

Thermal Segregation: Causes and Effects on In-Place Density and Fatigue Performance of Asphalt Mixtures

by

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Abstract

Previous research has demonstrated that an excessive loss of mix temperature during hauling and paving operations can cause significant reductions in the mix consistency and therefore, in its ability to be compacted appropriately. This construction-related problem has been called thermal segregation.

Twenty-eight asphalt paving projects were evaluated in the state of Alabama. Thermal profiles of the mat prior to its compaction were obtained by using the MOBA Pave-IR infrared bar. Based on the results, it was found that remixing operations were a key factor in the reduction of high temperature differentials.

Field cores were taken from each ALDOT Division in order to evaluate the effect of thermal segregation on in-place densities. The results indicated a negative effect of thermal segregation on mat in-place densities. Additionally, samples were collected in order to compare the laboratory fatigue performance between cold and hot spots in terms of fatigue cycles, initial stiffness and fracture energy. Based on the results of this study, the mix initial stiffness was determined to be the unique parameter affected by excessive air voids leading to the conclusion that cold spots can be more susceptible to fatigue cracking than hot spots.

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List of Abbreviations

AASHTO: American Association of State Highway and Transportation Officials

ALDOT: Alabama Department of Transportation

AMPT: Asphalt Mixture Performance Tester

APA: Asphalt Pavement Analyzer

CEDEX: Centro de Estudios y Experimentación de Obras Publicas

DOT: Department of Transportation

GLM: General Linear Model

GPS: Global Positioning System

HMA: Hot-Mix Asphalt

IDT: Indirect Tensile Testing

IR: Infrared bar

MAS: Maximum Aggregate Size

MEPDG: Mechanistic-Empirical Pavement Design Guide

MRD: Material Remixing Device

MTD: Material Transfer Devic

NCAT: National Center for Asphalt Technology

NCHRP: National Cooperative Highway Research Program

OGFC: Open-Graded Friction Course

SGC: Superpave Gyrotory Compactor

SMA: Stone-Matrix Asphalt

TDD: Temperature Differential Damage

TTI: Texas Transportation Institute

TxDOT: Texas Department of Transportation

WMA: Warm-Mix Asphalt

Chapter 1 : Introduction

Background

In 1995, the Washington State Transportation Center developed a research project (1) focused on the study of aggregate segregation of the asphalt mixes. During the study, researchers identified zones of low in-place densities due to poor compaction of the mat and that did not show signs of aggregate segregation. The poor compaction of the mat was attributed to a loss of consistency of the mix caused by the lack of uniformity of the mix temperature prior to its compaction. This type of segregation was called “thermal segregation”.

During the last 15 years, agencies and research centers have focused their attention on identifying the causes of thermal segregation and at the same time, on recommending the best construction practices for minimizing the loss of mix temperature during hauling and paving operations. Remixing devices have been mentioned by most of the studies as the most significant factor in the reduction of high temperature differentials although it is believed that other factors such as hauling time, type of mix and ambient temperature have a significant role on HMA thermal segregation. In addition to this, the negative effects of thermal segregation on in-place densities and pavement performance have been proposed as one of the main topics. Despite this, a few research projects about thermal segregation have included this topic in their scope due to absence of a methodology for locating cold and hot areas for subsequent evaluations.

Early research projects utilized infrared imaging for identifying high temperature differentials during paving operations. Although infrared imaging was suggested as an appropriate method for detecting this problem, recent studies have considered the use of infrared bars, capable of providing real-time temperature measurements of the mat during paving operations and an approximate location of thermally segregated areas. Updated versions of this equipment have allowed researchers to evaluate cold and hot spots as well as to monitor the performance of these zones in order to establish the real effects of thermal segregation on HMA pavement performance and durability. This research project is focused on the study

of thermal segregation of asphalt mixtures, its causes and effects on in-place density and pavement performance.

Objectives

The main objective of this research project includes the study of thermal segregation of asphalt mixtures, the causes associated with this problem and its potential effects on in-place density and pavement fatigue performance in the laboratory. Specifically, the objectives of this study are:

- To determine the causes and consequences of thermal segregation of asphalt mixtures by collecting field data from 28 paving projects constructed during 2010-2011 paving seasons in the state of Alabama
- To identify the most significant parameters in the development of thermally segregated areas
- To recommend the most appropriate construction practices to minimize the loss of mix temperature during asphalt hauling and paving operations
- To evaluate the effects of high temperature differentials on in-place densities for both surface and binder courses
- To analyze the effects of density on pavement fatigue life and mix stiffness in the laboratory, as a consequence of thermal segregation.

Scope of Work

This research project was executed in two stages. The first stage corresponded to the field evaluation which consisted in the evaluation of 28 paving projects in the state of Alabama. This stage allowed accomplishing the first four objectives of this study. During this stage, the MOBA PAVE-IR infrared bar and an infrared camera were utilized for obtaining thermal profiles of the mat prior to compaction and for obtaining infrared images at different stages of paving operations, respectively. Construction-related data were also collected for each paving project visited. One project per ALDOT Division was selected for the subsequent compaction analyses and fatigue laboratory testing. The compaction analyses consisted in the determination of in-place air voids by extracting field cores from both cold and hot areas, previously

located by a global positioning system (GPS) of the infrared bar. Afterwards, it was possible to compare the results obtained from both areas and to establish several conclusions about the effect of thermally segregated areas on in-place densities.

The second stage consisted in analyzing the effects of air voids on the fatigue performance of asphalt mixtures in the laboratory. Three parameters related to fatigue performance were selected for this purpose. The number of fatigue cycles and the mix initial stiffness were measured using the bending beam fatigue device in accordance to AASHTO T-321 whereas the fracture energy was determined from indirect tensile testing applied on field cores in accordance with sections 7, 8 and 11 of AASHTO T-283. Several statistical analyses were applied in order to find a possible correlation among air voids and the three aforementioned parameters.

Chapter 2 : Literature Review

Definition of Thermal Segregation

Thermal segregation of asphalt mixtures is a problem that occurs due to the placing of cool material over a prepared surface prior to its compaction. This problem is related to an excessive lack of uniformity of the compaction temperature on the mat that causes loss of consistency and workability. Thermal segregation, also known as temperature differential damage (TDD), is considered a construction-related problem since the loss of mix temperature in the field cannot be avoided, although its reduction is possible if good construction practices are applied.

According to a study developed by Stroup–Gardiner and Brown (2), the term “segregation” is defined as “*a lack of homogeneity in the hot-mix asphalt constituents of the in-place mat of such a magnitude that there is a reasonable expectation of accelerated pavement distress(es).*” Basically, this definition can be interpreted as the lack of homogeneity of coarse and fine aggregate, asphalt binder, additives or air voids in the asphalt mixture that causes accelerated pavement distresses.

Thermal segregation was defined by Henault and Larsen (3) as “*isolated areas of the mat that differ significantly from the main body of the mat in temperature.*” This type of segregation was identified during a field study performed by Read (1) during the 1995 paving season. The research was focused on identifying several causes related to HMA segregation under the assumption that segregation problems were caused only by aggregate segregation. After several measurements and tests, Read found that apparent segregated areas did not always present aggregate segregation but they showed low in-place densities produced by poor HMA compaction.

Causes of Thermal Segregation

Thermal segregation problems have been studied by means of the determination of areas with high temperature differentials commonly called “cold areas.” Overall, these areas have lower temperatures than the surrounding mat leading to higher air voids and therefore, to accelerated aging of the asphalt binder.

Figure 2.1 shows a typical thermal profile containing high temperature differentials on the mat, prior to its compaction.

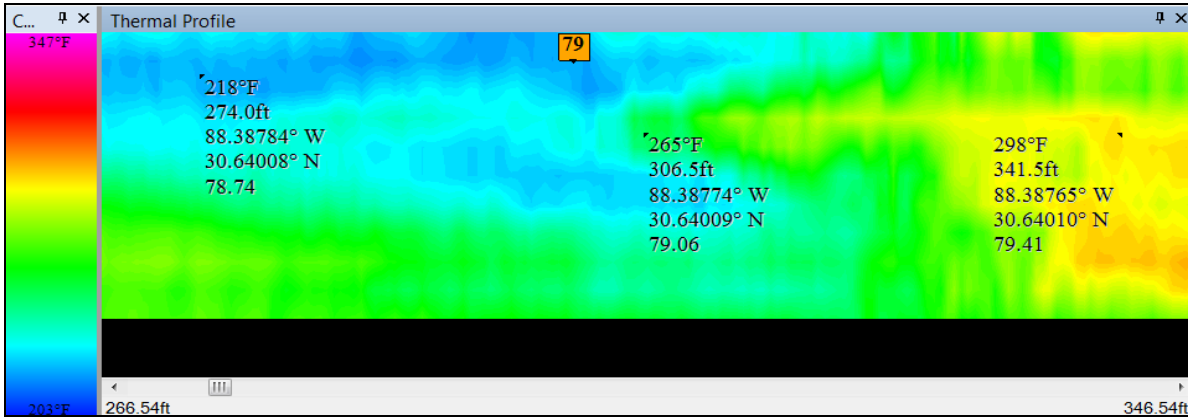


Figure 2.1: PAVE-IR Infrared Bar Thermal Profile with High Temperature Differentials

Many researchers have established that cold areas are mainly caused by the loss of temperature of HMA during hauling and paving operations. Myers, Mahoney and Stephens (4) found that the combined effect of the last discharge coming from a haul unit and the first discharge coming from the next truck can produce cold areas, since the material in the perimeter of the hauling unit and near the surface tended to cool quicker than material near the core of the load.

