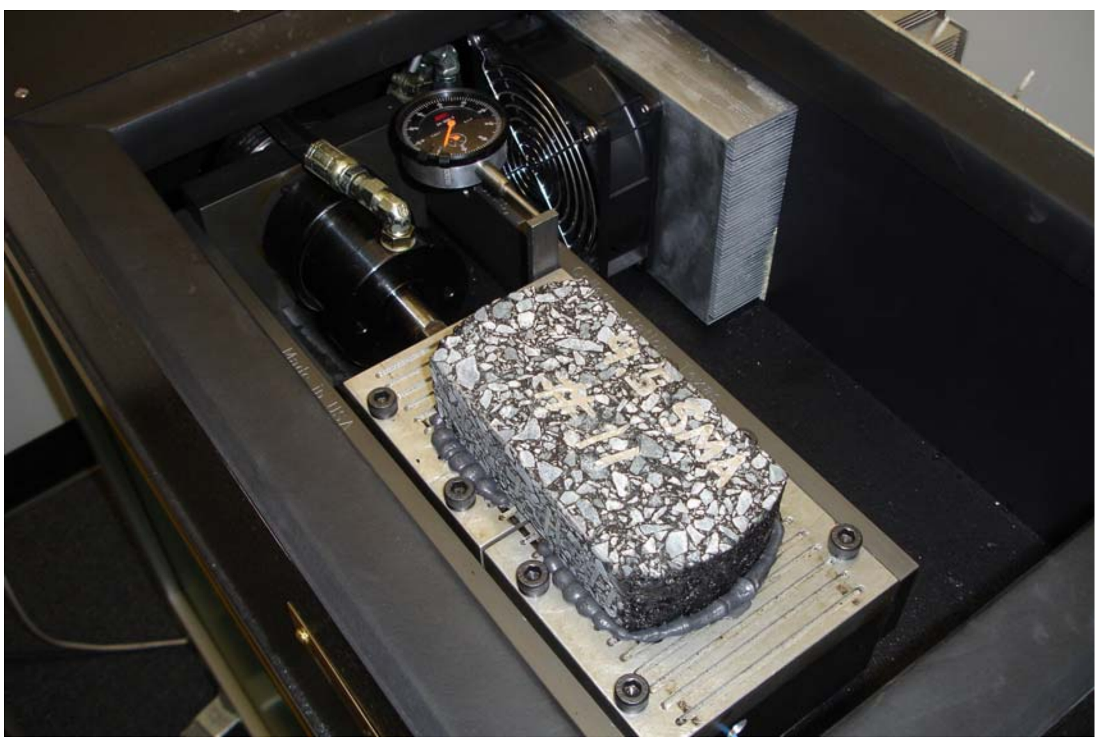


Figure 4.7 – Overlay Tester Device

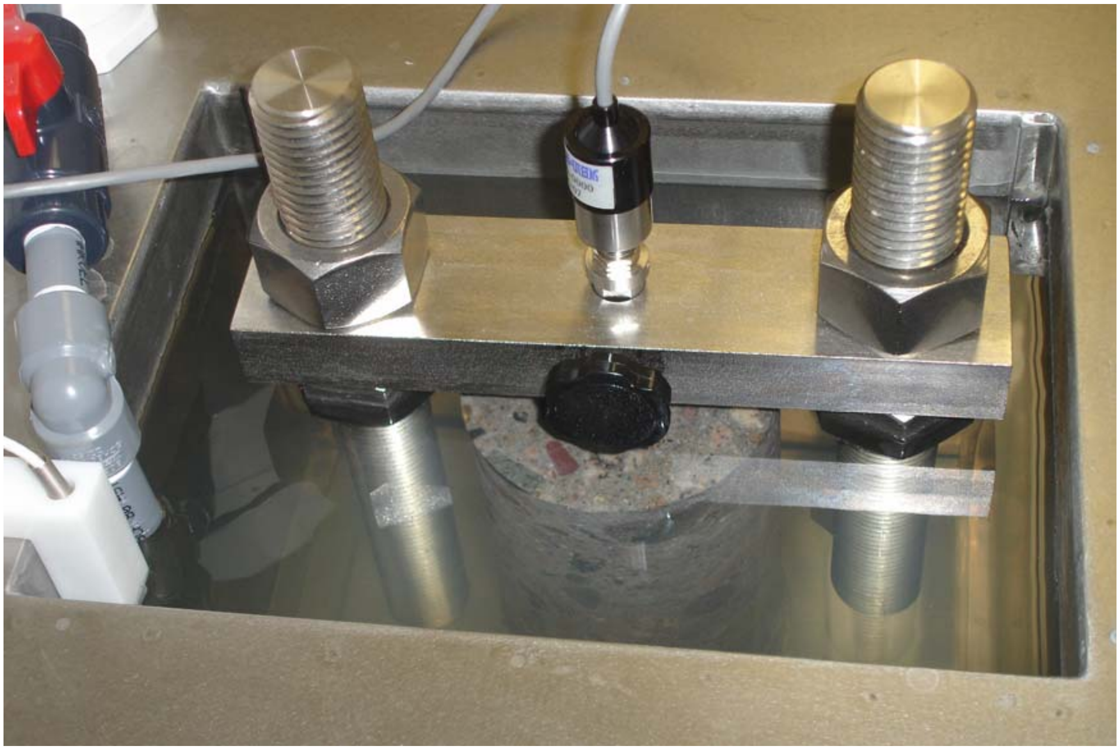


Figure 4.8 – Coefficient of Thermal Expansion Test Device

### 4.6 Analysis of Generated Test Results

The data generated from the field test sections, laboratory tests, and monitoring of the test sections were analyzed and used to calibrate the prediction methodology. An advantage to reflective cracking failures, as opposed to traditional fatigue cracking, is that a majority of reflective cracking problems generally occur within the first two years after the bituminous overlay has been placed. The test sections selected in the study were all tested and overlaid between 2006 and 2007. This allowed for an accurate calibration of the prediction methodology for both the vertical and horizontal pavement deflection modes. The final calibration of the prediction methodology provided the required information for the development of the Decision Tree System for bituminous overlays for PCC pavements.

### 4.7 Development of Decision Tree

With the lack of time, and for the most part, lack of trained personal, in most cases state agencies would not have the ability to conduct the analytical calculations required in the prediction methodology developed in this research study. However, the development of an easier and more practical selection procedure would most often be utilized. Therefore, a Decision Tree system was developed from the analysis described in Section 4.6. The Decision Tree system asks for the pavement characteristics (i.e. – PCC slab length, deflections at the PCC joint, proposed bituminous thickness overlaying the PCC, and traffic level in ESAL's) to recommend the bituminous overlay system (with or without an asphalt interlayer) and the fatigue requirements of the respective mixture(s).

### 4.8 Conclusions and Recommendations

The test results and analysis generated, as well as the extensive Literature Review and National Survey results, were summarized in the Conclusions. The Conclusions also include the general findings that resulted in the development of the Decision Tree system. Based on the outcome and results of the research study, recommendations for future research are provided.

# CHAPTER 5 – FIELD EVALUATION AND TESTING

Field test sections are a necessary requirement for any reflective cracking study due to the complex nature of the deformations that occur at the PCC joint/crack area. Vertical deflections, created by applied traffic loading, and horizontal deflections, resulting from temperature cycling, result in a coupled affect that would be extremely difficult, if not impossible to duplicate in a laboratory setting.

Another factor often forgotten when comparing laboratory evaluations to field performance is the affect the asphalt plant production has on the mixture characteristics. With asphalt suppliers consistently using RAP in a majority of their mixtures, as well as differences in mixing efficiency and asphalt binder stiffening that occurs in the batch plant pug mill or the drum, bituminous mixtures produced in the laboratory do not adequately represent plant produced asphalt mixtures, especially with respect to cracking resistance. Therefore, any type of model calibration involving field performance and asphalt material testing also requires that the asphalt mixture testing be conducted on material sampled at the paver.

Therefore, field test sections were selected to help provide the necessary field and material inputs for the prediction methodology. Each of the test sections were required to be constructed by 2007 in order to allow for some time where reflective cracking could potentially occur. As indicated in the National Survey conducted as part of this research study, a majority of the state agencies that responded to the survey indicated that their composite pavements failed within 2 to 4 years. Therefore, a 2 year period between the time of construction and final analysis was determined to be appropriate to represent how the bituminous overlay design and material selection performed.

The last important parameter included in the study was test sections that included asphalt interlayers to mitigate the reflective cracking. As indicated in the national survey, asphalt interlayer mixtures had the best track record of mitigating reflective cracking. Therefore, test sections were selected that included asphalt interlayer mixtures to help provide a final recommendation on the appropriate pavement system to mitigate reflective cracking.

### 5.1 Rt 34N – New Jersey

The first test site of the research study was New Jersey State Route 34 Northbound, between mileposts 0.3 and 7.6, located in Wall Township, New Jersey. Route 34 is a two-lane composite pavement, originally consisting of 228.6 mm (9 inches) of concrete overlaid with 63.5 to 150 mm (2.5 to 4.5 inches) of hot mix asphalt. The concrete pavement consists of 12.2 m (40 ft) slabs with 19mm (3/4 inch) expansion joints and 31.75 mm (1.25 inches) diameter stainless steel dowel bars. The first hot mix asphalt overlay originally consisted of a 9.5mm nominal aggregate gradation with an AC20 asphalt binder. Recent overlays, which were placed as part of a mill and replace rehabilitation treatment, used a 12.5mm Superpave mix with a PG64-22 asphalt binder. The predominant pavement distress associated on this roadway was reflective cracking, with longitudinal cracking and slight rutting (Figure 5.1a and b).

(a) (b)

Figure 5.1a and 1b – Typical Reflection Cracking at the Joint Area of the Composite Pavement Test Section

In the summer of 2006, a maintenance resurfacing program consisting of using a reflective crack interlayer mix (RCRI) was specified. The project limits of the resurfacing project were from milepost 0.2 to milepost 7.6. The pavement design recommendation consisted of milling to a constant depth of 76.2 mm (3 inches) prior to the overlay. Photos of some of the construction are shown in Figures 5.2 and 5.3.

Figure 5.2 – Photo of Reflective Crack Relief Interlayer (RCRI) Mix, shown in far lane, and Existing HMA Patch Over Deteriorated Reflective Crack

Figure 5.3 – Paving RCRI Mixture on Rt 34N in New Jersey

The selected asphalt overlay for the project consisted of the following:

- Section #1: Milepost 0.3 to 2.5, consists of 25mm (1 inch) of a reflective crack relief interlayer mix (RCRI), overlaid by 50 mm (2 inches) of a 12.5mm Superpave HMA (NJDOT 12.5M76), which in turn is overlaid by 38.1 mm (1.5 inches) of a 9.5mm Superpave mix (NJDOT 9.5H76).
- Section #2: Milepost 2.5 to 4.5, consists of 76.2 mm (3 inches) of a 12.5mm Superpave HMA overlaid by 38.1 mm (1.5 inches) of a 9.5mm Superpave HMA.
- Section #3: Milepost 4.5 to 7.6, consists of 25mm (1 inch) of a reflective crack relief interlayer mix (RCRI), overlaid by 50 mm (2 inches) of a 12.5mm

Superpave HMA, which in turn is overlaid by 38.1 mm (1.5 inches) of a 9.5mm Superpave mix.

Both the 9.5mm and 12.5mm Superpave mixes contained a PG76-22 asphalt binder. The 12.5mm Superpave mixture was designed using an $N_{design}$ level of 75 gyrations, while the 9.5mm Superpave mixture was designed using an $N_{design}$ level of 100 gyrations. The RCRI mixture, marketed under the name Strata®, contained a highly polymerized asphalt binder specially designed for high deflection-type applications and manufacturered by SemMaterials. The mixture design information of the asphalt mixtures is shown in Table 5.1. The 9.5mm and 12.5mm Superpave mixtures are the most common asphalt overlay mixes used in New Jersey for composite pavement maintenance. The composite pavement is supported by an uncrushed gravel base layer, which in turn rests on a silty sand subgrade soil.

Table 5.1 – HMA Mixture Volumetric Design Properties

| Mixture Design Property | Mixture Type | | |
|---|---|---|---|
| | 12.5mm | 9.5mm | RCRI |
| Binder Content (%) | 4.7% | 5.1% | 7.5% |
| VMA (%) | 15.1% | 15.7% | 19.1% |
| $G_{mm}$ (g/cm$^3$) | 2.58 | 2.513 | 2.425 |
| $G_{sb}$ (g/cm$^3$) | 2.779 | 2.716 | 2.726 |
| Percent Passing | | | |
| 19mm | 100 | 100 | 100 |
| 12.5mm | 93.7 | 100 | 100 |
| 9.5mm | 85.2 | 94.7 | 100 |
| 4.75mm | 52.9 | 61.4 | 93 |
| 2.36mm | 32.4 | 41.2 | 71 |
| 1.18mm | 25.1 | 29.8 | 51 |
| 0.6mm | 19.1 | 20 | 36 |
| 0.3mm | 10.3 | 10 | 21 |
| 0.075mm | 3.5 | 5 | 8.6 |

### 5.1.1 Traffic Conditions on Rt 34N

Portable Weigh-in-Motion Sensors (WIM) and Automatic Vehicle Classifiers (AVC) were used to characterize the traffic loading conditions on the test site. Data was collected over a seven-day period and used to determine the axle load spectra that could be expected at the test site. The axle load spectra was collected so that future expansion of the prediction methodology, or even future use in the Mechanistic Empirical Pavement Design Guide (MEPDG), could be conducted.

#### *5.1.1.1 Vehicle Class Distribution (VCD)*

The Vehicle Class Distribution for Rt 34N in New Jersey is shown in Figure 5.4. Overall, the one-way Average Daily Traffic (ADT) was measured as 8,840 vehicles. The summary of the VCD was as followed:

- 91.6% automobiles;
- 2.7% light trucks (FHWA Class 4 and 5); and
- 5.7% heavy trucks (FHWA Class 6 and Greater)

#### *5.1.1.2 Hourly Distribution*

The Hourly Distribution (HRD) of the traffic stream on Rt 34N in New Jersey was measured to evaluate peak flow traffic volume and it respective time of the day. This is an important factor in the general performance of bituminous materials as the time of the day will influence the general range in temperature that is associated with the highest volume of traffic. For example, if an asphalt pavement undergoes it greatest

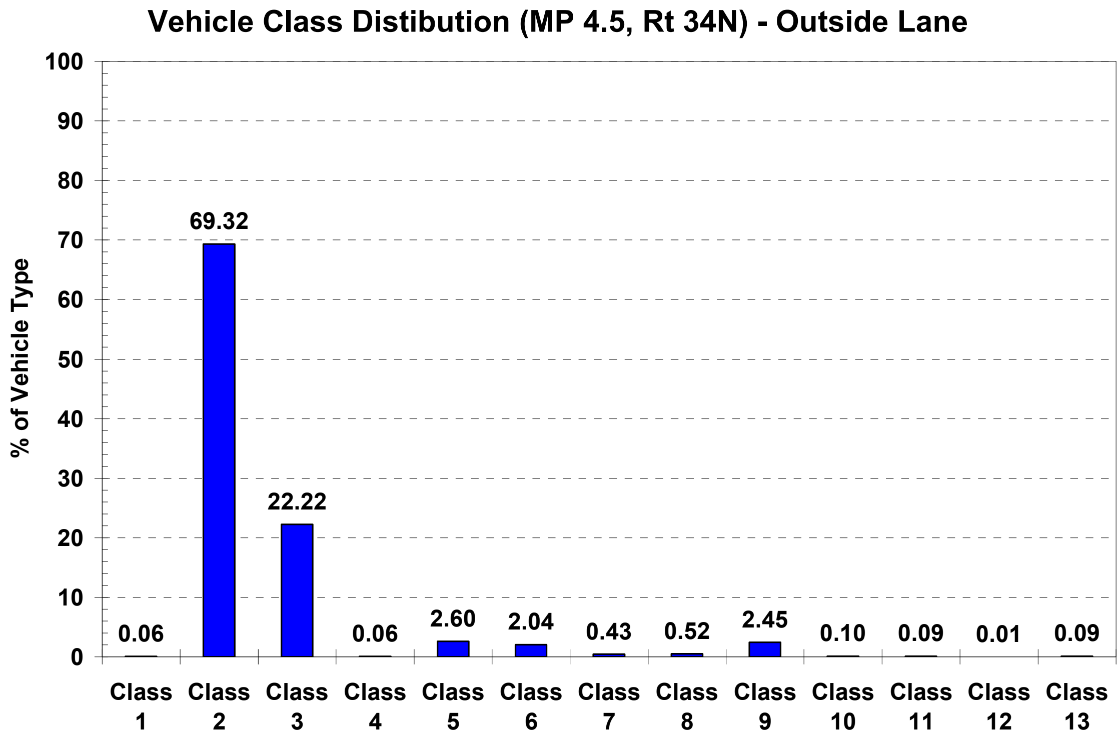



Figure 5.4 – Vehicle Class Distribution (VCD) for Rt 34N in New Jersey

traffic volume in the afternoon, rutting susceptibility may become an issue as the asphalt layer is generally it hottest at this time. In contrast, fatigue cracking may be attributed to higher traffic volumes during colder time periods of the day (usually during the morning rush hour). The HRD for Rt 34N in New Jersey is shown in Figure 5.5. As can be seen in the figure, the highest level of traffic is occurring during the morning rush hours, which may increase the potential of cracking in the test section.

*5.1.1.3 Axle Load Spectra*

The Axle Load Spectra (ALS) for the traffic on Rt 34N was collected using the WIM-AVC system. Each axle weight and count per vehicle class are shown in Figure

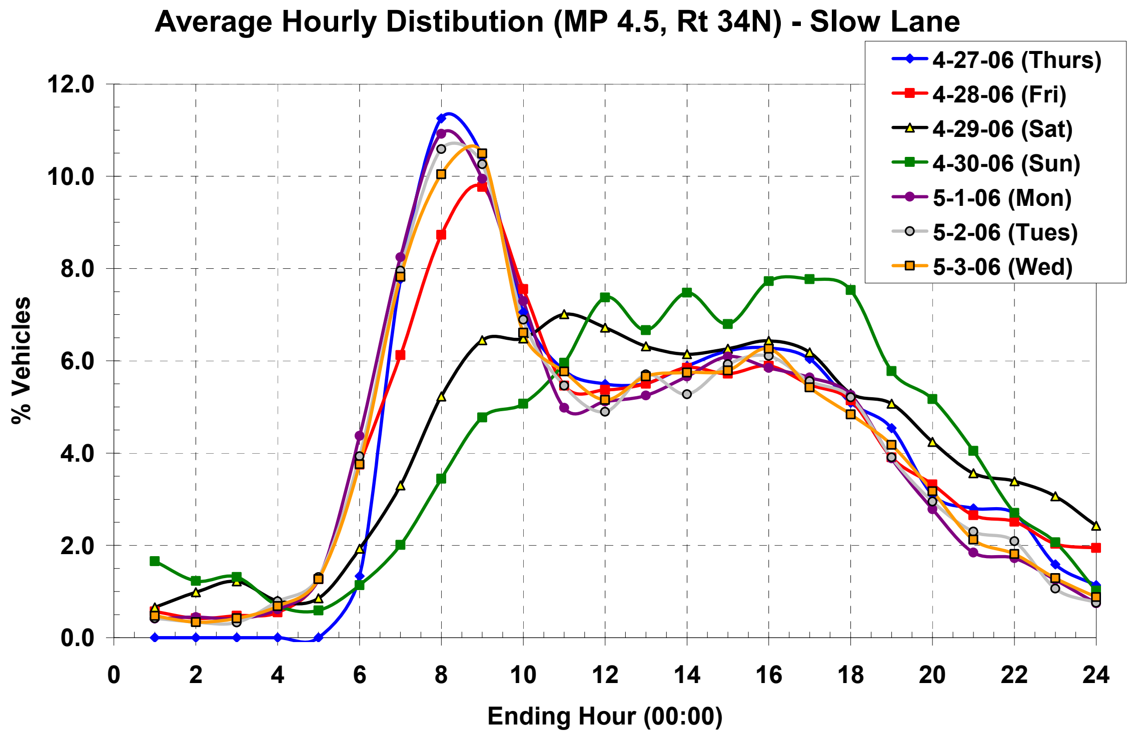



Figure 5.5 – Hourly Distribution on Rt. 34N in New Jersey

5.6. As shown in Figure 5.6, the majority of traffic stream (percent vehicles) is due to the non-truck traffic (FHWA Classification Class 1 to 3). However, the ALS does indicate that the truck traffic does provide significant axle loading. For example, the Class 6 truck count indicates an average axle load over 12,000 lbs with 540 daily occurrences (180 Class 6 trucks times 3 axles).

*5.1.1.4 Summary of Traffic Conditions on Rt. 34N*

Detailed hourly distribution, vehicle and axle distribution were measured, resulting in the following average traffic loading:

- Average Daily Traffic (ADT) = 8,840 vehicles

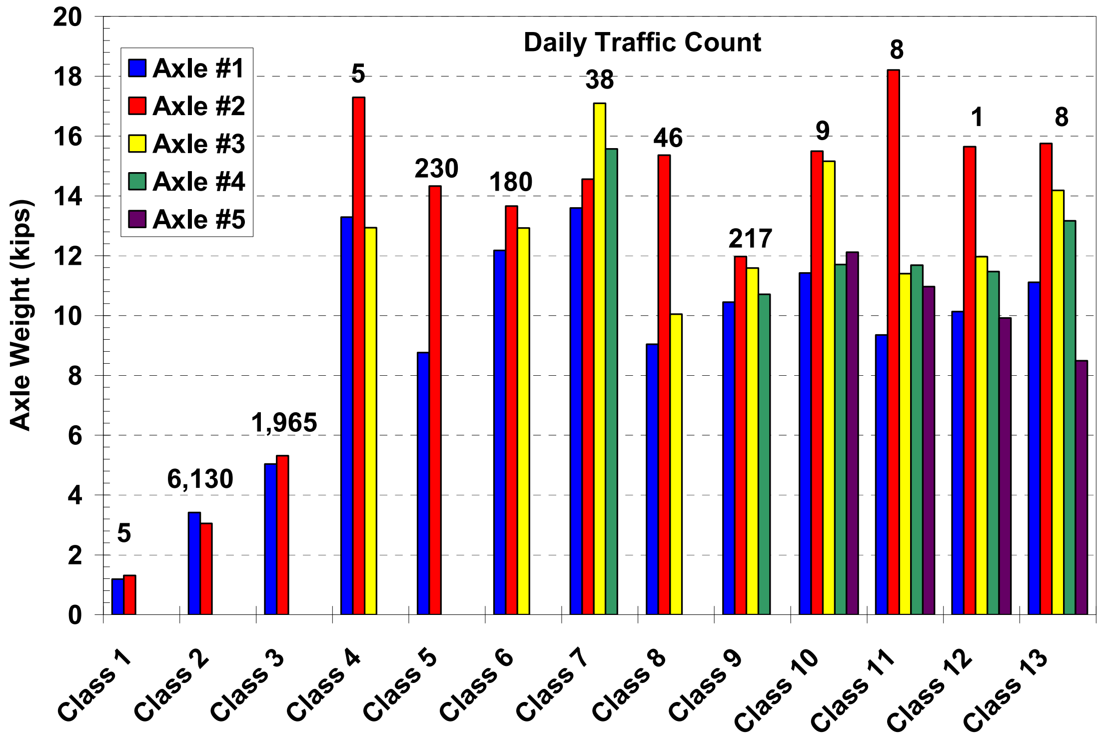



Figure 5.6 – Axle Load Spectra (ALS) for Rt. 34N in New Jersey

- Automobile Axle Loads: Ave. = 16.9 kN (3.8 kips); Max. = 25.8 kN (5.6 kips)
- Light Truck Axle Loads: Ave. = 59.2 kN (13.3 kips); Max. = 91.2 kN (20.5 kips)
- Heavy Truck Axle Loads: Ave. = 57.8 kN (13 kips); Max. = 108.5 kN (24.4 kips)

5.1.2 Visual Distress Survey

Although the opportunity was not available for all test sections, a Visual Distress Survey (VDS) was conducted to assess the existing pavement distresses on Rt 34N in New Jersey. The VDS was conducted in accordance to the *Distress Identification Manual for the Long-Term Pavement Performance Project* (2003). Table 5.2 summarizes the VSD results. It should be noted that Lane #1 represents the inside (fast/passing) lane and Lane #3 is the acceleration/deceleration lane. It should also be noted that the longitudinal joints in the PCC pavement had reflected to the surface of the asphalt overlay and resulted in areas of medium-severity longitudinal cracking.

Table 5.2 – Visual Distress Survey for Rt 34N in New Jersey

| Number of Transverse Cracks | Lane #1 | Lane #2 | Lane #3 | Right Shoulder |
|---|---|---|---|---|
| Low Severity | 65 | 37 | 2 | 3 |
| Moderate Severity | 46 | 59 | 4 | 13 |
| High Severity | 414 | 474 | 12 | 52 |
| Patched Transverse Joints | 10 | 9 | 0 | 0 |

5.1.3 Falling Weight Deflectometer Testing

Falling Weight Deflectometer (FWD) testing was conducted before and after the transverse joints at three load magnitudes; 28.91, 44.48, and 71.17 kN (6.5, 10, and 16 kips). The FWD testing was used to assess the Load Transfer Efficiency (LTE) and the vertical deflections at the joint due to the applied loads. The LTE has been shown to be related to shear stress in the vicinity of the PCC joint/crack area. Meanwhile, the vertical deflections are related to the bending stress resulting in tensile stress/strain at the bottom of the HMA layer.

The measured vertical deflections at the PCC joint/crack, normalized to 9,000 lbs for comparison purposes, are shown in Figure 5.7. On average, the vertical deflection normalized to 9,000 lbs was 6.08 mils with a standard deviation of 1.47 mils.

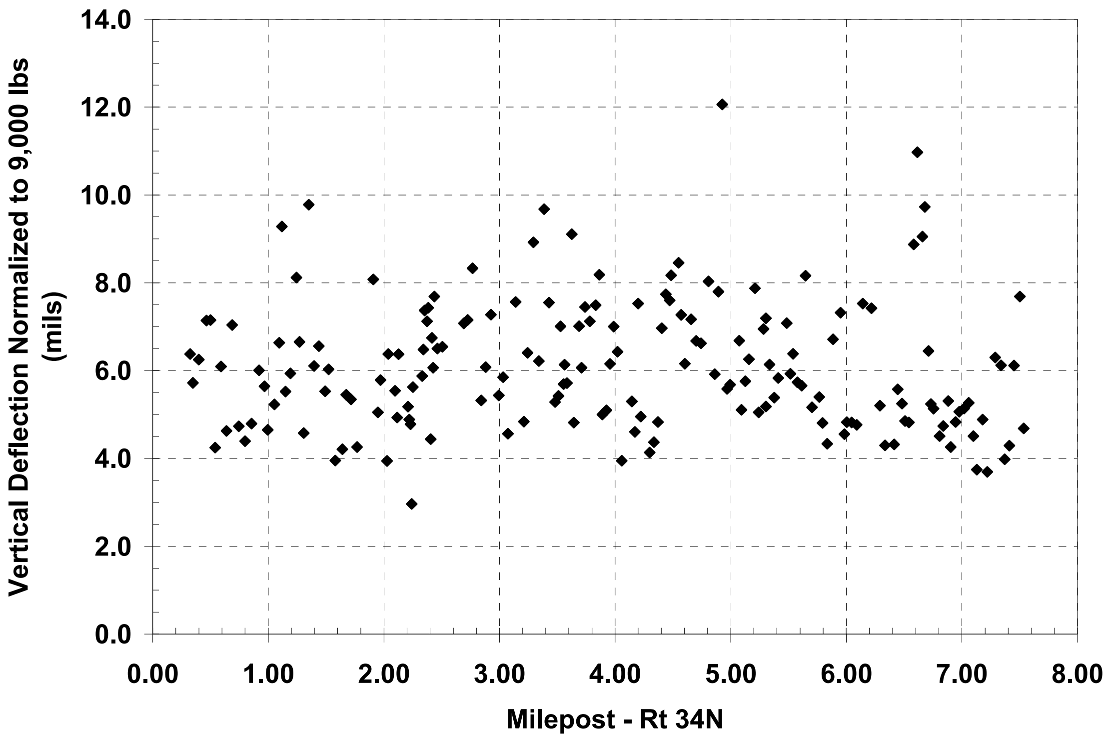



Figure 5.7 – Vertical Deflections at PCC Joint/Crack

The measured Load Transfer Efficiency (LTE) at the PCC joint/crack is shown in Figure 5.8. The measured LTE resulted in an average LTE of 77.1% with a standard deviation of 14.1%.

Since there are three test sections located on Rt 34N, the test results were calculated and compared for each test section. The Falling Weight Deflectometer (FWD) measured parameters for the different test sections were as follows:

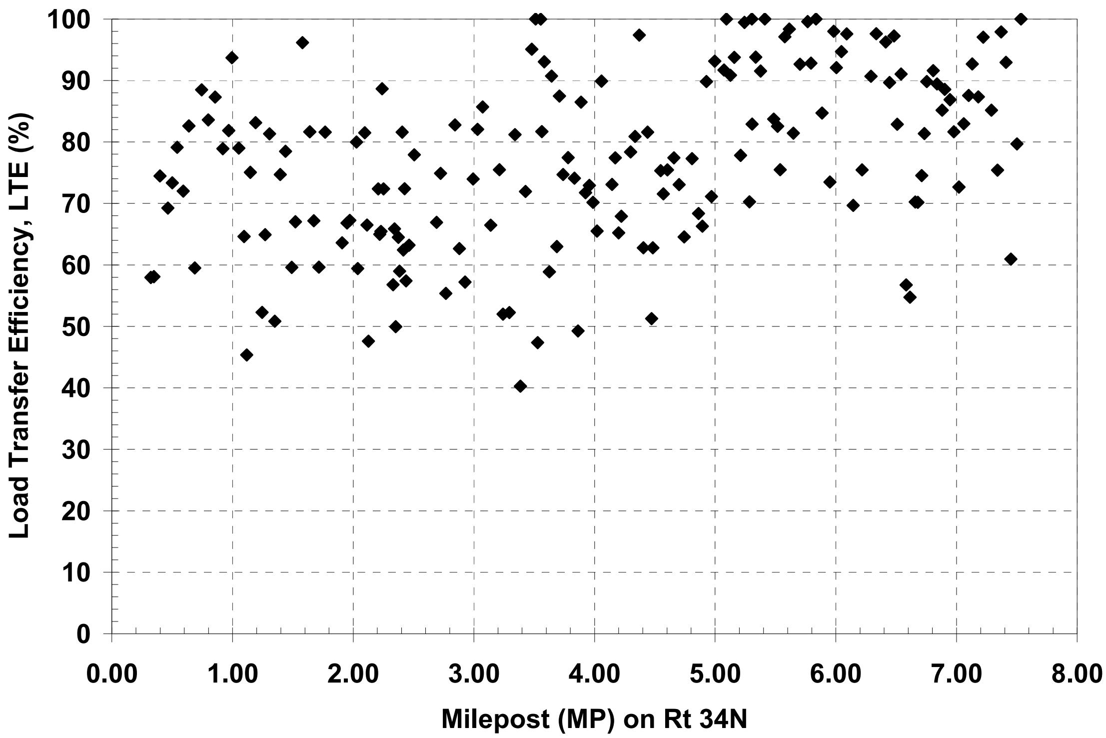



Figure 5.8 – Load Transfer Efficiency for Rt 34N in New Jersey

- Test Section #1 (MP 0.3 to 2.5): Average LTE = 71% with an average vertical deflection of 5.8 mils when normalized to 9 kips;
- Test Section #2 (MP 2.5 to 4.5): Average LTE = 73% with an average vertical deflection of 6.4 mils when normalized to 9 kips; and
- Test Section #3 (MP 4.5 to 7.4): Average LTE = 85% with an average vertical deflection of 6.0 mils when normalized to 9 kips.

It should be noted that the FWD testing was conducted during the month of April 2006 with average air and pavement surfaces temperatures of 63 and 74°F, respectively. Therefore, it was assumed that minimal to no PCC joint/crack friction had occurred that would have increased LTE and reduced the vertical deflection.

5.1.4 – PCC Vertical Joint Deflection Spectra

When conducting FWD testing at different load magnitudes, the relationship between the vertical deflection at the joint (measured immediately under the load) and the applied load can be represented by a linear regression, thereby, allowing for the direct substitution of any known applied axle load to determine the resultant deflection at the joint. The relationship for each Test Section, along with the average, is shown as Figure 5.9.

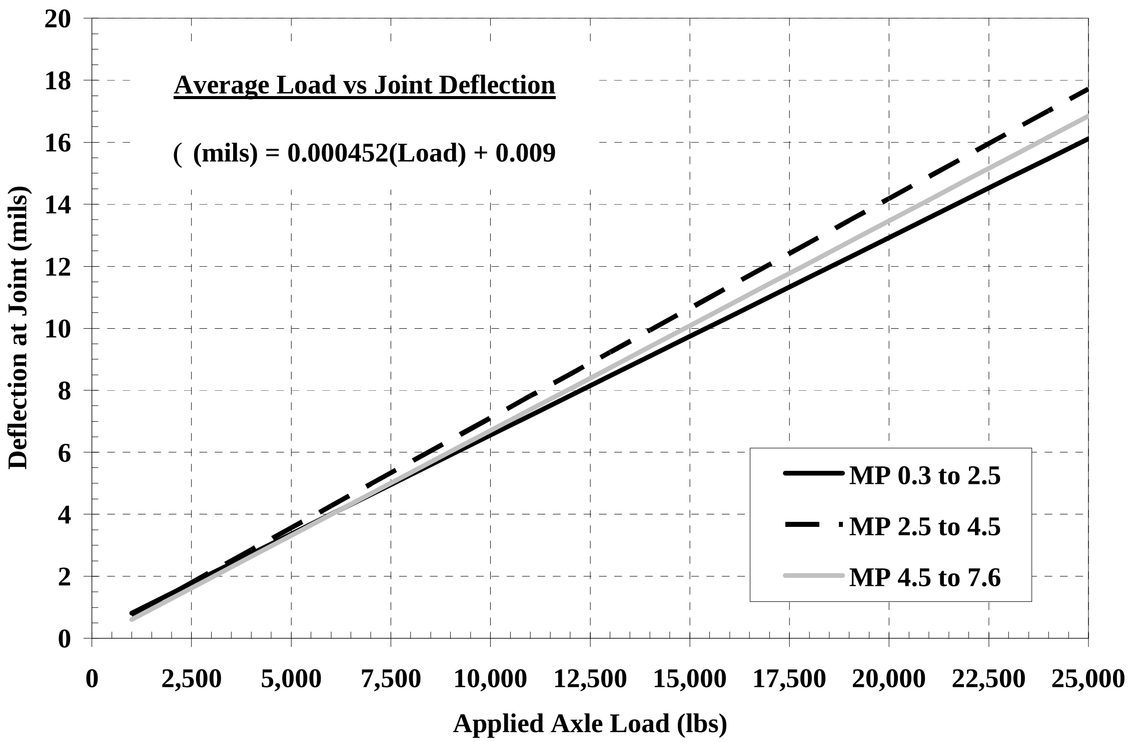



Figure 5.9 – Applied Load vs Resultant PCC Joint/Crack Vertical Deflection

Once the relationship between applied load and vertical joint deflection is established, the axle load measurements recorded by the WIM/AVC sensors can be directly inputted into the regression equation to generate a PCC Vertical Joint Deflection Spectra. In this research, a PCC Vertical Joint Deflection Spectra is defined as the amount and magnitude of vertical deflections that are assumed to occur due to daily traffic volumes. The PCC Vertical Joint Deflection Spectra developed for Test Section #1 is shown as Figure 5.10. Figure 5.10 provides valuable information regarding the magnitude of the vertical deflection due to the individual axle load. More detail on the use of the PCC Vertical Joint Deflection Spectra and how it can be applied to laboratory testing is discussed later in Chapter 6.

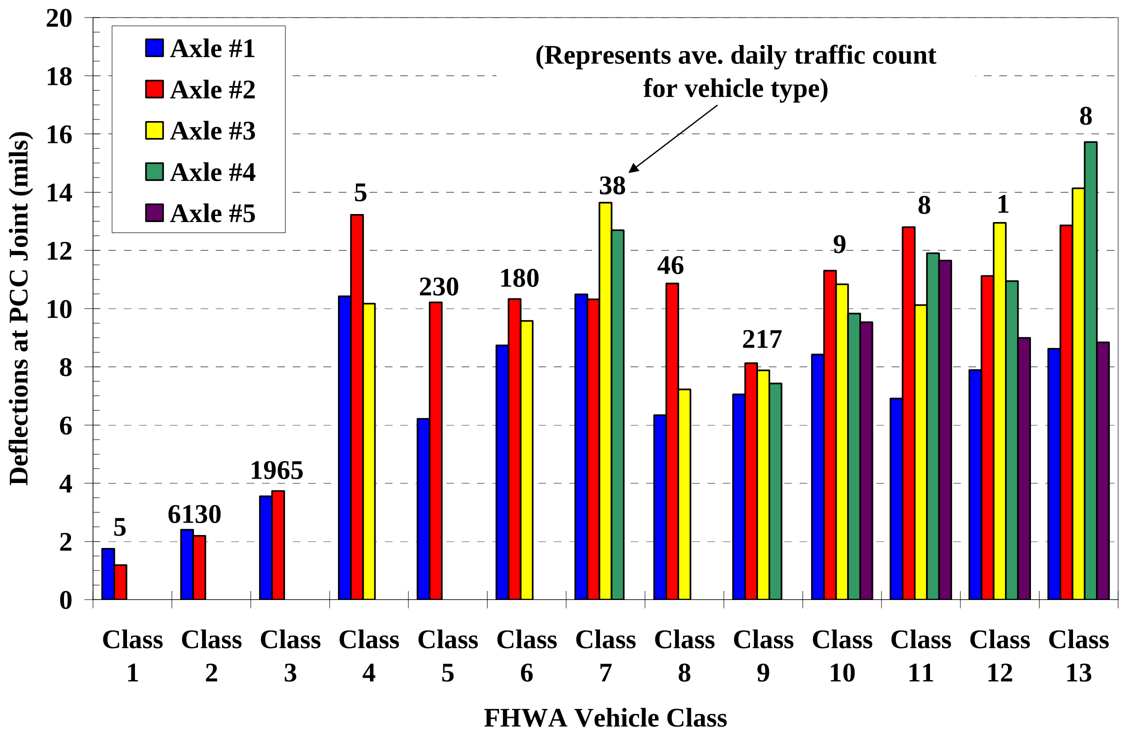



Figure 5.10 – PCC Vertical Joint Deflection Spectra for Test Section #1

5.1.5 Coring - Concrete Test Results

Immediately after the FWD testing, full-depth pavement cores were taken for laboratory analysis (Figure 5.11). Of particular interest was the Coefficient of Thermal Expansion (CTE) of the underlying concrete pavement, as measured in accordance with AASHTO Designation: TP60-06, *Standard Method of Test for Coefficient of Thermal Expansion of Hydraulic Cement Concrete*. The CTE is a parameter that, along with effective slab length ($L_{eff}$), maximum 24 hour temperature difference ($\Delta T$), and the PCC/Base friction factor ($\beta$), can provide an estimate of the expected horizontal movement at the concrete slab joint ($\Delta L$) due to daily temperature changes, as shown in Equation 5.1.

$$\Delta L = CTE(L_{eff})(\Delta T)(\beta) \tag{5.1}$$

Figure 5.11 – Cross Sectional View of PCC Core Taken from Rt 34N in New Jersey

Six cores were tested according to AASHTO TP60. The average CTE of extracted cores from Rt 34N were determined to be 12.34E-6 cm/cm/$^{o}$C, with a standard deviation of 0.28E-6 cm/cm/$^{o}$C. Since the temperature of the underlying concrete slabs was not recorded, the Enhanced Integrated Climatic Model, EICM (Larson and Dempsey, 2006) was used to estimate temperature profiles in the pavement. Interpolated weather data from neighboring weather stations were used to generate 5 years (1998 and 2003) worth of historical climate information for the test site. The average 24-hour temperature difference determined was 6$^{o}$C (10.8$^{o}$F) at the surface of the concrete pavement for a 114.5 mm (4.5 inch) asphalt overlay. Figure 5.12 shows an example of the thermal prediction output provided by the Enhanced Integrated Climactic Model (EICM). The PCC/Base Friction Factor (β) was estimated from comparable PCC pavements overlying aggregate base course material at the Long Term Pavement Project (LTPP) concrete pavement test sites (Khazanovich and Gotliff, 2002). An average β assumed for the analysis was 0.76. Therefore, using Equation (5.1) and the appropriate input parameters, it was concluded that the maximum horizontal movement that would be expected in a 24-hour period is 0.67 mm (0.026 inches).

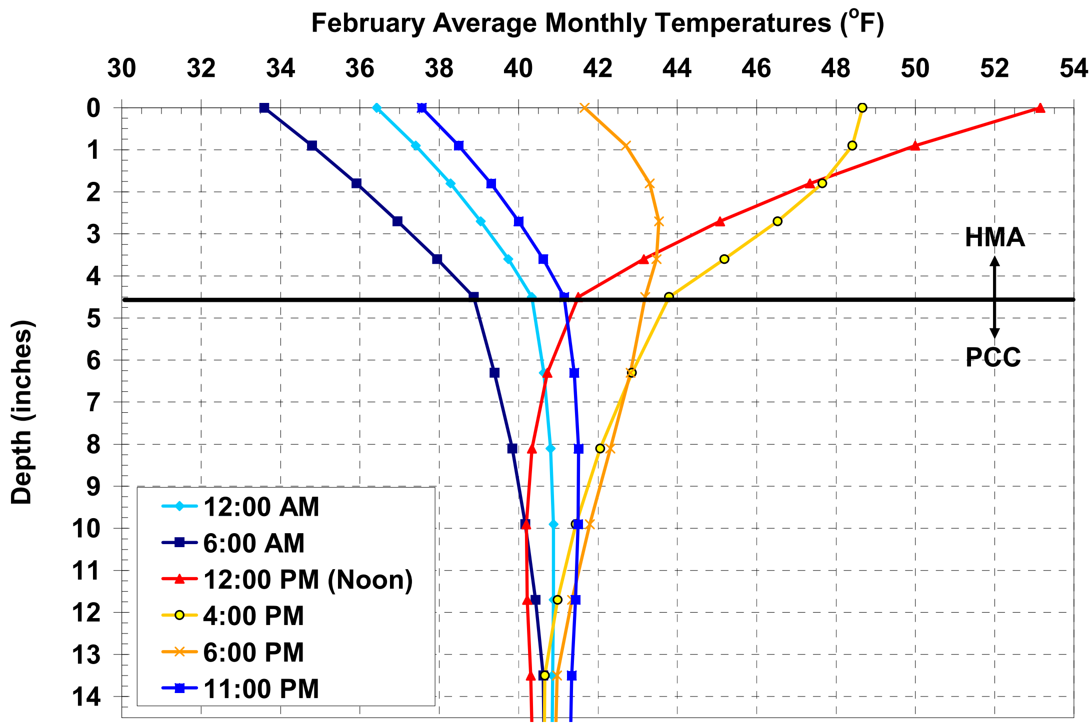



Figure 5.12 – An Example of the Predicted Thermal Distribution on Rt 34N in New Jersey

### 5.1.6 Dynamic Cone Penetrometer (DCP) Testing

After the cores were extracted from the pavement, Dynamic Cone Penetrometer (DCP) testing was conducted through the core hole into the unbound material. The DCP testing was used to evaluate the general bearing capacity (California Bearing Ratio, CBR) of the underlying unbound materials, as well as the general thickness of each of the unbound aggregate/subgrade layers. Table 5.3 shows the corresponding CBR values of the underlying unbound material.

Table 5.3 – Dynamic Cone Penetrometer (DCP) Test Results

| Core No. | Lane | Milepost | Depth into Granular Layer (in.) and CBR Value | | | | |
|---|---|---|---|---|---|---|---|
| 1 | Lane 2 | 2.11 | 0.0 - 20.7 (40%) | | | | |
| 2 | Lane 2 | 2.21 | 0.0 - 5.9 (25%) | 5.9 - 15.4 (33%) | 15.4 - 18.7 (24%) | | |
| 3 | Lane 2 | 2.39 | 0.0 - 5.5 (12%) | 5.5 - 13.6 (22%) | 13.6 - 17.2 (16%) | 17.2 - 22.5 (43%) | |
| 9 | Lane 2 | 4.62 | 0.0 - 6.7 (54%) | 6.7 - 12.0 (40%) | 12.0 - 20.9 (34%) | | |
| 11 | Lane 2 | 5.50 | 0.0 - 5.5 (47%) | 5.5 - 19.6 (61%) | 19.6 - 21.4 (40%) | | |
| 13 | Lane 2 | 6.23 | 0.0 - 4.7 (37%) | 4.7 - 9.7 (28%) | 9.7 - 17.5 (40%) | 17.5 - 19.7 (30%) | |

In general, the interpretation of the DCP test results indicates that there exists relatively good supporting material underneath the PCC slabs. The average results for the two identified unbound layers are shown below;

- Unbound Layer #1:
    - Thickness: Approximately 5.7 inches
    - CBR Value: Approximately 35.8%
- Unbound Layer #2:
    - Thickness: Approximately 10.25 inches
    - CBR Value: Approximately 42.3%

### 5.1.7 Pavement Performance on Rt 34N in New Jersey

Within the first seven months after construction, reflective cracking was observed (March 2007). Figures 5.13 and 5.14 show pictures of reflective cracking from the Rt 34N test section in New Jersey from MP 0.2 to 2.5 and MP 2.5 to 4.5 sections, respectively. The Visual Distress Survey (VSD) conducted indicated that seven months after construction had completed;

- Test Section #1 (MP 0.2 to 2.5): 16.4% of the transverse joints/cracks had reflected through (7% of total joint length)
- Test Section #2 (MP 2.5 to 4.5): 9% of the transverse joints/cracks had reflected through (3.6% of the total joint length)
- Test Section #3 (MP 4.5 to 7.4): 2% of the transverse joints/cracks reflected through (0.33% of total joint length)

Follow-up Visual Distress Surveys had been conducted since the first observation of reflective cracking in March 2007. Table 5.4 shows the measured cracking on the test location as of March 2009.

Table 5.4 – Transverse Cracking Measurements on Rt 34N in New Jersey (March 2009)

| | South Strata Section (MP 0.2 to 2.5) | Control Section[1] (MP 2.5 to 4.5) | North Strata Section (MP 4.5 to 7.6) |
|---:|:---:|:---:|:---:|
| Total Pavement Length (ft) = | 12,144 | 9,230 | 16,368 |
| NJDOT Reported Transverse Joint Spacing (ft) = | 40 | 40 | 40 |
| Number of Transverse Joints = | 305 | 232 | 410 |
| Number of Transverse Cracks = | 76 | 44 | 55 |
| % of Transverse Joints Cracked = | 25.0 | 19.0 | 13.4 |
| Lane Width (ft) = | 11 | 11 | 11 |
| Total Transverse Joint Length (ft) = | 6,701 | 5,099 | 9,024 |
| Total Measured Crack Length (ft) = | 542 | 353 | 279 |
| % Joints Length Cracked (%) = | 8.1 | 6.9 | 3.1 |

(a) (b)

Figure 5.13 – Early Reflective Cracking in Test Section MP 0.2 to 2.5

### 5.2 Rt 202S – New Jersey

The second test section of the research study was New Jersey State Route 202 Southbound, between mileposts 13.4 and 17.03, located in Hunterdon County, New Jersey. Rt 202S generally consists of two mainline lanes and a right shoulder. At the time of the overlay, the mainline is comprised of rigid (i.e. – Portland cement concrete, PCC) pavement, while the right shoulder generally consisted of asphalt pavement. The PCC slabs in the mainline lanes were approximately 78 ft in length and approximately 8 inches thick.


Figure 5.14 – Early Reflective Cracking in Test Section MP 2.5 to 4.5

With the donation of experimental asphalt binder from SemMaterials, four experimental test sections were constructed on Rt 202S. The main purpose of the Rt 202S test sections was to evaluate more flexible overlay materials to overlay reflective crack relief interlayers (RCRI). In the case of Rt 202S, the RCRI mixture was the Strata product developed and manufactured by SemMaterials. Figure 5.15 shows the cross-sections of the different sections evaluated. Table 5.5 contains the mixture design information for the asphalt mixtures shown in Figure 5.15.

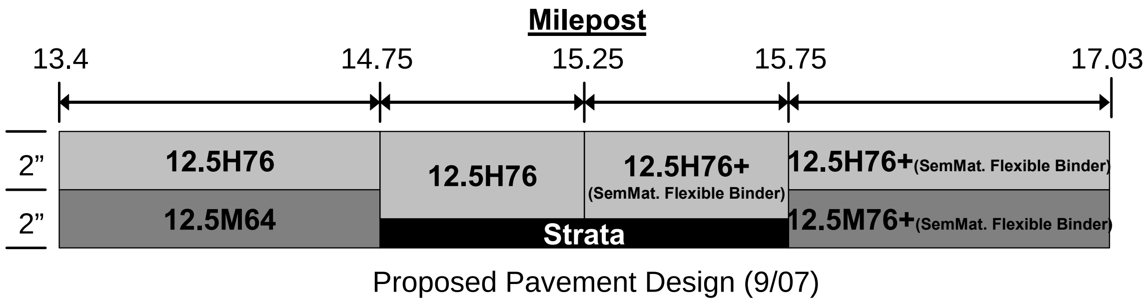



Figure 5.15 – Cross Section of Research Test Sections on Rt. 202S in New Jersey

Table 5.5 – Mixture Design Parameters for Rt 202S Asphalt Mixtures

| Mixture Design Property | Mixture Type | | |
|---|---|---|---|
| | 12M76 | 12H76 | RCRI |
| Binder Content (%) | 5.5% | 5.1% | 8.5% |
| VMA (%) | 15.9% | 15.7% | 18.9% |
| $G_{mm}$ (g/cm$^3$) | 2.616 | 2.513 | 2.421 |
| $G_{sb}$ (g/cm$^3$) | 2.822 | 2.83 | 2.726 |
| Percent Passing | | | |
| 19mm | 100 | 100 | 100 |
| 12.5mm | 94.3 | 93.5 | 100 |
| 9.5mm | 80.7 | 81.8 | 100 |
| 4.75mm | 49.2 | 51.5 | 91 |
| 2.36mm | 34.6 | 34.6 | 65 |
| 1.18mm | 24.6 | 23.8 | 52 |
| 0.6mm | 17.7 | 17 | 42 |
| 0.3mm | 11 | 7.5 | 27 |
| 0.075mm | 5.8 | 5.2 | 7.2 |

It should be noted that the “M” and “H” designations shown in Table 5.5 mean Moderate ($N_{design}$ = 75 gyrations) and Heavy ($N_{design}$ = 100 gyrations) traffic as defined by Superpave. The mixtures in Figure 5.15 noted as 12M76+ and 12H76+ are identical in gradation and volumetric properties to the 12M76 and 12H76 shown in Table 5.5. However, the asphalt binder used is different than a typical polymer-modified PG76-22.

The asphalt binder, shown as 76+, was an experimental fatigue resistant asphalt binder called XFB. The XFB binder was specially formulated by SemMaterials to increase the fatigue cracking resistance of HMA overlays placed on RCRI mixtures.

### 5.2.1 Traffic Conditions on Rt 202S

Portable Weigh-in-Motion Sensors (WIM) and Automatic Vehicle Classifiers (AVC) were used to characterize the traffic loading conditions on the Rt 202S test site. Data was collected over a seven-day period and used to determine the axle load spectra that could be expected at the test site. The axle load spectra was collected so that future expansion of the prediction methodology, or even future use in the Mechanistic Empirical Pavement Design Guide (MEPDG), could be conducted.

#### *5.2.1.1 Vehicle Class Distribution*

The Vehicle Class Distribution for Rt 202S in New Jersey is shown in Figure 5.16. Overall, the one-way Average Daily Traffic (ADT) was measured as 10,178 vehicles. The summary of the VCD was as followed:

- 94.5% automobiles;
- 1.9% light trucks (FHWA Class 4 and 5); and
- 3.6% heavy trucks (FHWA Class 6 and Greater)

#### *5.2.1.2 Hourly Distribution*

The Hourly Distribution (HRD) of the traffic stream on Rt 202S in New Jersey was measured to evaluate peak flow traffic volume and it respective time of the day.

This is an important factor in the general performance of bituminous materials as the time of the day will influence the general range in temperature that is associated with the highest volume of traffic. The Hourly Distribution is shown in Figure 5.17. As opposed to Rt 34N, the peak traffic flow on Rt 202S is towards the evening (6:00PM).

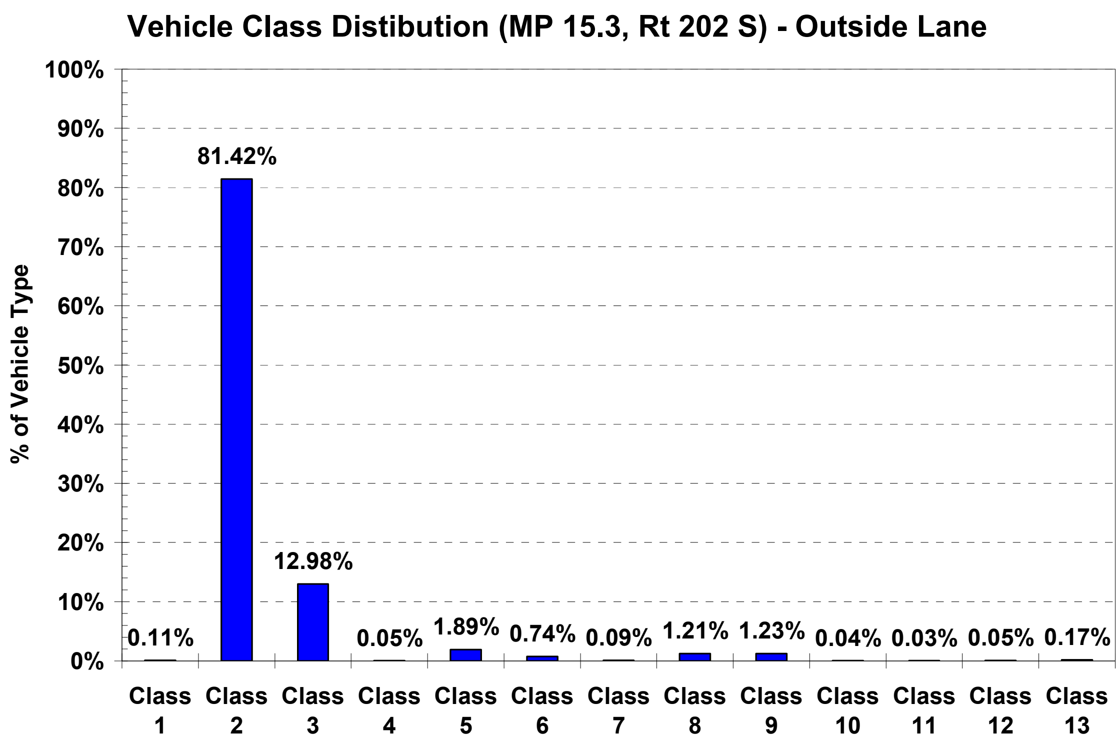



Figure 5.16 – Vehicle Class Distribution for Rt 202S

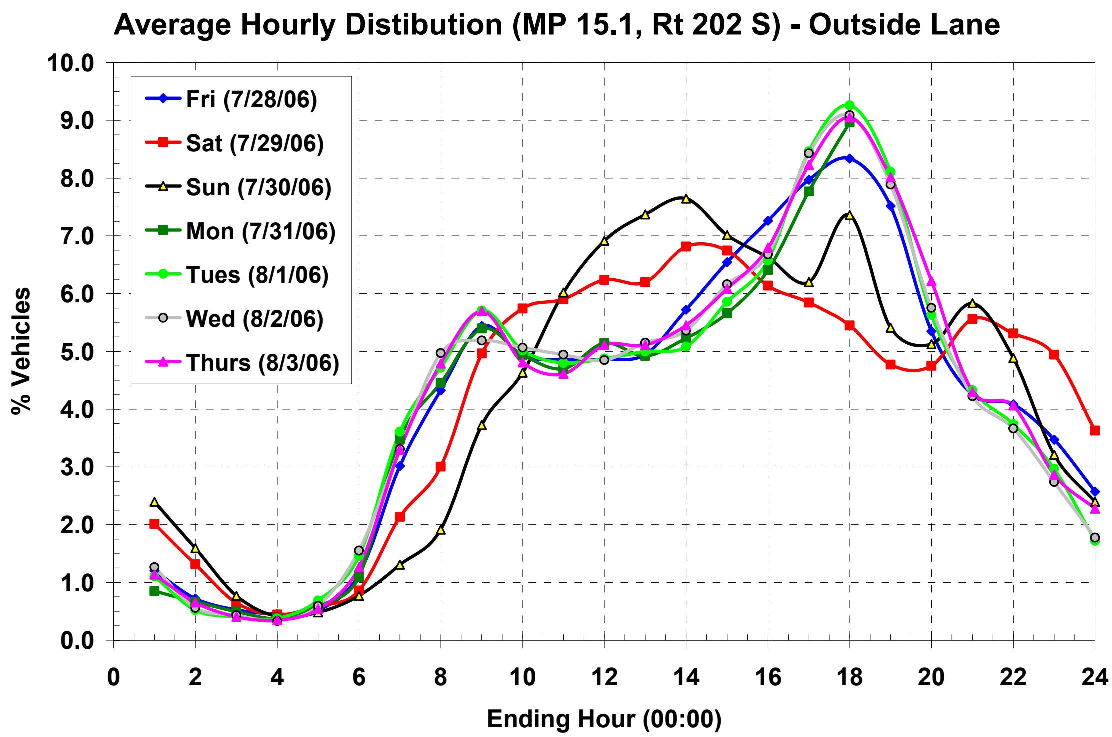



Figure 5.17 – Hourly Distribution for Rt 202S

### *5.2.1.3 Axle Load Spectra*

The Axle Load Spectra (ALS) for the traffic on Rt 202S was collected using the WIM-AVC system. Each axle weight and count per vehicle class are shown in Figure 5.18. As shown in Figure 5.18, the majority of traffic stream (percent vehicles) is due to the non-truck traffic (FHWA Classification Class 1 to 3). However, the ALS does indicate that the truck traffic does provide significant axle loading. For example, almost all of the truck traffic (light and heavy) have axle weights greater than 12,000 lbs. Although the total number of trucks is lower than that measured on Rt. 34N, the average axle weights are heavier.

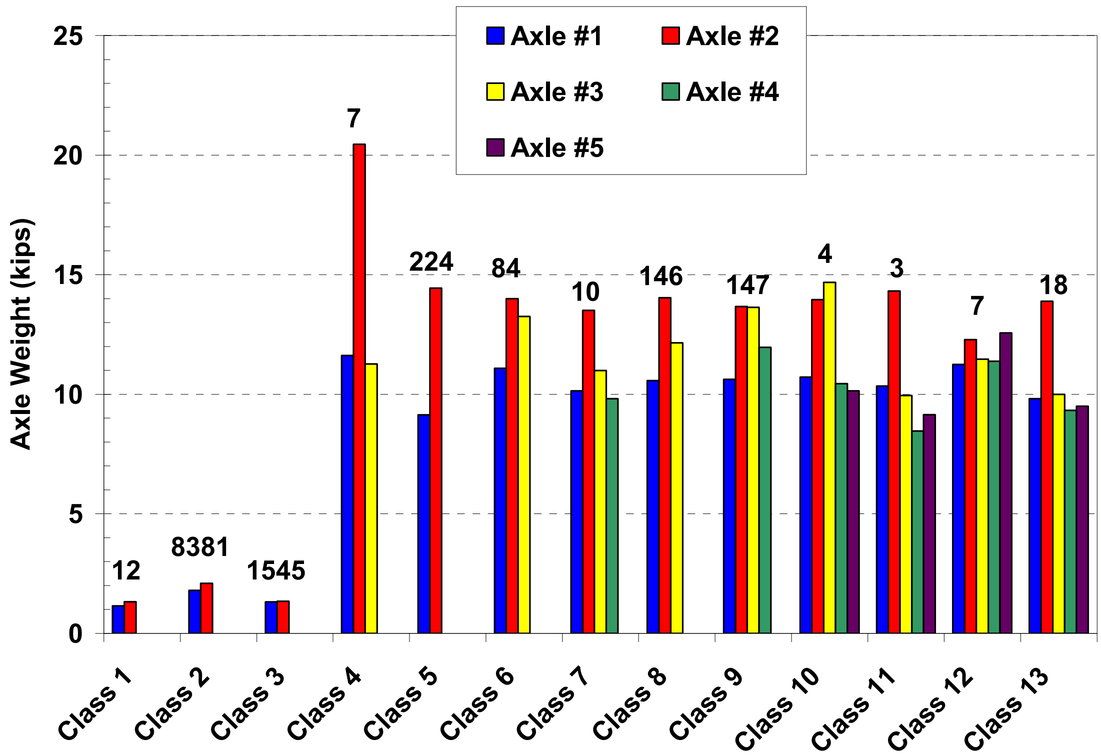



Figure 5.18 – Axle Load Spectra for Rt 202S

*5.2.1.4 Summary of Traffic Conditions on Rt. 202S*

Detailed hourly distribution, vehicle and axle distribution were measured, resulting in the following average traffic loading conditions:

- Average Daily Traffic (ADT) = 10,177 vehicles
- Automobile Axle Loads: Ave. = 7.0 kN (1.6 kips); Max. = 9.8 kN (2.2 kips)
- Light Truck Axle Loads: Ave. = 59.5 kN (13.4 kips); Max. = 112.9 kN (25.4 kips)
- Heavy Truck Axle Loads: Ave. = 51.5 kN (11.6 kips); Max. = 98.1 kN (24.4 kips)

### 5.2.2 Visual Distress Survey

A Visual Distress Survey (VDS) was conducted to assess the existing pavement distresses on Rt 202S in New Jersey. The VDS was conducted in accordance to the *Distress Identification Manual for the Long-Term Pavement Performance Project* (2003). Table 5.6 summarizes the VSD results. It should be noted that due to time constraints on the project site, the VDS was only conducted on Lane #2 (outside lane) of Rt. 202S.

Table 5.6 – Visual Distress Survey for Rt 202S in New Jersey

| Number of Transverse Cracks | Lane #2 |
|---|---|
| Low Severity | 1 |
| Moderate Severity | 26 |
| High Severity | 41 |
| Spalled or Patched Transverse Joints | 99 |

### 5.2.3 Falling Weight Deflectometer Testing

Falling Weight Deflectometer (FWD) testing was conducted before and after the transverse joints at three load magnitudes; 28.91, 57.82, and 71.17 kN (6.5, 13, and 16 kips). The FWD testing was used to assess the Load Transfer Efficiency (LTE) and the vertical deflections at the joint due to the applied loads.

The measured vertical deflections at the PCC joint/crack, normalized to 9,000 lbs for comparison purposes, are shown in Figure 5.19. Since Rt 202S was divided into four (4) different test sections, each test section is noted in Figure 5.19.

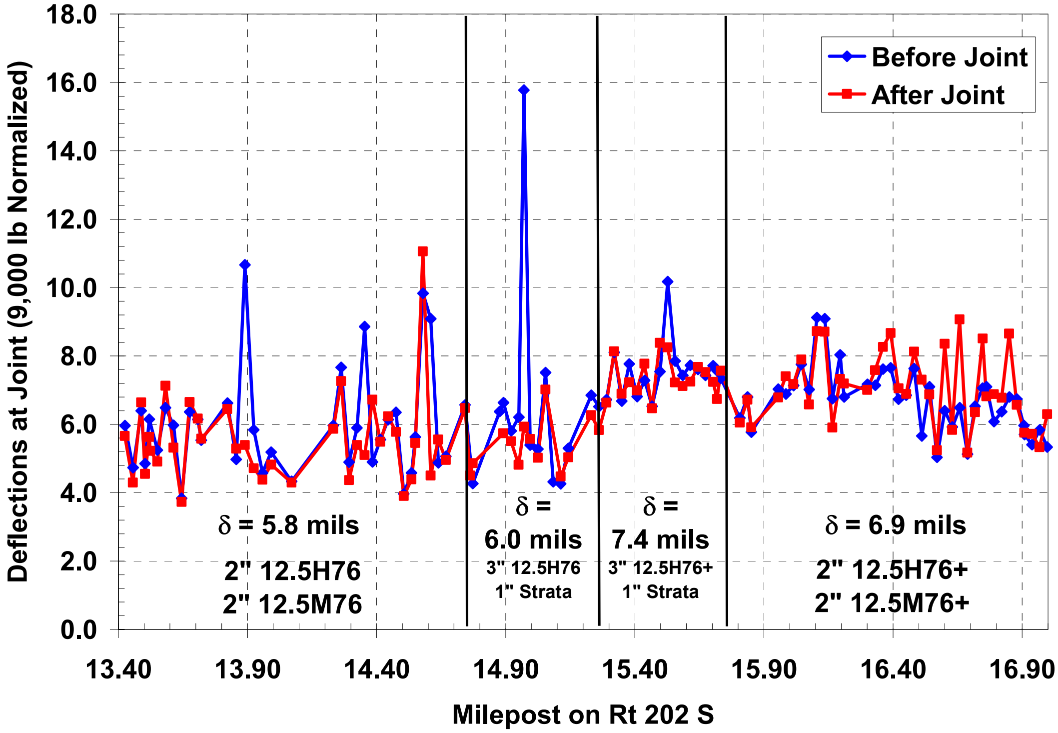



Figure 5.19 – Vertical Deflection at the PCC Joint/Crack Normalized to 9,000 lbs

The vertical deflections for the different test sections are summarized below:

- Test Section #1 (2" 12H76 over 2" 12M76) = 5.8 mils
- Test Section #2 (3" of 12H76 over 1" RCRI) = 6.0 mils
- Test Section #3 (3" of 12H76+ over 1" RCRI) = 7.4 mils
- Test Section #4 (2" 12H76+ over 12M76+) = 6.9 mils

The measured Load Transfer Efficiency (LTE) at the PCC joint/crack is shown in Figure 5.20, also with the different test sections noted. The LTE for the different test sections are summarized below:

- Test Section #1 (2" 12H76 over 2" 12M76) = 74.2%
- Test Section #2 (3" of 12H76 over 1" RCRI) = 75.7%
- Test Section #3 (3" of 12H76+ over 1" RCRI) = 79.9%

- Test Section #4 (2" 12H76+ over 12M76+) = 69.8%

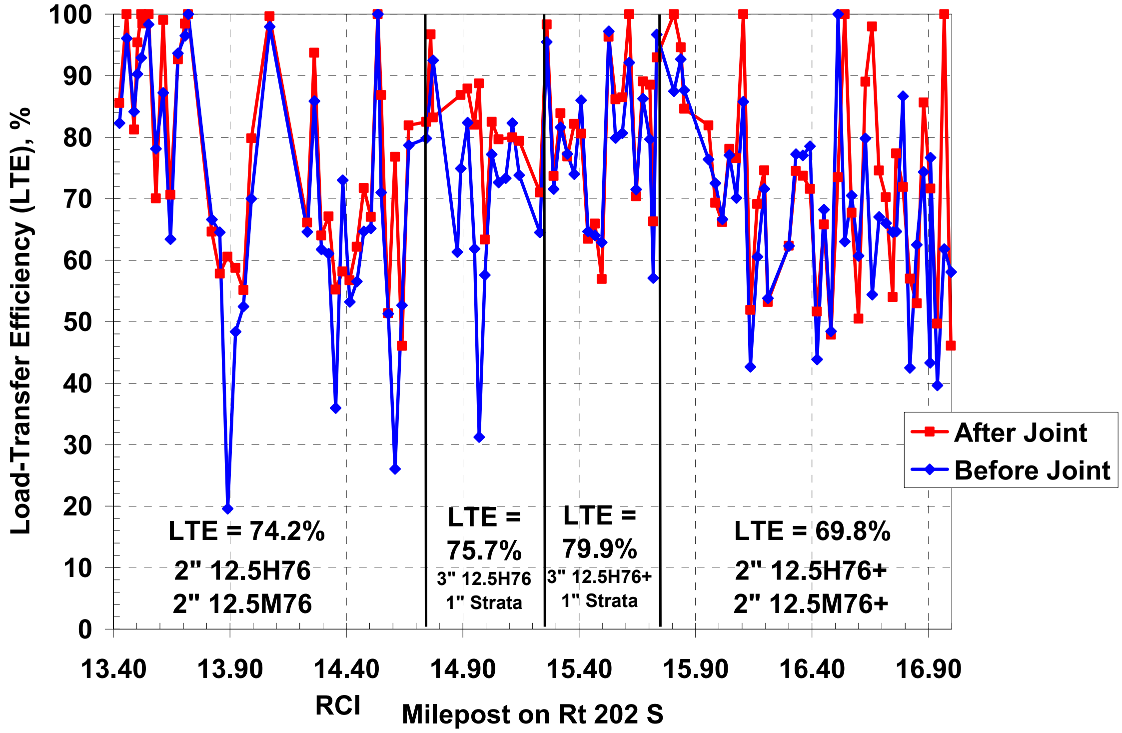



Figure 5.20 – Load Transfer Efficiency (LTE) of the Different Test Sections on Rt. 202S

It should be noted that the FWD testing was conducted during April 2007 with the average air temperature and pavement surface temperatures of 57 and 62$^{o}$F, respectively. Therefore, it was assumed that minimal to no PCC joint/crack friction had occurred that would have increased the LTE and/or reduced the vertical deflection.

### 5.2.4 PCC Vertical Joint Deflection Spectra

When conducting the FWD testing at different load magnitudes, the relationship between the vertical deflection at the joint (measured immediately under the load) and the applied load can be represented by a linear regression, thereby, allowing for the direct substitution of any known applied axle load to determine the resultant deflection at the

PCC joint/crack. The relationship for each test section, along with the average, is shown as Figure 5.21.

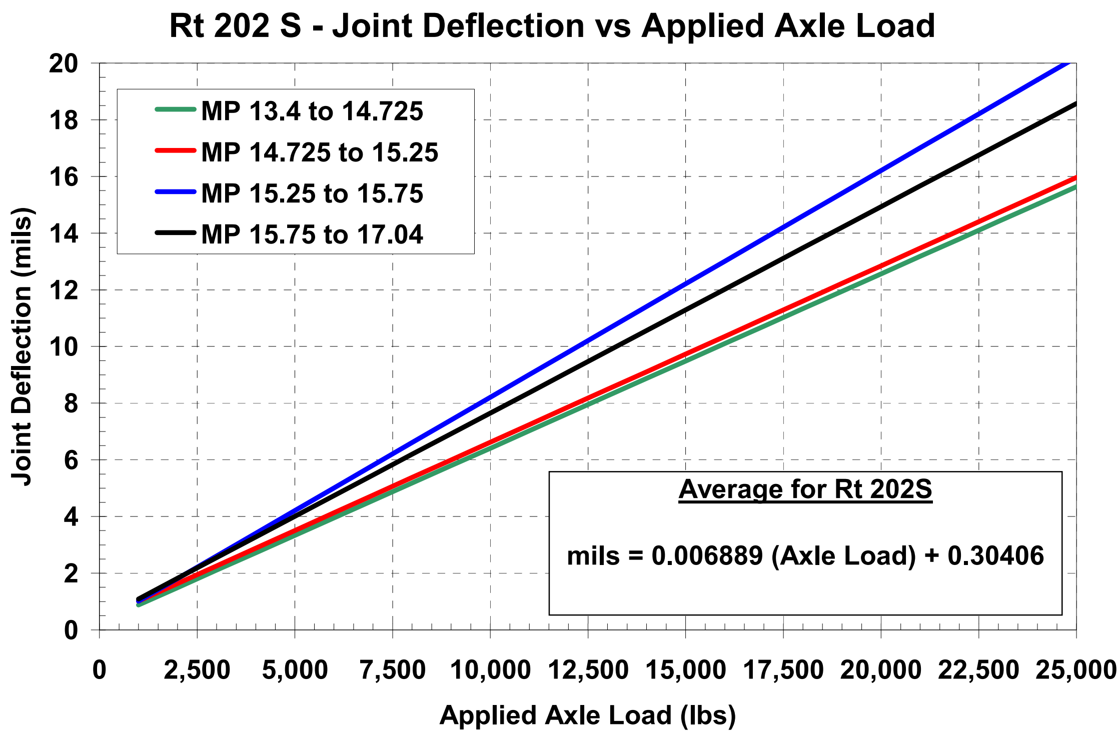



Figure 5.21 – PCC Joint Deflection vs Applied Axle Load for Rt 202S in New Jersey

As shown prior for the Rt 34N test section, the relationship between the FWD Applied Load and the Vertical Deflection at the PCC Joint/Crack can be assumed to represent the PCC vertical joint/crack deflection from axle loading during daily trafficking. This was defined earlier as the PCC Vertical Joint Deflection Spectra. The average PCC Vertical Joint Deflection Spectra generated for Test Section #2 (Rt 202S in New Jersey) is shown in Figure 5.22.

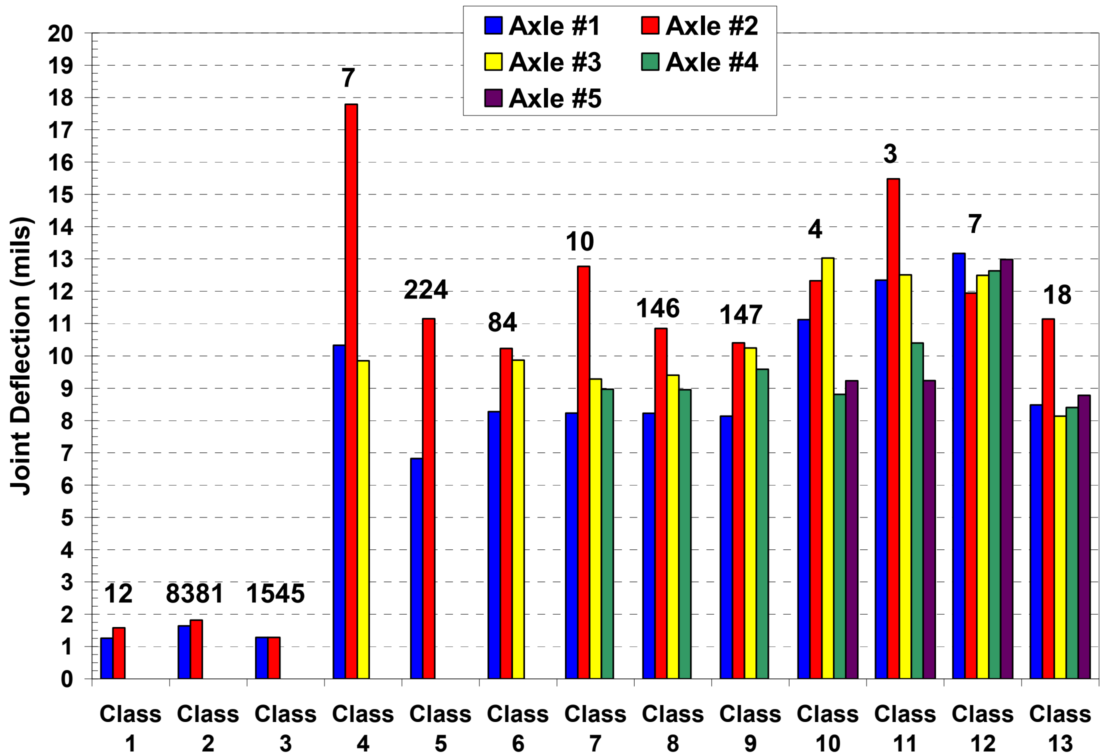



Figure 5.22 – PCC Vertical Joint Deflection Spectra for Test Section #2 (Rt 202S)

### 5.2.5 Coring – Concrete Test Results

Immediately after the FWD testing, full-depth pavement cores were taken for laboratory analysis (Figure 5.23). Of particular interest is the Coefficient of Thermal Expansion (CTE) of the PCC pavement. The CTE parameter, along with the effective PCC slab length, maximum 24 hour temperature difference, and PCC/Base friction factor, provide an estimate of the expected horizontal movement at the PCC joint/crack (shown earlier as Equation 5.1).

Figure 5.23 – Cross Sectional View of PCC Core Taken from Rt 202S in New Jersey

Five PCC cores were extracted from the pavement on Rt 202S in New Jersey for Coefficient of Thermal Expansion Testing (AASHTO TP60). The average CTE of the extracted cores from Rt 202S was determined to be 11.77E-06 cm/cm/$^{o}$C, with a standard deviation of 0.28E-06 cm/cm/$^{o}$C. Since temperature probes were not used on site to measure the temperature profile of the pavement, the Enhanced Integrated Climatic Model (EICM) was used to predict temperature profiles using interpolated weather data from neighboring weather stations. The average 24-hour temperature difference determined was 5.7$^{o}$F (3.2$^{o}$C) at the surface of PCC pavement (or at the bottom of the proposed HMA overlay). Figure 5.24 shows an example of the thermal prediction output. The procedure described earlier under Section 5.1.5 was used to conduct the calculations to estimate the average horizontal movement that would be expected in a 24-hour period.

The estimated, expected horizontal movement at the PCC joint/crack was 0.026 inches (0.067 cm).

### 5.2.6 Dynamic Cone Penetrometer (DCP) Testing

After the cores were extracted from the pavement, Dynamic Cone Penetrometer (DCP) testing was conducted through the core hole into the unbound material. The DCP testing was used to evaluate the general bearing capacity (California Bearing Ratio, CBR) of the underlying unbound materials, as well as the general thickness of each of the unbound aggregate/subgrade layers. Table 5.7 shows the corresponding CBR values of the underlying unbound material.

Table 5.7 – Dynamic Cone Penetrometer (DCP) Testing of Rt 202S in New Jersey

| Core No. | Lane | Milepost | Depth into Granular Layer (in.) and CBR Value | | | | |
|---|---|---|---|---|---|---|---|
| 2 | Lane 2 | 16.50 | 0.0 - 7.9 (39%) | 7.9 - 11.8 (18%) | 11.8 - 16.2 (44%) | 16.2 - 21.8 (12%) | 21.8 - 26.2 (48%) |
| 4 | Lane 2 | 15.42 | 0.0 - 9.3 (25%) | 9.3 - 13.0 (10%) | 13.0 - 19.1 (21%) | 19.1 - 23.8 (38%) | 23.8 - 26.3 (87%) |
| 6 | Lane 2 | 14.50 | 0.0 - 5.1 (34%) | 5.1 - 22.0 (21%) | 22.0 - 26.2 (53%) | | |
| 8 | Lane 2 | 13.41 | 0.0 - 3.4 (73%) | 3.4 - 11.9 (120%) | | | |

In general, the interpretation of the DCP test results indicates that there exists variable supporting material underneath the PCC slabs (i.e. – some locations good while some average to poor). The average results for the two identified unbound layers are shown below;

- Unbound Layer #1:
  - Thickness: Approximately 12.4 inches
  - CBR Value: Approximately 40%
- Unbound Layer #2:
  - Thickness: Approximately 13.9 inches
  - CBR Value: Approximately 43.3%

### 5.2.7 Pavement Performance on Rt 202S in New Jersey

Visual Distress Surveys (VSD) were conducted on three occasions during the research study; 1) March 2008, 2) September 2008, and 3) March 2009. During each one of the visits, zero cracking was observed for all of the sections. Besides poor construction, poor backfilling of extracted cores, and poor sawing and sealing practices, Figures 5.25, 5.26, and 5.27, the composite pavement section is performing well.

Figure 5.25 – Rubber Cord Paved into Pavement Surface of Rt 202S in New Jersey

Figure 5.26 – Poor Backfilling of Extracted Core from Rt 202S in New Jersey

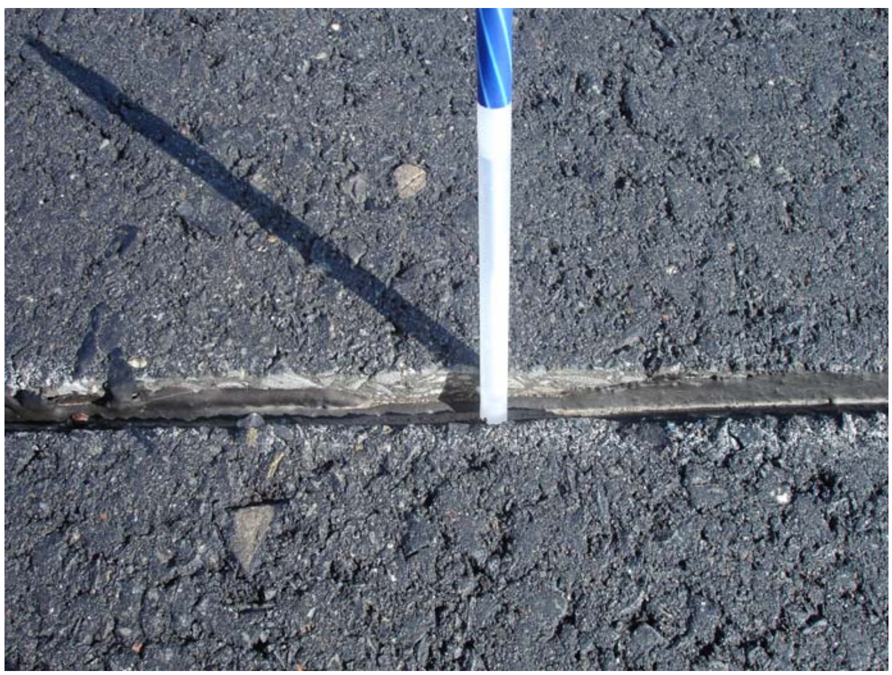


Figure 5.27 – Poor Sealing of Sawed Joints from Rt 202S in New Jersey

## 5.3 Interstate 495 – Massachusetts

The project began in Franklin, MA approximately 1.5 miles south of the Route I-495/King Street interchange, and proceeded 9.77 miles south along Route I-495, ending in Mansfield, MA, approximately 1100 feet north of the Route I-495/Route I-95 interchange. A Windshield Survey consisting of a visual pavement evaluation of the existing PCC surface was conducted from the right travel lane at approximately 30 mph, while occasionally stopping in the break down lane to observe the existing pavement condition in more detail. MassHighway's Automatic Road Analyzer (ARAN) was used to record the IRI (ride quality), transverse profile (ruts), and pavement surface condition. The ARAN's video logging system was also used to record a permanent video tape of the PCC surface condition and roadway right-a-way. Windshield Surveys conducted by MassHighway and the contractor showed minimal faulting and midslab, transverse cracking. If such problems had occurred in the past, MassHighway had implemented corrective actions for these issues. Therefore, it was concluded that this project would be a good candidate for an asphalt overlay containing a Reflective Crack Relief Interlayer (RCRI). Figure 5.28a and 5.28b shows pictures from the site location prior to the HMA overlay.