Myers, Mahoney and Stephens also observed that the accumulation of cold material on paver hopper wings contributed significantly to the formation of cold areas. When trucks are placing the mix in the paver hopper, one portion of the material tends to flow directly to the slat conveyors and at the same time, the other portion tends to remain in the hopper wings until they are tilted toward the center of the hopper. During this process, the mix that remains in the hopper wings cools down faster than the material that flows directly to the slat conveyors increasing the formation of cold areas.

A research developed by CEDEX and SACYR in Spain (5) demonstrated that paver stops are critical in the formation of cold joints. When a paver stops, the mix that remains in the paver hopper, the slat conveyors and the augers in front of the screed cools down quickly leading to a loss of consistency of the mix and increasing the risk of severe transverse cracking. They observed that a 10 min paver stop can produce temperature differentials higher than 122°F (50°C).

Another identified cause of thermal segregation is related to the use of material transfer/remixing devices (known as MTD/MRD). A study by the Colorado Department of Transportation (6) in 2005 included the analysis of two types of transfer devices: Material Transfer Devices (whose function is to pick up and to remix the material), and the Windrow Elevator (whose function is only to pick up windrowed mix and convey it to the paver hopper). Results demonstrated the presence of high temperature differentials once every 42 tons due to the absence of a remixing process. As expected, the use of MTDs and Windrow elevators reduced the presence of cold areas once every 182 tons and once every 197 tons respectively, concluding that remixing processes are necessary to reduce high temperature differentials.

This study also reported the analysis of two types of mix gradations by comparing the frequency of cold areas for each one. According to the results, the coarser gradation showed the presence of cold areas once each 68 tons whereas the finer mix registered the presence of cold areas once each 214 tons. This led to the conclusion that coarser mixes cool down quicker than finer mixes. Based on this, it is expected that mixes like Stone Matrix Asphalt (SMA) and Open Graded Friction Courses (OGFC) cool down faster than well-graded mixes since they are characterized by high asphalt contents, high air voids and a considerable presence of coarse aggregate. High air voids makes the binder more accessible to the environmental effects, causing a rapid viscosity increase and variability in the consistency of the mix and reducing the time available for compaction.

Once the mat is spread by the paver, the time available for compaction becomes dependent on the mix temperature and layer thickness. Thicker asphalt layers lose temperature more slowly than thinner layers since they present smaller surface-volume ratios. In 2005, Flexible Pavements of Ohio (7) published a relationship between time for compaction and layer thickness for three different job mix formula temperatures using Pavcool software, developed by the Minnesota DOT. They obtained a time of compaction of 7 minutes for a 1.25in lift thickness and a mix temperature of 275°F whereas for a 2in layer thickness and the same mix temperature, the time available for compaction increased to 17 minutes. This fact led the authors to conclude that by increasing the layer thickness, the time available for compaction

increases for the same mix temperature. Therefore, surface courses (generally thinner than binder courses) are more susceptible to a rapid loss of temperature and thus, more vulnerable to temperature segregation.

HMA thermal segregation is even more critical when paving projects are executed under conditions associated with long hauling times, cold or rainy conditions and initial mix temperatures that do not meet the specifications of job mix formula temperature. For example, Amirkhanian and Putman (8) found that when hauling times exceeded 70 minutes, the percent of pavement affected by the high temperature differentials increased notably (119°F for nighttime projects and 73°F for daytime projects).

The aforementioned factors contribute to the reduction of time available for compaction and increase the risk of low-density zones in the pavement.

Manifestations of Thermal Segregation in Paving Projects

Areas with high temperature differentials can be manifested in different shapes and signs. Many researchers have tried to associate these shapes with different causes of thermal segregation to identify the real problem and to establish quick corrective actions.

Amirkhanian and Putman (8) found that zones of the mat with high temperature differentials showed two typical shapes identified with an infrared camera. The first type corresponded to a streak described by researchers as a “*long, narrow band of cooler material that runs parallel to the direction of the paver*” as shown in Figure 2.2:

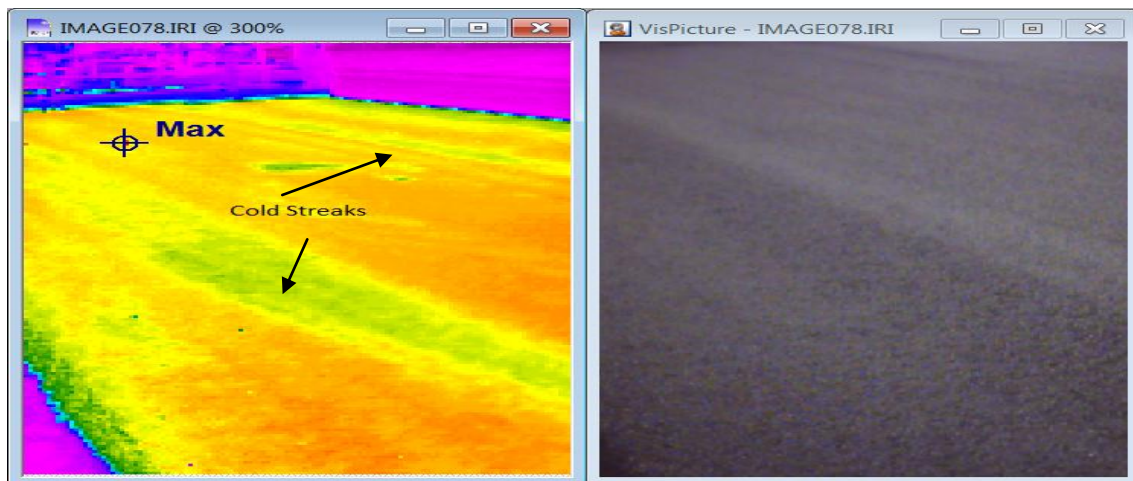


Figure 2.2: Band of cooler material parallel to the direction of the paver

They associated this manifestation to an incorrect functioning of the paver screed whose function is to control the layer thickness and to provide a smooth surface. The paver screed must maintain a uniform mix temperature along the screed to avoid the formation of cold streaks.

The second identified shape is described as a “cold spot” or a “localized area of low temperature material” (Figure 2.3) basically caused by the remixing of cold material with warmer material during the paving process.



Figure 2.3: Cold Spot (Infrared Image taken from I-R Analysis Software)

These two types of manifestations correspond to low density areas that can cause accelerated pavement damage such as potholes, raveling, permanent deformation and moisture induced damage.

Consequences of Thermal Segregation on HMA In-Place Density

The lack of temperature uniformity in the asphalt concrete is a type of segregation that negatively affects pavement performance.

This lack of uniformity reduces consistency and workability of the paving mixture which leads to low density zones generally called “cold spots.” The low density zones are more permeable to air and water that cause accelerated aging of the binder and an increase of moisture susceptibility leading to a

progressive loss of durability and bond between the aggregate and the binder. During the first few years of service, these areas tend to show evidence of fatigue cracking and moisture damage reducing the life of the pavement.

A research developed by CEDEX and SACYR in Spain (5) evaluated the effects of thermal segregation on certain characteristics of HMA that included density, Cantabro abrasion loss (dry and wet condition) and stiffness in indirect tension. Researchers took samples in zones where temperature differentials were higher than 27°F (15°C) and compacted specimens using the Marshall compaction method. The results showed that decreasing the compaction temperature caused a decrease in density and stiffness modulus as well as an increase in Cantabro abrasion loss. Based on this, the researchers demonstrated the negative effect of temperature segregation on material properties.

In 2000, a study conducted by the Washington State Transportation Center (1) evaluated seventeen paving projects to create a procedure that used temperature differentials to obtain density profiles on the compacted mat. Three to four longitudinal density profiles per project were performed using nuclear density gauges for determining density ranges (maximum-minimum density) and density drops (average-minimum density). The temperature differentials, identified with an infrared camera, were used to define where density profiles had to be performed. Density profiles showed that where temperature differentials exceeded 25°F, 90% of in-place densities failed to meet the minimum density criteria. On the other hand, the 80% of the in-place densities met or exceeded the density specification criteria where temperature differentials were less than 25°F.

Levels of Thermal Segregation

Since the study of thermal segregation began, many researchers have tried to establish levels or ranges of temperature differentials in order to define a specification that can be used for quality assurance and quality control purposes.

In 2000, a study performed under the National Cooperative Highway Research Program, NCHRP (9) proposed different levels of temperature segregation as a function of temperature differentials between the hottest and coldest temperature using thermal images as follows:

- Not segregated: areas with temperature differentials less than 18°F (10°C)
- Low level segregation: discrete areas with a mean temperature between 19.8°F (11°C) and 28.8°F (16°C) cooler than surrounding mat
- Medium level segregation: discrete areas with a mean temperature between 30.6°F (17°C) and 37.8°F (21°C) cooler than surrounding mat
- High level segregation: discrete areas with a mean temperature of more than 37.8°F (21°C) cooler than surrounding mat.

Stroup-Gardner and Brown also proposed a pay reduction factor when low level segregation was identified. For medium level segregation, they suggested pay reduction factors or the removal and replacement of the identified segregated area plus 50ft on either side of this area. For high level segregation, they only proposed removal and replacement of the identified segregated area plus 50ft on either side of the area.

During a study developed by Colorado Department of Transportation (6) in 2005, Gilbert defined “cold areas” as those locations with a temperature differential of more than 25°F from the surrounding area. These areas were identified using a thermal camera and a handheld infrared surface-temperature thermometer.

A specification created by the Texas DOT (10) established two levels of segregation that appears to be more flexible than those levels established by the above studies. These levels of thermal segregation are:

- Moderate thermal segregation: areas that have a maximum temperature differential greater than 25°F (14°C) but that does not exceed 50°F (28°C)
- Severe thermal segregation: areas that have a maximum temperature differential greater than 50°F.

These levels have been utilized by new technologies like infrared bars to provide users the number of profiles with low, moderate or severe thermal segregation along the evaluated pavement area.

Relationship between Aggregate Segregation and Thermal Segregation

The significance of “segregation” in the context of asphalt paving corresponds to the lack of uniformity in the mix constituents of the in-place mat. This lack of uniformity can manifest as a lack of homogeneity of the aggregates or the mat temperature. Various studies have investigated the possible relationship between aggregate segregation and temperature segregation.

This relationship was studied by the Connecticut Department of Transportation (3). Henault and Larson were interested in determining a relationship between thermal segregation and particle segregation and how an infrared camera could be used to identify particle segregation on HMA. During the study, they extracted HMA cores at locations identified as cold and normal areas in order to evaluate air voids, gradation and asphalt content.

By comparing the percent of material passing the No. 4 and No. 8 sieve obtained for each core with the limits proposed by Brown (11) for detecting aggregate segregation (differences greater than 8% passing these sieves), they found that material from cold areas did not always present aggregate segregation. Taking this fact into account, Henault and Larsen demonstrated the independency between thermal segregation and particle segregation and that fact that one of these conditions can exist without the presence of the other.

A similar procedure was followed by the Washington State Transportation Center in 1998 (1). Researchers determined the gradation and the asphalt content of the cores obtained at cold and normal spots without finding aggregate segregation leading to the conclusion about the independency between both types of segregation.

Overall, both types of segregation have different causes but their effects are the same: high air voids, low density zones, accelerated aging of the asphalt binder and eventually, premature pavement distresses.

Identification of Thermal Segregation during Paving Projects

Since thermal segregation appeared as an important issue in the asphalt paving industry, many attempts have been made to quickly identify this problem in the field and how to correct it. The identification of thermally segregated areas was not an easy task until the application of infrared imaging technology during pavement construction.

In 1998, a study developed by the Washington Department of Transportation (1) included the use of an infrared image camera and a temperature probe to measure mat temperatures behind the paver prior to compaction. During the same year, the Connecticut Department of Transportation (3) performed the evaluation of eleven projects whose secondary objective was to implement the use of infrared cameras to improve HMA construction methods and to reduce the effects of thermal segregation. Both studies concluded that infrared imaging technology constituted an appropriate way to detect zones with high temperature differentials.

The function of these cameras is to take infrared images that allow measuring the surface temperature of the analyzed material based on the radiation that it emits. Infrared cameras provide images that associate mat surface temperatures with a range of colors (Figure 2.4). This allows collecting temperature measurements without the necessity to be in physical contact with the material and at the same time, to obtain quick, reliable and precise measurements. However, due to its high cost, the high number of pictures necessary in the field, the limited dimensions of the camera's field of view and the fact that it is difficult to match temperature readings with their exact location in the field, researchers continued to search for alternatives that offer the same or better efficiency in cost and data collection.

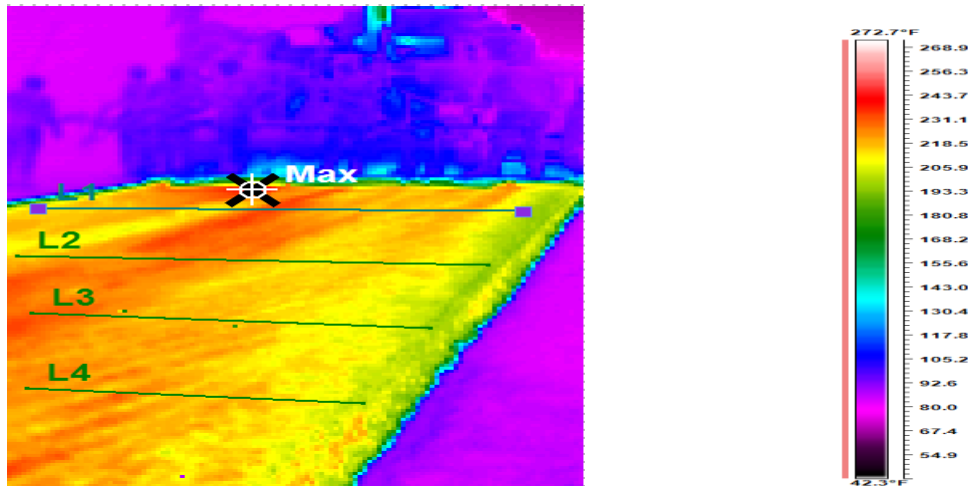


Figure 2.4: Image taken with an infrared camera (Source: RAZ IR Analysis Software)

In 2005, the Colorado Department of Transportation (6) thermal segregation research included as one of its objectives to compare the measurements obtained with the high cost infrared camera with the less costly infrared gun. The idea of this comparison was to check the equality of the readings by applying a paired t-test. The statistical analysis concluded that both devices reported equal results demonstrating that inexpensive infrared guns can be used as an appropriate alternative.

The principle of function of temperature guns (technically called pyrometers) is similar to the infrared camera. Basically, infrared guns are composed of an optical system and a detector. This optical system channelizes the thermal radiation emitted by the material into the detector and then, this thermal radiation is related to temperature based on Stefan-Boltzmann law (12). This law states that the energy radiated by a black body is directly proportional to the fourth power of the black body temperature.

Due to the limitations of the infrared cameras and the temperature guns, researchers began to investigate the development of other type of equipment that allows detecting high temperature differentials. In 2000, Stroup-Gardiner (13) developed the first prototype of an infrared sensor bar that could be mounted to the back of the paver for the acquisition of the temperature data. Additionally, the research included the creation of software that would allow the analysis of the measurements taken by each sensor. Basically, the prototype was composed by a transverse line of infrared sensors, a global positioning system (GPS) and a computer data acquisition device. After several applications of this

prototype on existing pavement surfaces, the system was determined to be efficient regarding data acquisition, data analysis and location of thermally segregated areas.

In 2003-2004, the Texas Transportation Institute (TTI) (14) developed a similar prototype called Pave IR-System for paving project evaluations. The first version of this equipment consisted of 10 temperature sensors placed along a transversal bar attached behind a paver. The function of these sensors (Figure 2.5) was to create a transversal temperature profile of the mat and to send the information to a computer whose function was to collect the information in real time during paving operations. Additionally, the TTI designed software provided basic statistics for collected data.