(a)

(b)

Figure 5.28 – PCC Pavement on Interstate 495 Prior to HMA Overlay a) Typical PCC Joint Condition; b) HMA Patched Joint

The following composite pavement design was selected for the PCC overlay:

- Leveling Course: 2" of dense-graded HMA, 30% RAP, PG52-33 + SBR 3% Latex
- Reflective Crack Relief Interlayer (MassHighway Stress-Absorbing Membrane, SAMI): 1" of SAMI
- Intermediate Course: 2" of 19mm dense-graded HMA, 30% RAP, PG52-33 + 3% SBR Latex
- Surface Course: 1.5" of 9.5 mm Asphalt-Rubber Gap-Graded Mixture

The Leveling and Intermediate Course mixtures were the identical mixture; 19 mm nominal aggregate size, produced from the same job mix formula (5.2% AC), aggregates, asphalt binder, and RAP content. The RCRI mixture had 100% passing the 9.5 mm sieve and contained 8.3% asphalt binder. The asphalt binder of the RCRI is highly polymerized to enhance the flexural performance of the mixture. As part of the mixture design process, the RCRI mixture must achieve a minimum flexural fatigue life, while maintaining rutting resistance, commonly measured in either the Asphalt Pavement Analyzer (AASHTO TP63) or Hveem Stability Test. The design volumetric and gradation properties of the mixes are shown in Table 5.8.

Construction of the overlay began in the summer of 2007 and continued until late October 2007, at which time approximately three (3) miles of the southern end of the project had only received the 2" Leveling Course mixture, and was scheduled to receive RCRI and Intermediate Course in Spring 2008. Neither section, 2" Overlay (Leveling Course only) nor the 5" Overlay with RCRI, had yet to receive the Surface Course at the time of this study.

Table 5.8 – Volumetric and Gradation Properties of HMA Mixture from I-495 in Massachusetts

| Mixture Design Property | Mixture Type | |
|---|---|---|
| | 19mm DGA | RCRI |
| Binder Content (%) | 5.2% | 8.3% |
| VMA (%) | | |
| $G_{mm}$ (g/cm$^3$) | | |
| $G_{sb}$ (g/cm$^3$) | | |
| Percent Passing | | |
| 25mm | 100 | 100 |
| 19mm | 95.2 | 100 |
| 12.5mm | 74.8 | 100 |
| 9.5mm | 62.7 | 100 |
| 4.75mm | 50.6 | 91.5 |
| 2.36mm | 40.5 | 75.0 |
| 1.18mm | 29.8 | 53.1 |
| 0.6mm | 20.9 | 36.8 |
| 0.3mm | 13.3 | 24.3 |
| 0.075mm | 3.7 | 7.6 |

### 5.3.1 Traffic Conditions on Interstate 495 in Massachusetts

Unlike the previous test sections evaluated in the research study, the Massachusetts Department of Transportation (MassHighway) did not have access to weigh-in-motion and automated vehicle classification systems to provide detailed traffic and axle load information. Therefore, MassHighway provided traffic information in the form of Equivalent Single Axle Load (ESAL's). Traffic values collected by MassHighway at the overlay location were as follows:

- Left Lane = 763.3 ESAL's per day
- Middle Lane = 1204.7 ESAL's per day
- Right Lane = 747 ESAL's per day

where an ESAL is an Equivalent Single Axle Load of 18,000 lbs.

### 5.3.2 Falling Weight Deflectometer Testing

Falling Weight Deflectometer (FWD) testing was conducted on the two different sections, 2” HMA Overlay Section and the 5” HMA Overlay Section, to evaluate the vertical deflections at the PCC joint/crack and the load transfer efficiency (LTE) of the PCC joint/crack. Falling Weight Deflectometer (FWD) testing was conducted before and after the transverse joints at three load magnitudes; 28.9, 42.3, and 80.1 kN (6.5, 9.5, and 18 kips). The test results were then normalized to 9,000 lbs for comparative purposes and are shown in Figure 5.29. The results clearly show a higher level of vertical deflection occurring in the 2-inch Overlay Section than in the 5-inch Overlay Section. On average, the 2-inch section witnessed a vertical deflection at the PCC joint/crack of 12.2 mils when normalized to 9,000 lbs, and on average the 5-inch Overlay Section resulted in a 8.75 mils deflection when normalized to 9,000 lbs. One would assume that as the vertical deflection increases, the potential for reflective cracking would also increase.

The Load Transfer Efficiency (LTE) was measured to assess the integrity of the PCC joint/crack of the PCC pavement. On average, the 2-inch Overlay Section had a slightly higher LTE (79.3%) than the 5-inch Overlay Section (75%). However, the 2-inch Overlay Section had a higher degree of variability (standard deviation of 13.1%) than the 5-inch Overlay Section (5.5%). The measured LTE results are shown in Figure 5.30.

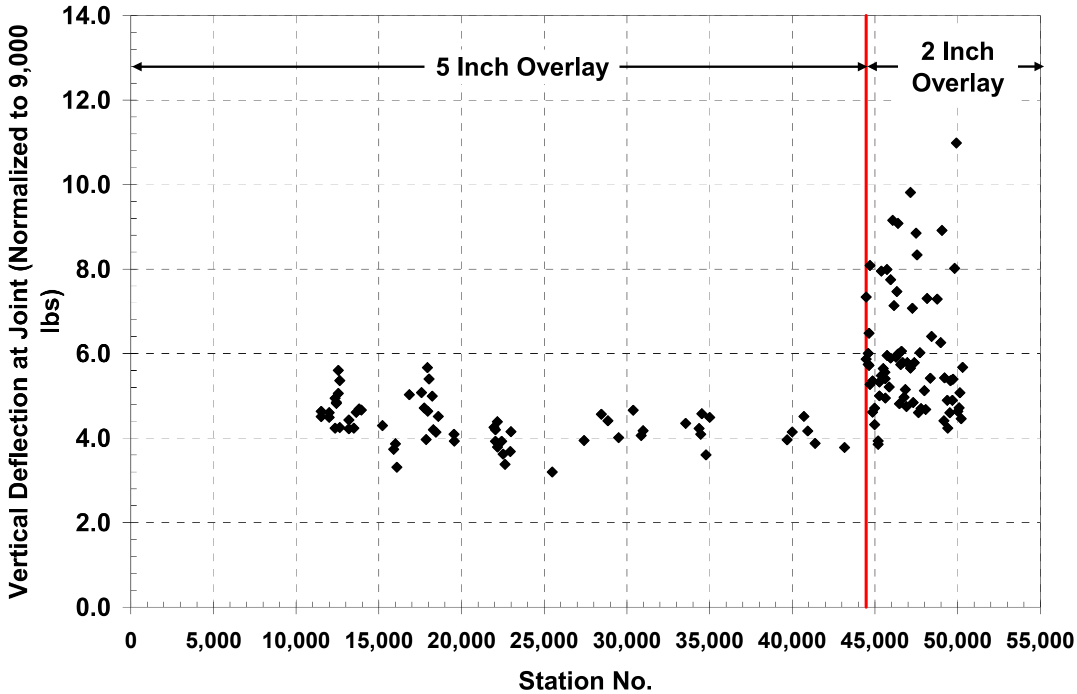



Figure 5.29 – Vertical Deflection at PCC Joint/Crack Normalized to 9,000 lbs

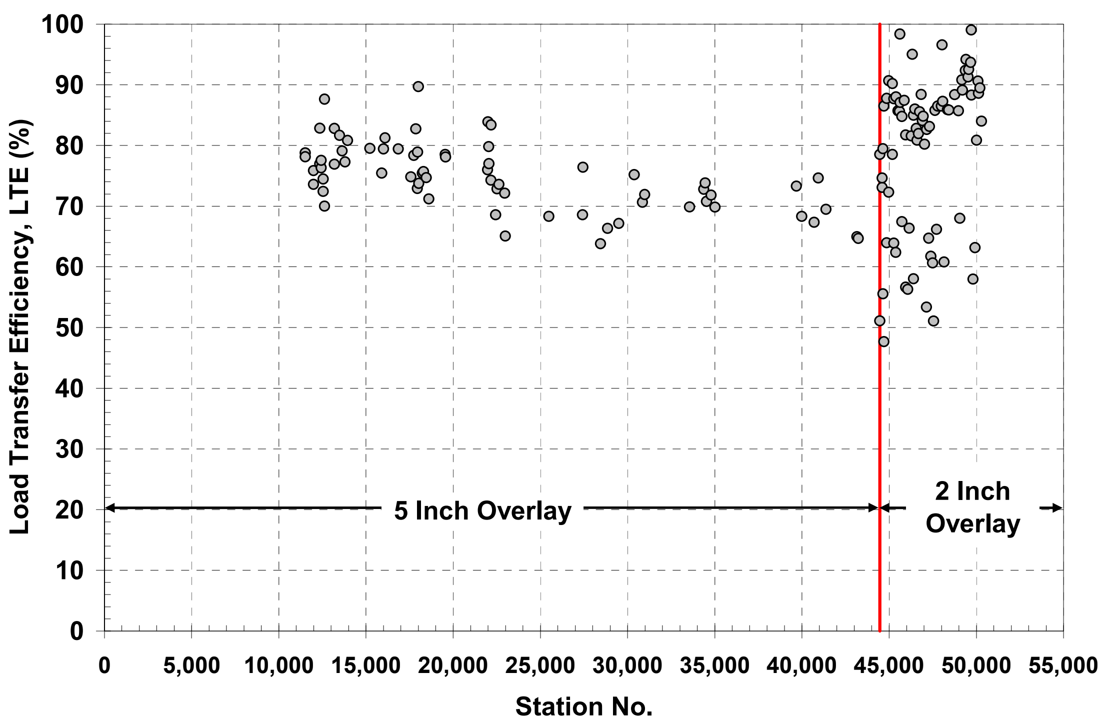



Figure 5.30 – Load Transfer Efficiency (LTE) at PCC Joint/Crack

As shown earlier, the FWD test results, used in combination with measured traffic information, can provide a pavement designer with valuable information regarding the resulting PCC joint/crack movement due to the applied traffic loading. Figure 5.31 shows the relationship developed for both the 2-inch and 5-inch Overlay Sections. Figure 5.31 clearly shows that the 2-inch Overlay Section is more susceptible to traffic loading than the 5-inch section. On average, the vertical deflections in the 2-inch Overlay Section are approximately 27% higher than the 5-inch Overlay Section.

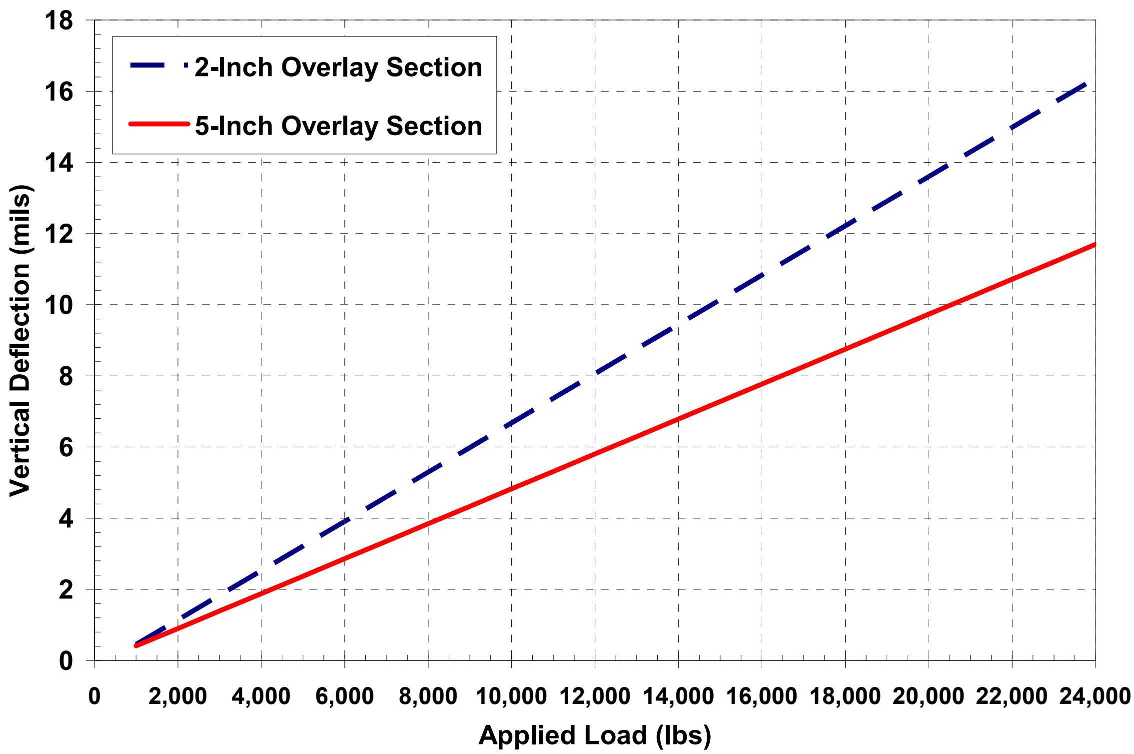



Figure 5.31 – Applied Load vs PCC Vertical Deflection Relationship for Interstate 495

However, unlike the test sections in New Jersey, axle load spectra were not provided, simply ESAL's. Therefore, using the trendlines shown in Figure 5.31, the application of an ESAL would result in 8.75 mils of vertical deflection in the 5-Inch Overlay section and 12.21 mils of vertical deflection in the 2-Inch Overlay section.

### 5.3.3 Coring – Concrete Test Results

Immediately after the FWD testing, full-depth pavement cores were taken for laboratory analysis (Figure 5.32). Of particular interest is the Coefficient of Thermal Expansion (CTE) of the PCC pavement. The CTE parameter, along with the effective PCC slab length, maximum 24 hour temperature difference, and PCC/Base friction factor, provide an estimate of the expected horizontal movement at the PCC joint/crack (shown earlier as Equation 5.1).

Figure 5.32 – Photo of PCC Core Taken from Interstate 495 in Massachusetts

Four PCC cores were taken from Interstate 495 and tested for the Coefficient of Thermal Expansion (CTE). The PCC cores were evaluated using the CTE test procedure outlined in AASHTO TP60-06 and determined to have a CTE of 10.95E-6 mm/mm/$^{o}$C, with a standard deviation of 0.25E-6 mm/mm/$^{o}$C. Since the temperature of the underlying concrete slabs was not recorded, the Enhanced Integrated Climatic Model, EICM (Larson and Dempsey, 2006) was used to estimate temperature profiles in the pavement. Interpolated weather data from neighboring weather stations were used to generate 5 years (1998 and 2003) worth of historical climate information for the test site. The average 24-hour temperature difference determined was 5$^{o}$F for the 5-Inch Overlay section and 7$^{o}$F for the 2-Inch Overlay section at the surface of the concrete pavement. Figures 5.33 and 5.34 shows the predicted temperature distributions, generated by the Enhanced Integrated Climactic Model (EICM), for the month of December. For calculation of PCC joint/crack horizontal deformation, the PCC/Base Friction Factor (β) was estimated from comparable PCC pavements overlying aggregate base course material at the Long Term Pavement Project (LTPP) concrete pavement test sites (Khazanovich and Gotliff, 2002). An average β assumed for the analysis was 0.76. Therefore, using Equation (5.1) and the appropriate input parameters, it was concluded that the maximum horizontal movement that would be expected in a 24-hour period is 0.53 mm (0.021 inches) for the 5-Inch Overlay section and 0.74 mm (0.029 inches) for the 2-Inch Overlay section.

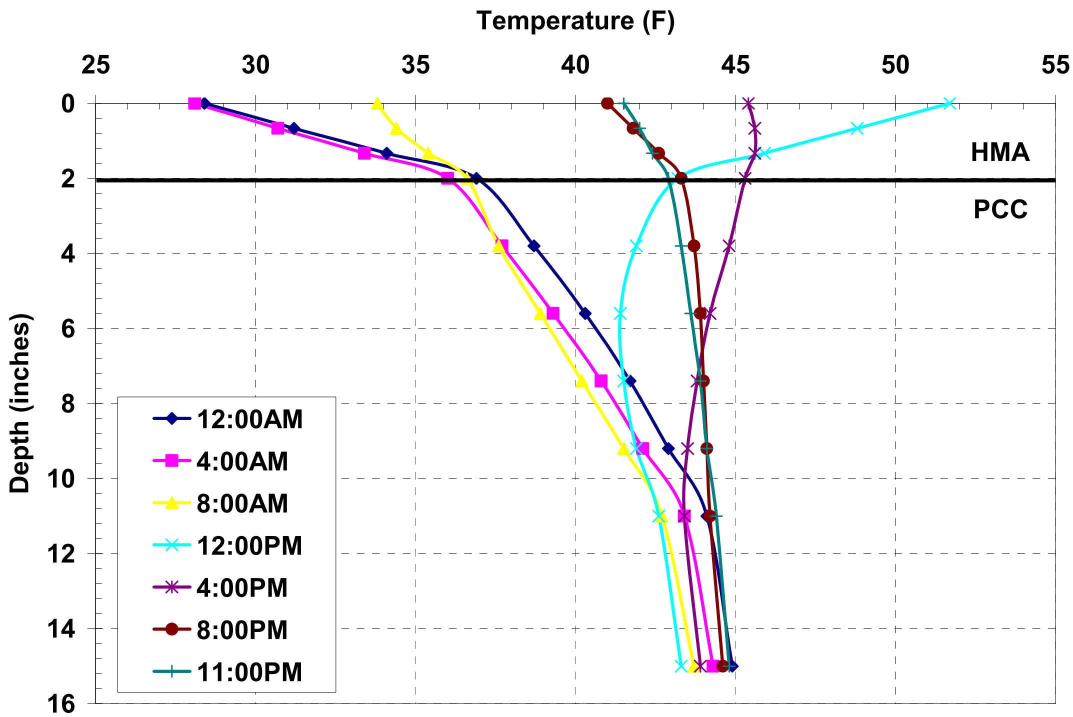



Figure 5.33 – Predicted Temperature Profiles for 2-Inch Overlay on I-495 MA

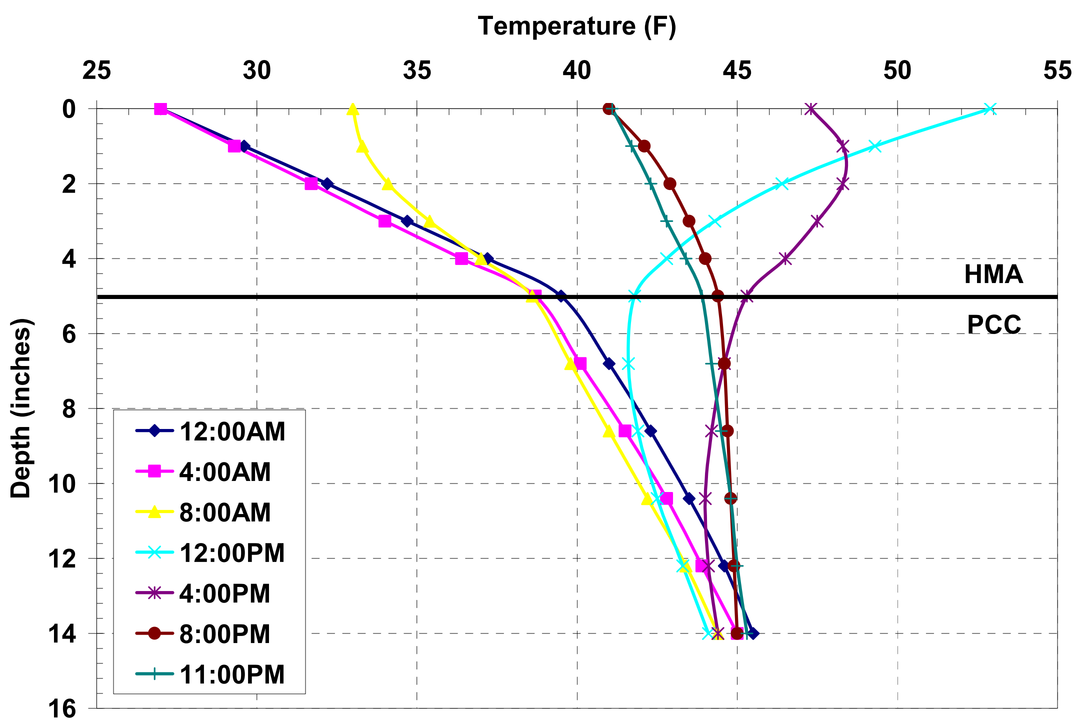



Figure 5.34 – Predicted Temperature Profiles for 5-Inch Overlay on I-495 MA

5.3.4 Pavement Performance on Interstate I495 in Massachusetts

Within the first two months after paving had stopped in late October 2007, cracking was observed in the 2" Leveling Course Section, while limited cracking had been observed in the 5" Overlay with RCRI by early Spring 2008. An official visual distress survey (crack count) was conducted by MassHighway approximately 8 months after the construction had originally stopped and showed that:

- 2" Leveling Course Section (Average: 77.6% of transverse joints cracked)
  - Left Lane = 99% of transverse joints cracked
  - Middle Lane = 56.3% of transverse joints cracked
  - Right Lane = 77% of transverse joints cracked
- 5" Overlay with RCRI (Average: 8.2% of transverse joints cracked)
  - Left Lane = 14.2% of transverse joints cracked
  - Middle Lane = 7.6% of transverse joints cracked
  - Right Lane = 2.8% of transverse joints cracked

Figure 5.35a and b shows pictures of the cracking observed from the surface (Figure 5.35a) and from extracted cores (Figure 5.35b). Cores taken in the 2" Leveling Course section confirmed that the crack had propagated through the entire 2" thick HMA layer. What is interesting to note in Figure 5.35b is that cracking had occurred above and below the RCRI layer, while the RCRI layer remained intact.

(a)

(b)

Figure 5.35 a) Reflective Cracking on the Pavement Surface of I495; b) Cracking Above and Below the RCRI Layer

### 5.4 Interstate 476 Pennsylvania - Pennsylvania Turnpike

The rehabilitation project consisted of milling off 3.5 inches of aged and cracked hot mix asphalt from an aging PCC pavement on Interstate 476 in Pennsylvania. The project limits for Interstate 476 only include the southbound section from milepost 95 to 105. Photos of the PCC pavement after milling are shown as Figures 5.36 and 5.37. The photos clearly show the poor condition of the existing PCC pavement, as well as the previous HMA overlay that had been placed over it.

Figure 5.36 – Milled and Unmilled Surface of Interstate 476 in Pennsylvania

Figure 5.37 – Exposed Surface of Underlying PCC Pavement on Interstate 476 in Pennsylvania

The pavement rehabilitation called for 2 to 3 inches of a leveling course (12.5mm, 100 gyration design), 1 inch of a reflective crack relief interlayer, and 2 inches of a 12.5mm dense-graded mix as the surface course (75 gyration and 100 gyration design mixes). Gradation and volumetric properties of the hot mix asphalt designs for the Interstate 476 is shown in Table 5.9. Both 12.5mm dense-graded mixtures had identical aggregate materials and gradations and only differed in asphalt content, and therefore, VMA.

Table 5.9 – Gradation and Volumetric Properties for Hot Mix Asphalt Mixtures on Interstate 476 in Pennsylvania

| Mixture Design Property | Mixture Type | | |
|---|---|---|---|
| | 12.5mm DGA, 100 Gyr. | 12.5mm DGA, 75 Gyr. | RCRI |
| Binder Content (%) | 5.2% | 5.5% | 8.0% |
| VMA (%) | 15.0% | 15.6% | 17.8% |
| $G_{mm}$ (g/cm$^3$) | 2.464 | 2.453 | 2.331 |
| $G_{sb}$ (g/cm$^3$) | 2.633 | 2.633 | 2.585 |
| Percent Passing | | | |
| 25mm | 100 | 100 | 100 |
| 19mm | 100 | 100 | 100 |
| 12.5mm | 95 | 95 | 100 |
| 9.5mm | 87 | 87 | 100 |
| 4.75mm | 57 | 57 | 91 |
| 2.36mm | 39 | 39 | 71 |
| 1.18mm | 26 | 26 | 52 |
| 0.6mm | 18 | 18 | 35 |
| 0.3mm | 12 | 12 | 21 |
| 0.075mm | 4.5 | 4.5 | 7.9 |

### 5.4.1 Traffic Conditions on Interstate 476 in Pennsylvania

Similar to the Massachusetts location, the Pennsylvania Turnpike did not have access to weigh-in-motion and automated vehicle classification systems to provide detailed traffic and axle load information. Therefore, the Pennsylvania Turnpike provided the average following traffic information:

- % Trucks = 23%
- Annual Growth Rate = 2.7%
- Annual Daily Traffic = 16,517
- Calculated ESAL's = 13,000,000

### 5.4.2 Falling Weight Deflectometer Testing

Falling Weight Deflectometer (FWD) testing was conducted on Interstate 476 in Pennsylvania to evaluate the vertical deflection and load transfer efficiency (LTE) at the PCC joint/crack area. Unfortunately, due to traffic control issues and time constraints, limited FWD testing was conducted. Two different sections were evaluated; 1) Rigid section where FWD testing was conducted on the bare PCC surface, and 2) Composite section where FWD testing was conducted at the reflective crack area above the PCC joint/crack.

### 5.4.3 Coring – Concrete Test Results

Immediately after the FWD testing, full-depth PCC pavement cores were taken for laboratory analysis. Of particular interest is the Coefficient of Thermal Expansion (CTE) of the PCC pavement. The CTE parameter, along with the effective PCC slab length, maximum 24 hour temperature difference, and PCC/Base friction factor, provide an estimate of the expected horizontal movement at the PCC joint/crack (shown earlier as Equation 5.1).

Only one PCC core was taken from Interstate 476 and tested for the Coefficient of Thermal Expansion (CTE). The PCC core was evaluated using the CTE test procedure outlined in AASHTO TP60-06 and determined to have a CTE of 10.95E-6 mm/mm/$^{o}$C, with a standard deviation of 0.25E-6 mm/mm/$^{o}$C. Since the temperature of the underlying concrete slabs was not recorded, the Enhanced Integrated Climatic Model, EICM (Larson and Dempsey, 2006) was used to estimate temperature profiles in the pavement. Interpolated weather data from neighboring weather stations were used to generate 5 years (1998 and 2003) worth of historical climate information for the test site. The average 24-hour temperature difference determined was 4.7°F for the I476 pavement section. For calculation of PCC joint/crack horizontal deformation, the PCC/Base Friction Factor (β) was estimated from comparable PCC pavements overlying aggregate base course material at the Long Term Pavement Project (LTPP) concrete pavement test sites (Khazanovich and Gotliff, 2002). An average β assumed for the analysis was 0.76. Therefore, using Equation (5.1) and the appropriate input parameters, it was concluded that the maximum horizontal movement that would be expected in a 24-hour period is 0.013 inches (0.33 mm).

# CHAPTER 6 – LABORATORY EVALUATION AND TESTING

As discussed during Chapter 2, Literature Review, the Flexural Beam Fatigue (AASHTO T321) and the Overlay Tester (TxDOT Tex-248-F) were found to represent the most representative laboratory testing devices to simulate field movements for the evaluation of HMA mixtures. The Flexural Beam Fatigue represents the vertical PCC joint/crack movement associated with traffic loading, while the Overlay Tester represents the horizontal expansion and contraction movements associated with climatic loading (non-traffic loading related distress).

During construction of the pavement sections evaluated in this study, loose hot mix asphalt was sampled from the asphalt plants and brought back to the laboratory for evaluation. In particular, the Flexural Beam Fatigue and Overlay Tester were used to assess the cracking resistance of the different asphalt mixtures. The Flexural Beam Fatigue test is utilized to determine the cracking resistance due to the load associated vertical deflections. HMA mixtures located at the bottom, middle, and surface of the HMA overlay cross section would need to be evaluated as vertical deflections occur throughout the pavement cross-section. Meanwhile, the Overlay Tester was only used to evaluate mixtures that were placed immediately over the PCC or at as first paving lift on a composite pavement (i.e. – Leveling Course). The Dynamic Modulus test (AASHTO TP62-07) was also used to evaluate the stiffness properties of the different asphalt mixtures. In general, asphalt mixtures that are found to have lower modulus or stiffness values at lower and intermediate temperatures are typically less susceptible to undergo cracking distress. Meanwhile, asphalt mixtures with higher modulus at higher temperatures are less susceptible to permanent deformation.

## 6.1 – Flexural Beam Fatigue Test

Fatigue testing was conducted using the Rutgers Asphalt/Pavement Laboratory's (RAPL) Flexural Beam Fatigue device manufactured by IPC (Figure 6.1). The device is capable of applying haversine and sinusoidal strain- and stress-controlled waveforms. The device is also capable of applying user defined strain-controlled waveforms (i.e. – double-hump, triple-hump, triangular, etc.). The unit is contained in an environmental chamber capable of controlling temperatures from 0 to 60$^{o}$C.

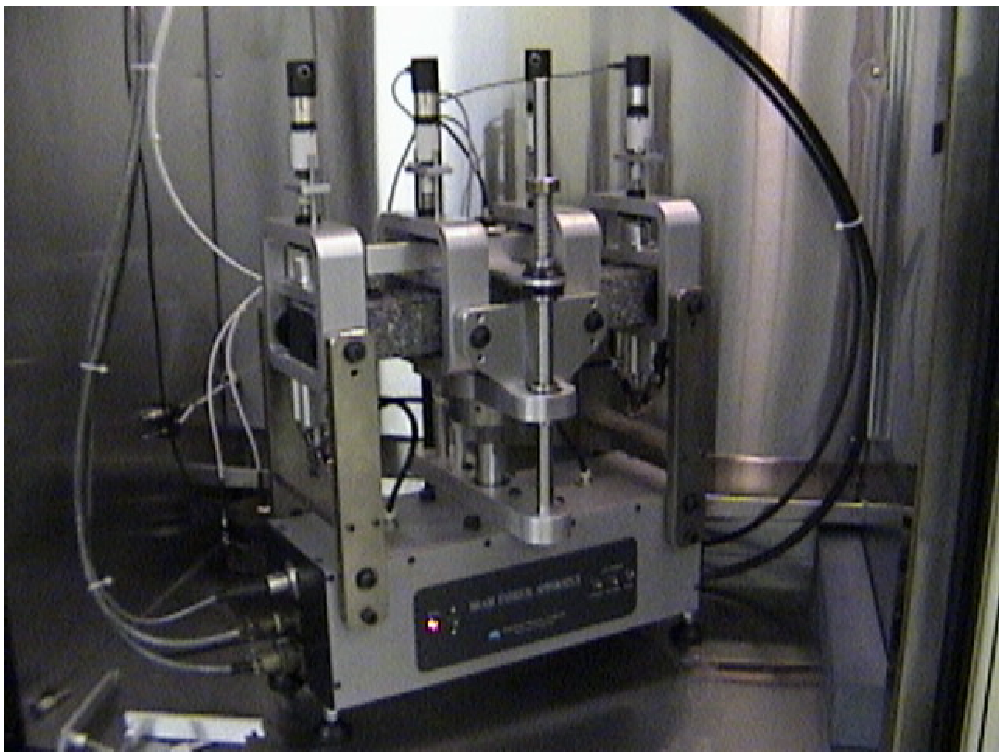


Figure 6.1 – Flexural Beam Fatigue Device for AASHTO T321

Throughout the test, the flexural stiffness of the sample was calculated and recorded. The stiffness of the beam was plotted against the load cycles and the resulting data was fitted to an exponential function as recommended by AASHTO T321 (Equation 6.1):

$$E = E_i e^{bN} \tag{6.1}$$

where,

E (also known as S) = flexural stiffness after the n load cycles;
$E_i$ (also known as $S_o$) = initial flexural stiffness;
e = natural algorithm to the base e
b = constant from regression analysis
N = number of load cycles

Equation (6.1) was modified to determine the number of loading cycles to achieve 50% of the initial flexural stiffness ($N_{f,50\%}$). This was conducted for four different applied tensile strain levels to provide a regression equation in the form of Equation (6.2) and Equation (6.3).

$$N_f = k_1 \varepsilon_t^{k_2} \tag{6.2}$$

$$N_f = k_1 \left(\frac{1}{\varepsilon_t}\right)^{k_2} \left(\frac{1}{E}\right)^{k_3} \tag{6.3}$$

where,

$N_f$ = number of loading repetitions until fatigue failure (50% of the initial stiffness)
$k_1$, $k_2$, $k_3$ = regression coefficients depending on material type and test conditions
$\varepsilon_t$ = tensile strain
E = initial flexural stiffness

The applied tensile strain levels used for this study varied and depended on the asphalt mixture type. In general, typical dense-graded mixtures were tested between 200 and 1,000 micro-strains, while reflective crack relief interlayer type mixtures were tested between 1,000 and 2,000 micro-strains. The test conditions utilized, and recommended by AASHTO T321, for the study were as follows:

- Test temperature = 15°C;

- Haversine waveform;
- Strain-controlled mode of loading; and
- Loading frequency = 10 Hz;

### 6.1.1 Flexural Beam Fatigue Test Results – Rt 34N, New Jersey

Three asphalt mixtures were sampled and tested using the Flexural Beam Fatigue test. The final test results are shown in Figure 6.2. The test results clearly show the superior fatigue properties of the RCRI mixture at comparable strain levels (900 microstrains or 0.0009 in/in), and at elevated microstrains, when compared to the dense-graded mixtures.

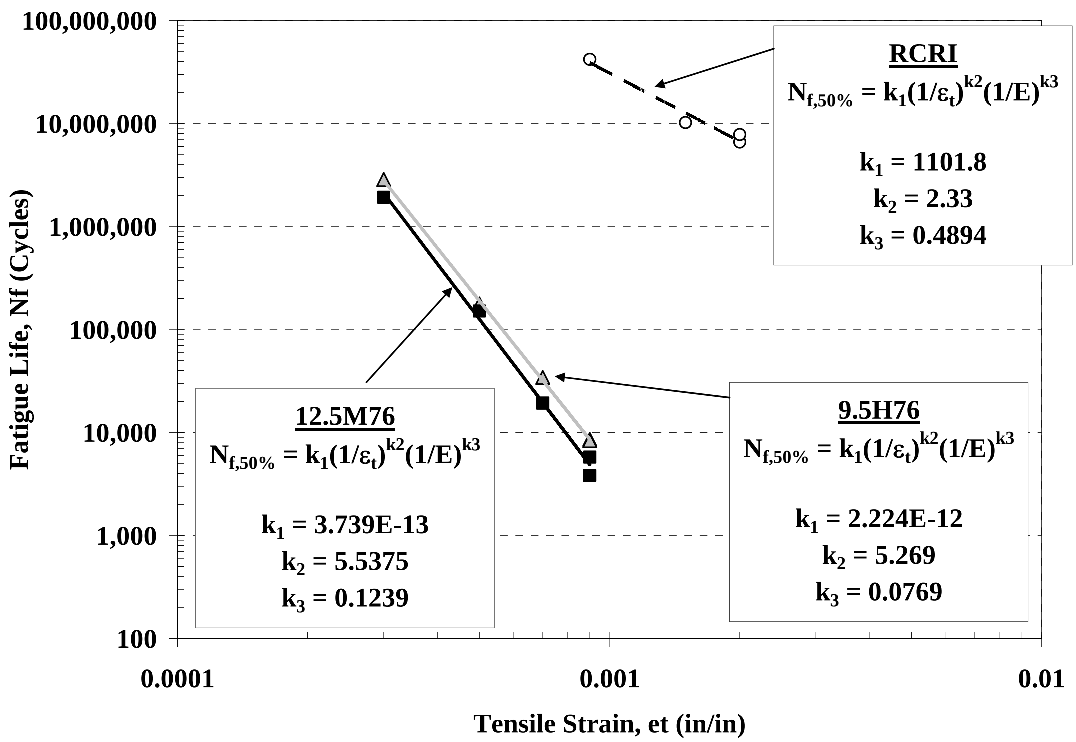



Figure 6.2 – Flexural Beam Fatigue Life of Asphalt Mixture from Rt 34N, New Jersey

The results of Figure 6.2 clearly indicate the benefit of utilizing the RCRI mixtures when attempting to build asphalt pavements over pre-existing PCC pavements. However, with the flexural fatigue life of the dense-graded mixtures being so poor, it remains to be seen whether or not the dense-graded mixtures can withstand any of the residual vertical deflections.

### 6.1.2 Flexural Beam Fatigue Results – Rt. 202S, New Jersey

Five different asphalt mixtures sampled from the asphalt supplier's plant were brought back to the laboratory and tested for their respective flexural fatigue life. Figure 6.3 shows the resultant fatigue life of the individual mixes. The fatigue life coefficients of the different mixes are shown in Table 6.1. As described earlier, this project utilized an experimental binder called Experimental Flexural Binder (XFB) in some of the surface (wearing) course mix, as opposed to the typical PG76-22 asphalt binders commonly used in New Jersey. The results again show the superior performance of the Reflective Crack Relief Interlayer (RCRI) mixture over the dense-graded mixes. Meanwhile, the fatigue resistance was also found to improve when the XFB asphalt binder was substituted straight for the PG76-22 asphalt binder. This indicates that the fatigue resistance of asphalt mixture can be improved by using the appropriate asphalt binder type, not just through adjusting the volumetrics of the asphalt mixture (i.e. – air voids, effective asphalt content, etc.).