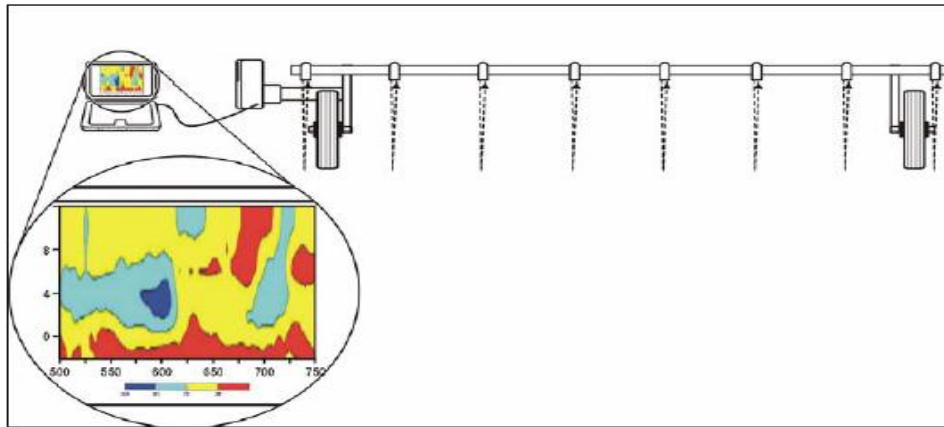


Figure 2.5: Function of Pave-IR Infrared bar (Source: Texas Transportation Institute¹⁴)

Basically, this equipment and its software were designed exclusively for the study of thermal segregation as well as for its application for quality assurance/quality control during paving operations. Like temperature guns, infrared cameras need to be handled by an operator whereas infrared bars can be operated without operator attendance. During the last few years, the Texas Transportation Institute has worked on improving of the Pave-IR System. In 2008, TTI (15) decided to include a Global Positioning System (GPS) in order to provide a more accurate positioning of recorded information.

Infrared bars solved the problems with infrared cameras regarding temperature reading location, field of view and data collection. As a result of this research and development, the Texas Department of Transportation (TxDOT) (10) has encouraged the use of infrared bars as part of the quality control performed by contractors.

HMA Thermal Segregation: Effects on Flexible Pavement Performance and Durability

The presence of high air void contents on asphalt pavements has a significant effect on their durability during their performance period causing the early presence of distresses. One of the most serious effects of low in-place densities is the reduction of the fatigue life of pavements (16, 17). Asphalt surface layers with high air void contents caused by high temperature differentials are more susceptible to hardening and oxidation of the asphalt binder than lower and upper binder courses, due to access to air. The increased rate of aging of the binder results in an increase of mix stiffness and makes the asphalt layer more brittle, reducing its fatigue life (18). Basically, the presence of low in-place densities reduces the ability of the asphalt layer to withstand tensile stresses produced by traffic loads whose magnitude is critical at the bottom of the asphalt layer (19).

There are a few studies that have investigated the direct relationship between HMA thermal segregation and the fatigue life of pavements. Overall, researchers were interested in assessing the effects of high temperature differentials in pavements by monitoring the evaluated projects for many years after construction. However, many research projects have used laboratory testing to establish the relationships between low in-place densities and fatigue performance by varying the air void contents for a same mix.

Harvey et al. (20) studied the effects of air voids and asphalt content on mixture fatigue performance by applying bending beam fatigue tests on several mixes, varying the aforementioned parameters. Regarding air voids, the laboratory study considered the compaction of specimens at low (1 to 3%), medium (4 to 6%) and high (7 to 9%) air void contents. The results indicated that the mixtures with the same asphalt content (5%) suffered a 30% of reduction of fatigue life when only air voids were increased by 1%. On the other hand, when only asphalt content was reduced by 1% from its target value (5%), the fatigue life was diminished to only 12%. This demonstrated a greater influence of air voids in fatigue cycles.

As part of the same study, Harvey et al. (16) evaluated several fatigue models that included independent variables such as air voids, asphalt content and micro-strain levels. Other variables like volume of aggregate, volume of binder and voids filled with binder were not as significant. The model

with air voids, asphalt content and strains was found to be the best to predict the fatigue life with an R^2 value of 0.916. Overall, Harvey et al. concluded that high air void contents considerably reduce pavements fatigue life.

Blankenship and Anderson (21) evaluated the relationship between HMA densities and pavement durability by applying several performance tests that included beam fatigue testing on specimens compacted at different density and asphalt content levels. According to their results, lower air voids led to an increase in cycles to failure. This behavior was more evident for specimens tested at lower micro-strain levels (350 and 400 micro-strains). In general, an HMA density increment of 1.5% produced an 8% increase in the HMA fatigue life.

Probably the most direct evaluation of fatigue response to varying material properties was developed in the Westrack Road Test in Nevada. In 1998, Epps et al. (17) reported the first results of field and laboratory experiments that included the evaluation of 34 test sections (26 sections originally placed and one year later, 8 failed sections were replaced with new sections). The objective of the study was to create a performance-related specification based on the effects of materials and construction variability on pavement performance. These variations considered modifications of aggregate gradation, asphalt content and in-place density. Regarding in-place density, three different air void content levels were proposed as low (4%), medium (8%) and high (12%). At the same time, this factor was combined with the different gradations and 3 different asphalt content values proposed in the research program. According to the field performance monitoring during the first 2 years, the largest concentration of fatigue cracking appeared in the test sections with high initial air voids and in minor amount in those test sections with low asphalt contents.

Bending beam fatigue testing was also performed using samples sawed from slabs extracted from the test sections. Based on linear regression models, results showed that the influence of air voids and asphalt content was greater than the effect of aggregate gradation, consistent with the field performance results. They suggested that in-place densities was the most significant factor on asphalt pavement fatigue life

since a good compaction process provides lower air voids between particles avoiding zones of high tensile strain concentrations that can increase the probability of accelerated fatigue deterioration.

On the other hand, one study contradicted the above studies and concluded that bending beam fatigue tests are insensitive to change in air voids. Mogawer et al. (19) analyzed the effects of HMA density in terms of rutting and fatigue cracking as well as the relationship between density and mix stiffness. The results obtained from bending beam fatigue tests did not show a clear tendency between density and the number of fatigue cycles.

Regarding the influence of density on HMA permanent deformations, there is a general consensus that increasing HMA density reduces rutting. Mogawer et al. (19) utilized the Asphalt Pavement Analyzer (APA) rut and flow number tests to study the influence of air voids on HMA permanent deformations. In contrast with the bending beam fatigue tests, both rutting test results were found to be sensitive to density changes. Based on the results, Mogawer concluded that low densities increase the risk of premature HMA permanent deformation.

As part of the same study, researchers utilized the dynamic modulus (AASHTO TP 62) to analyze the effects of air voids on mix stiffness, as well as to predict rutting damage over a period of time by using the Mechanistic-Empirical Pavement Design Guide (MEPDG) software (23). The results obtained by using the Asphalt Mixture Performance Tester (AMPT) showed that mix stiffness increases with an increasing of mixture density. Based on the MEPDG rutting distress model, rut depth decreases with an increasing of mix density.

However, several studies have demonstrated that fatigue life is less sensitive to air voids than moisture damage susceptibility. Research at Purdue University (22) studied the effects of density on moisture damage and permeability of HMA pavements. Bending beam fatigue tests were executed using beams compacted at different density levels (90% to 96%). Additionally, several specimens were moisture-conditioned to between 55 and 70% saturation to analyze the effects of moisture damage on pavement fatigue life. Results showed that the number of cycles was insensitive to changes in air void contents.

However, del Pilar and Haddock (22) found that the number of cycles to failure for the moisture conditioned specimens decreased by 75% relative to unconditioned specimens.

Additionally, they applied a laboratory wheel tracking device (PurWheel) to simulate the conditions associated with rutting and stripping in the field. Specimens were conditioned to a saturation period of 20min prior to testing. This allowed studying the combined effect of moisture and density changes on HMA rut depth. Based on results, an increment of air voids produced high rut depths.

Several studies have also examined the influence of HMA density on pavement performance and durability in terms of stiffness. Del Pilar and Haddock (22) utilized the dynamic modulus obtained for each density level to study the relationship between air voids and mix stiffness. For this purpose, specimens were compacted to different density levels in both unconditioned and moisture conditioned states. Based on the results, a reduction of air voids from 10% to 4% increases the stiffness by 67%, leading the authors to conclude that a reduction of air void contents increases HMA dynamic modulus. Additionally, researchers found that air voids were significant to initial stiffness, according to bending beam fatigue results.

Indirect Tension Testing and Fracture Energy

During the last few years, several studies have suggested that the current Superpave volumetric mix design should be complemented with a set of simple laboratory performance tests that can ensure reliable mix performance under different traffic and environmental conditions and that can be used as part of quality control and quality assurance programs in paving projects.

One of the simple tests proposed for predicting fatigue cracking is the indirect tension test (IDT). This test method was suggested by several researchers taking into account many advantages such as the simplicity of its execution as well as the fabrication of the specimens with the Superpave Gyratory Compactor (SGC). Additionally, the stress-state in the center of a specimen subjected to indirect tension reflects a similar behavior to the stress-state at the bottom of the asphalt layer in a pavement structure (24). The aforementioned characteristics represent common disadvantages of the bending beam fatigue

testing. The indirect tensile tests are used to create tensile stresses along the diametral axis of a SGC specimen by the application of a vertical compressive load along its thickness. The load is applied through “*two diametrically opposed, arc shaped rigid platens*” (25) at a constant deformation rate until the specimen fails along its vertical diametral axis, as shown in Figure 2.6.



Figure 2.6: IDT Specimen Testing

Although the principal objective of an IDT test is to measure the indirect tensile strength of an asphalt specimen, there are other engineering parameters that can be obtained from this testing method and that can be useful to study the behavior of any mixture. One of these parameters corresponds to the fracture energy until failure. Wen (24) defined fracture energy as “*the sum of the strain energy and the dissipated energy due to structural changes (such as microcracking)*”. This definition considers that the calculation of the resistance of an asphalt mixture to fatigue cracking including its resistance to both deformation and damage. Mathematically, the fracture energy is commonly defined as “*the area under the stress-strain curve in the loading portion*” (24) as shown in Figure 2.7.

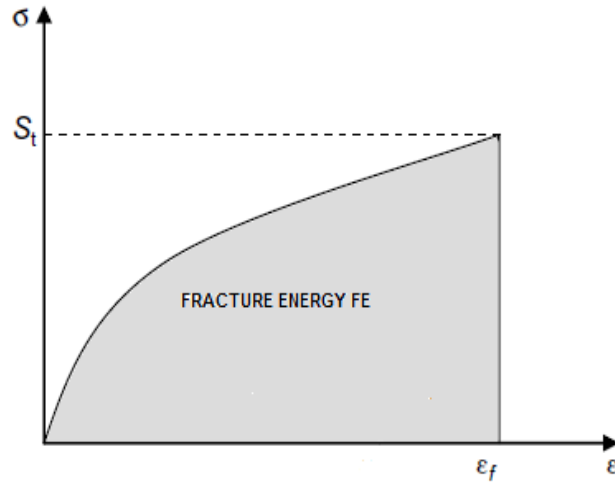


Figure 2.7: Fracture Energy at Peak Load

Wen (24) studied the application of the fracture energy as a simple performance indicator for fatigue cracking in asphalt pavements. This was performed by correlating the percent of cracking measured in the Westrack test sections with several engineering parameters obtained from the application of IDT testing on 6" (150mm) diameter field cores extracted from those test sections. The samples had a thickness of 1 ½" (38.1mm). Prior testing, specimens were conditioned to a testing temperature of 20°C and then, failed under a loading rate of 2"/min (50.8mm/min). Gauge-point mounted LVDT's were used to measure horizontal and vertical deformations on the faces of the specimens. Deformation measurements were divided by the gauge length (2 inches) to determine both vertical and horizontal strains. Based on the results, the fracture energy was the only parameter that showed a very good correlation with fatigue cracking. This correlation demonstrated that high magnitudes of fracture energy are associated to low air void contents and therefore, to less fatigue cracking. These results coincided with the idea proposed by Epps (17) which mentioned that fatigue cracking levels increased significantly as air void content increased. Finally, Wen proposed fracture energy as an "*excellent indicator of the resistance of mixtures to fatigue cracking*". He recommended the IDT testing as a simple performance test that can be used as a complement of the current Superpave mix design.

Roque and Koh (26) compared the fracture energy of dense and open-graded asphalt mixtures obtained from indirect tensile tests with the recently created dog-bone direct tension to study the

suitability of both test methods for predicting accurately the fracture energy of HMA. They found that although the indirect tensile strength and the failure strain were different between two test methods, the resulting fracture energy was the same. They stated that the first two parameters were susceptible to the differences on the loading rate in the plane of failure whereas the fracture energy showed to be independent from the stress state and the loading conditions. They strongly recommended both testing methods for determining accurately the fracture energy.

According to the available literature, fracture energy represents a valid engineering parameter to analyze the effects of low in-place densities on the fatigue life of pavements, as a consequence of thermal segregation.

Chapter 3 : Field Evaluation and Data Collection

The asphalt thermal segregation study required the evaluation of different paving projects as well as the execution of laboratory testing that could simulate the effects of thermally segregated areas in pavement performance and durability. This section presents a detailed description about the methodology and procedures followed during the development of this study.

Field Evaluation

The field evaluation of thermal segregation was performed in 28 paving projects in the state of Alabama during the 2010-2011 paving season. Field data were collected from three paving projects per Division, except for the 3rd Division, where four paving projects were evaluated to include an open-graded friction course in the evaluation. The evaluated projects were selected based on the following requirements:

- Hot-mix asphalt, warm-mix asphalt, stone-matrix asphalt or open-graded friction courses
- Lower, upper binder courses or surface courses (uniform layers in thickness)
- Minimum paving length: 2,500ft
- Only main lanes under constant traffic loads (shoulders not included)

The figure 3.1 shows the distribution of the evaluated roads in the state of Alabama.

Once the projects were selected, it was possible to visit the construction site where the field data were collected, before and during the paving operations. The appendix A shows the data collected for each evaluated project. The field data collection included the following aspects:

1. Job mix formula compaction temperature
2. Mix maximum aggregate size (MAS) and type of binder grade
3. Layer: lower/upper binder courses or surface (wearing) courses
4. Layer thickness
5. Hauling time and distance

6. Type of hauling units: end dump, live bottom or belly dump trucks
7. Truck bed cover during hauling operations
8. Type of material remixing device (MRD)/material transferring device (MTD)
9. Type of paver
10. Weather conditions
11. Ambient temperature
12. Existing surface temperature

EVALUATION OF THERMAL SEGREGATION

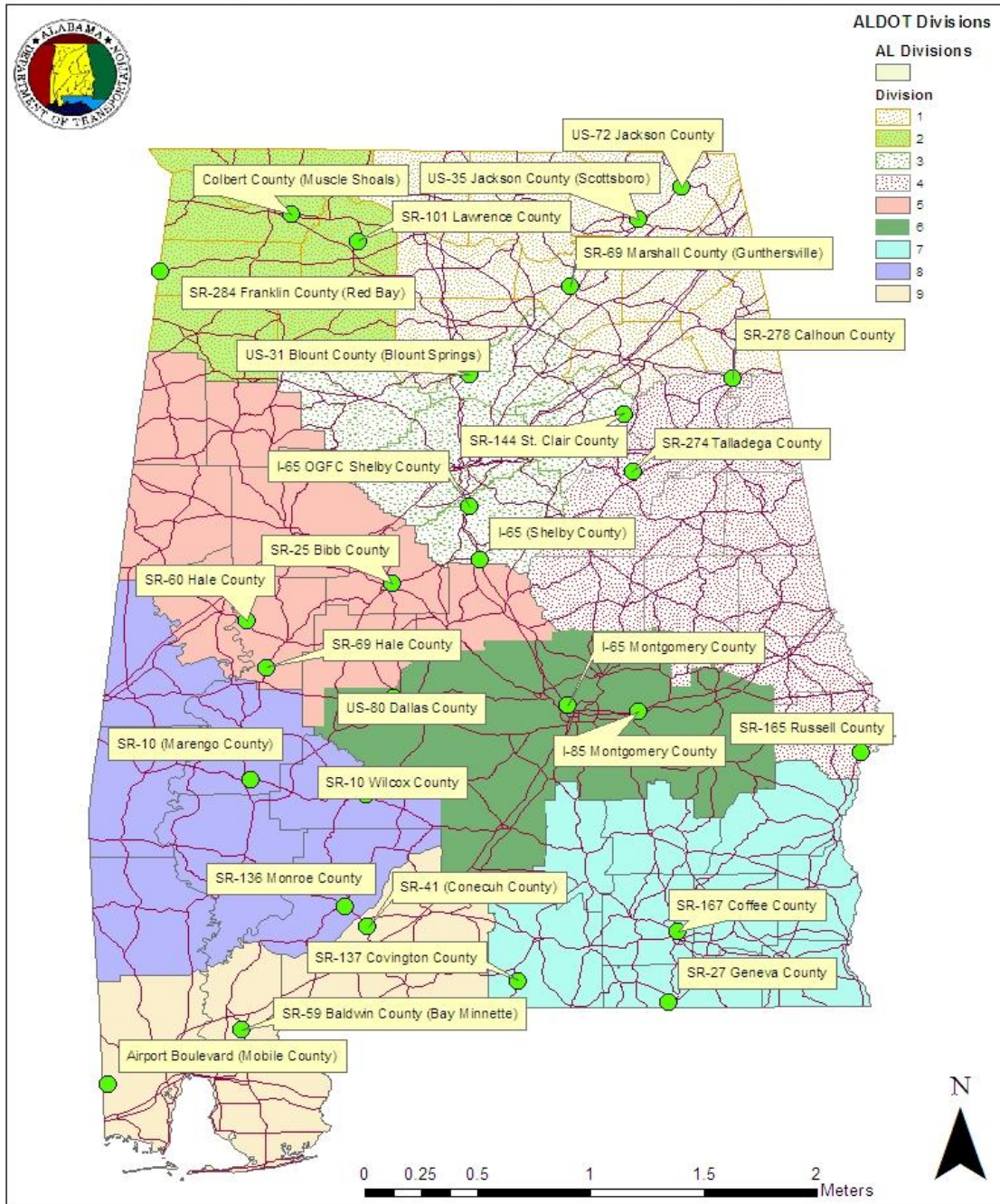


Figure 3.1: Asphalt Thermal Segregation Evaluated Sections

The first five parameters were provided by both ALDOT inspectors and the contractor before the beginning of the paving operations. The information regarding the equipment used for the paving activities was collected in the field before the execution of the paving operations as well. Finally, the

ambient temperature and the existing surface temperature were measured during the paving operations with a hand-held weather station and a handheld infrared gun, respectively.