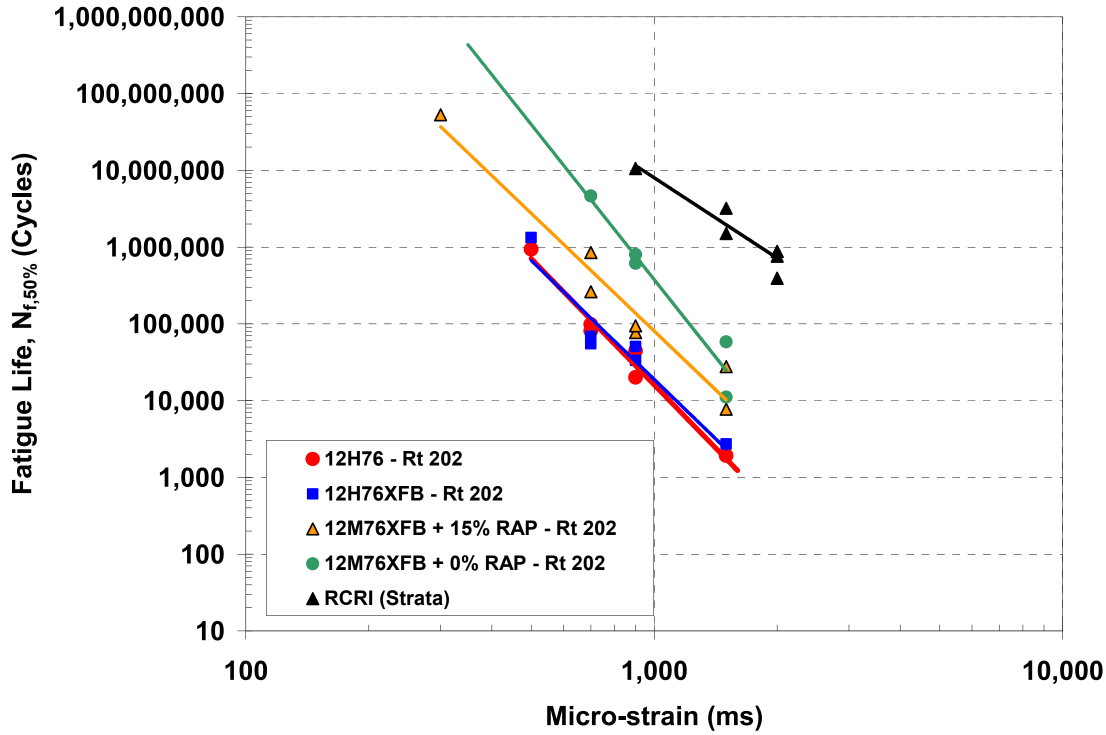



Figure 6.3 – Flexural Beam Fatigue Life of Asphalt Mixture from Rt 202S, New Jersey

Table 6.1 - Flexural Fatigue Life Coefficients of Rt 202S Asphalt Mixtures

| Asphalt Mixture | Flexural Fatigue Life Coefficients | | |
|---|---|---|---|
| | $k_1$ | $k_2$ | $k_3$ |
| RCRI (Strata) | 1.45E+37 | 4.8026 | 2.959 |
| 12M76XFB + 0% RAP | 3.30E+57 | 8.7039 | 4.6225 |
| 12M76XFB + 15% RAP | 3.60E+41 | 5.7779 | 3.3492 |
| 12H76XFB + 15% RAP | 2.38E+30 | 5.4204 | 1.6499 |
| 12H76 + 15% RAP | 3.75E+23 | 5.5045 | 0.4765 |

### 6.1.3 Flexural Beam Fatigue Results – I495, Massachusetts

Flexural Fatigue testing was conducted using the Flexural Beam Fatigue test procedure outlined in AASHTO T321, *Determining the Fatigue Life of Compacted Hot-Mix Asphalt (HMA) Subjected to Repeated Flexural Bending*. The applied tensile strain levels used for the fatigue evaluation were; 300, 500, 700, and 900 micro-strains for the Leveling/Intermediate mixture; 900, 1200, 1500, and 2000 for the XFB mixture; and 1500, 1750, and 2000 microstrains for the RCRI mixture. Since loose mix was not available for sampling, laboratory test specimens were made with raw materials (aggregates, RAP, and asphalt binder). Samples were tested after short-term aging following the procedures outlined in AASHTO R30, *Mixture Conditioning of Hot-Mix Asphalt (HMA)* in an effort to age the asphalt mixture samples to similar aging conditions found in the field. The test conditions utilized were those recommended by AASHTO T321 and were as follows:

- Test temperature = 15$^{o}$C;
- Sinusoidal waveform;
- Strain-controlled mode of loading; and
- Loading frequency = 10 Hz;

The final test results are shown in Figure 6.4. The Flexural Fatigue results clearly show the difference in fatigue resistance between the RCRI mixture and the Leveling/Intermediate mixture used on the I495 composite pavement. This discrepancy in flexural fatigue resistance creates an incompatibility that initiates what is called a "crack jump". This occurs on RCRI/SAMI overlays where the reflective crack does not initiate at the bottom of the RCRI/SAMI layer, but at the bottom of the HMA overlay that is placed immediately on top of the RCRI/SAMI layer.

### 6.1.4 Flexural Beam Fatigue Results – I476, Pennsylvania

Loose mix from the project was sampled and brought back to the laboratory of SemMaterials, LLC in Tulsa, OK for testing. The flexural beam fatigue test results are shown in Figure 6.5. The results indicate that lowering gyration level from 100 design gyrations to 75 design gyrations increased the flexural fatigue resistance. This was expected as the asphalt content increased by 0.4% by simply reducing the design gyration level. The test results also indicate that the RCRI mixture has superior fatigue resistance over the dense-graded mixtures, by almost four orders of magnitude.

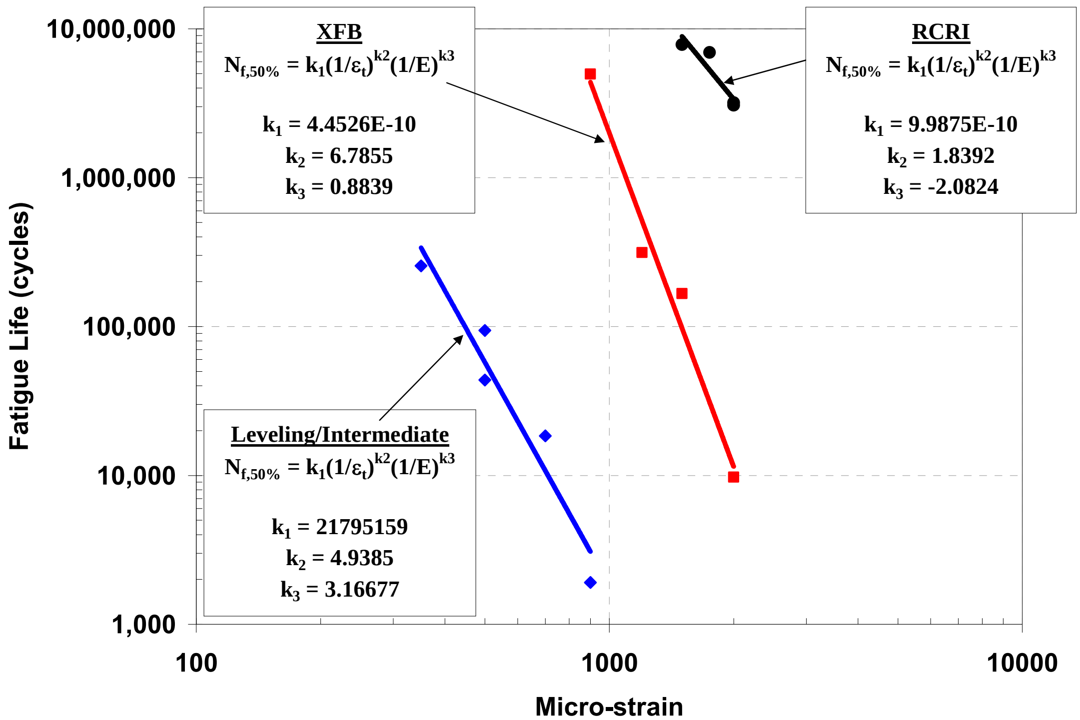



Figure 6.4 – Flexural Beam Fatigue Life of Asphalt Mixture from I495, Massachusetts

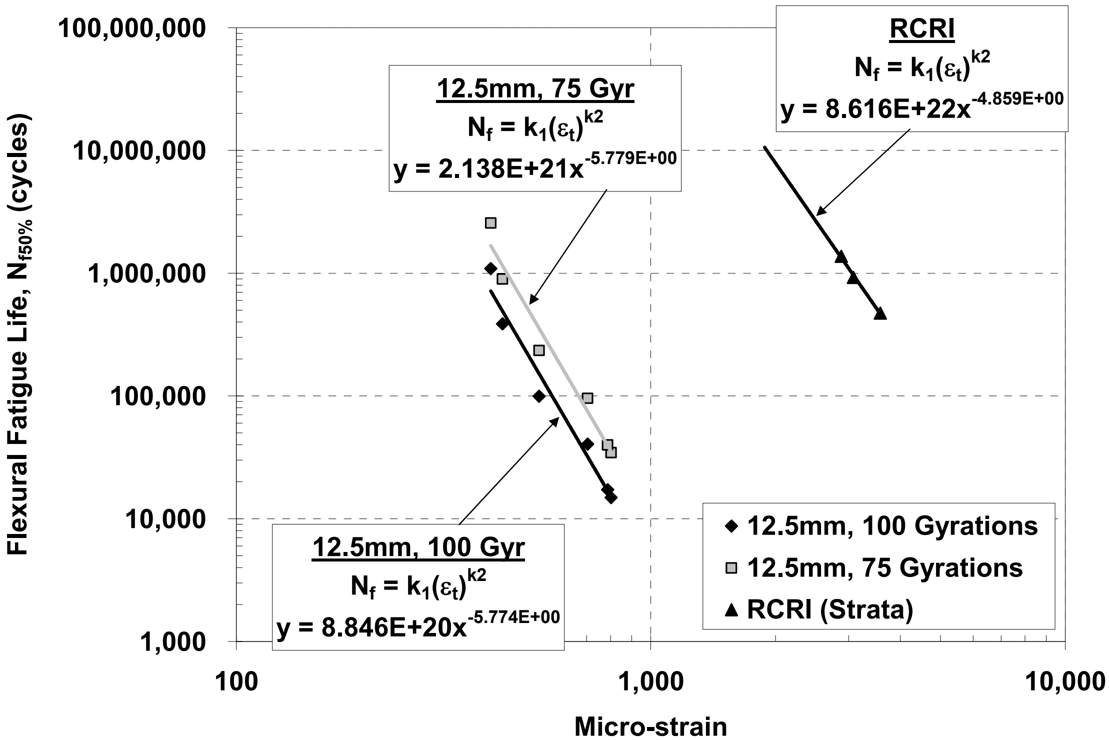



Figure 6.5 – Flexural Beam Fatigue Life of Asphalt Mixtures from I476, Pennsylvania

### 6.1.5 Additional HMA Mixtures from New Jersey

Additional HMA mixtures were sampled from different New Jersey asphalt plants during construction. Each mixture's flexural beam fatigue properties were measured and determined in accordance with Equation 6.3. A summary of material coefficients, as defined in Equation 6.3, are shown in Table 6.2.

Table 6.2 – Flexural Fatigue Coefficients of Various HMA Mixtures in New Jersey

| Asphalt Mixture Type | Flexural Fatigue Coefficients | | |
|---|---|---|---|
| | k1 | k2 | k3 |
| Rich Bottom Layer | 2.02E-08 | 3.684432 | -2.435533 |
| High Performance Thin Overlay | 9.73E-07 | 2.313353 | -2.738322 |
| 9.5mm SMA | 2.18E-09 | 3.643022 | -2.935182 |
| NJDOT 9.5H76 | 1.75E-07 | 1.289634 | -3.686605 |
| NJDOT 12.5H76 | 1.66E-07 | 2.906064 | -2.017857 |
| NJDOT 12.5M64 | 3.57E-13 | 4.543909 | -3.491312 |

## 6.2 TTI Overlay Tester Results

The TTI Overlay Tester is a relatively new test method developed by the Texas Transportation Institute, TTI (Germann and Lytton, 1979; Zhou and Scullion, 2005). The test device simulates the expansion and contraction movements that occur in the joint/crack vicinity of PCC pavements. Although this test procedure is essentially a fatigue-type test, it currently represents the best method to truly simulate horizontal joint movements of PCC pavements in the laboratory (Figure 6.6).

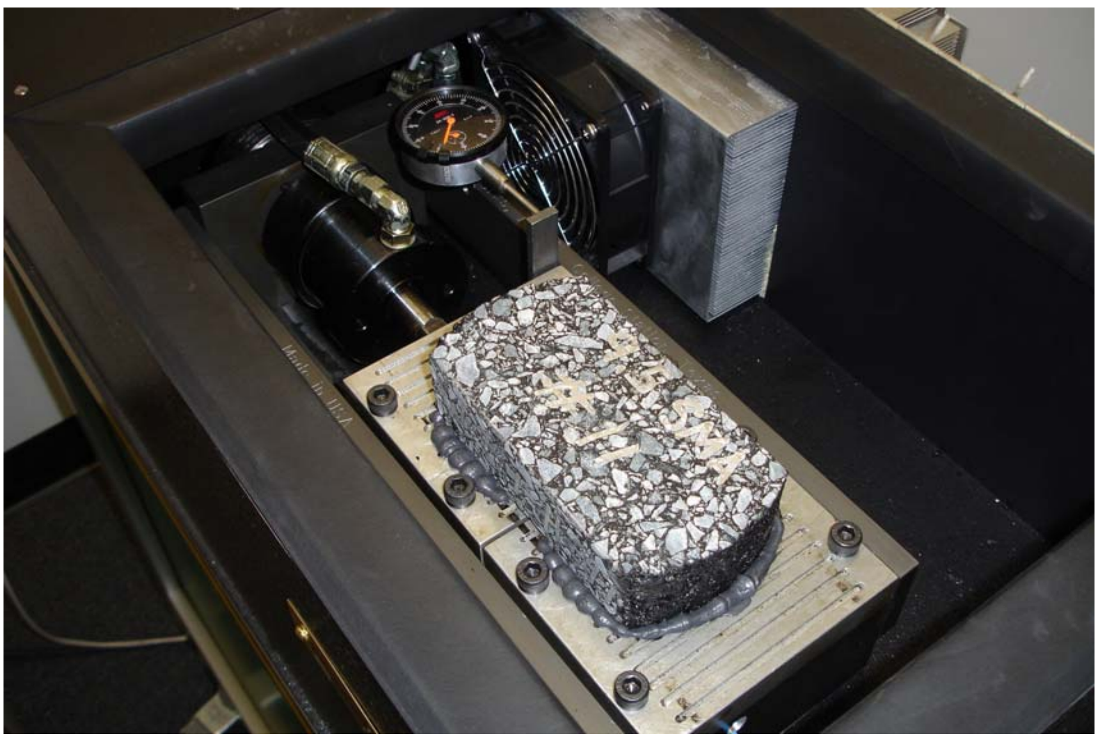


Figure 6.6 – Photo of the TTI Overlay Tester

### 6.2.1 Description of Methodology for Determining Testing Parameters

The TTI Overlay Tester has the capability of measuring the fatigue cracking resistance of hot mix asphalt specimens under temperature and deformation characteristics similar to field conditions. The horizontal deflection mode of reflective cracking is dictated by the expansion and contraction movements of the PCC slabs due to temperature cycling, and can be calculated using Equation (6.4).

$$\Delta L = CTE(L_{eff})(\Delta T)(\beta) \tag{6.4}$$

where,

CTE = coefficient of thermal expansion
$\Delta T$ = maximum 24-hour temperature difference
$\beta$ = PCC/Base friction factor
$L_{eff}$ = effective PCC joint spacing
$\Delta L$ = expected horizontal movement at the PCC slab joint due to daily temperature changes

In this scenario, the most critical condition would be when the temperature is already cold and there is a cooling cycle (i.e. – 4:00PM to 4:00AM in the month of February) (Bozkurt and Buttlar, 2002). And since the expansion and contraction is dependent on the temperature change, the same composite pavement with a thicker HMA overlay will expand and contract less due to the affect of thermal insulation. One of the difficulties in utilizing Equation 6.4 is the determining the temperature of the asphalt material at the surface of exiting PCC pavement, as well as determining the maximum temperature difference within a 24-hour time period. In substitution of actual field measurements, an alternative prediction methodology currently being used in the Mechanistic Empirical Pavement Design Guide (MEPDG) can be utilized.

The Enhanced Integrated Climatic Model (EICM) is a one-dimensional coupled heat and moisture flow model initially developed for the FHWA and adapted for use in the *Mechanistic-Empirical Pavement Design Guide* (MEPDG) developed under NCHRP Project 1-37A. In the MEPDG, the EICM is used to predict or simulate the changes in behavior and characteristics of pavement and unbound materials in conjunction with environmental conditions over many years of service. The research conducted in

NCHRP Project 9-23, "Environmental Effects in Pavement Mix and Structural Design Systems," validated the temperature profile and moisture predictive capabilities of the EICM (Zapata and Houston, 2008).

The PCC/base friction factor, β, in Equation 6.4 adjusts the unrestrained movement of a slab at a joint to a lower value as a result of slab base friction. Friction coefficients calculated during FHWA-RD-02-088, *Evaluation of Joint and Crack Load Transfer* (Khazanovich and Gotlif, 2003) are shown in Figure 6.7. The PCC/base friction factor coefficients were determined for nine PCC LTPP test sections. One can observe that only one section (133019) resulted in a very low friction factor. For all other sections, the friction factor ranges from 0.34 to 0.8. By utilizing pavement sections in close vicinity to the pavement sections in this study (Ohio - 390204 and Pennsylvania - 421606), an average PCC/base friction factor of 0.76 is calculated and can be used for determining the horizontal deformation test criteria in the TTI Overlay Tester.

| Section | State | Base Type | Coefficient of Thermal Expansion, mm/mm/°C (inch/inch/°F) | Joint Spacing, m (ft) | Effective Joint Spacing, m (ft) | Slope k mm/°C (inch/°F) | Friction Factor |
|---|---|---|---|---|---|---|---|
| 133019 | Georgia | AGG | 1.00 (0.56) | 6 .00(19.68) | 6.0 (19.68) | 0.0068 (0.00015) | 0.11 |
| 204054 | Kansas | CTB | 1.09 (0.61) | 9.00 (29.52) | 4.5 (14.76) | 0.0111 (0.00024) | 0.40 |
| 274040 | Minnesota | AGG | 1.02 (0.57) | 8.10 (26.57) | 4.05 (13.28) | 0.0161 (0.00035) | 0.69 |
| 390204 | Ohio | AGG | 0.95 (0.53) | 4.50 (14.76) | 4.5 (14.76) | 0.0194 (0.00043) | 0.81 |
| 421606 | Pennsylvania | AGG | 1.24 (0.69) | 13.95 (45.76) | 6.9 (22.63) | 0.0339 (0.00075) | 0.70 |
| 484142 | Texas | AGG | 0.95 (0.53) | 18.15 (59.53) | 6.0 (19.68) | 0.0111 (0.00024) | 0.35 |
| 484143 | Texas | CTB | 0.95 (0.53) | 18.15 (59.53) | 6.0 (19.68) | 0.0172 (0.00038) | 0.53 |
| 493011 | Utah | CTB | 0.95 (0.53) | 4.50 (14.76) | 4.5 (14.76) | 0.0139 (0.00031) | 0.58 |
| 833802 | Manitoba | CTB | 1.03 (0.58) | 4.50 (14.76) | 4.5 (14.76) | 0.0149 (0.00033) | 0.57 |

1 mm/mm/°C = 0.56 inch/inch/°F
1 mm/°C = 0.22 inch/°F

Figure 6.7 – PCC/Base Friction Factors for SMP LTPP Sections (Khazanovich and Gotlif, 2003)

### 6.2.2 TTI Overlay Tester – Rt 34N, New Jersey

The TTI Overlay Tester was used to determine the fatigue resistance of the asphalt mixtures in the horizontal deflection mode. As stated earlier, the magnitude of the horizontal deflection at the PCC joint/crack is a function of the coefficient of thermal expansion (CTE), effective PCC slab length, and 24 hour temperature change (Equation 6.4). Therefore, to maximize the data from the TTI Overlay Tester, asphalt mixtures should be tested under similar conditions as would be expected in the field (i.e. – temperature of asphalt mixture and magnitude of horizontal deflection).

The test was conducted using protocol established for the Texas Department of Transportation (TxDOT Tex-248-F), although a test temperature of 15$^{o}$C was used instead of 25$^{o}$C to better represent New Jersey conditions. The horizontal deflection of 0.026 inches (67mm), determined from the Coefficient of Thermal Expansion testing on the PCC cores and the resulting joint deflection calculated using Equation 6.4, was used for the testing. The test results from the Overlay tester were:

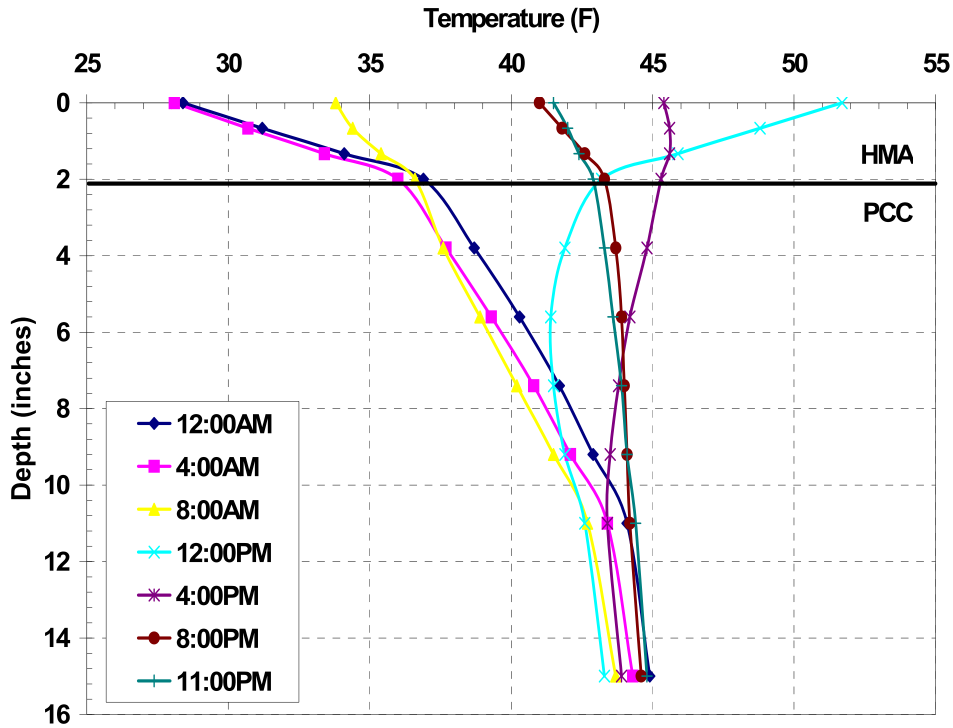
- 12.5mm Superpave Mixture: 22 Cycles
- 9.5mm Superpave Mixture: 24 Cycles
- RCRI Mixture: 46,502 Cycles

It should be noted that the final test results of the RCRI mixture were extrapolated after testing was stopped at 3,000 cycles. The TTI Overlay Tester results again illustrate the superior fatigue resistance of the RCRI mixtures. The TTI Overlay Tester results for the two Superpave mixes were typical for most of the Superpave mixtures currently being placed in New Jersey.

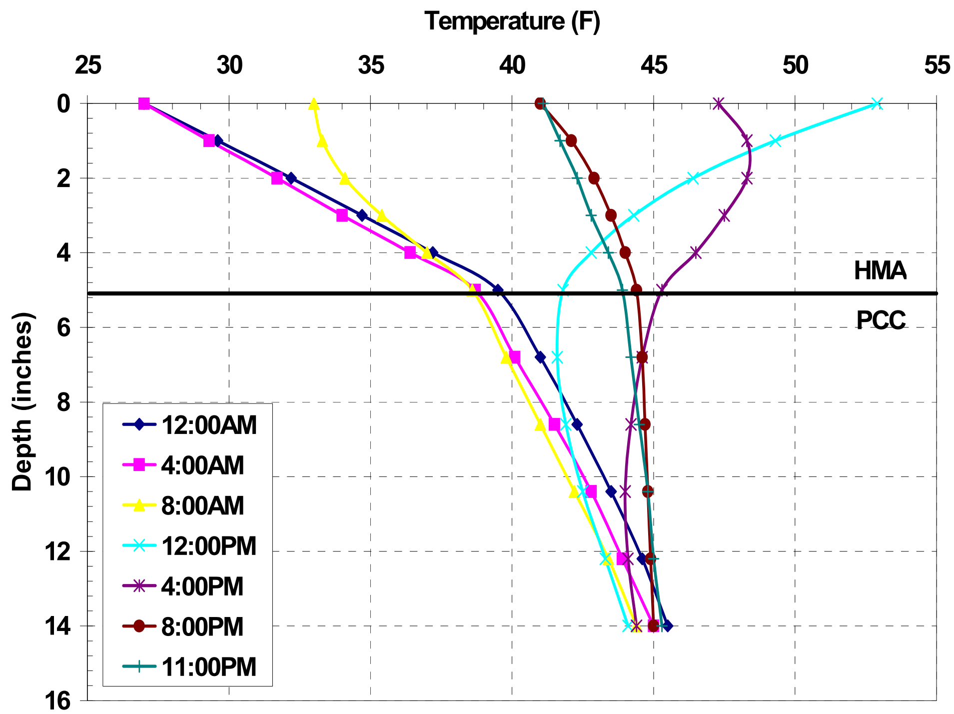
To put the test results in perspective, albeit empirical, it would only take approximately 22 days of climatic conditions that create a 6$^{o}$C drop in temperature, within a 24-hour period, at the surface of the PCC for the 12.5mm Superpave mixture to achieve cracking failure.

### 6.2.3 TTI Overlay Tester Results – I495, Massachusetts

Interstate 495 in Massachusetts provided an interesting evaluation on the affects of pavement and HMA overlay parameters on the fatigue cracking resistance of HMA mixtures due to horizontal expansion and contraction. Since the expansion and contraction is dependent on the temperature change, the same composite pavement with a thicker HMA overlay will expand and contract less due to the affect of thermal insulation. This can be seen in Figure 6.8, where the 2” Overlay and 5” Overlay Sections of I495 were modeled for a day in December 2007. The Enhanced Integrated Climatic Model was used to model the temperature distribution in the pavement sections, and a joint spacing of 25 meters (78.8 ft) was used as the effective joint spacing, as determined from the MassHighway crack survey. A β value of 0.76 was used for the PCC/Base Friction Factor. For the laboratory evaluation, the average monthly maximum 24-hour temperature change and pavement temperature were determined and shown in Table 1 for the 2” and 5” Overlay Sections. On average, a $2^{o}$F difference in temperature change at the top of the PCC pavement was observed between the two overlay sections. There was also a $0.8^{o}$F difference in average temperature between the overlay sections. For the laboratory evaluation, the average monthly maximum 24-hour temperature change and pavement temperature were determined and shown in Table 6.3 for the 2” and 5” Overlay Sections. On average, a $2^{o}$F difference in temperature change at the top of the PCC pavement was observed between the two overlay sections. There was also a 0.8°F difference in average temperature between the overlay sections.




(a)




(b)

Figure 6.7 a) Pavement Temperature Profile for 2" Section; b) Pavement Temperature Profile for 5" Section on I595 in Massachusetts

Table 6.3 - Maximum Monthly Temperatures for Pavement Sections

| Summary of Temperature for 2-Inch Overlay Section | | | |
|---|---|---|---|
| Month | Max. 24-hr Temperature Change ($^{o}$F) at Top of PCC | Average Temperature at Top of PCC ($^{o}$F) | Average Temperature at Middle of HMA ($^{o}$F) |
| November | 6.9 | 51.7 | 51.2 |
| December | 5.9 | 41.5 | 40.6 |
| January | 7.0 | 36.0 | 35.0 |
| February | 7.2 | 37.9 | 37.8 |
| March | 8.2 | 42.0 | 42.3 |
| ***Average*** | ***7.0*** | ***41.8*** | ***41.4*** |

| Summary of Temperature for 5-Inch Overlay Section | | | |
|---|---|---|---|
| Month | Max. 24-hr Temperature Change ($^{o}$F) at Top of PCC | Average Temperature at Top of PCC ($^{o}$F) | Average Temperature at Middle of HMA ($^{o}$F) |
| November | 5.0 | 52.1 | 51.2 |
| December | 4.2 | 42.3 | 40.8 |
| January | 5.0 | 36.6 | 35.2 |
| February | 5.1 | 38.1 | 37.7 |
| March | 5.4 | 43.8 | 43.6 |
| ***Average*** | ***5.0*** | ***42.6*** | ***41.7*** |

Utilizing the modeled temperatures shown in Table 1, the horizontal deflection of the 2" and 5" Overlay sections were calculated using Equation (3). The calculated horizontal movements for the Overlay sections were:

- 2" Overlay Section: $\Delta L = 0.74$mm (0.029 inches)
- 5" Overlay Section: $\Delta L = 0.53$mm (0.021 inches)

The calculated $\Delta L$ and average temperature at the top of the PCC was then used in the Overlay Tester to assess the different mixtures' resistance to the horizontal joint movements. Utilizing the calculated $\Delta L$ and pavement temperatures determined earlier, the different mixtures were evaluated. The samples were tested in triplicate at measured in-place air voids and averaged for comparison purposes. The results of the Overlay Tester are shown in Figure 6.8.

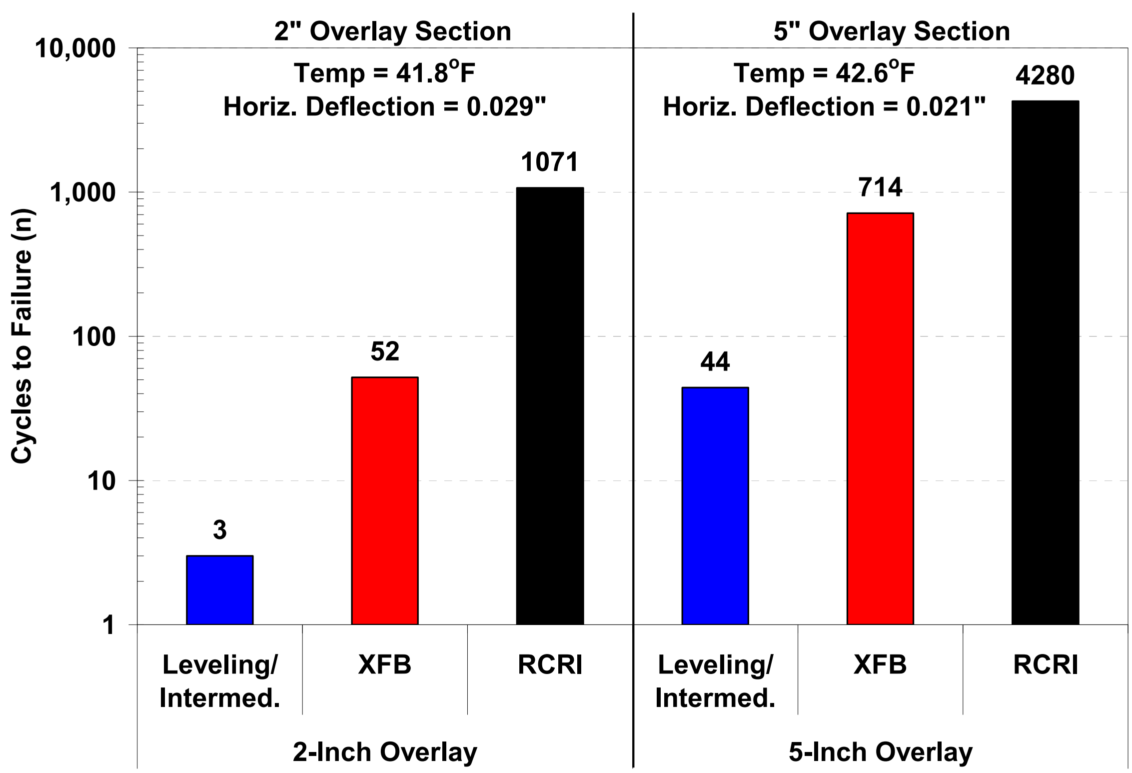



Figure 6.8 – Overlay Tester Test Results for 2" and 5" Overlay Sections

Figure 6.8 clearly shows the impact of the thermal insulation provided by the additional 76mm (3 inches) of HMA placed in the 5” Overlay Section. The test results in Figure 6.8 shows how susceptible the Leveling/Intermediate course mixture was to fatigue failure under the expected horizontal joint movements, especially in the 2” Overlay Section. The RCRI mixture shows exceptional resistance to the expected horizontal movements, even if it were placed in the 2” Overlay Section (just evaluated and shown for comparison purposes). The test results also show the benefit of utilizing an asphalt binder that is designed to be more resistant to fatigue cracking. The XFB mixture, having the identical JMF as the Leveling/Intermediate Course with simply a different asphalt binder, had an Overlay Tester fatigue resistance almost twenty times greater than the Leveling/Intermediate course mixture.

### 6.2.4 TTI Overlay Tester Results – I476 Pennsylvania

Limited loose mix collected during the asphalt overlay on I476 in Pennsylvania was used to prepare TTI Overlay Tester specimens. The Enhanced Integrated Climatic Model (EICM) was used to model the temperature distribution in the pavement sections, and a joint spacing of 14.2 meters (46.5 ft) was used as the effective joint spacing, as determined from the Pennsylvania Turnpike Authority crack survey. A $\beta$ value of 0.76 was used for the PCC/Base Friction Factor. For the laboratory evaluation, the average monthly maximum 24-hour temperature change and pavement temperature at the top of the PCC pavement (or bottom of the HMA overlay) were determined using the EICM. On average, the monthly 24-hour temperature change and PCC pavement surface temperatures were 4.7$^{o}$F and 54$^{o}$F, respectively. Utilizing Equation 6.4, the average expected horizontal deflection used in the laboratory testing was 0.013 inches.

The results of the TTI Overlay Tester testing on the I476 asphalt mixtures are shown in Figure 6.9. The results indicate that the RCI mixture evaluated at loading conditions similar to those experienced on I476 are far superior then the dense-graded mixtures. This is consistent with the both the flexural beam fatigue data and the TTI Overlay Tester data presented in this study. The test results also show that the dense-graded mixes from I476 outperformed those from the other test sections. This was most likely due to the fact that the effective slab length was shorter for the I476 PCC pavement than the other test sections. As a result, the average horizontal deflection was less.

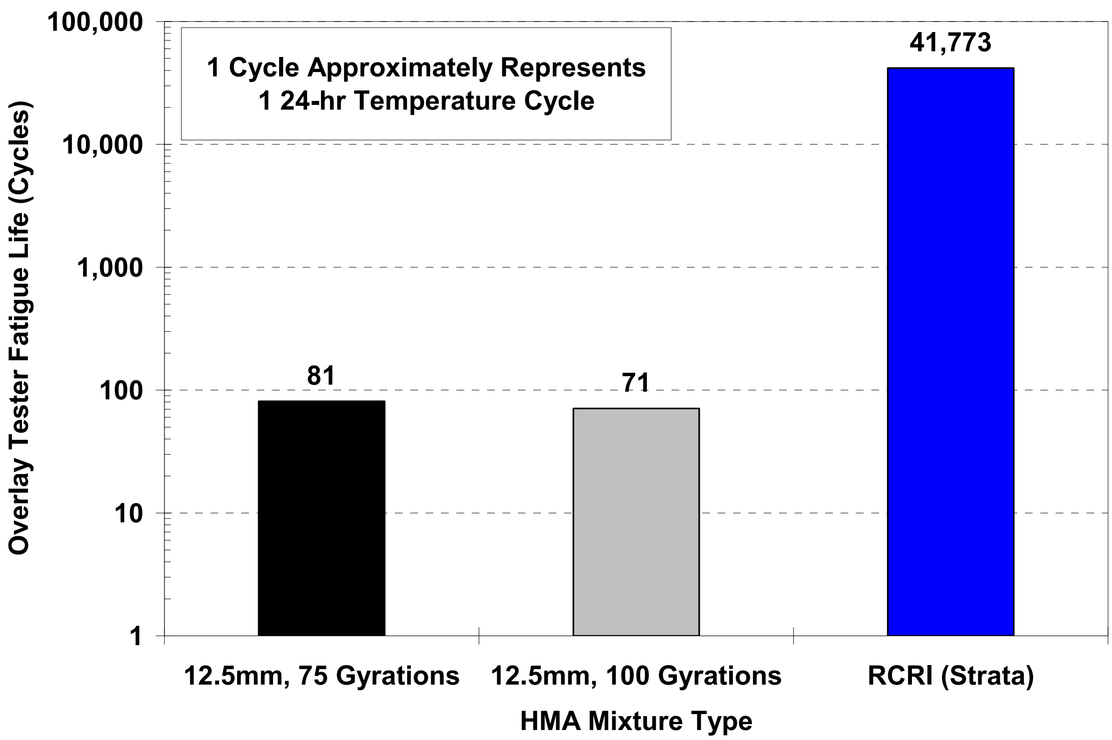



Figure 6.9 – TTI Overlay Tester Results for HMA Mixtures on I476, Pennsylvania

### 6.2.5 TTI Overlay Tester Results - Additional HMA Mixture

A fatigue cracking database of material properties using the TTI Overlay Tester was created in similar fashion. Loose mix was sampled from various projects throughout New Jersey and compacted to provide TTI Overlay Tester samples. Unlike previously where horizontal movements and pavement temperatures were calculated for each research test section, specimens were tested at different test temperatures and horizontal deformations. The resultant test data was plotted and fitted with a non-linear regression equation relating specimen temperature and horizontal deformation to fatigue life due to horizontal expansion and contraction. Equation 6.5 was developed to provide a prediction equation for future use and Figure 6.10 shows an example test data for a 12.5mm dense-graded asphalt mixture in New Jersey.

$$\mathrm{N_f} = \mathrm{k_1}(\mathrm{Temp})^{\mathrm{k_2}}(\Delta H)^{\mathrm{k_3}} \tag{6.5}$$

where,

$N_f$ = fatigue life in Overlay Tester (cycles);
Temp = specimen temperature ($^{o}$F);
$\Delta$H = horizontal deformation (inches); and
$k_1$, $k_2$, $k_3$ = material specific coefficients.

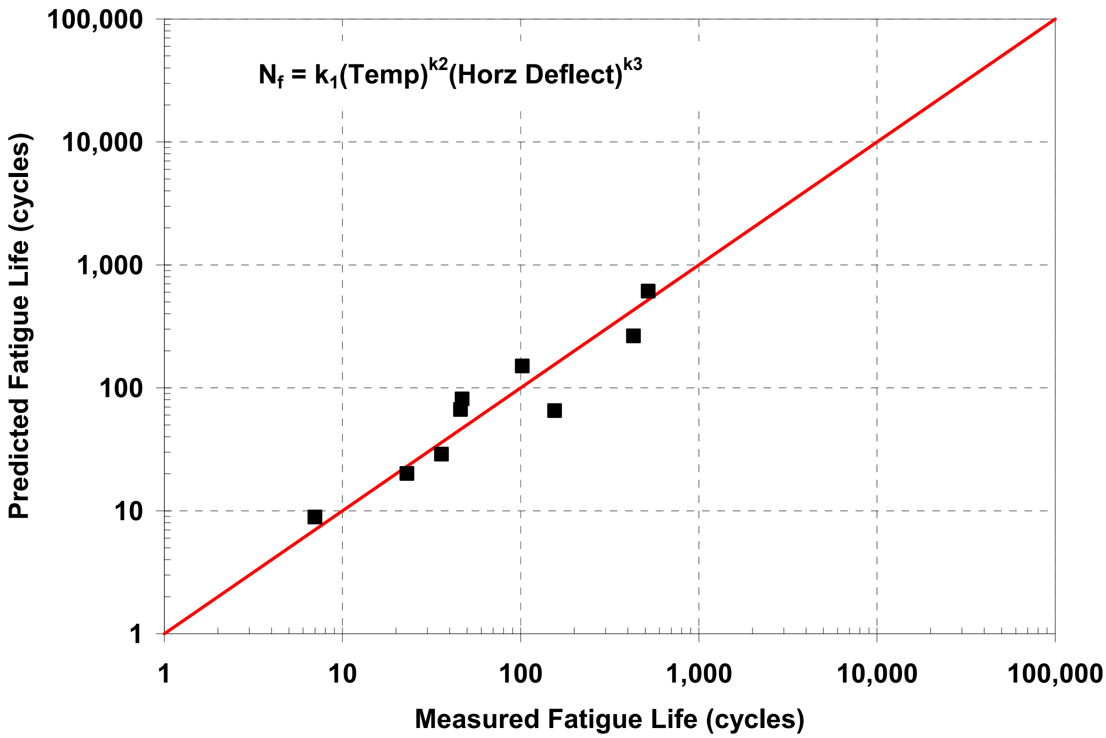



Figure 6.10 – Example of Horizontal Fatigue Life Determined in the TTI Overlay Tester

The advantage of generating a material database using the TTI Overlay Tester and Equation 6.5 is that HMA mixtures can be compared under different pavement conditions to determine the most cost effective HMA mixture to be placed immediately over the PCC pavement, or at the bottom of new HMA overlay. Table 6.4 contains the mixture specific coefficients of Equation 6.5 for the different New Jersey asphalt mixtures commonly placed over PCC/composite pavements.

Table 6.4 – HMA Material Specific Coefficients Generated by the TTI Overlay Tester (Horizontal Deflection Mode – Equation 6.5)

| HMA Mixture Type (NJDOT) | HMA Material Coefficients | | |
|---|---|---|---|
| | $k_1$ | $k_2$ | $k_3$ |
| Strata | 2.07E-07 | 3.491199 | -2.433821 |
| RBL | 2.02E-08 | 3.684432 | -2.435533 |
| HPTO | 9.73E-07 | 2.313353 | -2.738322 |
| 9.5mm SMA | 2.18E-09 | 3.643022 | -2.935182 |
| 9.5H76 | 1.75E-07 | 1.289634 | -3.686605 |
| 12H76 | 1.66E-07 | 2.906064 | -2.017857 |
| 12M64 | 3.57E-13 | 4.543909 | -3.491312 |

## 6.3 Dynamic Modulus (Stiffness) Testing

Dynamic modulus and phase angle data were measured and collected in uniaxial compression following the method outlined in AASHTO TP62, *Standard Test Method for Determining Dynamic Modulus of Hot-Mix Asphalt Concrete Mixtures*. The data was collected at three temperatures; 4, 20, and 35$^{o}$C (for the Neat asphalt binder only) and 45$^{o}$C (for polymer modified asphalt binders), using loading frequencies of 25, 10, 5, 1, 0.5, 0.1, and 0.01 Hz. All samples were tested in triplicate and averaged to develop a master stiffness curve.