The aforementioned information was collected in a field worksheet shown in Appendix B.

Temperature Data Collection

The temperature of the mat prior to its compaction was obtained with three different devices, capable of measuring the surface temperature of the placed mix. The MOBA Pave-IR infrared bar (Figure 3.2), a RAZ-IR infrared camera (Figure 3.3) and a Raytec Ranger hand-held infrared gun (Figure 3.4) were used as part of the temperature data acquisition.



Figure 3.2: MOBA Pave-IR bar



Figure 3.3: RAZ-IR Infrared camera



Figure 3.4: Raytec Ranger Hand-held Infrared Gun

MOBA Pave-IR Infrared Bar Data Acquisition

The MOBA Pave-IR infrared bar was used to determine real time surface temperatures of the mat behind the paver screed. Once the infrared bar was attached to the back of the paver, the information regarding the evaluated project was input in the bar operative system and then, the odometer (device to measure linear lengths) was placed on the paver wheel and calibrated by considering a distance of 200ft. The calibration would allow matching the thermal profiles with the stations defined in the project. Additionally, this would help obtaining a more accurate reading of the paver speed as well as the distance covered by the thermal profile. After the calibration of the odometer, it was possible to start the temperature readings.

The temperature range introduced in the Pave-IR operative system depended on the target compaction temperature of the mix being evaluated. Therefore, it was necessary to define if the mix was HMA, WMA or in some cases, SMA. Additionally, it was necessary to ignore some sensors (generally the sensors located at the ends of the infrared bar) during the data acquisition, especially in those paving projects where the lane width was within 10ft to 14ft. This would avoid obtaining temperature readings from materials outside of the newly paved edges that could give erroneous results. All the thermal profiles

satisfied the minimum paving length of 2500ft, except for the SR-278 paving project due to problems with the HMA production plant.

The MOBA Pave-IR infrared bar draws a transverse thermal profile of the mat that allows checking the distribution of the mix temperature along the paved length. The infrared bar software records transverse temperature measurements every 6in longitudinally by utilizing twelve sensors spaced every one foot. More readings could be taken but it would result in an overwhelming amount of data. A Global Positioning System (GPS) was incorporated during the measurements to locate the exact position of the hot and cold areas and thus, to obtain field cores from the desired locations using the coordinates provided by the GPS.

Once the field evaluation was finished, the thermal profiles were analyzed in the Pave Project Manager Software (31). This tool allowed determining the magnitude of the mat temperature as well as its location along the evaluated length. Additionally, the software was used to determine the levels of segregation of each project. Basically, the Pave Project Manager divides the evaluated section into 150ft thermal profiles. For each profile, the minimum and the maximum temperature is used to determine the highest temperature differential as follows:

$$\Delta T = T_H - T_L$$

Equation 1

Where:

ΔT : temperature differential (°F)

T_H : highest temperature in a 150ft thermal profile (°F)

T_L : lowest temperature in a 150ft thermal profile (°F)

It is important to remember that the infrared bar determines the levels of thermal segregation as defined by the Texas Department of Transportation (TxDOT) specifications. These specifications divide the levels of segregation in three categories as follows:

- No segregation: temperature differential less than 25°F (14°C)
- Moderate thermal segregation: temperature differential between 25°F (14°C) and 50°F (28°C)

- Severe thermal segregation: temperature differential greater than 50°F (28°C)

Based on these levels, the Pave Project Manager reports the amount of profiles that presents “no segregation”, “moderate thermal segregation” and “severe thermal segregation” as a percentage of the total number of profiles.

$$\% \text{ of Thermal Segregation} = \left(\frac{\text{No. of Profiles with/without Thermal Segregation}}{\text{Total No. of Thermal Profiles}} \right) * 100 \quad \text{Equation 2}$$

Where:

% of Thermal Segregation: percentage of moderate, severe or no thermal segregation along the evaluated length

No. of Profiles with/without Thermal Segregation: number of profiles with no thermal segregation, moderate or severe thermal segregation along the evaluated length

Total No. of Thermal Profiles: total number of 150ft thermal profiles along the evaluated length

Infrared Images Acquisition

Infrared images with the RAZ-IR camera were also used to detect thermally segregated areas. For this purpose, infrared pictures at the back of the paver were taken every 50ft along a total paving length of 2500ft (50 images total). Additionally, several pictures of the mat at the paver screed, the MTD/MRD hopper and conveyor belt (depending on the type of MTD/MRD), and the truck bed were taken to evaluate the mix temperature during hauling, remixing/transferring and paving operations.

Before the images were taken, it was necessary to introduce the emissivity of the asphalt in the infrared camera to obtain more accurate readings of surface temperature. The recommended emissivity value of asphalt varies between 0.90 and 0.98 (32, 33 and 34). For this research, an emissivity value of 0.95 was used.

Hand-held Infrared Gun Temperature Readings

Three temperature gun readings were obtained every 50ft along the paver screed, as shown in Figure 3.5, to draw a temperature profile of the mat prior to its compaction. These readings were taken by a field technician at the back of the paver, along a paving length of 2500ft (150 temperature gun readings).

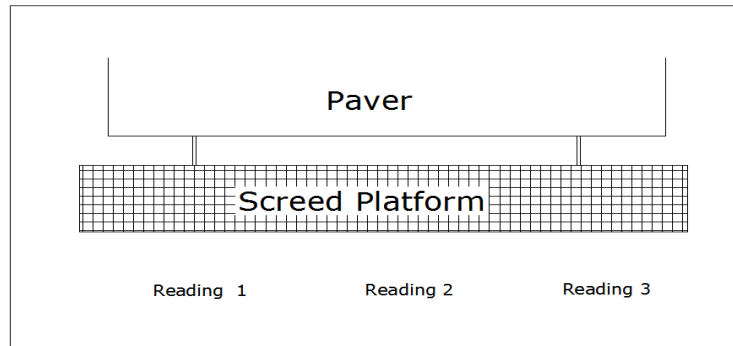


Figure 3.5: Hand-held Infrared Gun Reading Locations

Temperature gun readings were documented in a field worksheet shown in Appendix B.

Analysis of Field Data

As aforementioned, one of the objectives of this study was to identify the most significant parameters on the development of thermally segregated areas during paving operations. In order to accomplish this objective, a General Linear Model (GLM) was executed using a 95% confidence level (p -value = 0.05) and the independent variables shown in Table 3.1 as follows:

Table 3.1 General Linear Model Independent Variables

Independent Variables	GLM Categories	Number of Projects
Material Transfer/Remixing Device	Roadtec SB-2500	16
	Blaw-Knox MC-330	9
	Weiler 1250A	2
	No MRD/MTD	1
Mix Type	Dense-graded HMA	21
	WMA	3
	SMA	3
	OGFC	1
Type of Layer/MAS	Binder 1" MAS	3
	Surface/Binder 3/4" MAS	14
	Surface 1/2" MAS	9
	Surface 3/8" MAS	2
Hauling Time	≤ 20min	4
	21 min ≤ HT ≤ 40min	10
	> 40min	14
Time of Day	Daytime	23
	Nighttime	5
Paver Stop (*)	Accumulated time in minutes	28
Paver Speed (*)	Speed in ft/min	28

(*): measured by the MOBA infrared bar

The GLM analysis was performed in Minitab 16, relating the aforementioned variables with each level of segregation (no segregation, moderate segregation and severe segregation), as the dependent variable of the statistical analysis. It is important to mention that GLM was utilized because some variables in this analysis were qualitative and they could not be used for a regression model. Although layer thickness could be a potential significant parameter, it was not included in the GLM for two reasons. The first reason is related to the wide range of layer thicknesses that were collected from the paving projects. Eleven different thicknesses were registered during the field data collection. They varied from 0.88in (OGFC wearing course) to 3.0in (WMA upper binder course). Eleven of the 28 projects reported a layer thickness of 1.50in whereas 7 projects reported a layer thickness of 1.25in. The remaining 10 paving projects considered 9 different thicknesses. The great difference between the number of observations for each layer thickness led to an unbalanced analysis in the GLM executed by Minitab 16. Comparative

statistical analyses gain reliability when sample sizes are equal. Excessive differences in sample size lead to unreliable analyses. For this reason, Minitab was not able to compute the analyses when this variable was included.