The collected modulus values of the varying temperatures and loading frequencies were used to develop Dynamic Modulus master stiffness curves and temperature shift factors using numerical optimization of Equations 6.5 and 6.6 (Bonaquist and Christensen, 2005). The reference temperature used for the generation of the master curves and the shift factors was 20$^{o}$C.

$$\log|E^*| = \delta + \frac{(Max - \delta)}{1 + e^{\beta + \gamma\left\{\log\omega + \frac{\Delta E_a}{19.14714}\left[\left(\frac{1}{T}\right) - \left(\frac{1}{T_r}\right)\right]\right\}}} \tag{6.6}$$

where:

$|E^*|$ = dynamic modulus, psi

$\omega_r$ = reduced frequency, Hz

*Max* = limiting maximum modulus, psi

$\delta$, $\beta$, and $\gamma$ = fitting parameters

$$\log[a(T)] = \frac{\Delta E_a}{19.14714}\left(\frac{1}{T} - \frac{1}{T_r}\right) \tag{6.7}$$

where:

a(T) = shift factor at temperature T

$T_r$ = reference temperature, °K

T = test temperature, °K

$\Delta E_a$ = activation energy (treated as a fitting parameter)

Although the dynamic modulus (E*) parameter in itself does not directly measure the fatigue cracking or rutting resistance of asphalt mixtures, the dynamic modulus does provide a general guideline as how asphalt mixtures will generally perform. For example, asphalt mixtures with higher modulus at higher temperatures will generally be more resistant to permanent deformation. Meanwhile, asphalt mixtures with lower modulus values at lower and intermediate temperatures will generally be more resistant to fatigue and low temperature cracking.

The dynamic modulus value is required when conducting general pavement response analysis, such as when using linear elastic analysis programs and the Mechanistic Empirical Pavement Design Guide (MEPDG). These programs use the modulus (or stiffness) to determine the resultant stress and strains due to applied loading. The dynamic modulus is also used in Equation 6.3 as a HMA material parameter affecting the flexural fatigue performance of asphalt mixtures.

### 6.3.1 Dynamic Modulus Test Results – Rt 34N, New Jersey

The master stiffness curves of the asphalt mixtures sampled from the Rt 34N – New Jersey test section are shown in Figure 6.11. The results indicate that the RCRI mix has intermediate and low temperature (shown in the master stiffness curves as intermediate to high loading frequencies) modulus values much lower than the 9.5mm and 12.5mm Superpave mixes. As stated earlier, asphalt mixtures that are capable of obtaining lower stiffness properties at intermediate to low temperatures tend to be more resistant to cracking. This is consistent with the Flexural Beam Fatigue and TTI Overlay Tester results.

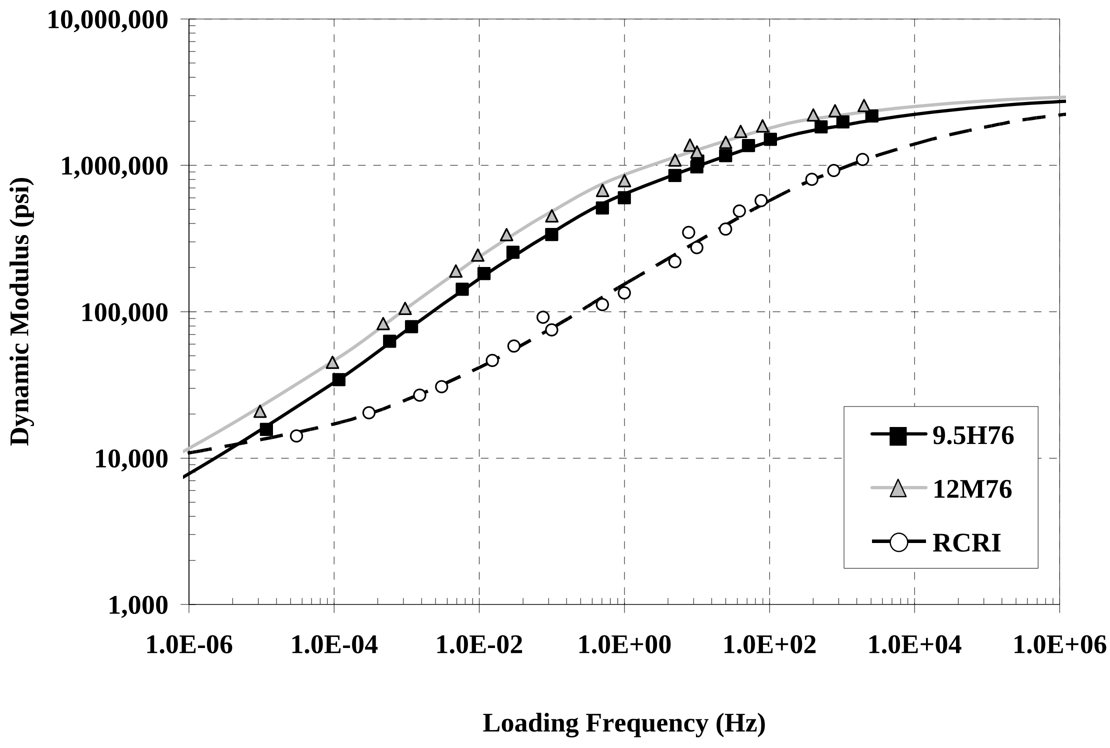



Figure 6.11 – Dynamic Modulus Test Results of Asphalt Mixtures on Rt 34N, New Jersey

### 6.3.2 Dynamic Modulus Test Results – I495 Massachusetts

Dynamic modulus and phase angle data for the I495 Massachusetts test section were measured and collected in uniaxial compression following the method outlined in AASHTO TP62-07, *Standard Test Method for Determining Dynamic Modulus of Hot-Mix Asphalt Concrete Mixtures*. The data was collected at three temperatures; 4, 20, and 45$^{o}$C (except for the RCRI which used 35$^{o}$C), using loading frequencies of 25, 10, 5, 1, 0.5, 0.1, and 0.01 Hz. Samples were tested in triplicate after short-term oven aging following the procedures outlined in AASHTO R30. The short term aging was required since the mixtures used for the I495 test section were produced in the laboratory and not during plant production like the other test sections.

The collected modulus values at the varying temperatures and loading frequencies were used to develop Dynamic Modulus master stiffness curves and temperature shift factors using numerical optimization as described in Equations 6.5 and 6.6. The resulting master stiffness curves for the different mixtures are shown in Figure 6.12. The master stiffness curves show that the RCRI mix has intermediate and low temperature (intermediate to high loading frequencies) stiffness' much lower than the Leveling/Intermediate Course mix. Asphalt mixtures that are capable of obtaining lower stiffness properties at intermediate to low temperatures tend to be more resistant to cracking. On the contrary, the master stiffness curve of the XFB mixture was far more comparable to that of the RCRI mixture, showing better compatibility to the RCRI mixture than the Leveling/Intermediate mix. Again, the dynamic modulus test results compare favorably to the fatigue cracking results shown earlier in the Flexural Beam Fatigue and TTI Overlay Tester results.

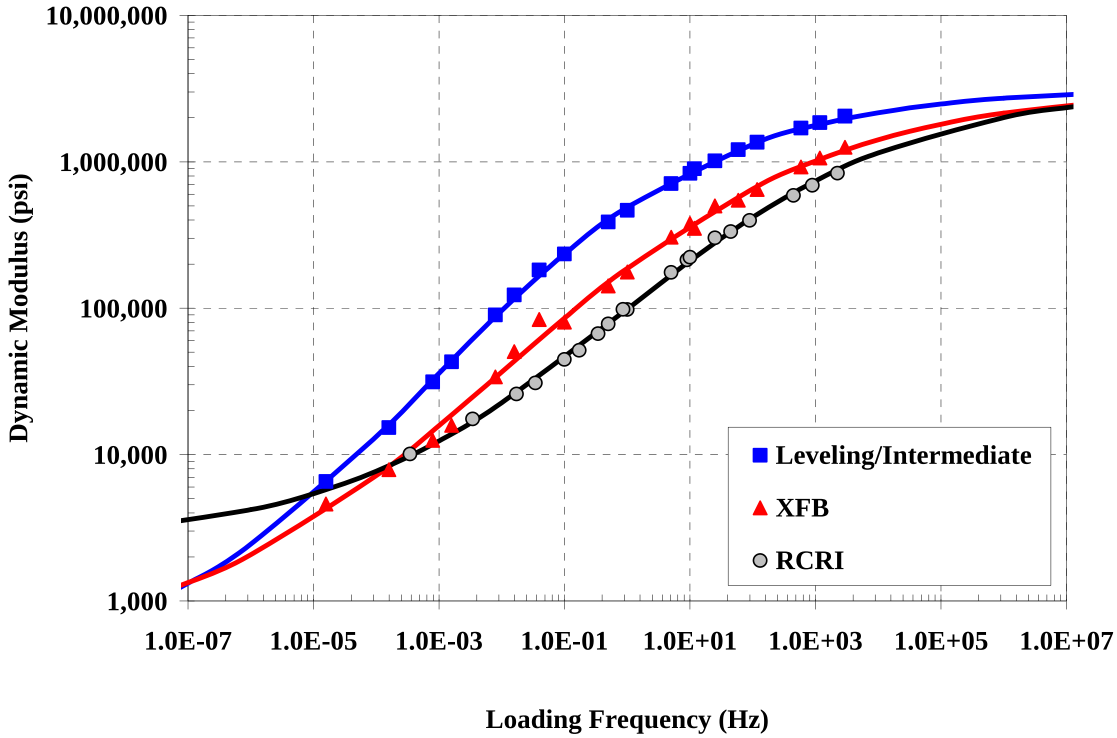



Figure 6.12 – Master Stiffness Curves of the Asphalt Mixtures from I495 Massachusetts Test Section

6.3.3 Dynamic Modulus Test Results – Rt 202S New Jersey

Dynamic modulus and phase angle data for the Rt 202S New Jersey test section were measured and collected in uniaxial compression following the method outlined in AASHTO TP62-07, *Standard Test Method for Determining Dynamic Modulus of Hot-Mix Asphalt Concrete Mixtures*. The data was collected at three temperatures; 4, 20, and 45$^{o}$C (except for the RCRI and 12M64 which used 35$^{o}$C), using loading frequencies of 25, 10, 5, 1, 0.5, 0.1, and 0.01 Hz.

The resultant dynamic modulus master curves for the tested asphalt mixtures are shown in Figure 6.13. Again, the RCRI mixture obtained the lowest dynamic modulus values at the intermediate and low test temperatures (shown in Figure 6.13 as the intermediate and higher loading frequencies). The asphalt mixtures containing the XFB asphalt binder and designed using 75 gyrations ("M" mixes) obtained lower dynamic modulus values at the lower and intermediate test temperatures (intermediate and higher lower frequencies). It is also interesting to note that as the RAP content increased in the 12M76 XFB mixes, the intermediate and low temperature stiffness also increased. Once again, the ranking of dynamic modulus at the intermediate and low test temperatures (intermediate and high loading frequencies) compared well with the Flexural Beam Fatigue and TTI Overlay Tester results.

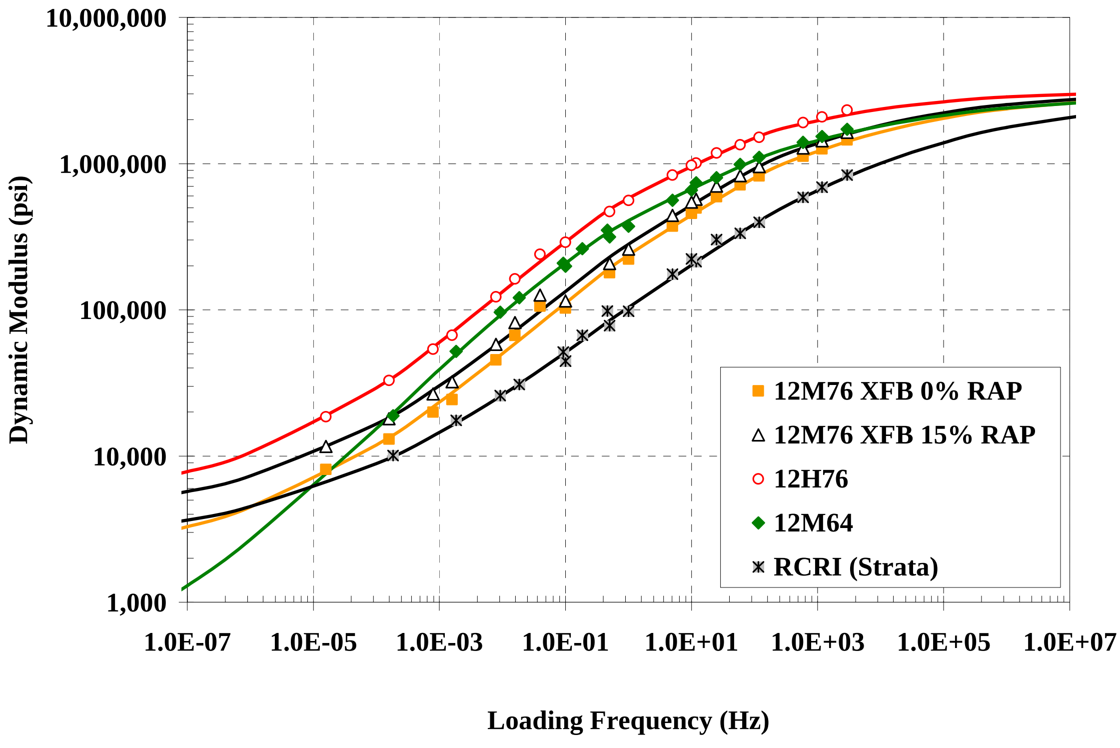



Figure 6.13 – Master Stiffness Curves of Asphalt Mixtures from the Rt 202S New Jersey Test Section

# CHAPTER 7 – DEVELOPMENT OF ANALYSIS METHOD

The proposed analysis method to evaluate reflective cracking potential of hot mix asphalt overlays on PPC/composite pavements utilizes field measured/estimated movements and evaluates asphalt mixtures under identical conditions in the laboratory. Although pavement engineers have attempted to conduct this type of analysis for years, the proposed method in this study has the advantage of utilizing known (from field data) or measured displacements in laboratory based performance tests. In most occasions, pavement designers relied heavily on elastic layer and finite element analysis with general fatigue relationships to predict the fatigue cracking potential of asphalt mixtures.

The following proposed analysis method separates the vertical and horizontal modes of deflection and evaluates them independently in an attempt to resolve each deflection mode one at a time. Vertical bending related reflective cracking is evaluated in the field using the Falling Weight Deflectometer (FWD) and measured traffic load, along with the Flexural Beam Fatigue and Dynamic Modulus test in the laboratory. Horizontal deformation related reflective cracking is evaluated using the structural characteristics of the pavement (i.e. – pavement thickness, PCC effective slab length, regional climatic conditions) and the coefficient of thermal expansion of the PCC in the field, along with the TTI Overlay Tester in the laboratory. Proper HMA mixture selection for the PCC/composite pavement requires the HMA overlay mixture placed directly on the PCC, or at the bottom of the new HMA overlay, to withstand both horizontal and vertical deflections without cracking. Meanwhile, asphalt mixture placed in the overlaying layers must primarily resist the residual vertical movements in the pavement section located in the immediate vicinity of the PCC joint/crack.

Based on field and laboratory studies, as well as the Literature Review and National Survey, the pavement design consists of some type of stress-absorbing membrane (SAMI) or reflective crack relief interlayer (RCRI) mixture that are designed to withstand the vertical and horizontal movements. Meanwhile, HMA mixtures overlaying the SAMI or RCRI must still be able to resist the residual vertical deflections associated with vertical joint deflections at the PCC joint/crack due to traffic loading. Chapter 7 discusses the proposed methodology for optimal mixture design, while demonstrating the effectiveness of the RCRI/flexible overlay design to mitigate reflective cracking in PCC/composite pavements.

## 7.1 Vertical Deflection Mode – Deflection Spectra Approach

When conducting Falling Weight Deflectometer (FWD) testing at different load magnitudes, the relationship between the vertical deflection at the joint (measured immediately under the load) and the applied load can be represented by a linear regression, thereby, allowing for the direct substitution of any known applied axle load to determine the resultant deflection at the joint. Figure 7.1 shows the relationship between applied load (applied by the Falling Weight Deflectometer) and the measured vertical deflection at the PCC joint/crack for the Rt 34N New Jersey test section.

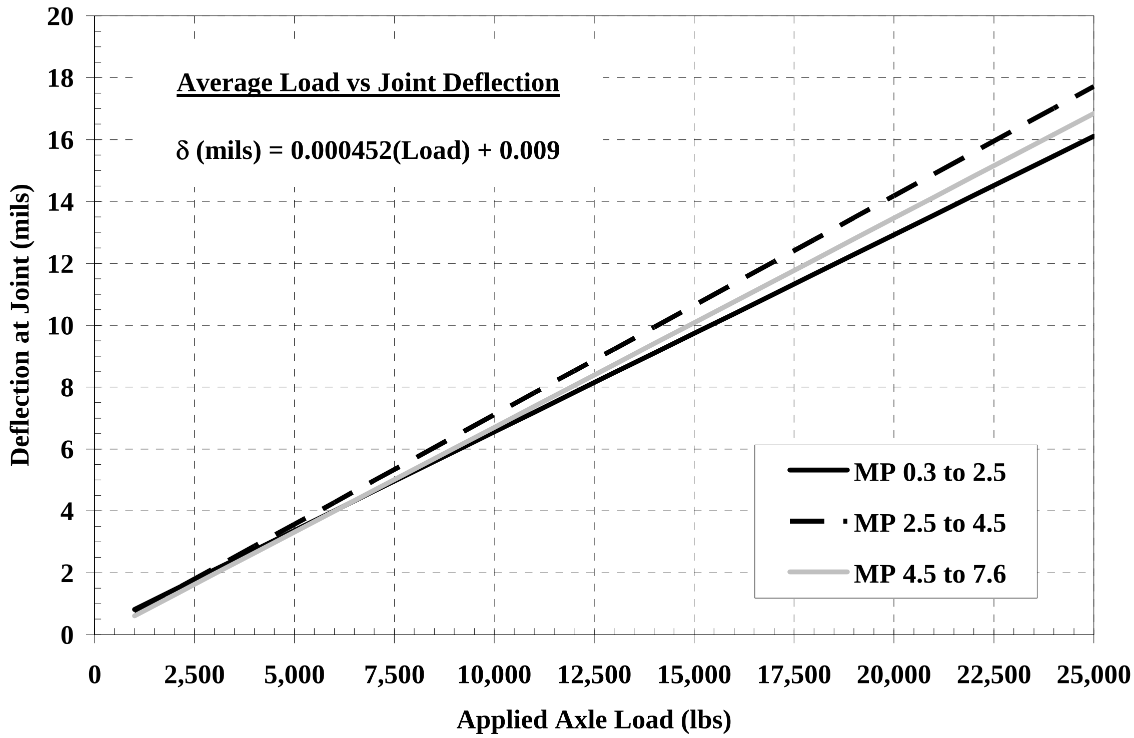



Figure 7.1 – Relationship Between FWD Load Magnitude and Vertical Deflection at PCC Joint for Rt 34N New Jersey Test Section

Once the relationship between applied load and vertical joint deflection is established, the axle load measurements recorded by WIM/AVC sensors or Equivalent Single Axle Load (ESAL) counts can be directly inputted into the regression equation to generate a PCC Vertical Joint Deflection Spectra. In this research study, a PCC Vertical Joint Deflection Spectra is defined as the amount and magnitude of vertical deflections that are assumed to occur due to daily traffic volumes. The PCC Vertical Joint Deflection Spectra developed for Test Section #1 of Rt 34N New Jersey is shown as Figure 7.2. The Deflection Spectra shown in Figure 7.2 can now be used to help determine the magnitude and number of vertical deflections to cause reflective cracking in the HMA overlay.

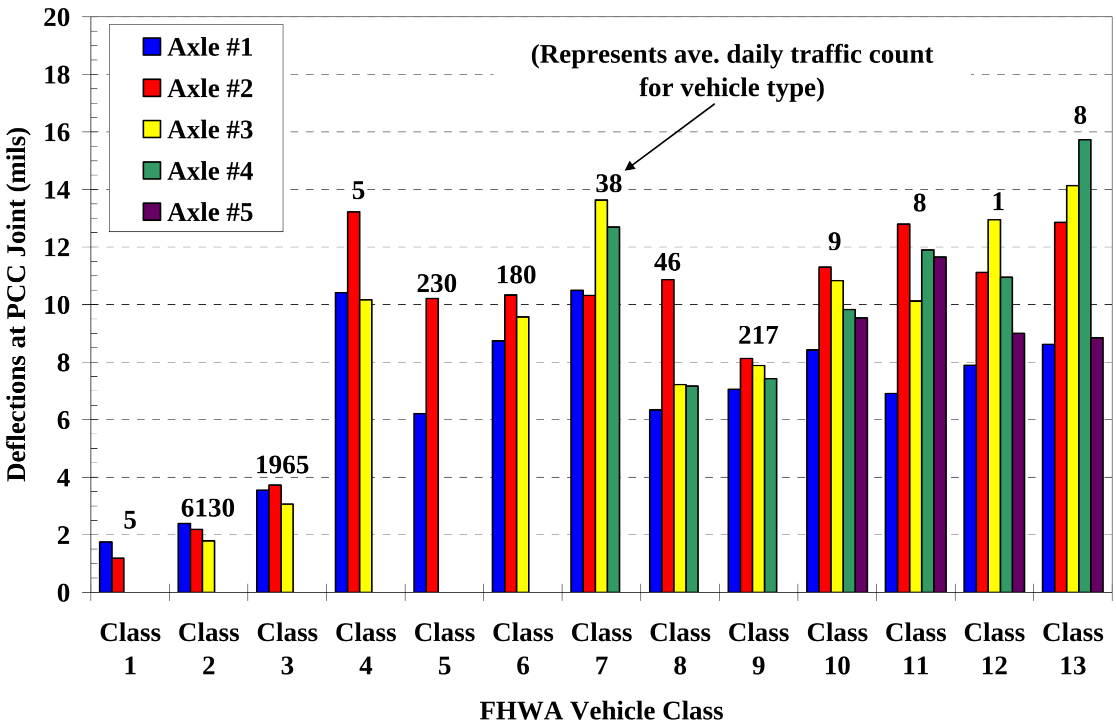



Figure 7.2 – PCC Vertical Joint Deflection Spectra for Test Section #1

The reflective cracking fatigue analysis using the Deflection Spectra Approach is based on the cumulative damage concept originally developed by Miner (1945), shown as Equation 7.1. The fatigue damage is calculated as the ratio of the predicted number of traffic repetitions to the allowable number of load repetitions. This is also the current fatigue cracking analysis format used in the Mechanistic Empirical Pavement Design Guide (Applied Research Associates, 2004).

$$D = \sum_{i=1}^{T} \frac{n_i}{N_i} \tag{7.1}$$

where:

$D$ = damage;
$T$ = total number of periods;
$n_i$ = actual traffic for period $i$; and
$N_i$ = allowable failure repetitions under conditions prevailing in period $i$.

The Deflection Spectra Approach relies on the following pieces of information:

1. Measurement of axle load spectra from WIM and AVC sensors.
2. Relationship between applied load and joint deflection.
3. Conversion of joint deflections to applied tensile micro-strain for the Flexural Beam Fatigue test.
4. Fatigue life relationship of asphalt mixtures using the Flexural Beam Fatigue test.

Both 1), 2) and 4) were described earlier. The conversion of joint deflections to applied tensile micro-strain is accomplished by using data calculations of the Flexural Beam Fatigue test device. For this study, it is the device manufactured by IPC. The equation that relates joint deflection to tensile micro-strain in the Flexural Beam Fatigue test is shown as Equation (7.2).

$$\varepsilon_t = \frac{12\delta h(1E6)}{\left(3G_O^2 - 4G_i^2\right)} \tag{7.2}$$

where:

$\varepsilon_t$ = applied tensile strain during Flexural Beam Fatigue test
d = peak deflection of beam (or joint deflection)
h = average height of Flexural Beam Fatigue specimen
$G_O$ = outer gauge length of Flexural Beam Fatigue test
$G_i$ = inner gauge length of Flexural Beam Fatigue test

The PCC Vertical Deflection Joint Spectra, as illustrated in Figure 7.2, is inputted into Equation 7.2 to convert the daily vertical joint deflections to applied tensile strains in the Flexural Beam Fatigue test. Once the tensile strains are determined, the tensile strains can then be inputted into the Fatigue Life relationship for the asphalt mixtures, described in Section 6.1, to determine the allowable number of failure repetitions ($N_i$). The master stiffness curve of the asphalt mixture, described in Section 6.3, is used to vary the asphalt mixture modulus with monthly temperature changes. The daily axle count is then used to determine $n_i$ for that respective applied axle load. This allows for the determination of the Damage Ratio ($n_i/N_i$). An example of a daily calculation for the 12.5mm Superpave mixture from Rt 34N New Jersey is shown as Table 7.1.

Table 7.1 – Deflection Spectra Approach Example Calculations

| FHWA Vehicle Class | # of Applied Loads per Axle ($n_i$) | Determined Joint Deflection (mils) | Applied Tensile Micro-strain (□) | Allowable # of Failure Repetitions ($N_i$) | Fatigue Damage Ratio, ($[n_i/N_i]$x 100) |
|---|---|---|---|---|---|
| Class 1 | 5 | 0.84 | 42.58 | 100,769,842,698 | 4.9618E-09 |
| | 5 | 0.92 | 46.72 | 60,251,665,892 | 8.29853E-09 |
| Class 3 | 1965 | 3.42 | 173.74 | 41,830,828 | 0.004697492 |
| | 1965 | 3.61 | 183.19 | 31,197,116 | 0.006298659 |
| | 1965 | 2.52 | 127.90 | 228,049,862 | 0.000861654 |
| Class 7 | 38 | 9.17 | 465.89 | 177,515 | 0.021406626 |
| | 38 | 9.82 | 498.67 | 121,824 | 0.03119258 |
| | 38 | 11.52 | 585.29 | 50,185 | 0.075719482 |
| | 38 | 10.50 | 533.27 | 84,024 | 0.045225192 |
| Class 9 | 217 | 7.06 | 358.54 | 757,074 | 0.028629971 |
| | 217 | 8.08 | 410.49 | 357,882 | 0.060564649 |
| | 217 | 7.82 | 397.44 | 427,964 | 0.050646777 |
| | 217 | 7.24 | 367.57 | 659,619 | 0.032859881 |
| Class 11 | 8 | 6.32 | 320.99 | 1,397,059 | 0.000593083 |
| | 8 | 12.27 | 623.31 | 35,418 | 0.023394085 |
| | 8 | 7.70 | 390.95 | 468,820 | 0.001767356 |
| | 8 | 7.89 | 400.63 | 409,461 | 0.002023565 |
| | 8 | 7.40 | 376.13 | 580,665 | 0.001426936 |

The last thing that needs to be considered prior to applying the Deflection Spectra Approach to the pavement section is the concept of residual deflection/strain. Residual deflection/strain represents the decrease in the vertical deflection at the PCC joint due to the asphalt overlay thickness and stiffness. For example, consider a PCC pavement has been overlaid with an asphalt overlay with two different HMA mixtures (1 layer each). Prior to the asphalt overlay, Falling Weight Deflectometer (FWD) testing as conducted at the PCC joint and a vertical joint deflection of 10 mils was measured. Due to the stiff, confining nature of the asphalt overlay, the vertical deflection at the PCC joint is no longer 10 mils, but a reduced amount dependent on the asphalt overlay thickness and general stiffness of the asphalt overlay.

To evaluate the reduction in vertical deflection at the PCC joint due to the asphalt overlay, a review of FWD vertical deflection records of composite pavements before and after an asphalt overlay was conducted. The resultant equation relating the remaining (called residual) vertical deflection versus asphalt overlay thickness is shown in Figure 7.3. Figure 7.3 clearly shows that as the asphalt overlay thickness increases, the residual vertical strain in the pavement (as measured from the surface of the asphalt overlay) decreases. Due to experimental error in the field measurements, the residual vertical strain does not converge back to 100% when a 0.0 inch asphalt overlay is applied. Therefore, the regression equation (Equation 7.3) shown in Figure 7.3 was used in the tensile strain calculations at the bottom of the individual asphalt lifts.

$$\text{Residual Vertical Strain (mils)} = -6.9458\left(\text{HMA Overlay Thickness}\right) + 93.374 \qquad (7.3)$$

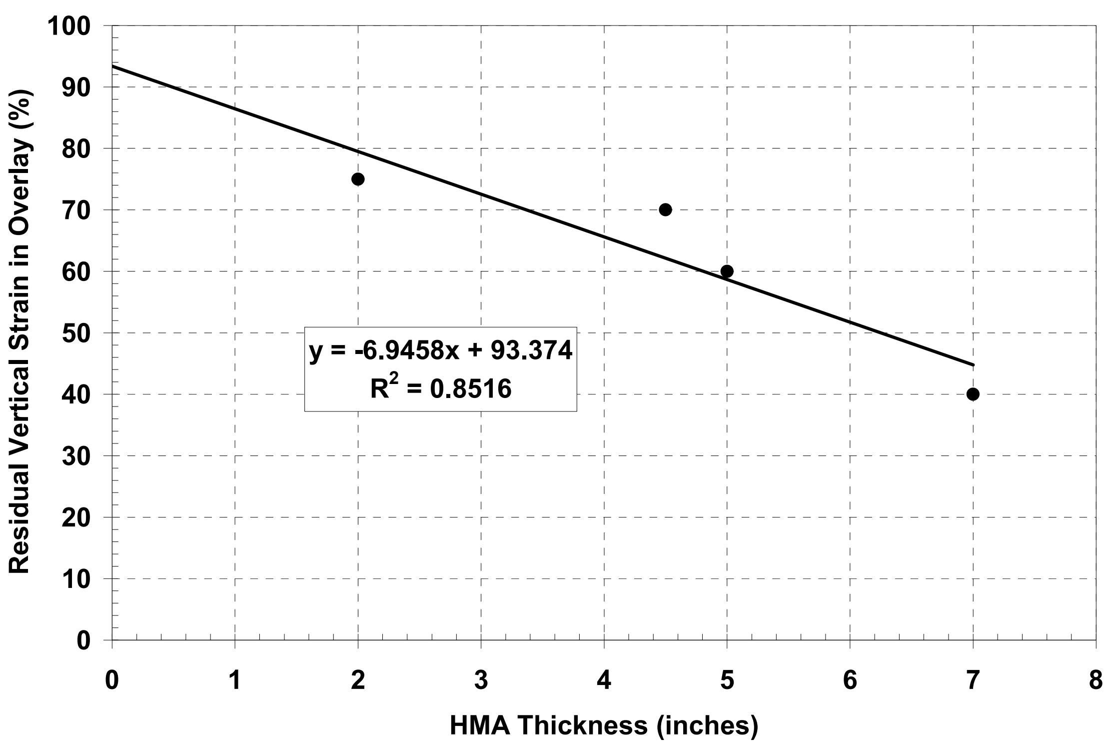



Figure 7.3 – Determination of Residual Vertical Strain from FWD Field Testing

The last concept that needs to be addressed is the "measured" cracking. Since reflective cracking commonly occurs from the bottom of the asphalt overlay and propagates upward to the surface, the "measured" or observed cracking is actually due to the fatigue cracking susceptibility of the surface or wearing course asphalt mixture. Therefore, it should be noted that the accuracy of the prediction methodology shown in the upcoming sections should be based on the comparison between the observed or measured reflective cracking to the predicted cracking of the surface or wearing course asphalt mixture.

### 7.1.1 Application of Deflection Spectra Approach – Rt 34N New Jersey

The Deflection Spectra Approach was used to evaluate the reflective cracking resistance of the asphalt mixtures on Rt 34N New Jersey. Three different sections were placed at the Rt 34N test section, shown below.

- Section #1: Milepost 0.3 to 2.5, consists of 25mm (1 inch) of a reflective crack relief interlayer mix (RCRI), overlaid by 50 mm (2 inches) of a 12.5mm Superpave HMA (NJDOT 12.5M76), which in turn is overlaid by 38.1 mm (1.5 inches) of a 9.5mm Superpave mix (NJDOT 9.5H76).
- Section #2: Milepost 2.5 to 4.5, consists of 76.2 mm (3 inches) of a 12.5mm Superpave HMA overlaid by 38.1 mm (1.5 inches) of a 9.5mm Superpave HMA.
- Section #3: Milepost 4.5 to 7.6, consists of 25mm (1 inch) of a reflective crack relief interlayer mix (RCRI), overlaid by 50 mm (2 inches) of a 12.5mm Superpave HMA, which in turn is overlaid by 38.1 mm (1.5 inches) of a 9.5mm Superpave mix.

Prior to the application of the asphalt overlay, the contractor milled off three (3) inches of the existing asphalt pavement to provide a level surface for new overlay. The constant milling depth resulted in areas where the existing asphalt pavement (new plus old) was variable. To determine the exact amount of asphalt overlay remaining, the Center for Advanced Infrastructure and Transportation (CAIT) at Rutgers University conducted Ground Penetrating Radar (GPR) testing. The results are shown in Figures 7.4 through 7.6 and were used in the calculation of the residual tensile strains in the Deflection Spectra Approach.

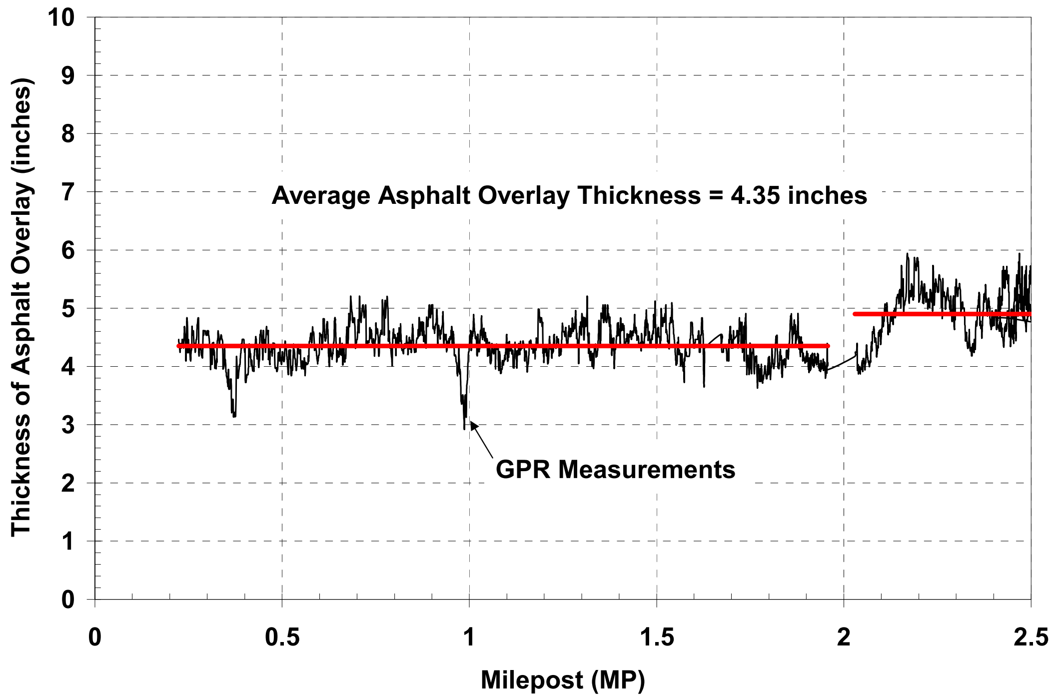



Figure 7.4 – Ground Penetrating Radar (GPR) Test Results for Section #1 of Rt 34N New Jersey

The presentation of the analysis is conducted in a manner that determines at what time a percent of the transverse joints show reflective cracking. To accomplish this, the PCC joint/crack vertical deflections are sorted in descending order. The time and magnitude of the transverse cracking will be dependent on the severity of the vertical deflections at that respective joint/crack (i.e. – the worst PCC joints will result in the earliest reflective cracking). Predetermined percentages of the transverse joints (i.e. – 2.5, 5, 10, 25, and 50%) are used for analysis and graphical presentation. As an example,

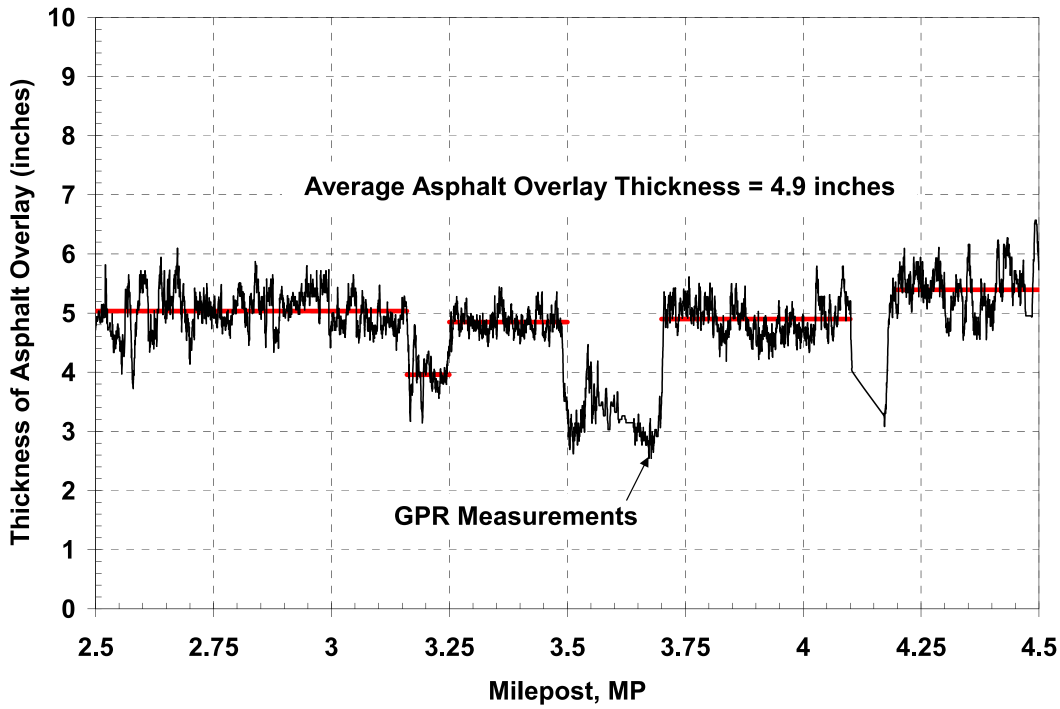



Figure 7.5 – Ground Penetrating Radar (GPR) Test Results for Section #2 of Rt 34N New Jersey

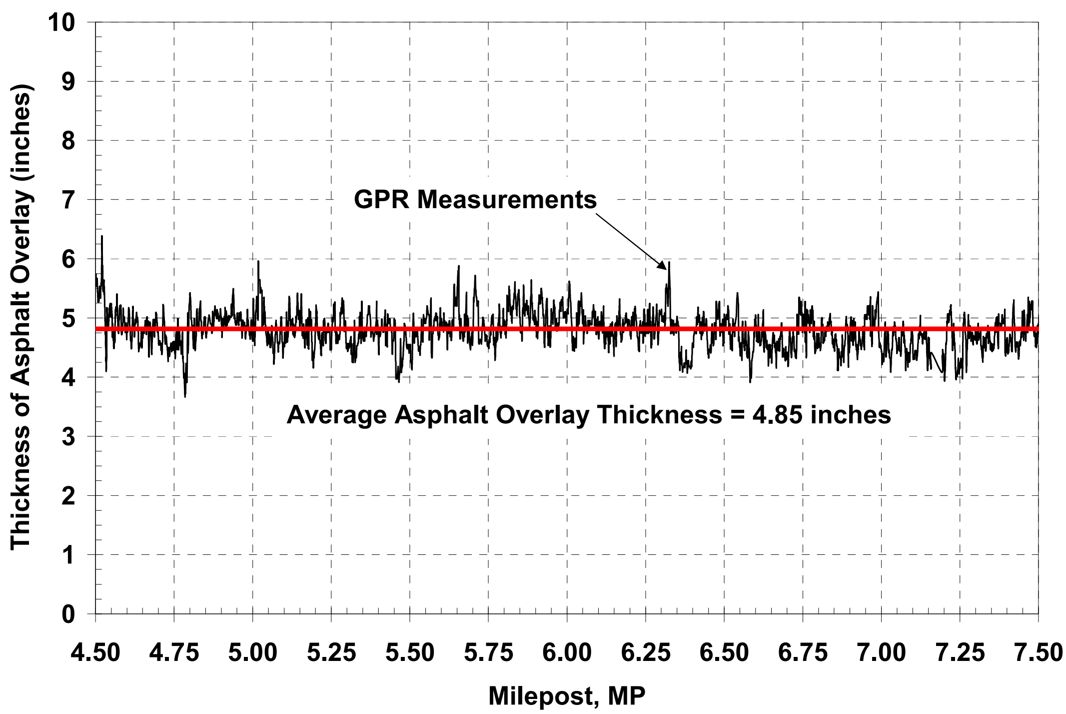



Figure 7.6 – Ground Penetrating Radar (GPR) Test Results for Section #3 of Rt 34N New Jersey

the time to when 2.5% of the transverse joints show reflective cracking are based on the average vertical deflection characteristics of the worst 5% of the PCC joints/cracks. An example of the Deflection Spectra Approach calculations are shown in Figure 7.7 for determining the time when 2.5% of the transverse joints will crack for the 12M76 asphalt mixture on Rt 34N New Jersey.

| | # of Vehicle | Load (kips) | (mils) | Micro-strain | Residual Strain | Fatigue Life | Fatigue Damage Ratio |
|---|---|---|---|---|---|---|---|
| Class 1 | 5 | 1.189 | 1.79 | **85** | **73** | 5,040,751,369 | 9.91916E-08 |
| | 5 | 1.311 | 1.93 | **91** | **79** | 3,389,970,828 | 1.47494E-07 |
| Class 2 | 6130 | 3.416 | 4.23 | **200** | **173** | 43,249,560 | 0.014172399 |
| | 6130 | 3.048 | 3.83 | **181** | **156** | 75,222,793 | 0.008148461 |
| | 6130 | 1.981 | 2.66 | **126** | **109** | 565,870,868 | 0.001083198 |
| Class 3 | 1965 | 5.034 | 6.01 | **284** | **245** | 6,229,459 | 0.031543671 |
| | 1965 | 5.311 | 6.31 | **298** | **257** | 4,740,830 | 0.041448436 |
| | 1965 | 3.690 | 4.54 | **214** | **185** | 29,568,236 | 0.006645645 |
| Class 4 | 5 | 13.292 | 15.06 | **711** | **614** | 38,376 | 0.013354582 |
| | 5 | 17.296 | 19.45 | **918** | **793** | 9,308 | 0.055059296 |
| | 5 | 12.942 | 14.68 | **693** | **599** | 44,264 | 0.011578183 |
| Class 5 | 230 | 8.759 | 10.09 | **476** | **412** | 352,287 | 0.06514583 |
| | 230 | 14.328 | 16.20 | **764** | **661** | 25,655 | 0.894569426 |
| Class 6 | 180 | 12.179 | 13.84 | **653** | **565** | 61,277 | 0.293952197 |
| | 180 | 13.661 | 15.47 | **730** | **631** | 33,130 | 0.54369623 |
| | 180 | 12.923 | 14.66 | **692** | **598** | 44,622 | 0.403665345 |
| Class 7 | 38 | 13.597 | 15.40 | **727** | **628** | 33,980 | 0.111829086 |
| | 38 | 14.558 | 16.45 | **776** | **671** | 23,555 | 0.161322118 |
| | 38 | 17.096 | 19.24 | **908** | **784** | 9,911 | 0.383393373 |
| | 38 | 15.572 | 17.56 | **829** | **716** | 16,398 | 0.231734829 |
| Class 8 | 46 | 9.040 | 10.40 | **491** | **424** | 298,337 | 0.015502603 |
| | 46 | 15.365 | 17.34 | **818** | **707** | 17,624 | 0.262433221 |
| | 46 | 10.047 | 11.51 | **543** | **469** | 170,660 | 0.027100626 |
| | 46 | 10.161 | 11.63 | **549** | **474** | 160,704 | 0.028779601 |
| Class 9 | 217 | 10.450 | 11.95 | **564** | **487** | 138,468 | 0.156533797 |
| | 217 | 11.973 | 13.62 | **643** | **555** | 67,111 | 0.322970692 |
| | 217 | 11.591 | 13.20 | **623** | **538** | 79,800 | 0.271616679 |
| | 217 | 10.715 | 12.24 | **577** | **499** | 121,229 | 0.17879326 |
| Class 10 | 9 | 11.423 | 13.01 | **614** | **531** | 86,237 | 0.010602066 |
| | 9 | 15.496 | 17.48 | **825** | **713** | 16,837 | 0.054302536 |
| | 9 | 15.160 | 17.11 | **807** | **698** | 18,941 | 0.048269355 |
| | 9 | 11.704 | 13.32 | **629** | **543** | 75,761 | 0.012068011 |
| | 9 | 12.116 | 13.77 | **650** | **562** | 62,975 | 0.014518245 |
| Class 11 | 8 | 9.350 | 10.74 | **507** | **438** | 249,716 | 0.003318053 |
| | 8 | 18.211 | 20.46 | **965** | **834** | 7,047 | 0.117571463 |
| | 8 | 11.400 | 12.99 | **613** | **530** | 87,160 | 0.009506339 |
| | 8 | 11.684 | 13.30 | **628** | **542** | 76,458 | 0.010836937 |
| | 8 | 10.966 | 12.51 | **590** | **510** | 107,187 | 0.007730177 |
| Class 12 | 1 | 10.133 | 11.60 | **547** | **473** | 163,058 | 0.000788501 |
| | 1 | 15.646 | 17.65 | **833** | **720** | 15,984 | 0.008043594 |
| | 1 | 11.971 | 13.62 | **642** | **555** | 67,173 | 0.001914048 |
| | 1 | 11.471 | 13.07 | **617** | **533** | 84,341 | 0.001524431 |
| | 1 | 9.912 | 11.36 | **536** | **463** | 183,263 | 0.000701569 |
| Class 13 | 8 | 11.110 | 12.67 | **598** | **517** | 99,972 | 0.007752159 |
| | 8 | 15.753 | 17.76 | **838** | **724** | 15,407 | 0.050303426 |
| | 8 | 14.181 | 16.04 | **757** | **654** | 27,114 | 0.028583092 |
| | 8 | 13.171 | 14.93 | **705** | **609** | 40,307 | 0.019227399 |
| | 8 | 8.494 | 9.80 | **463** | **400** | 414,164 | 0.001871237 |
| | | | | | **0** | | |
| | | | | | | **Percent of Fatigue Life Used (Daily) =** | **4.94550567** |
| | | | | | | **Days Until Cracking Observed (Days) =** | **20** |

Figure 7.7 – Example of Deflection Spectra Approach Calculations for Rt 34N New Jersey, Section #1

The final comparisons between the measured and predicted reflective cracking from the three different test sections on Rt 34N in New Jersey are shown in Figures 7.8 through 7.10. The results of the Deflection Spectra Approach show;

1. The predicted reflective cracking life of the 12M76 asphalt mixture is much faster than that of the measured field cracking. As discussed earlier, this is most likely due to the fact that the 12M76 mixture may have actually cracked earlier but it is obviously not possible to "look into the pavement" to validate.
2. The predicted reflective cracking life of the 9.5H76 asphalt mixture is in good comparison to the measured field cracking for all three test sections. If the Deflection Spectra Approach methodology is valid, good agreement between the surface course mixture (9.5H76) and the predicted results would be expected.
3. The test results clearly indicate the superior fatigue cracking performance of the reflective crack relief interlayer (RCRI) mixture over the conventional dense-graded asphalt mixtures.

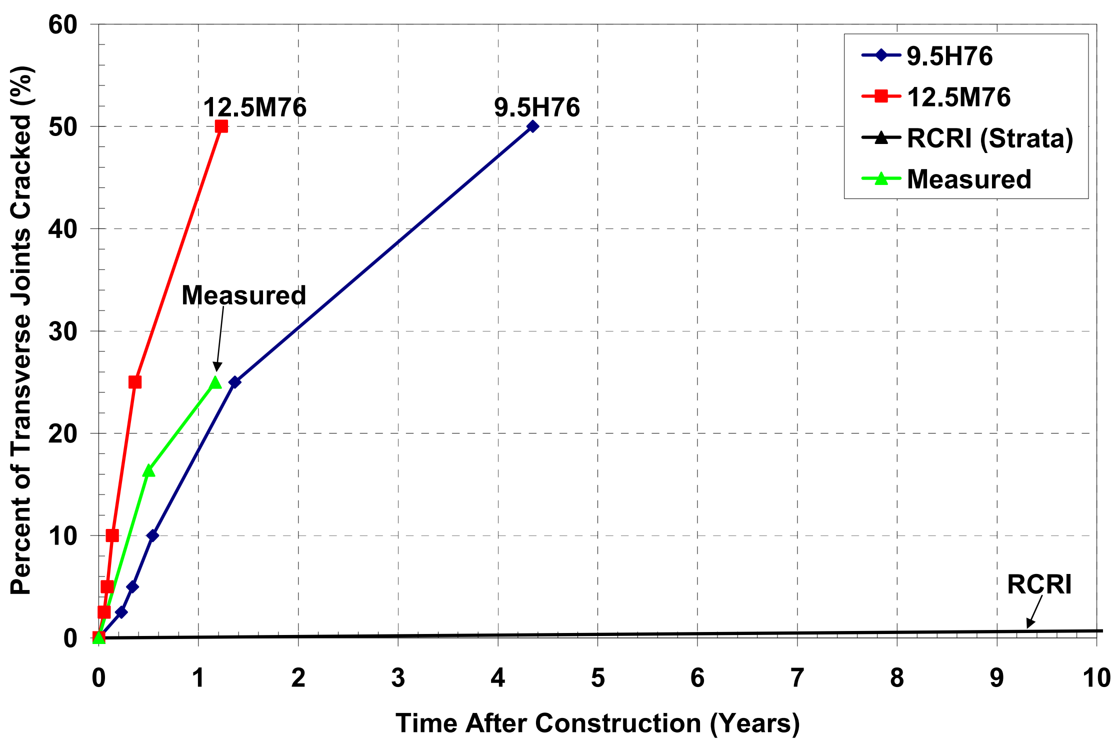



Figure 7.8 – Predicted versus Measured Reflective Cracking of Test Section #1 on Rt 34N New Jersey

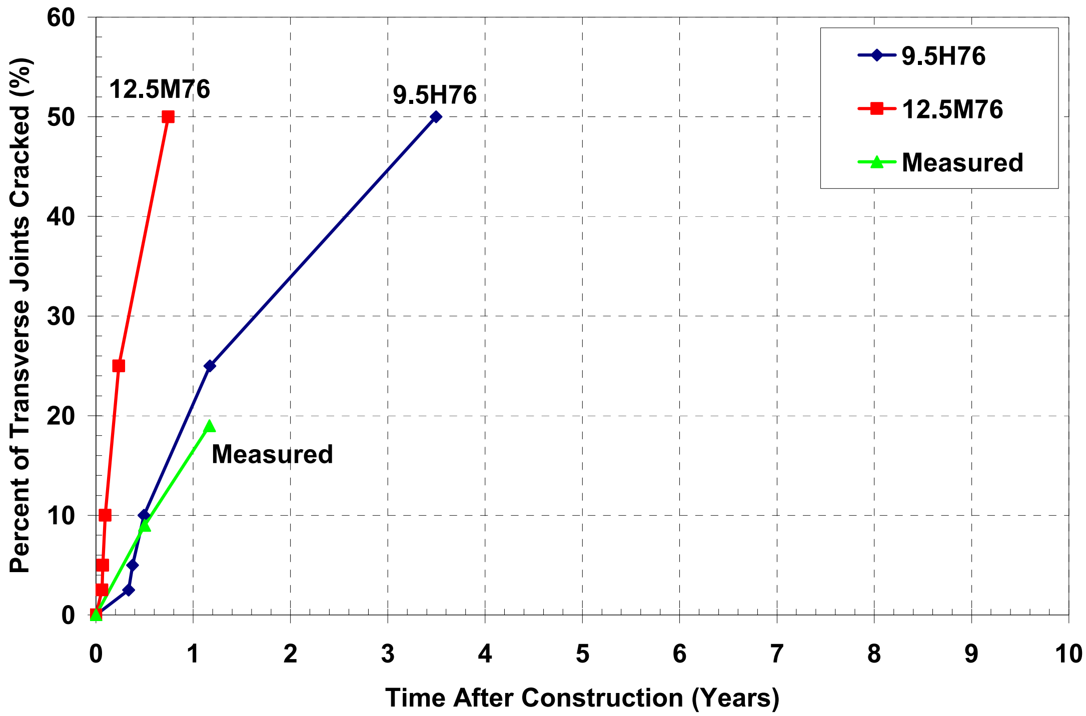



Figure 7.9 – Predicted versus Measured Reflective Cracking of Test Section #2 on Rt 34N New Jersey

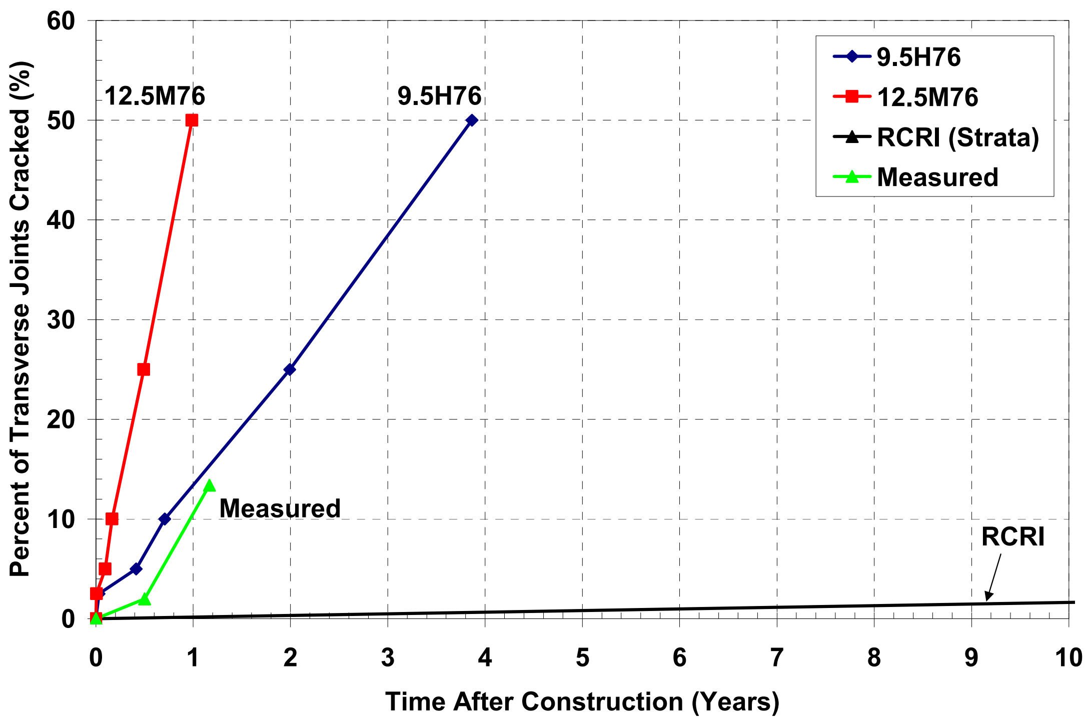



Figure 7.10 – Predicted versus Measured Reflective Cracking of Test Section #3 on Rt 34N New Jersey

### 7.1.2 Application of Deflection Spectra Approach – Rt 202S New Jersey

The Deflection Spectra Approach was used to evaluate the reflective cracking resistance of the asphalt mixtures on Rt 202S New Jersey. With the donation of experimental asphalt binder from SemMaterials, four experimental test sections were constructed on Rt 202S. The main purpose of the Rt 202S test sections was to evaluate more flexible overlay materials to overlay reflective crack relief interlayers (RCRI). In the case of Rt 202S, the RCRI mixture was the Strata product developed and manufactured by SemMaterials. Figure 7.11 shows the cross-sections of the different sections evaluated.

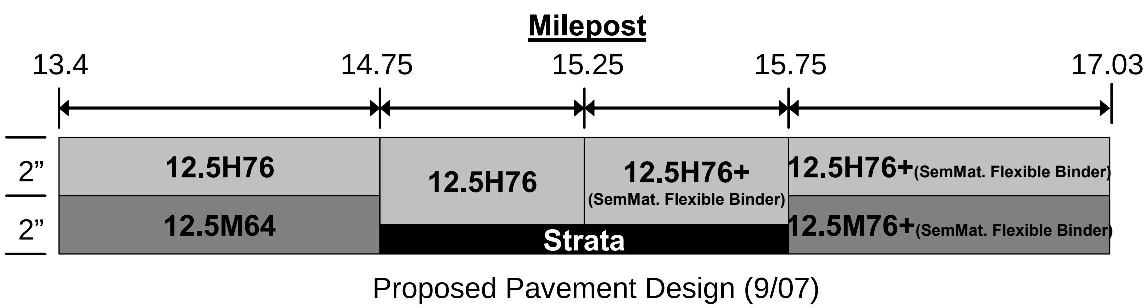



Figure 7.11 – Cross Section of Research Test Sections on Rt. 202S in New Jersey

Application of the asphalt overlay was placed directly on the existing PCC pavement. Therefore, no milling of any existing HMA surface was conducted resulting in the final asphalt overlay thickness solely being that of the new asphalt overlay.

The identical methodology previously discussed in Section 7.1.1 was utilized to evaluate the reflective cracking potential of the four different test sections on Rt 202S New Jersey. An example of the Deflection Spectra Approach calculations are shown in Figure 7.12 for determining the time when 2.5% of the transverse joints will crack for the 12H76 asphalt mixture on Rt 202S New Jersey (within MP 14.75 to 15.25).

| | # of Vehicle | Load (kips) | (mils) | Micro-strain | Residual Strain | Fatigue Life | Fatigue Damage Ratio |
|---|---|---|---|---|---|---|---|
| Class 1 | 12 | 1.148 | 2.09 | **98** | **85** | 1.71E+41 | 7.01542E-39 |
| | 12 | 1.323 | 2.29 | **108** | **94** | 1.06E+41 | 1.13149E-38 |
| Class 2 | 8381 | 1.793 | 2.85 | **135** | **116** | 3.53E+40 | 2.37136E-35 |
| | 8381 | 2.095 | 3.21 | **151** | **131** | 1.95E+40 | 4.30628E-35 |
| | 8381 | 1.781 | 2.84 | **134** | **116** | 3.63E+40 | 2.30878E-35 |
| Class 3 | 1545 | 1.318 | 2.29 | **108** | **93** | 1.07E+41 | 1.43841E-36 |
| | 1545 | 1.345 | 2.32 | **109** | **95** | 1.00E+41 | 1.54048E-36 |
| | 1545 | 1.748 | 2.80 | **132** | **114** | 3.89E+40 | 3.97123E-36 |
| Class 4 | 7 | 11.619 | 14.50 | **684** | **591** | 9.57E+36 | 7.31256E-35 |
| | 7 | 20.448 | 24.97 | **1178** | **1018** | 6.15E+35 | 1.13773E-33 |
| | 7 | 11.263 | 14.08 | **664** | **574** | 1.11E+37 | 6.30029E-35 |
| Class 5 | 224 | 9.141 | 11.56 | **546** | **472** | 3.00E+37 | 7.45821E-34 |
| | 224 | 14.433 | 17.84 | **842** | **727** | 3.36E+36 | 6.65843E-33 |
| Class 6 | 84 | 11.086 | 13.87 | **654** | **566** | 1.20E+37 | 7.00716E-34 |
| | 84 | 13.997 | 17.32 | **817** | **706** | 3.90E+36 | 2.15292E-33 |
| | 84 | 13.257 | 16.44 | **776** | **671** | 5.07E+36 | 1.65528E-33 |
| Class 7 | 11 | 10.143 | 12.75 | **602** | **520** | 1.83E+37 | 6.00315E-35 |
| | 11 | 13.510 | 16.74 | **790** | **683** | 4.63E+36 | 2.37554E-34 |
| | 11 | 10.991 | 13.75 | **649** | **561** | 1.25E+37 | 8.80486E-35 |
| | 11 | 9.813 | 12.36 | **583** | **504** | 2.14E+37 | 5.12835E-35 |
| Class 8 | 147 | 10.570 | 13.26 | **626** | **541** | 1.51E+37 | 9.76524E-34 |
| | 147 | 14.041 | 17.37 | **820** | **708** | 3.84E+36 | 3.82484E-33 |
| | 147 | 12.149 | 15.13 | **714** | **617** | 7.73E+36 | 1.90278E-33 |
| | 147 | 11.318 | 14.14 | **667** | **577** | 1.09E+37 | 1.35399E-33 |
| Class 9 | 148 | 10.619 | 13.31 | **628** | **543** | 1.47E+37 | 1.00513E-33 |
| | 148 | 13.671 | 16.93 | **799** | **691** | 4.37E+36 | 3.38424E-33 |
| | 148 | 13.633 | 16.89 | **797** | **689** | 4.43E+36 | 3.33851E-33 |
| | 148 | 11.967 | 14.91 | **704** | **608** | 8.31E+36 | 1.78153E-33 |
| Class 10 | 5 | 10.712 | 13.42 | **633** | **547** | 1.41E+37 | 3.53942E-35 |
| | 5 | 13.953 | 17.27 | **815** | **704** | 3.96E+36 | 1.26206E-34 |
| | 5 | 14.682 | 18.13 | **856** | **739** | 3.10E+36 | 1.61476E-34 |
| | 5 | 10.447 | 13.11 | **619** | **535** | 1.59E+37 | 3.14124E-35 |
| | 5 | 10.141 | 12.75 | **602** | **520** | 1.83E+37 | 2.72604E-35 |
| Class 11 | 4 | 10.348 | 12.99 | **613** | **530** | 1.67E+37 | 2.40044E-35 |
| | 4 | 14.315 | 17.70 | **835** | **722** | 3.50E+36 | 1.14298E-34 |
| | 4 | 9.946 | 12.52 | **591** | **510** | 2.01E+37 | 1.98833E-35 |
| | 4 | 8.462 | 10.76 | **508** | **439** | 4.32E+37 | 9.24971E-36 |
| | 4 | 9.149 | 11.57 | **546** | **472** | 2.99E+37 | 1.33719E-35 |
| Class 12 | 7 | 11.244 | 14.05 | **663** | **573** | 1.12E+37 | 6.24717E-35 |
| | 7 | 12.286 | 15.29 | **722** | **624** | 7.32E+36 | 9.56296E-35 |
| | 7 | 11.468 | 14.32 | **676** | **584** | 1.02E+37 | 6.8693E-35 |
| | 7 | 11.385 | 14.22 | **671** | **580** | 1.06E+37 | 6.63178E-35 |
| | 7 | 12.564 | 15.62 | **737** | **637** | 6.57E+36 | 1.06496E-34 |
| Class 13 | 18 | 9.811 | 12.36 | **583** | **504** | 2.15E+37 | 8.38235E-35 |
| | 18 | 13.890 | 17.19 | **811** | **701** | 4.05E+36 | 4.44396E-34 |
| | 18 | 9.998 | 12.58 | **593** | **513** | 1.96E+37 | 9.16889E-35 |
| | 18 | 9.323 | 11.78 | **556** | **480** | 2.74E+37 | 6.581E-35 |
| | 18 | 9.502 | 11.99 | **566** | **489** | 2.50E+37 | 7.20064E-35 |
| | | | | | **0** | | |
| | | | | | **Percent of Fatigue Life Used (Daily) =** | | **3.30082E-32** |
| | | | | | **Days Until Cracking Observed (Days) =** | | **3.03E+33** |

Figure 7.12 - Example of Deflection Spectra Approach Calculations for Rt 202S New Jersey, Section MP 14.75 to 15.25

The final comparisons between the measured and predicted reflective cracking from the three different test sections on Rt 202S in New Jersey are shown in Figures 7.13 through 7.16. The results of the Deflection Spectra Approach show;

1. The predicted reflective cracking performance and the measured results were identical. The Deflection Spectra Approach indicates that all four of the test sections should not result in reflective cracking failure (i.e. – transverse cracking over the existing PCC joints/cracks). The analysis results coincide with the visual distress surveys conducted at Rt 202S New Jersey.

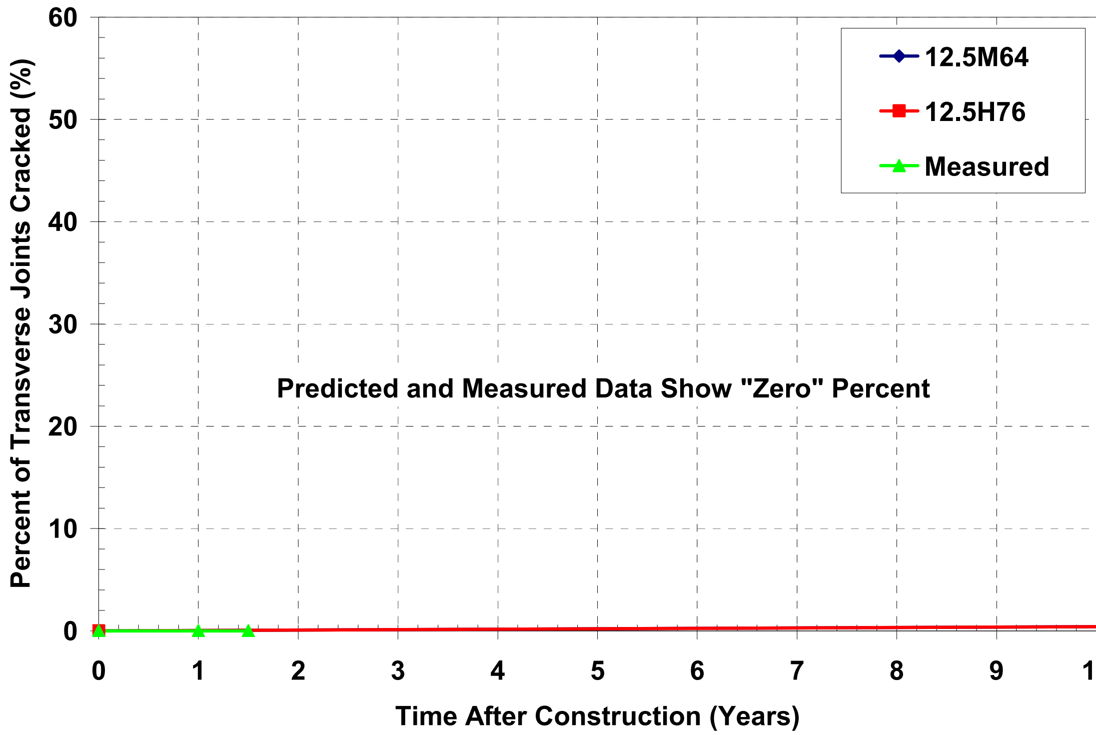



Figure 7.13 - Predicted versus Measured Reflective Cracking of Test Section mp 13.4 to 14.75 of Rt 202S New Jersey

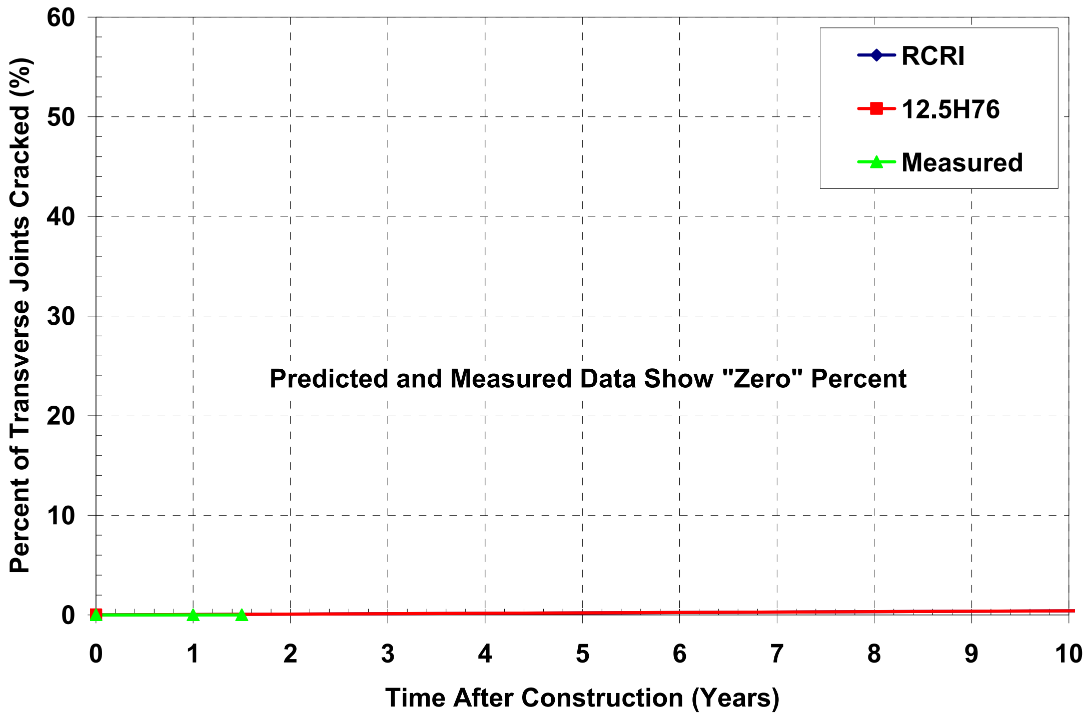



Figure 7.14 – Predicted versus Measured Reflective Cracking of Test Section MP 14.75 to 15.25 of Rt 202S New Jersey

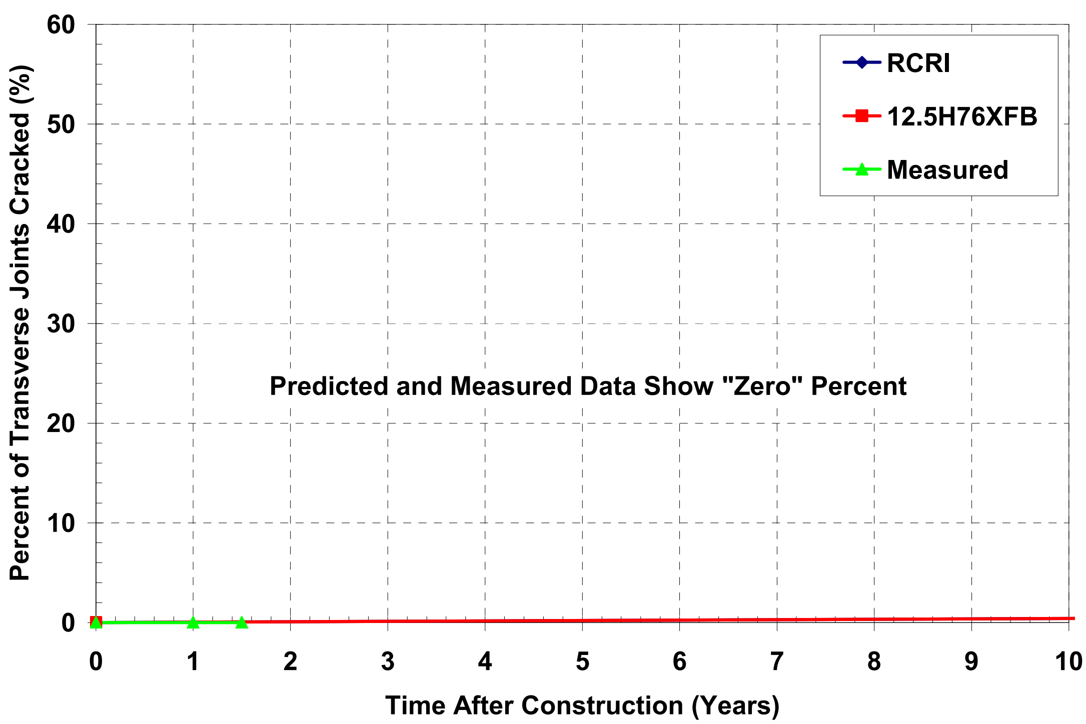



Figure 7.15 – Predicted versus Measured Reflective Cracking of Test Section MP 15.25 to 15.75 of Rt 202S New Jersey

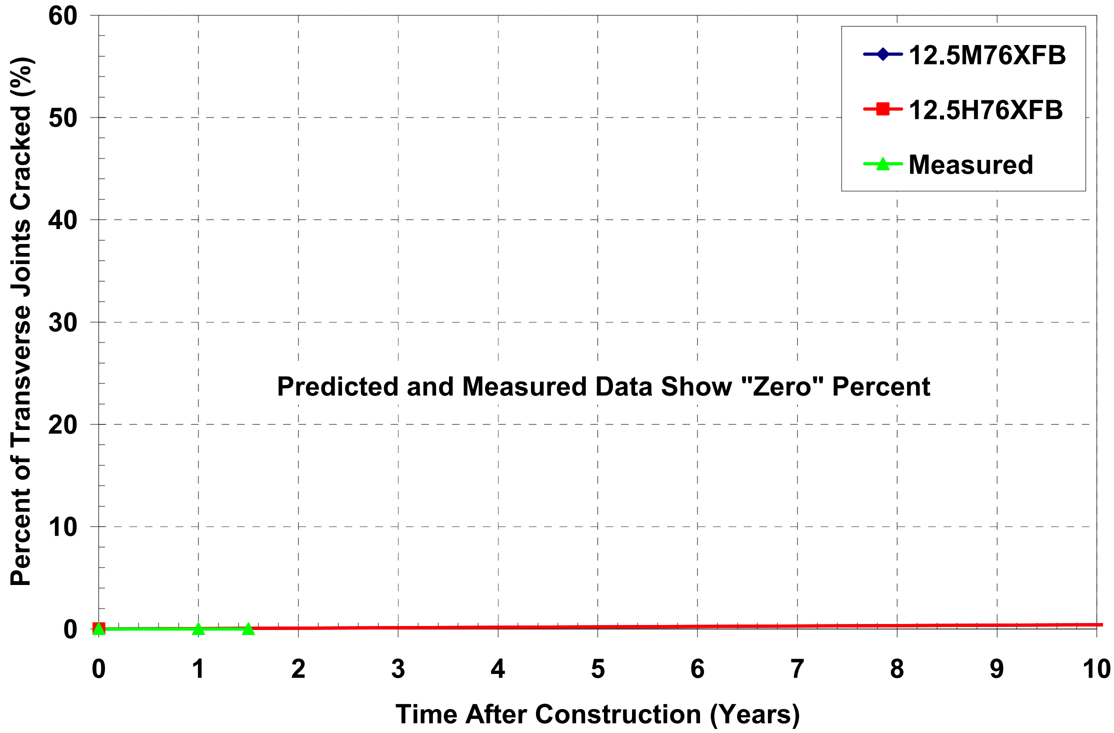



Figure 7.16 – Predicted versus Measured Reflective Cracking of Test Section MP 15.75 to 17 of Rt 202S New Jersey

7.1.3 Application of Deflection Spectra Approach for I495 Massachusetts

The Deflection Spectra Approach was applied to two (2) test sections on Interstate 495 in Massachusetts. The first test section consisted of only a 2" lift of a 19mm dense-graded HMA with 30% RAP and a PG52-33 + SBR 3% Latex asphalt binder. The second test section consisted of an asphalt overlay of five inches comprising of the following:

- Leveling Course: 2" of 19mm dense-graded HMA, 30% RAP, PG52-33 + SBR 3% Latex;
- Reflective Crack Relief Interlayer (MassHighway Stress-Absorbing Membrane, SAMI): 1" of SAMI; and
- Intermediate Course: 2" of 19mm dense-graded HMA, 30% RAP, PG52-33 + 3% SBR Latex.

Unlike the previous two pavements (Rt 34N and Rt 202S), the Massachusetts State Highway Association (MassHighway) did not have axle load spectra available for the traffic analysis. Instead, MassHighway had daily Equivalent Single Axle Load (ESAL's) daily counts. This simplifies the Deflection Spectra Approach calculations as only an 18,000 lb axle load is utilized in the calculations. Both test sections were placed immediately on the existing PCC pavement. Therefore, no asphalt overlay thickness corrections were required to for the Deflection Spectra calculations.

The Deflection Spectra Approach analysis for the 2-Inch and 5-Inch test sections on I495 in Massachusetts is shown in Figures 7.17 and 7.18. The results show;

1. Both the Deflection Spectra Approach and field measurements for the 2-Inch test section indicate the Leveling course performed poorly in fatigue. The predicted value of 95.5% of the transverse joints failing in fatigue cracking matches closely with the 77.6% measured by the Massachusetts Department of Transportation.
2. For the 5-Inch test section, the Leveling course fails very quickly compared to the RCRI mixture and Intermediate course. This is primarily due to the materials location in the immediate vicinity of the PCC joint/crack. The RCRI mixture shows almost an infinite fatigue life where the mixture would appear to never crack under the loading and deflection conditions on I495 in Massachusetts. Meanwhile, the predicted cracking of the Intermediate course compares favorably to the measured field cracking. As mentioned earlier, if the Deflection Spectra Approach provides accurate results, then the measured field cracking should correspond to the asphalt layer placed on the surface. In the case of I495, this was the Intermediate course as the surface course was scheduled to be placed in the following paving season.
3. The rankings of the 5-Inch test section match the visual observations of field cores that were taken through the surface cracks (Figure 7.19). As shown in Figure 7.19, the Leveling and Intermediate course mixes have cracked completely through each lift while the RCRI layer is completely intact. This is a classical example of "Crack Jumping" where the reflective cracking has actually "jumped" over the RCRI layer and propagated into the layer overlaying it. In this case of I495 in Massachusetts, cracking occurred in the Leveling course first and then the reflective crack "jumped" the RCRI layer into the Intermediate course where it propagated to the pavement surface.

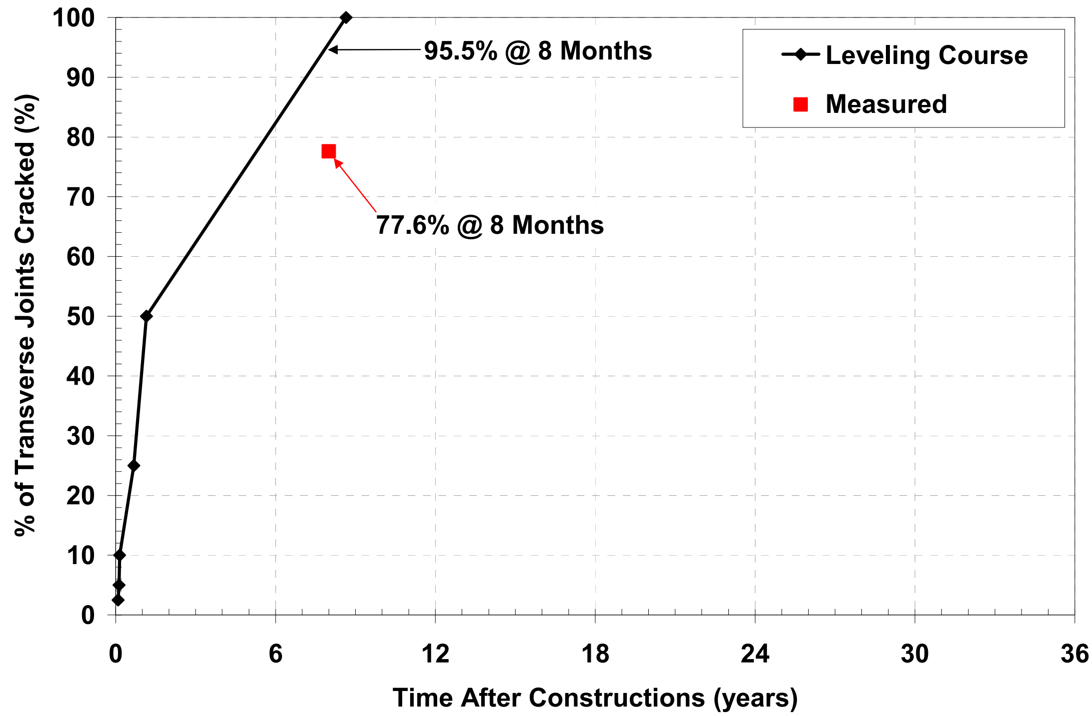



Figure 7.17 – Predicted versus Measured Reflective Cracking of the 2-Inch Overlay Test Section on I495 in Massachusetts

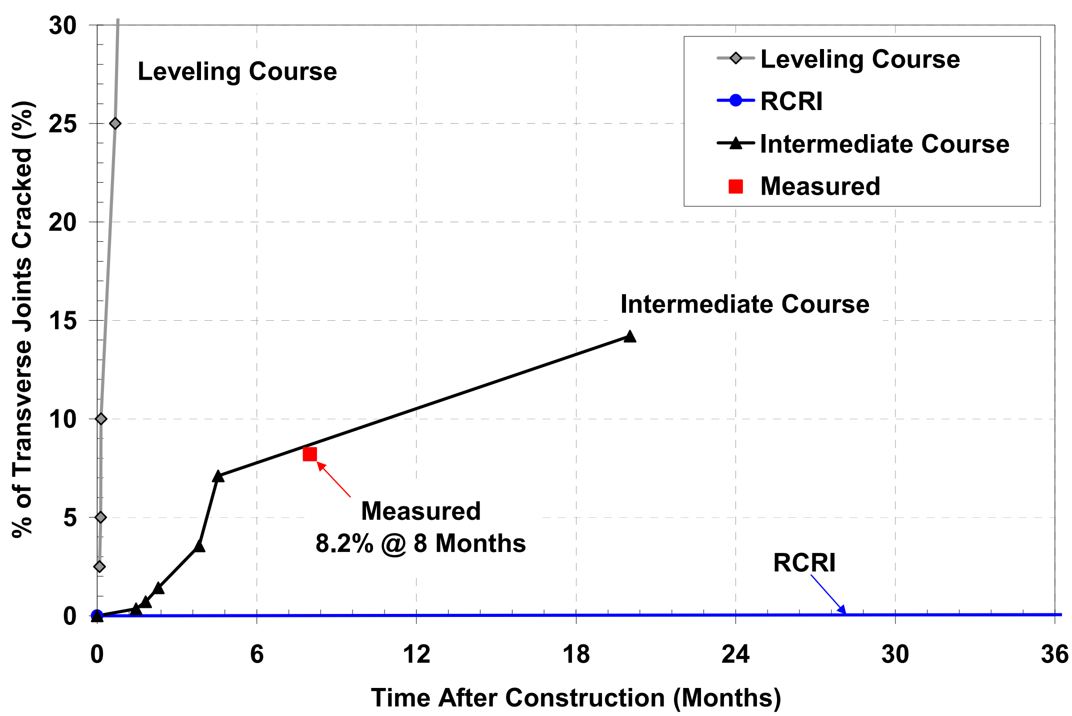



Figure 7.18 – Predicted versus Measured Reflective Cracking of the 5-Inch Overlay Test Section on I495 in Massachusetts

Figure 7.19 – Photo of Core Taken Through Surface Crack on I495 in Massachusetts

### 7.1.4 Application of Deflection Spectra Approach for I476 Pennsylvania

The Deflection Spectra Approach was used to evaluate the reflective cracking potential of the asphalt overlay placed on I476 in Pennsylvania. I476 in Pennsylvania contained two different test sections where different asphalt mixtures were used as a surface course; 75 gyration design and 100 gyration design. Unfortunately, the I476 pavement section had the least amount of Falling Weight Deflectometer testing conducted when compared to any of the other test sections. Not to mention, minimal asphalt material was available for testing, therefore, the flexural beam fatigue data collected for the analysis was actually provided by SemMaterials.