The second reason is associated to the fact that the MOBA PAVE-IR infrared bar takes temperature readings within 2 to 5 seconds after the mix is laid down behind the paver screed. At this stage, the effect of the layer thickness in thermally segregated areas is not significant and therefore, it was not included in the GLM.

Once the GLM was executed, Tukey-Kramer multiple comparisons were applied to the most significant parameters ($p\text{-value} < 0.05$) for determining those means that were significantly different for each variable. This would allow defining, for example, which types of MTD/MRD's or mixes were more prone to mix temperature differentials.

Additionally, the analysis of thermal profiles and infrared images was included in order to demonstrate how each of the aforementioned variables can increase temperature differentials in the mix prior to its compaction.

Evaluation of Hot and Cold Areas

Field Samples

One of the objectives of this project was to evaluate the effect of high temperature differentials on in-place density by obtaining cores from different paving projects. The projects where field cores were taken are listed below.

- Division 7: SR-167 HMA samples (wearing course)
- Division 3: US-31 HMA samples (wearing course)
- Division 6: I-65 SMA samples (upper binder course)
- Division 9: SR-41 HMA samples (wearing course)

- Division 8: SR-10 HMA samples (wearing course)
- Division 1: US-35 WMA samples (upper binder course)
- Division 2: SR-101 HMA samples (wearing course)
- Division 5: SR-69 HMA samples (wearing course)
- Division 4: SR-278 HMA samples (wearing course)

The Pave-IR software was utilized to locate the cold and hot areas. After the evaluation of each project, it was necessary to analyze the thermal profiles provided by the MOBA bar and then it was possible to locate the zones with high and low temperature differentials. The GPS coordinates and the road stations identified in the field were the main references. However, the GPS coordinates were the most reliable source of information since, in some projects, the stations were not very clear and sometimes, not accurate. Taking this into account, the MOBA bar GPS coordinates (given in degrees) were introduced into Google Earth (35) to obtain an illustration of the approximate position of the core locations. These illustrations were printed and taken as another reliable reference. Afterwards, the coordinates given by Google Earth (provided in degrees, minutes and seconds) were converted to degrees and minutes since the GPS used in the field (Garmin GPS recreational-grade receiver) utilized this kind of coordinates. Physical references like road signs, intersections, rivers, etcetera constituted secondary references to support the GPS locations.

A minimum of three cores were extracted for both hot and cold areas to compare in-place densities and to define if the compaction of the mix at low temperatures decreased in-place densities in comparison to the spots compacted at normal compaction temperatures. For all the aforementioned projects, the extracted cores were taken within one month after construction. Figure 3.6 shows the common locations of the extractions along the paving width. It is important to mention that the distance between selected cold and hot spots varied for each project.

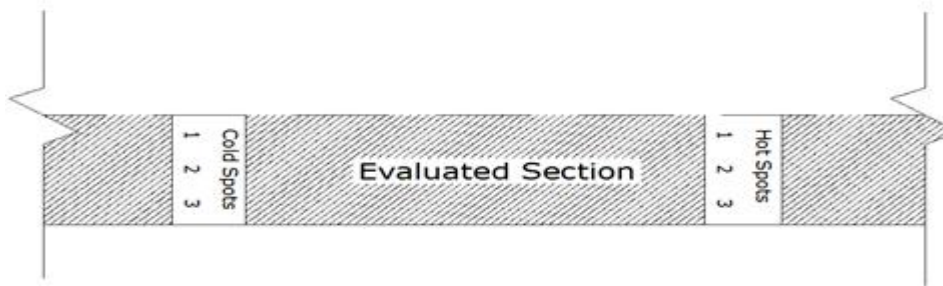


Figure 3.6: Typical Core Locations

The coring was performed by the materials laboratory personnel of each ALDOT Division. NCAT personal supervised the core extractions in seven of the nine projects (only the US-31 and the I-65 projects were not supervised).

Once the cores were extracted from the field, they were transported to NCAT’s laboratory for the volumetric analysis. In-place densities were determined as a percent of the theoretical maximum specific gravity (Gmm) as follows:

$$\%Gmm = \left(\frac{Gmb}{Gmm} \right) * 100$$

Equation 3

Where:

%Gmm: relative in-place density (%)

Gmb: bulk specific gravity determined with AASHTO T-166 (adimensional) ³⁶

Gmm: theoretical maximum specific gravity determined with AASHTO T-209 (adimensional) ³⁷

Regarding the determination of the theoretical maximum specific gravity (Gmm), it was necessary to remove aggregates from the cut faces of the cores where the aggregate was not coated with asphalt binder, prior to testing. This would avoid the absorption of water by these particles and thus, a possible alteration of the real Gmm value.

Analysis of In-Place Densities

In order to compare in-place densities obtained from both hot spots and cold spots, a 2 sample t-test was executed for each project in the Minitab 16 statistical software. The objective of this analysis was to examine if the average in-place densities corresponding to the hot spots were significantly different than average in-place densities of the cold spots, at a significance level of 95% (p-value = 0.05). The results and analysis are presented in Chapter 5.

Chapter 4 : Laboratory Testing Fatigue Performance Evaluation

Bending Beam Fatigue Testing

Mix from each of the 9 paving projects previously mentioned (one paving project per Division) was sampled for bending beam fatigue testing. HMA loose samples were obtained from seven projects whereas stone matrix asphalt (SMA) specimens were sampled only from one paving project. Finally, warm-mix asphalt (WMA) samples were obtained from only one project as well. For all the projects, the material was sampled from the augers that distribute the material in front of the paver screed.

Bending beam fatigue tests were executed in accordance with ASHTO T 321: “*Determining the Fatigue Life of Compacted Hot-Mix Asphalt (HMA) Subjected to Repeated Flexural Bending*” (38).

Basically, eight beam specimens were compacted for each project by using the material sampled in the field. The beams were compacted according to the field air voids obtained by analyzing the cores extracted from the hot spots and the cold spots in the field. Therefore, two target air void contents corresponding to the hot and cold spots on the project were used to prepare the beams. It is important to mention that only 7 of the 9 projects were evaluated since the difference of in-place air voids obtained from the SR-10 and SR-69 projects was less than 1.0% which was considered as an insignificant difference (20). Each specimen was compacted at the compaction temperature determined in the field by using a laboratory beam compactor (Figure 4.1) and then sawed to meet the required beam dimensions specified in AASHTO T 321 (63±6mm width, 50±6mm height and 380±6mm length). Specimens were prepared so that the air void contents were within ±1.0% of the target air void contents.



Figure 4.1: Beam Rolling Compactor

The bending beam fatigue apparatus (Figure 4.2) was located inside an environmental chamber, capable of maintaining the specimen at a constant temperature prior and during testing. Before the application of the bending loads, specimens were conditioned at a temperature of $20\pm 0.5^\circ\text{C}$ for a minimum of 2 hours before the test. After this stage, the beams were placed in the loading device, capable of providing a repeated sinusoidal loading at a certain frequency as well as of forcing the specimens back to its original position at the end of each load pulse. Additionally, specimens were subjected to four-point bending. Table 4.1 shows the established parameters for the bending beam fatigue tests.