The comparison of the measured field cracking and the Deflection Spectra predicted results are shown in Figures 7.20 and 7.21 for the 100 Gyration surface course mix and 75 Gyration surface course mix. The analysis and field results indicate;

1. The Leveling course mix for the test sections should crack relatively early when compared to the RCRI and Surface course mixes. The Deflection Spectra Approach predicted that cracking would occur at 100% of the transverse joints/crack area in the Leveling course within the first 2 years.
2. Both the Measured and Predicted reflective cracking in the 75 Gyration design Surface were significantly lower than the 100 Gyration design Surface course mix. This clearly indicates the importance of utilizing asphalt mixture with better flexural fatigue resistance, as shown earlier in Figure 6.5. As in the case for the 100 and 75 Gyration design mixes, the reduction of 25 design gyrations in the gyratory compactor increased the asphalt content of the surface course mix 0.3%.
3. The RCRI mixture had superior fatigue resistance than the conventional dense-graded asphalt mixtures used as the Leveling and Surface courses.
4. Overall, the Deflection Spectra and Measured results compared favorably to one another, with the Deflection Spectra results over-predicting the time to reflective cracking on I476 in Pennsylvania. Again, this was most likely due to a lack of FWD vertical deflection data, as well as lack of asphalt material for mixture testing.

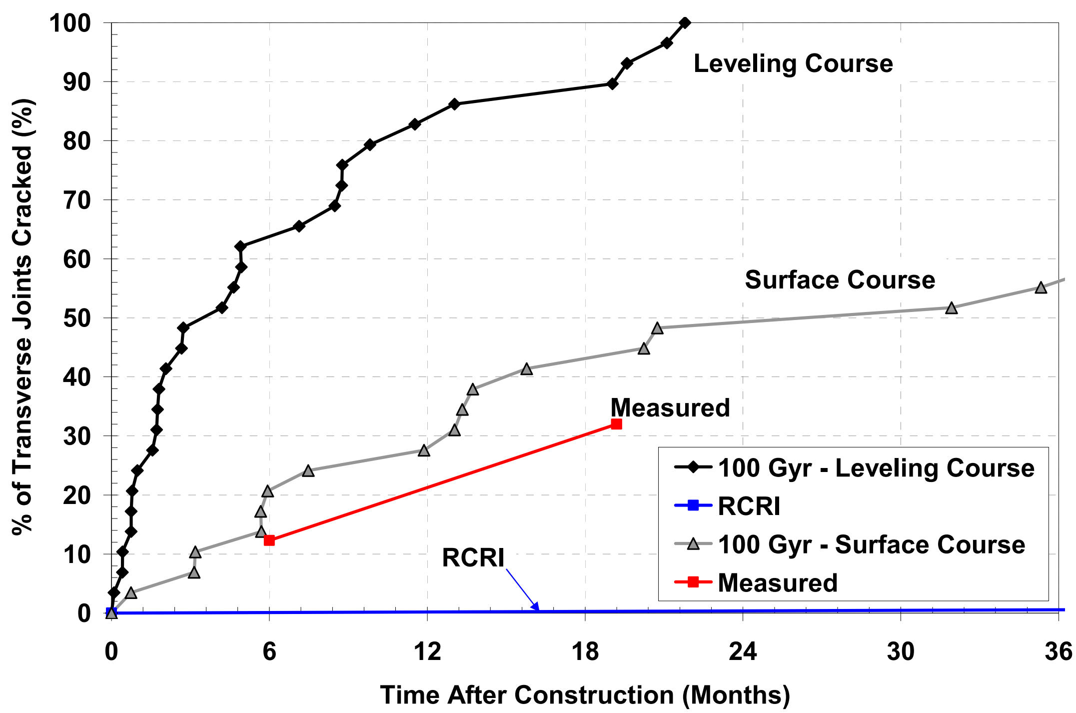



Figure 7.20 – Predicted versus Measured Reflective Cracking of the 100 Gyration Surface Course Mix on I476 in Pennsylvania

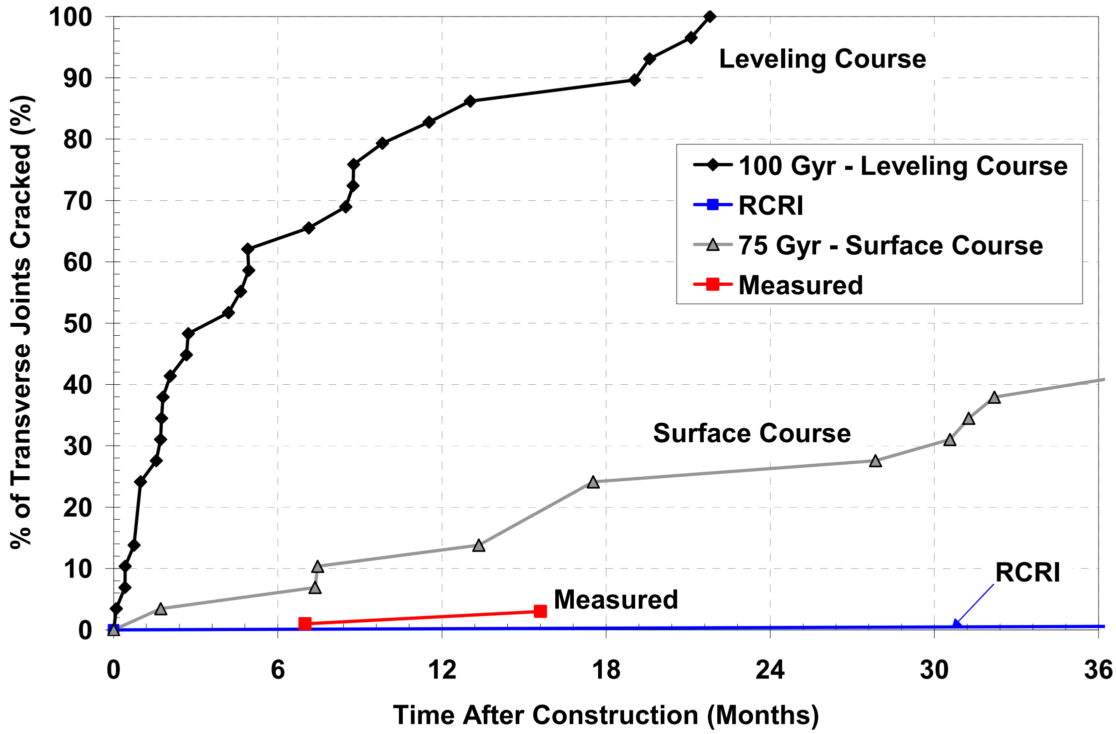



Figure 7.21 – Predicted versus Measured Reflective Cracking of the 75 Gyration Surface Course Mix on I476 in Pennsylvania

7.1.5 Summary of Deflection Spectra Approach Results

A comparison of the Deflection Spectra Approach and Measured reflective cracking for the eleven (11) test sections was conducted to determine the relative accuracy of the proposed methodology. The data comparison was conducted for each of the test section locations using the measured percent (%) of transverse joints cracked at the last time of measurement.

The results of the Deflection Spectra Approach and Measured reflective cracking are shown in Table 7.2, as well as the calculated Percent (%) Difference between the Deflection Spectra Approach and the Measured results. Table 7.2 indicates that if all of the test sections were used, the average percent difference between the Deflection Spectra

Approach predicted and measured is 57%. However, if the I476, 75 Gyration Surface Course section is eliminated assuming it to be an outlier, the average percent difference drops to 9.3%, which is very good when comparing the fracture/cracking resistance of asphalt mixtures.

Table 7.2 – Comparison Between the Deflection Spectra Approach and Measured Reflective Cracking of Eleven (11) Test Sections in Study

| % of Transverse Joints Cracked | | % Difference from Measured | Test Section |
|---|---|---|---|
| Deflection Spectra | Measured | | |
| 20.5 | 25 | -18.0 | Rt 34N, Section #1 |
| 25 | 19 | 31.6 | Rt 34N, Section #2 |
| 15 | 13.4 | 11.9 | Rt 34N, Section #3 |
| 0 | 0 | 0.0 | Rt 202S, MP 13.4 to 14.75 |
| 0 | 0 | 0.0 | Rt 202S, MP 14.75 to 15.25 |
| 0 | 0 | 0.0 | Rt 202S, MP 15.25 to 15.75 |
| 0 | 0 | 0.0 | Rt 202S, MP 15.75 to 17 |
| 95.5 | 77.6 | 23.1 | I495, 2-Inch Section |
| 8.8 | 8.2 | 7.3 | I495, 5-Inch Section |
| 44 | 32 | 37.5 | I476, 100 Gyration Surface Section |
| 19 | 3 | 533.3 | 1476, 75 Gyration Surface Section |

## 7.2 Horizontal Deflection Mode – TTI Overlay Tester Analysis

The TTI Overlay Tester has the capability of measuring the fatigue cracking resistance of hot mix asphalt specimens under temperature and deformation characteristics similar to field conditions. The horizontal deflection mode of reflective cracking is dictated by the expansion and contraction movements of the PCC slabs due to temperature cycling, and can be calculated using Equation (7.3).

$$\Delta L = CTE(L_{eff})(\Delta T)(\beta) \tag{7.3}$$

where,

CTE = coefficient of thermal expansion
$\Delta T$ = maximum 24-hour temperature difference
$\beta$ = PCC/Base friction factor
$L_{eff}$ = effective PCC joint spacing
$\Delta L$ = expected horizontal movement at the PCC slab joint due to daily temperature changes

In this scenario, the most critical condition would be when the temperature is already cold and there is a cooling cycle (i.e. – 4:00PM to 4:00AM in the month of February) (Bozkurt and Buttlar, 2002). And since the expansion and contraction is dependent on the temperature change, the same composite pavement with a thicker HMA overlay will expand and contract less due to the affect of thermal insulation. One of the difficulties in utilizing Equation 6.4 is the determining the temperature of the asphalt material at the surface of exiting PCC pavement, as well as determining the maximum temperature difference within a 24-hour time period. In substitution of actual field measurements, an alternative prediction methodology currently being used in the Mechanistic Empirical Pavement Design Guide (MEPDG) can be utilized.

The Enhanced Integrated Climatic Model (EICM) is a one-dimensional coupled heat and moisture flow model initially developed for the FHWA and adapted for use in the *Mechanistic-Empirical Pavement Design Guide* (MEPDG) developed under NCHRP Project 1-37A. In the MEPDG, the EICM is used to predict or simulate the changes in behavior and characteristics of pavement and unbound materials in conjunction with environmental conditions over many years of service. The research conducted in

NCHRP Project 9-23, "Environmental Effects in Pavement Mix and Structural Design Systems," validated the temperature profile and moisture predictive capabilities of the EICM (Zapata and Houston, 2008).

The PCC/base friction factor, β, in Equation 6.4 adjusts the unrestrained movement of a slab at a joint to a lower value as a result of slab base friction. Friction coefficients calculated during FHWA-RD-02-088, *Evaluation of Joint and Crack Load Transfer* (Khazanovich and Gotlif, 2003) are shown in Figure 7.22. The PCC/base friction factor coefficients were determined for nine PCC LTPP test sections. One can observe that only one section (133019) resulted in a very low friction factor. For all other sections, the friction factor ranges from 0.34 to 0.8. By utilizing pavement sections in close vicinity to the pavement sections in this study (Ohio - 390204 and Pennsylvania - 421606), an average PCC/base friction factor of 0.76 is calculated and can be used for determining the horizontal deformation test criteria in the TTI Overlay Tester.

| Section | State | Base Type | Coefficient of Thermal Expansion, mm/mm/°C (inch/inch/°F) | Joint Spacing, m (ft) | Effective Joint Spacing, m (ft) | Slope k mm/°C (inch/°F) | Friction Factor |
|---|---|---|---|---|---|---|---|
| 133019 | Georgia | AGG | 1.00 (0.56) | 6 .00(19.68) | 6.0 (19.68) | 0.0068 (0.00015) | 0.11 |
| 204054 | Kansas | CTB | 1.09 (0.61) | 9.00 (29.52) | 4.5 (14.76) | 0.0111 (0.00024) | 0.40 |
| 274040 | Minnesota | AGG | 1.02 (0.57) | 8.10 (26.57) | 4.05 (13.28) | 0.0161 (0.00035) | 0.69 |
| 390204 | Ohio | AGG | 0.95 (0.53) | 4.50 (14.76) | 4.5 (14.76) | 0.0194 (0.00043) | 0.81 |
| 421606 | Pennsylvania | AGG | 1.24 (0.69) | 13.95 (45.76) | 6.9 (22.63) | 0.0339 (0.00075) | 0.70 |
| 484142 | Texas | AGG | 0.95 (0.53) | 18.15 (59.53) | 6.0 (19.68) | 0.0111 (0.00024) | 0.35 |
| 484143 | Texas | CTB | 0.95 (0.53) | 18.15 (59.53) | 6.0 (19.68) | 0.0172 (0.00038) | 0.53 |
| 493011 | Utah | CTB | 0.95 (0.53) | 4.50 (14.76) | 4.5 (14.76) | 0.0139 (0.00031) | 0.58 |
| 833802 | Manitoba | CTB | 1.03 (0.58) | 4.50 (14.76) | 4.5 (14.76) | 0.0149 (0.00033) | 0.57 |

1 mm/mm/°C = 0.56 inch/inch/°F
1 mm/°C = 0.22 inch/°F

Figure 7.22 – PCC/Base Friction Factors for SMP LTPP Sections (Khazanovich and Gotlif, 2003)

### 7.2.1 Review of TTI Overlay Tester Results

A review of the TTI Overlay Tester results was conducted to evaluate the general material fracture properties under simulated field conditions of the respective test section. The results are shown below (Table 7.3). It is clear from the testing that the RCRI mixtures have superior fracture resistance performance over the conventional dense-graded mixtures. Based on the presented test results, it is clear that conventional dense graded asphalt mixture will have difficulty surviving the expected horizontal deflections at the PCC joint/crack area. However, the final performance of the dense-graded mixtures, with respect to the horizontal movement at the PCC joint/crack, will obviously depend on the effective PCC slab length, 24 hour temperature change at the surface of the PCC, and the coefficient of thermal expansion of the PCC.

Table 7.3 – Overall Results of Horizontal Deflection Testing at In-Situ Conditions

| Test Section Location | Predicted Horizontal Joint Movement | Simulated Performance (TTI Overlay Tester) | | | | |
|---|---|---|---|---|---|---|
| Rt 34N, New Jersey | 0.026 inches | 12.5M76<br>22 | 9.5H76<br>24 | RCRI<br>46,502 | N.A. | N.A. |
| Rt 202S, New Jersey | 0.026 inches | 12M64<br>41 | 12H76<br>26 | 12H76XFB<br>302 | 12M76XFB<br>568 | RCRI<br>3,205 |
| I495, MA | | 19mm Leveling | RCRI | | | |
| 2-Inch Overlay | 0.029 inches | 3 | 1,071 | N.A. | N.A. | N.A. |
| 5-Inch Overlay | 0.021 inches | 44 | 4,280 | | | |
| I476, Pennsylvania | 0.013 inches | 12.5mm 100 Gyr<br>71 | 12.5mm 75 Gyr<br>81 | RCRI<br>41,773 | N.A. | N.A. |

Even though the TTI Overlay Tester results indicate a relatively poor fatigue resistance with respect to the horizontal cracking mode, the results do indicate that as the horizontal deformation decreases, the fatigue life increases. Therefore, if the state agencies could pre-screen pavement conditions for potential horizontal related reflective cracking, the state agencies could predetermine whether or not dense-graded mixtures should be placed over top PCC pavements. In order to accomplish this, a parametric study was conducted that looked at the combination of coefficient of thermal expansion, effective PCC slab length, and 24 hour temperature change to limit the horizontal deflection at the PCC joint/crack to 0.01 inches and lower.

### 7.2.2 Parametric Study – Minimizing Horizontal Deflection-Related Cracking Potential

To evaluate this further, a theoretical design chart was developed to help assist state agencies in selecting whether or not conventional dense-graded mixtures should be placed on the PCC surface when constructing an asphalt overlay on PCC/composite pavements. Even though there will be differences in climatic conditions regarding different state agencies, the methodology should still be somewhat valid as the state agencies compensate for different climate conditions by selecting different PG graded asphalt binders suitable for their specific temperature conditions.

The parametric study looked at evaluating different combinations of effective PCC slab length, 24 hour change in PCC surface temperature, and coefficient of thermal expansion that would limit the horizontal deformation at the PCC joint/crack to 0.01 inches. A value of 0.01 inches was selected based on testing a number of different plant produced mixes in New Jersey. Laboratory testing showed that even at temperatures as low as 40$^{o}$F, asphalt mixtures were still able to achieve relatively long fatigue lives. Table 7.4 shows that database collected during the study.

Table 7.4 – Horizontal Deformation Fatigue Resistance of Conventional Dense-Graded Mixtures (Deflection of 0.01 Inches)

| Mixture Type | Test Temperature (F) | Fatigue Life (Cycles) |
|---|---|---|
| 9.5H76 | 40 | 482 |
| | 60 | 814 |
| | 80 | 1179 |
| 12.5H76 | 40 | 81 |
| | 60 | 265 |
| | 80 | 610 |
| 12.5M64 | 40 | 65 |
| | 60 | 412 |
| | 80 | 1524 |

The proposed guideline chart to limit horizontal deformation-related fatigue cracking due to the expansion and contraction of the PCC slabs (i.e. – horizontal movement at the PCC joint/crack) is shown in Figure 7.23. The chart requires knowledge of the effective PCC slab length, coefficient of thermal expansion of the PCC, and the maximum temperature change at the PCC surface (i.e. – bottom of the HMA overlay) in a 24 hour period. Most of the data is relative accessible except for the temperature change. Therefore, the Enhanced Integrated Climatic Model (EICM) was used to quickly evaluate the average relationship between asphalt overlay thickness and 24 hour temperature change at the PCC surface. The typical relationship discovered through simulations in the EICM is shown in Equation 7.4. By incorporating Equation 7.4, state agencies can now estimate what conditions will result in horizontal deflections of 0.01 inches or less, which would be desirable for placement of an asphalt overlay.

$$\Delta T = -0.5455(t_{HMA}) + 7.9545 \qquad (R^2 = 0.92) \tag{7.4}$$

where,

$\Delta T$ = maximum temperature change at PCC surface in 24 hours; and
$t_{HMA}$ = total thickness of asphalt mixture overlay.

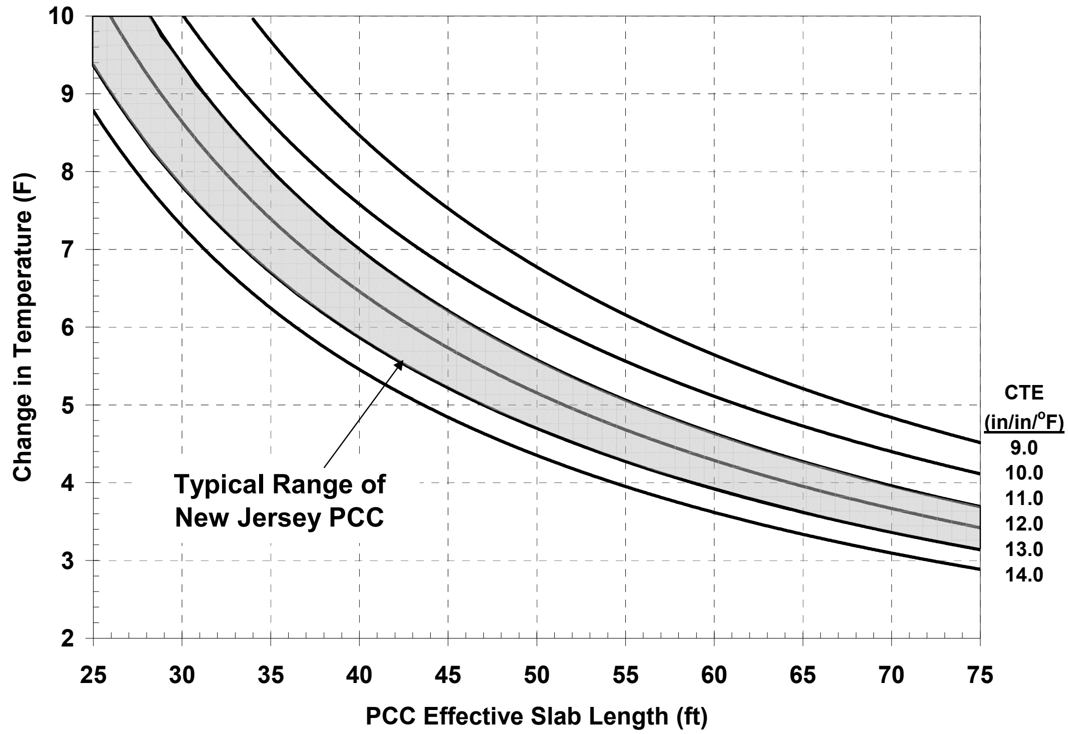



Figure 7.23 – Proposed Guideline to Limit Horizontal Deformation-Related Fatigue Cracking

An example of using the proposed guideline is as follows. A state agency would like to place a four inch asphalt overlay over an aging PCC pavement. The PCC pavement was designed with 60 ft slab lengths and the coefficient of thermal expansion of the PCC material is 12.0 x $10^{-6}$ in/in/$^{o}$F. Using the proposed guideline results in a maximum allowable change in PCC surface temperature ($\Delta$T) to be 4.3$^{o}$F (Figure 7.24). Once $\Delta$T is determined, it is then used in conjunction with Equation 7.4 to determine the asphalt overlay thickness required to limit cracking potential from horizontal joint deformations. In doing so, a value of 6.7 inches is calculated. The final check is to make sure the desired or design overlay thickness (4.0 inches) is less than the limiting cracking thickness (6.7 inches). For this example, this is not the case and the 4.0 inch asphalt overlay is not recommended. Working in reverse order indicates that if a 4.0 inch asphalt

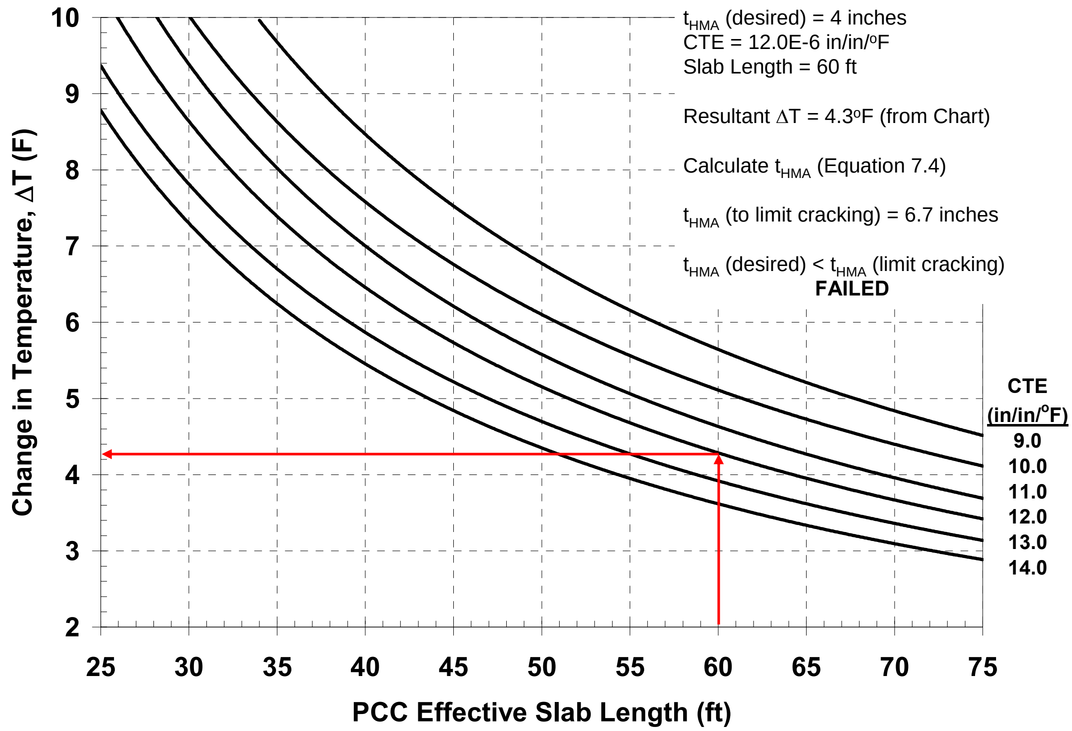



Figure 7.24 – Example of Using Proposed Guideline to Limit Horizontal Deformation-Related Fatigue Cracking – Required Asphalt Overlay Thickness

overlay is desired, only a PCC effective slab length of 45 ft or less would not result in horizontal PCC joint deflections greater than 0.01 inches (Figure 7.25).

### 7.2.3 Horizontal Deflection Testing of Reflective Crack Relief Type Asphalt Mixtures

Along with a number of conventional dense-graded asphalt mixtures, quite of few reflective crack relief interlayer mixtures were also evaluated using the TTI Overlay Tester. But unlike the dense-graded asphalt mixtures, the RCRI mixture all performed extremely well. In fact, the lowest number of cycles obtained during testing under simulated field conditions in the TTI Overlay Tester was 4,280 cycles from I495 in Massachusetts. If one assumes each cycle represents one 24 hour temperature cycle (conservative) than the I495 RCRI mixture would have a horizontal deflection fatigue life

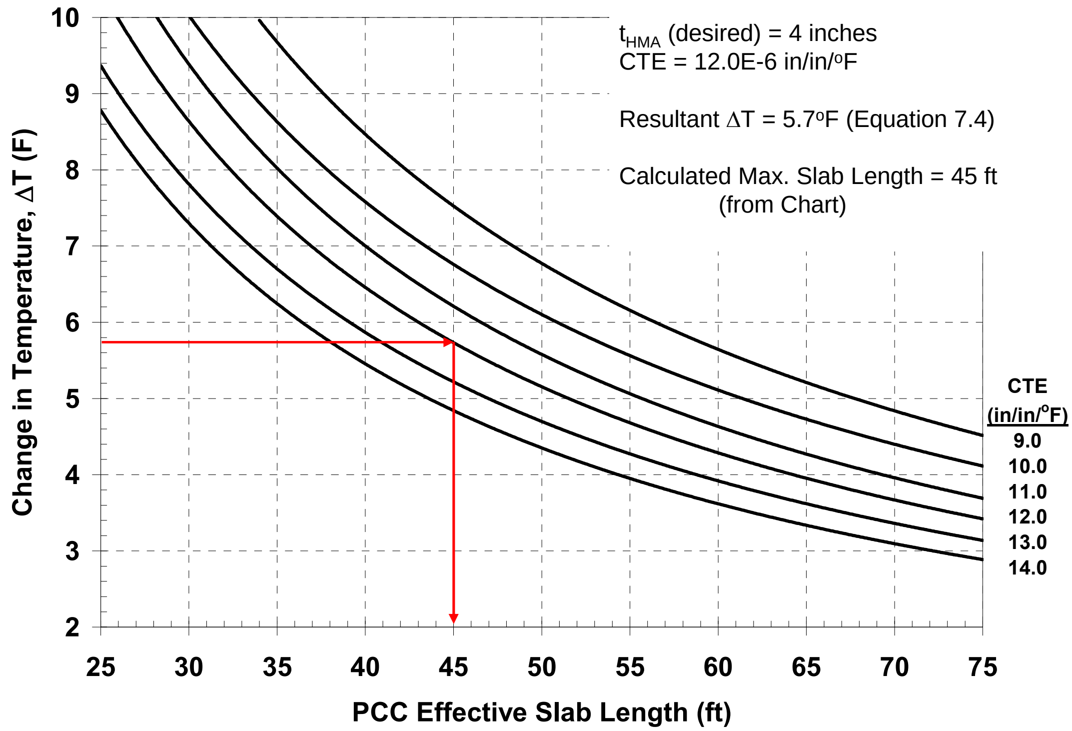



Figure 7.25 – Example of Using Proposed Guideline to Limit Horizontal Deformation-Related Fatigue Cracking – Determining Maximum Slab Length

life of over 11.7 years. The performance of the RCRI mixture was far superior to the 44 cycles of the dense-graded level course mixture at the same location. Additional TTI Overlay Tester evaluations were conducted on RCRI/dense-graded composite samples to determine how well the RCRI mixture absorbs the horizontal PCC joint/crack deflection when a dense-graded mixture is overlaid over it. As shown in the Deflection Spectra Approach, the use of RCRI mixtures does not necessary absorb the vertical deflections in the pavement structure as once thought, as residual vertical strains remain in the overlaying layers that must be overcome. The vertical strain absorption is actually accomplished more so due to the overlay thickness and general stiffness of the respective materials used in the overlay. However, is this the case regarding the horizontal deformations?

Test specimens were prepared in a manner to represent the “layering” that occurs in the field when RCRI mixtures are overlaid with conventional dense-graded asphalt mixtures. First, a two inch lift of an RCRI mixture, sampled from the Rt 34N New Jersey test section, was compacted in the gyratory compactor. The two inch RCRI lift was allowed to cool in the gyratory compactor for 2 hours before the dense-graded asphalt mixture was placed over top of it and compacted to two inches. This provided with a final specimen height of four (4) inches (2 inches of dense graded over 2 inches of RCRI). Test specimens were cut to achieve two different sized samples; 1) ½ inch RCRI overlaid by 1.5 inches of a dense-graded asphalt mixture, and 2) 1 inch of RCRI overlaid by 0.5 inches of a dense-graded asphalt mixture. The 1 inch of RCRI represents design and typical construction thickness, while the ½ inch of RCRI was evaluated to determine how poor construction practices (i.e. – lower thickness than specified) would impact the results. It should be noted that the dense-graded asphalt mixture used in this mini-experiment was a 12.5mm, coarse-graded Superpave mixture containing a PG76-22 asphalt binder and 15% RAP that was plant produced and sampled during production.

The TTI Overlay Tester testing parameters used in the mini-experiment consisted of 0.035 inches of horizontal deflection at a test temperature of 59$^{o}$F. The horizontal deflection was set at a higher level than typically predicted to occur in the field in an effort to try and fracture the RCRI mixture.

The test results of the RCRI horizontal deformation testing mini-experiment are shown in Figure 7.26. For a baseline comparison, the 12.5mm dense-graded mixture, shown as 12M76, was also tested under the identical testing parameters. The test results show that fatigue life of the 12.5mm dense-graded Superpave mixture was only 5 cycles.

**1.5" of 12M76**

**5 Cycles**

**0.5" of RCRI: 1.5" of 12M76**

**278 Cycles**

**1.0" of RCRI: 0.5" 12M76**

**2,800 Cycles**

**0.035" Opening**
**15°C (59°F)**

Figure 7.26 - Assessing RCRI Horizontal Strain Absorption and Thickness Impact on Fatigue Life Using the TTI Overlay Tester

When the RCRI mixture is placed below the dense-graded mixture, as is commonly done for composite pavements, the fatigue life of the composite specimen dramatically increases. For example, if the pavement is constructed according to specifications (i.e. – RCRI thickness equaling one inch), the horizontal deformation fatigue life of the specimen increased to greater than 2,800 cycles. In fact, the test was actually stopped at 2,800 cycles due to time constraints. Otherwise, the fatigue life would have probably been much higher. Visual inspection of the composite test specimen showed a slight crack developing in the RCRI layer, but it had not yet reached the dense-graded mixture when the test was stopped at 2,800 cycles. This clearly illustrates that the RCRI mixture absorbs almost the entire magnitude of horizontal deformation as the horizontal fatigue life increased from 5 cycles to over 2,800 cycles by simply using the RCRI mixture. This is extremely important as by eliminating any horizontal straining would allow a state agency to solely concentrate on asphalt mixtures that can resist the vertical straining.

Figure 7.26 also shows that when the RCRI mixture thickness is reduced in half (from one inch to ½ inch), an order of magnitude decrease in fatigue life can be expected. This is extremely important for state agencies to realize as quality control during construction can significantly decrease the service life of composite pavements.

## 7.3 Summary of Analysis Methodology

An analysis methodology was presented that looks at the vertical and horizontal modes of PCC joint/crack deflection individually and determines the cracking potential of asphalt mixture overlay due to the deflections. The analysis proposed a new method called the Deflection Spectra Approach to analyze the fatigue cracking potential of asphalt mixtures from the vertical deflections at the PCC joint/crack. The TTI Overlay Tester, along theoretical parametric studies, assessed the fatigue cracking potential of asphalt mixtures due to the horizontal deflections at the PCC joint/crack. The analysis procedures showed;

1. The Deflection Spectra Approach resulted in an excellent comparison to the field measured reflective cracking on the composite pavements. Overall, there was a 9.3% difference between the measured and predicted percent of transverse joints that underwent reflective cracking. This was based on the analysis of ten (10) different test sections (eliminating one as an outlier) over three different states in

the Northeast (Massachusetts, New Jersey, and Pennsylvania). If the outlier is allowed to stay in the dataset, the percent difference increases to 57%.

2. The Deflection Spectra Approach indicates that the use of reflective crack relief interlayer (RCRI) mixes do not necessarily absorb the vertical strain in the pavement structure as was commonly thought. In fact, the RCRI appears to simply be able to withstand the vertical bending associated with fatigue cracking that conventional asphalt mixture can not. This results in residual vertical strain in the pavement that conventional asphalt mixtures must withstand or reflective cracking will occur.
3. Test data from the TTI Overlay Tester indicates that conventional asphalt mixtures can not withstand the estimated horizontal field movements associated with the expansion and contraction of PCC joints/cracks. In fact, all of the test data indicates that conventional asphalt mixtures have difficulties surviving horizontal deformation movements greater than 0.01 inches. The lowest horizontal deformation estimated in the test sections was 0.013 inches with the resultant fatigue life measured as 71 cycles. Meanwhile, all of the RCRI mixes evaluated resulted in a fatigue life measured in the TTI Overlay Tester over 4,200 cycles for all of the test sections. Therefore, if conventional asphalt mixes are to be placed over PCC pavements, it is recommended that the maximum estimated horizontal movement at the PCC joint/crack be 0.01 inches.
4. Evaluating RCRI test specimens that had been overlaid with conventional asphalt mixes indicated that the RCRI mixtures, when placed to proper density and

thickness, can absorb almost all of the horizontal tensile strain developed due to the expansion and contraction at the PCC joint/crack.

5. By evaluating the vertical and horizontal modes of reflective cracking separately, it was possible to determine what the critical factors are that generate reflective cracking. The analysis indicated that if the horizontal deflection at the PCC joint/crack is to be greater than 0.01 inches, an RCRI-type mixture is required. The use of the RCRI mixture will absorb almost all of the horizontal deformation while still being able to withstand most vertical deformations in the pavement structure. Once the horizontal mode of reflective cracking is evaluated, the vertical mode can be assessed using the Deflection Spectra Approach. The test data collected from the test sections clearly showed that asphalt mixtures with greater flexural fatigue resistance are required for asphalt overlays on composite pavements. The asphalt mixtures from the Rt 202S New Jersey test section had superior flexural fatigue resistance when compared to the other dense-graded asphalt mixtures from the other test sections. This resulted in 0% of the transverse joints causing reflective cracking in the asphalt overlay. However, each pavement needs to be evaluated individually as the PCC joint integrity (i.e. – vertical deflection) and traffic loading conditions is generally site specific.

# CHAPTER 8 – DEVELOPMENT OF SIMPLIFIED ANALYSIS PROCEDURE

As discussed in Chapter 7, the vertical and horizontal modes of reflective cracking were analyzed. The detailed laboratory testing of asphalt mixtures sampled from the individual test sections in CHAPTER 6 clearly showed the general fracture limits of the asphalt mixtures and the typical fracture rankings of the different asphalt mixtures (i.e. – RCRI versus conventional dense-graded). Meanwhile, the analysis techniques shown in Chapter 7 also showed that the fracture rankings and field measured reflective cracking can be predicted with relatively good reliability when utilizing the Deflection Spectra Approach. The main purpose of CHAPTER 8 is to condense the analysis methods developed in CHAPTER 7 into a simplified analysis procedure that state agencies can implement. A flowchart describing the proposed HMA overlay design and asphalt mixture selection process for overlaying PCC/composite pavements is shown as Figure 8.1.

## 8.1 Step 1 – Assessing Horizontal Joint Movement

The first step in the simplified analysis procedure is to determine the magnitude of the horizontal movement at the PCC joint/crack. As defined in Equation 6.4, the horizontal movement is a function of the effective PCC slab length, maximum temperature change in a 24 hour temperature cycle, the coefficient of thermal expansion of the PCC, and the friction between the PCC slab and underlying unbound material (i.e. – subgrade soil or aggregate base). And as previously shown, most conventional asphalt mixtures are not capable of withstanding horizontal movements greater than 0.01 inches. Therefore, the first step in properly designing an asphalt overlay for a composite

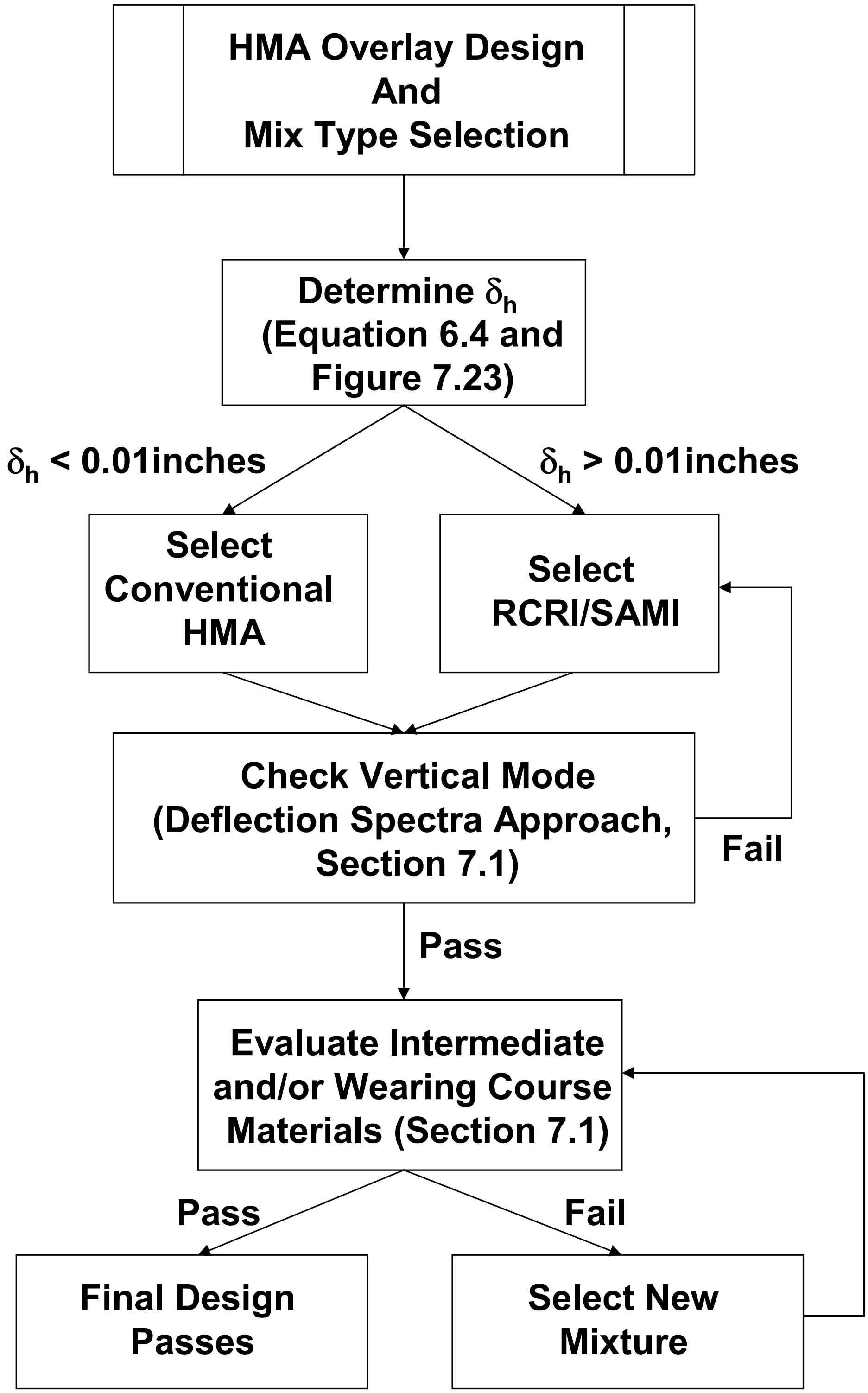



Figure 8.1 – Flowchart for Hot Mix Asphalt Overlay Design and Mixture Selection

pavement is to determine whether or not a reflective crack relief mixture is required. The general rule of thumb is if the estimated horizontal deformation is greater than 0.01 inches, a reflective crack relief interlayer should be used. The national survey conducted during the early stages of the research showed that reflective crack relief interlayers (RCRI) and stress-absorbing membranes (SAMI's) had the greatest chance of mitigating reflective cracking. Not to mention, the TTI Overlay Tester data indicated that RCRI asphalt mixtures also performed the best of all the asphalt mixtures evaluated. Therefore, it is recommended that the procedure outlined in Section 7.2.2 and Figure 7.23 be utilized in Step 1.

Two difficulties that state agencies may have during Step 1 is the determination of the coefficient of thermal expansion (CTE) of the PCC and the estimate of the 24 hour temperature change at the surface of the PCC. As mentioned during Section 7.2.2, the 24 hour temperature change at the surface of the PCC may be estimated using Equation 7.4 for typical HMA overlays thickness (2 to 7 inches of hot mix asphalt).

### 8.1.1 Estimation of the Coefficient of Thermal Expansion of PCC

The coefficient of thermal expansion of the PCC, if unable to be determined in accordance to AASHTO TP60 on field cores, may be estimated using the procedure outlined in Chapter 2 of the Mechanistic Empirical Pavement Design Guide manual (ARA, 2004). The estimation method uses a linear, weighted average of the constituent coefficient of thermal expansion (i.e. – aggregate and paste) values based on the relative volumes of the constituents (Equation 8.1). Table 8.1 provides typical coefficient of thermal expansion for various common PCC components and mixes. Using the respective state agency specifications for PCC mixes and known aggregate mineralogies, a state agency can estimate the coefficient of thermal expansion for PCC pavements.

$$CTE_{PCC} = CTE_{agg}(V_{agg}) + CTE_{paste}(V_{paste}) \tag{8.1}$$

where,

$CTE_{agg}$ = Coefficient of thermal expansion of the aggregate;
$V_{agg}$ = Volumetric proportion of the aggregate in the PCC mix;
$CTE_{paste}$ = Coefficient of thermal expansion of cement paste; and
$V_{paste}$ = Volumetric proportion of the paste in the PCC mix.

Table 8.1 – Typical Ranges for Coefficient of Thermal Expansion for Common Components and Concrete (Adapted from ARA, 2004a)

| **Material Type** | **Coefficient of Thermal Expansion, $10^{-6}$/°F** | **Concrete Coefficient of Thermal Expansion (made from this material), $10^{-6}$/°F** |
|---|---|---|
| **Aggregates** | | |
| Marbles | 2.2-3.9 | 2.3 |
| Limestones | 2.0-3.6 | 3.4-5.1 |
| Granites & Gneisses | 3.2-5.3 | 3.8-5.3 |
| Syenites, Diorites, Andesite, Basalt, Gabbros, Diabase | 3.0-4.5 | 4.4-5.3 |
| Dolomites | 3.9-5.5 | 5.1-6.4* |
| Blast Furnace Slag | | 5.1-5.9 |
| Sandstones | 5.6-6.7 | 5.6-6.5 |
| Quartz Sands & Gravels | 5.5-7.1 | 6.0-8.7 |
| Quartzite, Cherts | 6.1-7.0 | 6.6-7.1 |
| **Cement Paste (saturated)** | | |
| w/c = 0.4 to 0.6 | 10-11 | -- |
| **Concrete Cores** | | |
| Cores from LTPP pavement sections, many of which were used in calibration | N/A | $4.0*10^{-6}$– $5.5*10^{-6}$– $7.2*10^{-6}$ (Min – Mean – Max) |

### 8.1.2 Coefficient of Thermal Expansion for PCC – New Jersey Database

PCC cores from the various New Jersey test sections, as well as poured cylinders from a variety of New Jersey Department of Transportation (NJDOT) PCC projects, were sampled and tested for the coefficient of thermal expansion (CTE) in accordance with AASHTO TP60. The CTE values were determined using the automated system manufactured by the Gilson Equipment Company, shown earlier in Figure 4.8. The collected data is shown in Table 8.2. If all of the CTE values of the samples were averaged, an average coefficient of thermal expansion for New Jersey PCC mixes is 11.63 x $10^{-6}$ mm/mm/°C. It should be noted that this value represents the average for New Jersey materials and may not be valid for use in regional areas.

Table 8.2 – Coefficient of Thermal Expansion (CTE) for New Jersey PCC Mixes

| Location | Cylinder Type | CTE (mm/mm/°C) x 10-6 | | |
|---|---|---|---|---|
| | | (Average) | Std Dev | COV % |
| Rt 1 + 9-4T | Poured | 10.92 | 0.040 | 0.37 |
| Rt 46-47 56 Day HPC | Poured | 11.37 | 0.220 | 1.93 |
| Rt 46(47) | Poured | 11.61 | 0.199 | 1.71 |
| Rt 18-2F | Poured | 11.61 | 0.211 | 1.82 |
| Rt 31+518 | Poured | 11.74 | 0.219 | 1.87 |
| Rt 9-23E | Poured | 11.68 | 0.145 | 1.24 |
| Rt 78 6J | Poured | 10.80 | 0.137 | 1.27 |
| Rt 130 Collinswood | Poured | 11.45 | 0.157 | 1.37 |
| US Ave Br | Poured | 12.49 | 0.077 | 0.62 |
| ***Average*** | ***Poured*** | ***11.52*** | ***0.156*** | ***1.36*** |
| Rt 34N | Cores | 12.52 | 0.270 | 2.16 |
| Rt 202S | Cores | 11.17 | 0.280 | 2.51 |
| Rt 29N | Cores | 11.15 | 0.393 | 3.52 |
| I78 | Cores | 12.75 | 0.386 | 3.03 |
| ***Average*** | ***Cores*** | ***11.90*** | ***0.33*** | ***2.79*** |

## 8.2 Step 2 – Determine Fatigue Cracking Resistance of Asphalt Overlay

In Step 1, the selection of the asphalt mixture to be placed directly on the PCC or at the bottom of the new asphalt overlay is based on the magnitude of the horizontal deformation at the PCC joint/crack. If the horizontal deformation is less than 0.01 inches, a conventional asphalt mixture can be used. If the horizontal movement is greater than 0.01 inches, than a RCRI mixture is recommended. However, simply because the horizontal deformation is less than 0.01 inches, it does not mean that the conventional asphalt mixture will not fracture due to the vertical deformation associated with traffic loading. Therefore, Step 2 consists of using the Deflection Spectra Approach to verify whether or not the "bottom" layer/mixture of the new asphalt overlay will fracture.

As described in Section 7.1, the Deflection Spectra Approach needs the following inputs; 1) PCC joint/crack vertical deflection vs applied load data, 2) Flexural fatigue test data of proposed asphalt mixtures, and 3) axle load spectra or ESAL counts. With state agency budgets diminishing, there may be difficulties obtaining all of the required inputs for the Deflection Spectra Approach. Therefore, estimates may be able to be used to provide general guidance for asphalt mixture selection and overlay design.

### 8.2.1 Estimation of Vertical Joint Deflection for Applied ESAL's

During the field evaluation conducted during this research project, visual distress surveys were conducted in accordance to LTPP protocols (FHWA, 2003). With the use of the visual distress surveys, a general ranking was given to each pavement section. Along with the visual distress survey, the average vertical deflection at the PCC joint/crack (normalized to 18,000 lbs or 1 ESAL) was determined and correlated with the visual distress survey. Table 8.3 and 8.4 show the recorded values and the recommended values, respectively, for vertical deflection at the PCC joint/crack for use in the Deflection Spectra Approach. However, it should be noted that the accuracy of the predictions will be highly affected by accuracy of the vertical joint deflections.

Table 8.3 – Measured Visual Distress Condition Rating and Vertical Deflection at the PCC Joint/Crack Normalized to 18,000 lbs (1 ESAL)

| Pavement Test Section | Deflection @ 18 kips (mils) | PCC Visual Distress Condition |
|---|---|---|
| I78 | 17 | Poor |
| I495 - 5-Inch Section | 8.8 | Good |
| I495 - 2-Inch Section | 12.2 | Average |
| Rt 73 NB[1] | 8.3 | Good |
| Rt 73 NB[1] | 12.4 | Average |
| Rt 202 | 12.8 | Average |
| Rt 34N | 12.1 | Average |
| I476 - Section 1 | 14.8 | Poor |
| I476 - Section 2 | 13 | Poor |

[1] - Not included in Study

Table 8.4 – Recommended Vertical Deflection Values (at 18,000 lbs or 1 ESAL) as Defaults for Deflection Spectra Approach

| PCC Visual Distress Condition | Vertical Joint Deflection (mils) |
|---|---|
| Good | 8 |
| Average | 12 |
| Poor | 15 |

### 8.2.2 Estimation of Flexural Beam Fatigue Values

Flexural fatigue parameters are also required to be utilized in the Deflection Spectra Approach. During the development of the research project, a number of different asphalt mixtures were tested using the Flexural Beam Fatigue test (AASHTO T321). The test data was previously shown in Table 6.2. However, these mixes are native to New

Jersey and may not be applicable to other regional areas. Therefore, other means may be required to estimate fatigue life properties of asphalt mixtures.

Maupin and Freeman (1976) recommended using the indirect tensile test results to estimate the $k_1$ and $k_2$ material coefficients for determining the flexural fatigue life of asphalt mixtures using Equation 6.2. The prediction equations used are as follows:

$$\log k_1 = 7.92 - 0.122(\sigma_{IT}) \tag{8.2}$$

$$k_2 = 0.0374(\sigma_{IT}) - 0.744 \tag{8.3}$$

where,

$\sigma_{IT}$ = indirect tensile strength @ 72°F, psi
$k_1$, $k_2$ = material specific coefficients

Ghuzlan and Carpenter (2003) presented flexural beam fatigue test data for over 80 different asphalt mixtures sampled and tested from Illinois. Based on the data set developed, the authors recommended the following "average" values to be utilized with Equation 6.2; $k_1 = 10^{-10}$, and $k_2 = 4.5$.

## 8.3 Step 3 – Determine Fatigue Cracking Resistance of Asphalt Intermediate and/or Surface Course Mixes

The analysis required in Step 3 is identical to that of Step 2 (Section 8.2). The only difference is that the analysis is conducted on the intermediate and/or surface course. And once again, if the asphalt mixture exceeds the cracking limits established by the state agency, a new asphalt mixture type/design should be selected.

# CHAPTER 9 – CONCLUSIONS

The main purpose of the research project was to determine how to design hot mix asphalt overlays for use on composite/PCC pavements. With over 50% of the New Jersey Department of Transportation's (NJDOT) centerline miles consisting of composite pavements with less than desirable pavement lives, a better way for characterizing, designing, and selecting asphalt mixtures for overlaying these aging PCC pavements was a necessity. Based on the research conducted during this study, the following conclusions can be made:

- The major mechanism generating reflective cracking is the tensile stress/strain generated at the bottom of the asphalt overlay. The tensile stress/strain is a coupled resultant of vertical deflection at the PCC joint/crack associated with traffic loading and horizontal deflection at the PCC joint/crack associated with the expansion and contraction from environmental cycling.
- The shearing mechanism at the PCC joint/crack, commonly indexed with the measured Load Transfer Efficiency (LTE), is not a crack initiator but an accelerator. The energy required to initiate cracking is not capable of being generated from a "confined" shear mode. However, once a crack has initiated from the tensile stress/strain, poor LTE will accelerate the propagation of the crack to the pavement surface.
- The critical reflective cracking condition in composite/PCC pavements is when the air/pavement temperatures are already cold and the climate is under-going a cooling cycle. This creates an already brittle-like HMA layer that must be able to

withstand tensile straining caused by contraction occurring at the PCC joint/crack and material contraction.

- Low temperature asphalt binder grade was found to be related to the time until reflective cracking is observed. The survey results indicated that states that use a low temperature PG grade one to two grades lower than recommended by LTPPBind (at a 98% reliability level) for the HMA mixture immediately overlaying the PCC pavement, have a better chance at retarding reflective cracking longer.
- A number of reflective cracking mitigation methods have been attempted by the state agencies over the years. Statistically, the best performing mitigation methods were found to be the Stress Absorbing Membrane Interlayers (SAMI's) and the Reflective Crack Relief Interlayer mixes (Strata®-type mixes). The worst performing mitigation methods were found to be the paving fabrics and geogrids. However, it should be noted that even the best mitigation method only had a 50% success rate, when considering a successful method was defined as one that provided five years before reflective cracking was observed. This was most likely due to poor selection of brittle asphalt mixtures that overlaid the highly flexible SAMI/RCRI mixes.
- A new analysis approach for asphalt mixture selection/overlay design of composite/PCC pavements was developed and presented. The analysis approach requires the knowledge of; 1) Vertical deflection at the PCC joint/crack, 2) Magnitude of traffic loading, preferably as axle load spectra but also ESAL's can be used, and 3) Asphalt mixture properties measured by the Flexural Beam

Fatigue and Dynamic Modulus test. The predicted reflective cracking fatigue life was compared to the measured cracking from the test section monitored in the field. The predicted values matched the measured percent of transverse joints cracked quite well. Using all eleven test sections in the study, the average percent difference between the predicted and measured percent of cracked transverse joints was 57%. However, one of the test sections in I476 in Pennsylvania, where limited joint deflection data was collected, was determined to be an outlier in the data. By eliminating this point and only looking at ten test sections, the average percent difference between the predicted and measured dropped to 9.3%.

- The horizontal mode of the fatigue cracking response of asphalt mixture placed on composite/PCC pavements can be estimated using the TTI Overlay Tester. Analyzing data generated using the TTI Overlay Tester and field parameters estimated from pavement characteristics and climate conditions showed that typical dense graded mixtures utilized by state agencies have minimal horizontal deformation fatigue lives under typical field conditions. It was found that most dense graded mixtures could not withstand horizontal deformations as low as 0.01 inches without the rapid onset of cracking. Therefore, if dense graded asphalt mixtures are proposed to be used, it was recommended to check to determine if the horizontal deformation of the PCC joint/crack was greater than 0.01 inches. If so, the dense graded mixture should not be used. Meanwhile, reflective crack relief interlayer (RCRI) mixtures were found to have significant horizontal deformation fatigue lives and were found to absorb almost 100% of the horizontal deformation resulting from the expansion and contraction of PCC slabs. This is significant as RCRI mixtures can be placed at the bottom of an asphalt overlay to mitigate 100% of the horizontal deformation and then the designer can appropriately select asphalt mixture to withstand the residual vertical deformation remaining in the asphalt overlay in the area of the PCC joint/crack.

- A final asphalt overlay/mixture selection process was developed to allow state agencies to select more appropriate asphalt mixtures than can withstand the vertical and horizontal modes of deflection at the PCC joint/crack that result in reflective cracking. The process utilizes three main steps;
  - Estimate the magnitude of horizontal deformation to be expected for the proposed asphalt overlay thickness. Depending the results, either a conventional dense graded mixture or a reflective crack relief interlayer is recommended.
  - The next step evaluates the resistance to cracking due to the vertical deformation at the PCC joint/crack using the Deflection Spectra Approach. As stated earlier, simply because the asphalt mixture selected for placement immediately on top of the PCC pavement can withstand the climate-related horizontal movement, it does not mean it can withstand the traffic load associated vertical deflections. Depending on the vertical mode analysis, an RCRI-type mixture may need to be recommended to ensure both horizontal and vertical resistance to cracking.
  - The final step in the process is to once again utilize the Deflection Spectra Approach on asphalt mixtures that will be placed as intermediate and/or surface courses within the asphalt overlay. As shown in the Literature

Review, National Survey and test section data, asphalt mixture containing higher percentages of asphalt binder, as well as better low temperature asphalt binder properties, perform better in this zone of the asphalt overlay.

# CHAPTER 10 – RECOMMENDATIONS FOR FUTURE RESEARCH

The information gathered and developed during this research project was utilized to develop and recommend a hot mix asphalt overlay mixture selection and design procedure that can be utilized by state agencies for overlaying aging PCC/composite pavements. However, the research has shown that there are still areas that need to be evaluated to provide a better understanding on the mechanisms that initiate reflective cracking, as well as materials and procedures that can be utilized to mitigate reflective cracking. Recommendations for future research in these areas include the following;

- A better understanding regarding the impact of quality control during construction should be looked at in further detail. This includes the impact of poor bonding between the asphalt layers (i.e. – poor tack coats) and compacted densities of the asphalt mixtures.
- The evaluation of asphalt mixture types should be evaluated under the procedure, as well as in field trials. Warm mix asphalt, for example, promises decreased production and compaction temperatures while not sacrificing compaction (i.e. – density). It is generally agreed upon that a decrease in production temperatures will result in less oxidative aging of the asphalt mixture, thereby increasing the mixtures' resistance to cracking. Meanwhile, more and more state agencies are moving towards utilizing higher percentages of recycled asphalt pavement (RAP) in asphalt mixtures. Limited research has indicated that as the RAP percentages increase, the general durability/fatigue resistance decreases. If state agencies are going to move towards more environmentally friendly asphalt materials like warm

mix asphalt and higher RAP contents, further research should be conducted to determine if these materials will be beneficial or detrimental to mitigating reflective cracking.

- Additional work should be conducted to evaluate alternate field test methods that can provide information regarding vertical joint deflection. Currently, the best approach to determining vertical joint deflection is through the use of the Falling Weight Deflectometer. However, testing is often slow and requires lane closures. The development of a rapid field test that could possibly move at or close to highway speeds while still measuring vertical joint deflection would be a tremendous asset to the industry and understanding of reflection cracking.

**REFERENCES**

AASHTO Designation: R30, Standard Practice of Mixture Conditioning of Hot-Mix Asphalt (HMA), *Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part 1B: Specifications, 27th Edition, 2007*, American Association of State Highway and Transportation Officials, AASHTO.

AASHTO Designation: T321-07, Standard Method of Test for Determining the Fatigue Life of Compacted Hot-Mix Asphalt (HMA) Subjected to Repeated Flexural Bending, *Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part 2B: Tests, 27th Edition, 2007*, American Association of State Highway and Transportation Officials, AASHTO.

AASHTO Designation: TP60-06, Standard Method of Test for Coefficient of Thermal Expansion of Hydraulic Cement Concrete, *2007 AASHTO Provisional Standards*, American Association of State Highway and Transportation Officials, AASHTO.

AASHTO Designation: TP62-07, Standard Method of Test for Determining Dynamic Modulus of Hot-Mix Asphalt (HMA), *2007 AASHTO Provisional Standards*, American Association of State Highway and Transportation Officials, AASHTO.

Applied Research Associates, 2004a, *Chapter 2, Material Characterization,* National Cooperative Highway Research Program, NCHRP, Transportation Research Board, National Research Council, 83 pp.

Applied Research Associates, 2004b, *Guide for the Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures: Appendix II-1: Calibration of Fatigue Cracking Models for Flexible Pavements*, National Cooperative Highway Research Program, NCHRP, Transportation Research Board, National Research Council, 311 pp.

Blankenship, P., Drbohlav, J., and Iker, N., 2002, "Interlayer and Design Considerations to Retard Reflective Cracking", Proceedings of the 81st Annual Meeting of the Transportation Research Board, 22 pp.

Bonaquist, R. and D. Christensen, 2005,"Practical Procedure for Developing Dynamic Modulus Master Curves for Pavement Structural Design", *Transportation Research Record: Journal of the Transportation Research Board, No. 1929,* Transportation Research Board of the National Academies, Washington, D.C., pp. 208 – 217.

Bozkurt, D., 2002, *Three Dimensional Finite Element Analysis to Evaluate Reflective Cracking Potential in Asphalt Concrete Overlays*, Submitted in Partial Fulfillment of the Requirements for the Degree of Doctor of Philosophy in Civil Engineering, University of Illinois at Urbana-Champaign, 160 pp.

Bozkurt, D., Buttlar, W., and Dempsey, B., 2000, "Cost-Effectiveness of Reflective Crack Control Treatments in Illinois, "Proceedings of the 4th International RILEM

Conference – The Reflective Cracking in Pavements: Research in Progress, pp. 475 – 484.

Bozkurt, D., and Buttlar, W., 2002, "Three Dimensional Finite Element Modeling to Evaluate Benefits of Interlayer Stress Absorbing Composite for Reflective Crack Mitigation, Presented for the 2002 Federal Aviation Administration Airport Technology Transfer Conference, 16 pp.

Brown, R. and M. Anderson, 1998, "Revision of the Superpave Gyratory Compaction Standards", FHWA Mixture Expert Task Group, Baltimore, MD.

Burlie, R., Uzarowski, L., and Emery, J., 2000, "Use of Stone Mastic Asphalt to Mitigate Reflective Cracking", *Proceedings of the 4th International RILEM Conference – The Reflective Cracking in Pavements: Research in Practice*, pp. 279 – 288.

Button, J. and R. Lytton, 2007, "Guidelines for Using Geosynthetics with Hot Mix Asphalt Overlays to Reduce Reflective Cracking", Presented and Published at the 86th Annual Meeting of the Transportation Research Board, January 21st to 25th.

Button, J., 1989, "Overlay Construction and Performance Using Geotextiles", *Transportation Research Record No. 1248*, National Research Council, Washington D.C., pp. 24 – 33.

Buttlar, W., Dempsey, B., and Bozkurt, 1999, *Evaluation of Reflective Crack Control Policy*, Final Report Submitted to the Illinois Department of Transportation, Report No. ITRC FR 95/96-4.

*Distress Identification Manual for the Long-Term Pavement Performance Project, 4th Edition*, 2003, FHWA-RD-03-031, United States Department of Transportation, Federal Highway Administration, 169 pp.

ERES, 2004, *Material Characterization, Chapter 2 of the Manual for the Mechanistic-Empirical Pavement Design Guide*, National Cooperative Highway Research Program (NCHRP), 83 pp.

Germann, F.P. and R. L. Lytton, 1979, *Methodology for Predicting the Reflection Cracking Life of Asphalt Concrete Overlays*, Research Report FHWA/TX-79/09+207-5, College Station, Texas, March 1979.

Ghuzlan, K.A. and S.H. Carpenter, 2003, Traditional Fatigue Analysis of Asphalt Concrete Mixtures, Accepted for Presentation and Publication at the 2003 Annual Meeting of the Transportation Research Board, Washington, D.C.

Hammons, Michael, 1997, *Advanced Pavement Design: Finite Element Modeling for Rigid Pavement Joints, Report 1 – Background Investigation*, DOT/FAA/AR-95/85, Federal Aviation Administration, U.S. Department of Transportation, Washington D.C.

Hensley, M., 1980, "Open Graded Asphalt Concrete Base for the Control of Reflection Cracking", *Journal of the Association of Asphalt Paving Technologists*, Vol. 49, p. 72 – 96.

Larson, G. and B. Dempsey, 2006, *Enhanced Integrated Climatic Model, Version 3.2.*, Applied Research Associates (ARA).

Lee, J., Stokoe, K., Chen, D., Garrison, M., and Nam, B., 2005, "Monitoring Pavement Changes in a Rehabilitation Project with Continuous RDD Profiles", Proceedings of the 84th Annual Meeting of the Transportation Research Board, 23 pp.

Khazanovich, L. and Alex Gotlif, 2002, *Evaluation of Joint and Crack Load Transfer – Final Report, FHWA-RD-02-088,* Federal Highway Administration, 118 pp.

Kim, J., 1999, *Three Dimensional Finite Element Modeling of Multi-Layered Systems*, Submitted in Partial Fulfillment for the Requirements for the Degree of Doctor of Philosophy in Civil Engineering, University of Illinois at Urbana-Champaign, 282 pp.

Kohale, V. and Lytton, R., 2000, *Design of Asphalt Concrete Overlay to Mitigate Reflective Cracking*, Project No. 7256, Florida Department of Transportation, 89 pp.

Makowski, L., Bischoff, D., Blankenship, P., Sobczak, D., and Haulter, F., 2004, "Wisconsin Experiences with Reflective Crack Relief Projects", Proceedings of the 84th Annual Meeting of the Transportation Research Board, 16 pp.

Maupin, G.W. and J.R. Freeman, 1976, *Simple Procedure for Fatigue Characterization of Bituminous Concrete*, FHWA-RD-76-012.

Maurer, D. and Malasheskie, G., 1989, "Field Performance of Fabrics and Fibers to Retard Reflective Cracking", *Transportation Research Record No. 1248*, National Research Council, Washington D.C., pp. 13 – 23.

Miner, M. A., 1945, "Cumulative Damage in Fatigue", *Transactions of the American Society of Mechanical Engineering*, Vol. 67, pp. A159 – A164.

Mukhtar, M. and Dempsey, B., 1996, "Interlayer Stress Absorbing Composite (ISAC) for Mitigating Reflection Cracking in Asphalt Concrete Overlays", Project IHR-533, *Report No. UILU-ENG-96-2006*, Illinois Cooperative Highway Research Program, Illinois Department of Transportation.

Pais, J., 1999, *Reflective Cracking in Flexible Pavement Overlay Design*, Submitted in Partial Fulfillment of the Requirements of Doctor of Philosophy, University of Minho, Portugal, 203 pp.

Pais, J., Sousa, J., Way, G., and Stubstad, 2000, "An Overlay Method for Reflective Cracking", *Proceedings of Asphalt Rubber 2000 Conference*, 22 pp.

Scarpas, A., and De Bondt, A., 1996, "Finite Elements Simulation of Reflective Cracking in Asphaltic Overlays", Special Issue of the Journal HERON, Department of Civil Engineering, Delft University of Technology, Norway, Vol. 41, No.1.

Shuler, S., and Harmelink, D., 2004, "Reducing Reflection Cracking in Asphalt Pavements", *Proceedings of the 5th International RILEM Conference on Cracking in Pavements: Mitigation, Risk Assessment, and Prevention*, pp. 451 – 458.

Serfas, J. and De La Villegle,, B., 2000, "Fiber-Modified Asphalt Overlays", *Proceedings of the 4th International RILEM Conference – The Reflective Cracking in Pavements: Research in Practice*, pp. 227 – 239.

Sousa, J., Pais, J., Saim, R., and Way, G., and Stubstad, R., 2002, *Development of a Mechanistic Overlay Design Method Based on Reflective Cracking Concepts*, Report Submitted to Consulpav International, 16 pp.

Tan, Zhiming, 2004, "Mechanistic Analysis for Opening Reflective Cracking in Asphalt Overlays", *Proceedings of the 4th International RILEM Conference – The Reflective Cracking in Pavements: Research in Practice*, pp. 165 – 714.

Yang, H. H., 1993, *Pavement Analysis and Design*, Prentice Hall, Englewood Cliffs, New Jersey.

Zapata, C.E. and W.N. Houston, 2008, *Calibration and Validation of the Enhanced Integrated Climatic Model for Pavement Design*, NCHRP Report 602, National Cooperative Highway Research Program, Transportation Research Board, Washington D.C., 73 pp.

Zhou, F. and Sun, L., 2000a, "Optimal Thickness for Asphalt Overlay for Composite Pavements", *Proceedings of the 4th International RILEM Conference – The Reflective Cracking in Pavements: Research in Practice*, pp. 69 – 80.

Zhou, F. and Sun, L., 2000b, "Mechanistic Analysis of Reflective Cracking and Validation of Field Test", *Proceedings of the 4th International RILEM Conference – The Reflective Cracking in Pavements: Research in Practice*, pp. 81 – 92.

Zhou, F. and Scullion, T., 2005, "Overlay Tester: A Simple Performance Test for Thermal Reflective Cracking", *Journal of the Association of Asphalt Paving Technologists*, Vol. 74, pp.

Zhou, F. and T. Scullion, 2005, *Overlay Tester: A Rapid Performance Related Crack Resistance Test*, FHWA/TX-05/0-4467-2, Texas Transportation Institute, College Station, 85 pp.

# APPENDIX A – National Survey

State: ______________________________________________________________

Pavement Point of Contact: ________________________ Phone: _________________

email address: ________________________________________________________

1. What is your current in-service PCC pavement design
   Slab Length: ___________________________________
   Slab Thickness: _____________________________________
   Reinforcement Type: _________________________________
   Base Type: none ☐ granular ☐ cement treated ☐ bituminous ☐
   Other (please specify): ______________________________

   Transverse Joint Type: contraction ☐ construction ☐ expansion ☐ dowels ☐
   plain (no dowels) ☐

   Shoulder Pavement Type: HMA ☐ Concrete (Untied ) ☐ Concrete (Tied) ☐

2. Do you witness reflectice cracking on your composite pavements?
   Yes ☐ No ☐

   If yes, how soon after placement of the overlay does the reflective cracking appear?:

   | | |
   |---|---|
   | Less than 1 Year | ☐ |
   | 1 to 2 Years | ☐ |
   | 2 to 4 Years | ☐ |
   | Greater than 4 Years | ☐ |

3. What are the typical traffic levels (in ESAL's) where;
   a. Composite pavements are most commonly located?
   b. The greatest amount of reflective cracking has been observed?

   | | a) | b) |
   |---|---|---|
   | Low (< 0.3 Million ESAL's) | ☐ | ☐ |
   | Medium (3 to 30 Million ESAL's) | ☐ | ☐ |
   | High (> 30 Million ESAL's) | ☐ | ☐ |

4. Prior to designing the HMA overlay for the composite/PCC pavement, do you utilize any of the following:

   a. Falling Weight Deflectometer: ☐
   b. Ground Penetrating Radar: ☐
   c. Coring: ☐
   d. Dynamic Cone Penetrometer: ☐
   e. Visual Surveys: ☐
   f. Traffic Count/Vehicle Class: ☐
   g. Laboratory testing: ☐

If lab testing, please discuss what types: ______________________________

5. For HMA overlay design, do you use a design method or simply use a minimum thickness?

   Design Method (specify): _______________________________________________

   If minimum thickness method used, how do you select the minimum thickness?
   - Traffic ☐
   - Field testing ☐
   - Pavement condition ☐
   - Other(s) _________________________________________________________

   Typical/Minimum Thickness Description? (e.g. 5 inches of HMA - 2 inches of 12.5mm PG76-22 over 3 inches of 19mm PG64-22)

   ______________________________________________________

6. What treatment(s) are used to prepare the PCC pavement for overlay?
   a. No treatment ☐
   b. Repair of cracks ☐
   c. Repair of bad joints ☐
   d. Replacement of bad slabs or joints ☐
   e. Undersealing ☐
   f. Void filling ☐
   g. Crack and seat ☐
   h. Rubblizing ☐
   i. Edge Drains ☐
   j. Other (specifiy): _________________________________________________

7. For jointed PCC pavements, do you currently use a Saw & Seal method for your 1st Generation HMA overlays (first time PCC has been overlaid) to mitigate reflective cracking?

   Yes ☐ No ☐

   If yes, do you Saw & Seal the:
   - Transverse joint? ☐
   - Longitudinal joint? ☐

   If yes, what is the reservoir type?
   - Depth: _______________________
   - Width: _______________________
   - Sealant cup shape/dimensions: ___________________________

   If you use Saw & Seal, have you used Saw & Seal for 2nd and/or 3rd Generation HMA overlays (second or third time the composite pavement has been overlaid)?

   Yes ☐ No ☐

   Has this been successful?
   (please elaborate) ______________________________________________

   What type of sealant is specified (if any)? __________________________

8. Are you currently using or have you used any of the following to mitigate reflective cracking (please specify whether the use of the material was successful in retarding reflective cracking – greater than 5 years before cracking occurred)

| | | | | | |
|---|---|---|---|---|---|
| a. | Paving fabrics/geotextiles: | ☐ Successful? | Yes☐ | No | ☐ |
| b. | Geogrids (steel, fabric, fiberglass): | ☐ Successful? | Yes☐ | No | ☐ |
| c. | SAMI's: | ☐ Successful? | Yes☐ | No | ☐ |
| d. | Strata-type interlayer: | ☐ Successful? | Yes☐ | No | ☐ |
| e. | Crack-arresting layer: | ☐ Successful? | Yes☐ | No | ☐ |
| f. | Excessive overlay thickness | ☐ Successful? | Yes☐ | No | ☐ |
| g. | Others: | ☐ Successful? | Yes☐ | No | ☐ |

If used, why were the above mitigation methods selected?

Research ☐
Past experience ☐
Typically used for mitigation ☐
Other (specify): ____________________________________________

What criteria was used to select the above mitigation methods (if any)?

Visual survey ☐
Field evaluation/testing ☐
Traffic level ☐
Pavement structure ☐
Other (specify): ____________________________________________

Was a control section used for comparison?: Yes ☐ No ☐

Curriculum Vita

# Thomas A. Bennert

## Experience

1998 to Current

**Supervisor of Rutgers Asphalt/Pavement Laboratory (RAPL)/Senior Research Engineer,** Rutgers University, Department of Civil and Environmental Engineering, Center for Advanced Infrastructure and Transportation, Piscataway, NJ

1998 to Current

**Graduate Level Instructor for the Department of Civil and Environmental Engineering – Mechanistic Pavement Design**, Rutgers University, Department of Civil and Environmental Engineering, Piscataway, NJ

## Other Courses Taught:

2005 – Current

***Material Inputs for the Mechanistic-Empirical Pavement Design Guide***

- Original course outline and reference materials developed by FHWA "Design Guide Implementation Team – DGIT"
- Course taught to members of the NJDOT and their pavement design consultants

## Education

**Ph.D. in Civil Engineering**
Rutgers University, New Brunswick, New Jersey, October 2009
Major: Geotechnical Engineering

**Master of Science in Geotechnical Engineering**
Rutgers University, New Brunswick, New Jersey, May 1998

**Bachelor of Science in Civil Engineering**
Rutgers University, New Brunswick, New Jersey, May 1996

## Peer- Reviewed Publications:

Bennert, T., and R. Dongre, 2009, "A Backcalculation Method to Determine "Effective" Asphalt Binder Properties of RAP Mixtures", Submitted for Presentation and Publication at the 89th Annual Meeting of the Transportation Research Board.

Bennert, T., G. Reinke, W. Mogawer, and K. Mooney, 2009, ”Assessment of Workability/Compactability of Warm Mix Asphalt”, Submitted for Presentation and Publication at the 89th Annual Meeting of the Transportation Research Board.

Bennert, T., and Williams, S., 2009, “Precision of AASHTO TP62-07 for Use in the Mechanistic Empirical Pavement Design Guide for Flexible Pavements”, In Print in the Transportation Research Record.

Bennert, T., Worden, M., and Turo, M., 2009, “Field and Laboratory Forensic Analysis of Reflective Cracking on Massachusetts Interstate 495”, In Print in the Transportation Research Record.

Garg, N., Bennert, T., and Brar, H., 2009, “Performance of Hot Mix Asphalt Surface Under High Tire Pressure Aircraft Landing Gear Configuration at the FAA’s National Airport Pavement Test Facility”, Accepted for Publication in the Proceedings of the *7th International RILEM Symposium on Advanced Testing and Characterization of Bituminous Materials*, Rhodes, Greece, May 27th to 29th, 2009.

Bennert, T., and Maher, A., 2008, “A Practical Methodology for the Performance-Based Selection of Bituminous Mixtures for Concrete Overlays”, *Pavement Cracking: Mechanisms, Modeling, Detection, Testing and Case Histories*, Proceedings of the 6th RILEM International Conference on Cracking in Pavements, Chicago, IL, pp. 833 – 842.

Bennert, T., and Maher, A., 2008, “Field and Laboratory Evaluation of a Reflective Crack Relief Interlayer (RCRI) Mixture in New Jersey”, *Transportation Research Record No. 2084,* National Research Council, Washington, D.C., 114 - 123 pp.

Bennert, T., and Martin, J.V., 2008” Polyphosphoric Acid in Combination with Styrene-Butadiene-Styrene Block Copolymer : Laboratory Mixture Evaluation”, Presented and Published in the *Compendium of Papers for the 87th Annual Meeting of the Transportation Research Board*, Washington D.C., January 13th to January 17th, 2008.

Bennert, T. and Lynn, T., 2007, “Evaluation of Asphalt Interlayer Mixes to Mitigate Reflective Cracking in Composite Pavements”, Submitted to The Fifth International Conference on Maintenance and Rehabilitation of Pavements and Technological Control – MAIREPAV5, August 8th to 10th, 2007, Park City, Utah (In Print)

Bennert, T. and Maher, A., 2007, “Evaluation of the Current State of Flexible Overlay for Rigid/Composite Pavements in the United States”, *Transportation Research Record No. 1991,* National Research Council, Washington, D.C., 97 - 108 pp.

Bennert, T. and Cooley, L.A., 2006, “Comparison of Friction Properties of Friction-Course Pavement Systems During Winter Storm Events”, *International Journal of Pavements*, Vol. 5, No. 1-3, 152 – 163 pp.

Bennert, T., Maher, A., Bryant, M., and Smith, J., 2006, "Comparing Fine Aggregate Angularity (FAA) to Aggregate and HMA Performance Tests", *Transportation Research Record No. 1962,* National Research Council, Washington, D.C., 79 – 89 pp.

Vitillo, N., Bennert, T., Smith, J. and Maher, A., 2006, "Evaluation of Superpave Mix Design for Low Volume Roads in New Jersey", *Transportation Research Record No. 1946,* National Research Council, Washington, D.C., 139 – 146 pp.

Bennert, T., Fee, F., Sheehy, E., Jumikis, A., and Sauber, R., 2005, "Comparison of Thin-Lift Surface Course Mixes in New Jersey", *Transportation Research Record No. 1929,* National Research Council, Washington, D.C., 59 – 68 pp.

Bennert, T., Hanson, D., Maher, A., and Vitillo, N., 2004, "Influence of Pavement Surface Type on Tire/Pavement Generated Noise", *Journal of Testing and Evaluation, ASTM International*, Vol. 33, No. 2, 94 – 100 pp.

Rowe, G., Sauber, R., Bennert, T., Fee, F., and Smith, J., 2004, "The Performance of Long Life Pavement and Rehabilitation of Surface Cracking, I-287 New Jersey", *Proceedings of the International Symposium on Design and Construction of Long Lasting Asphalt Pavements*, p. 891 – 909 pp.

Maher, A, Bennert, T., Jafari, F., Douglas, S., and Gucunski, N., 2004, "Geotechnical Properties of Stabilized Dredged Material from New York-New Jersey Harbor", *Transportation Research Record No. 1874*, National Research Council, Washington, D.C., 2004, 86 – 99 pp.

Bennert, T.A., Maher, A., Papp Jr., W.J., and Gucunski, N., "Utilization of Construction and Demolition Debris under Traffic-Type Loading in Base and Subbase Applications.", Transportation Research Record No. 1714, National Research Council, Washington, D.C., 2000, 33 - 39.

Bennert, T. A., Maher, M. H., Jafari, F., and Gucunski, N., 1999, "Use of Dredged Sediments from the Newark Harbor for Geotechnical Applications," *Geotechnics of High Water Content Materials, ASTM STP 1374,* 1999, pp. 143-158.

Gaffney, D.A., Martin, S.M., Maher, M.H., and Bennert, T.A., 1999, "Dewatering Contaminated Fine-Grained Material Using Geotextiles.", *Proceedings of Geosynthetics '99*, Vol. 2, p. 1017 – 1032.

Papp Jr., W.J., Maher, M.H., Bennert, T.A., and Gucunski, N., 1998, "Behavior of Construction and Demolition Debris in Base and Subbase Applications.", *Recycled Materials in Geotechnical Applications, GSP No. 79*, 122 – 136 pp.

Maher, M.H., Bennert, T.A., Jafari, F., and Aagard, P., 1997, "Geotechnical Evaluation of Dredged Material from Newark Harbor.", *Proceedings of the 13th International Conference on Solid Waste Technology and Management*, Philadelphia, PA.