Figure 4.2: Bending Beam Fatigue Apparatus

Table 4.1 Bending Beam Fatigue Testing Parameters

Target Air Voids	Micro-strain Level	Number of tests	Frequency (Hz)	Test Temperature (°C)
Hot Spots	300	2	10	20±0.5
	600	2		
Cold Spots	300	2		
	600	2		

The failure criteria used for this project was associated to the initial stiffness (S_0) of the specimen. Based on AASHTO T 321, the initial stiffness of a specimen is reached at the first 50 load cycles. Thus, a stiffness reduction of 50% in regard to the initial stiffness represents the specimen failure and therefore, the end of the test. Equations 4, 5 and 6 show the determination of the stiffness, measured with the bending beam fatigue test.

$$\sigma_T = \frac{3aP}{bh^2}$$

Equation 4

Where:

σ_T : maximum tensile stress (Pa)

a: space between inside clamps (mm)

P: applied load (N)

b: average beam width (mm)

h: average beam height (mm)

$$\varepsilon_T = \frac{12\delta h}{3l^2 - 4a^2}$$

Equation 5

Where:

ε_T : maximum tensile strain (mm/mm)

δ : beam deflection at neutral axis (mm)

h: average beam height (mm)

l: average beam length (mm)

a: space between inside clamps (mm)

$$S = \frac{\sigma_T}{\varepsilon_T}$$

Equation 6

Where:

S: stiffness (Pa)

σ_T : maximum tensile stress (Pa)

ε_T : maximum tensile strain (mm/mm)

After the beams of the first four divisions were tested, it was found that the bending beam fatigue test was not sensitive to changes in air voids (22). For this reason, indirect tensile strength tests were executed as a complement of the bending beam fatigue tests. This would allow determining the fracture energy necessary to produce load-related cracking. This testing procedure was selected since it tends to be sensitive to changes in air voids (24), based on previous research. Despite the unexpected results, bending beam fatigue testing was still performed for the rest of the projects.

Analysis of Results: Bending Beam Fatigue Testing

In order to examine the relationship between the number of fatigue cycles and the air void content at each micro-strain level, it was necessary to apply regression analyses for each project. This would allow evaluating if the number of fatigue cycles (dependent variable) reported by the bending beam fatigue device was directly affected by the air void contents (independent variable). Additional regression analyses were executed for evaluating the relationship between air voids (independent variable) and initial stiffness (dependent variable) of the mix. Both analyses were performed at a confidence level of 95% by utilizing the Minitab 16 statistical software.

Indirect Tensile Strength Test and Fracture Energy

The indirect tensile strength was determined according to sections 7, 8 and 11 of the AASHTO T 283: “*Standard Method of Test for Resistance of Compacted Hot-Mix Asphalt to Moisture-Induced Damage*” (39). Due to the unexpected results from the bending beam fatigue testing, it was necessary to extract additional field cores from the initial four evaluated paving projects. These samples had an age between 11 and 12 months, except for the SR-10 field cores whose age was 5 months. For the rest of the projects, the same cores (age less than one month) used for in-place density determination were utilized for fracture energy.

Prior testing, it was necessary to condition the samples in an environmental chamber for a minimum of 4 hours at a temperature of 25°C. After conditioning, the samples were tested under a compression load, uniformly distributed along the specimen thickness. The applied loading rate was 2.0"/min, based on the ALDOT 361 testing specification (49) for moisture susceptibility. The loading device was a S5840 Geotest loading machine (Figure 4.3) composed by a load cell with a maximum load capacity of 2,500lb and an accuracy of ± 1 lb. A deformation gauge with an accuracy of ± 0.0001 in was used to determine the vertical deformation (ΔD) during the loading period.



Figure 4.3: S5840 Geotest Loading Machine for Indirect Tensile Strength

Both load cell and deformation gauge were able to transfer load and deformation readings to computer software. Thus, it was possible to determine the tensile stress and strain corresponding to each load increment by using equations 7 and 8, respectively.

$$\sigma_t = \frac{2P}{\pi DT} \quad \text{Equation 7}$$

Where:

σ_t : indirect tensile stress at the center of the specimen (psi)

P: maximum applied load (lb)

D: average diameter of the specimen (in)

T: average thickness of the specimen (in)

$$\varepsilon_t = \left(\frac{\Delta D}{D} \right) * \mu \quad \text{Equation 8}$$

Where:

ε_t : tensile strain at each load increment (in/in)

ΔD : specimen vertical deformation (in)

D: average diameter of the specimen (in)

μ : Poisson's ratio

For this study, a Poisson's ratio of 0.35 was used for determining the tensile strains at a test temperature of 25°C (46, 47, 48). Once the tensile stresses and strains were computed, it was possible to draw stress-strain plots (Figure 4.4) for determining the fracture energy. As aforementioned, the fracture energy is defined as the area under the stress-strain curve. Therefore, it was necessary to plot the trend-line that reported an acceptable coefficient of correlation ($R^2 > 0.95$). Then, it was possible to determine the best-fit equation of the trend-line and finally, this function could be integrated taking zero or the strain where the trend line intersects the "x" axis, as the lower limit and the tensile strain at the peak stress (ε_t) as the upper limit to determine the fracture energy (Equation 9).

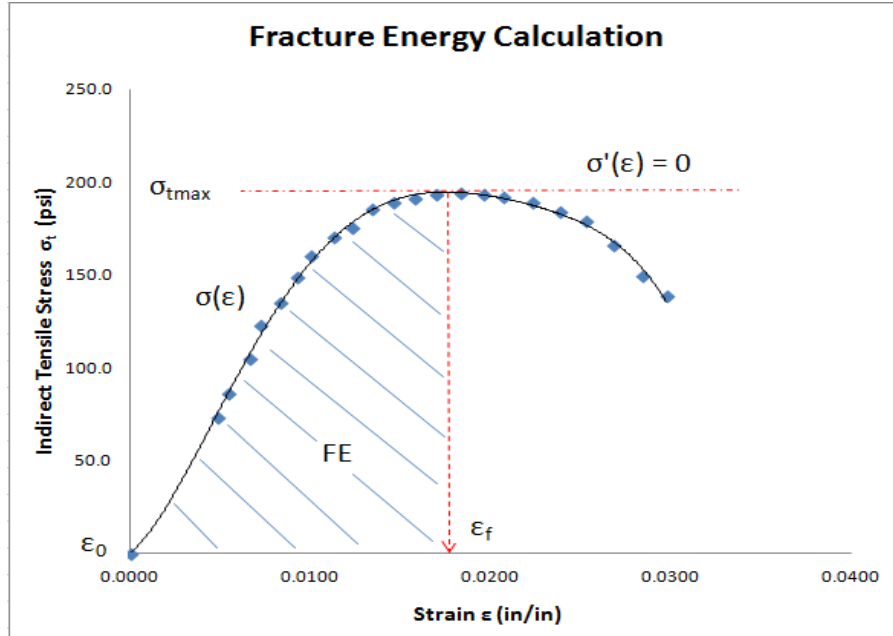


Figure 4.4: Typical Stress-Strain Plot from Indirect Tensile Strength Testing

$$FE = \int_{\epsilon_0}^{\epsilon_f} \sigma(\epsilon) d\epsilon$$

Equation 9

Where:

FE: fracture energy at the failure (psi)

$f(\epsilon)$: function that defines the stress-strain curve

ϵ_0 : zero or the tensile strain where the trend line intersects the “x” axis (in/in)

ϵ_f : tensile strain at the specimen failure (in/in)

It is important to mention that ϵ_f was determined by using the mathematical concept of the “extreme points” of a function. Basically, the first derivative of the equation that best describes the function “ $\sigma(\epsilon)$ ” was determined. Theoretically, the first derivative of a continuous function describes the slope of this function (41). Then, the first derivative “ $\sigma'(\epsilon)$ ” was equaled to zero in order to obtain the strain where the slope of the function was equal to zero. This would allow to obtain the strain (“x” value) where the slope of a function is equal to zero. By definition, this point corresponds to the “x” value (strain) associated to the maximum “y” value (maximum stress) (41). The aforementioned calculations were developed by utilizing an algorithm created in Matlab version 7.10.

Analysis of Results: Fracture Energy

A 2 sample t-test was applied for comparing the fracture energy of the field cores extracted from both hot spots and cold spots at a significance level of 95% (p-value = 0.05). The idea of this analysis was to determine if the specimens obtained from the hot-compacted areas had greater fracture energy than those specimens extracted from the cold areas. The results will be discussed in Chapter 6.

Chapter 5 : Field Evaluation Results and Discussion

This section includes the results obtained from the 28 paving projects evaluated during the 2010-2011 paving season in the state of Alabama. First, the results obtained from the General Linear Model (GLM) will be presented for identifying those parameters categorized as significant for the development of thermally segregated areas (Table 3.1), based on the levels of segregation reported by the PAVE-IR infrared bar. Then, the effect of each variable on HMA thermal segregation will be discussed by following the significance level obtained from the statistical analysis.

General Linear Model (GLM) Results

The tables 5.1, 5.2 and 5.3 show the results obtained from the GLM statistical analysis for the three different levels of thermal segregation: no thermal segregation, moderate thermal segregation and severe thermal segregation.

Table 5.1: General Linear Model Results No Thermal Segregation
 $\Delta T < 25^{\circ}\text{F} / R^2 = 66.07\%$

Independent Variables	Degrees of Freedom D.F	p-value
Paver Speed	1	0.606
Paver Stop	1	0.887
MTD/MRD (*)	3	0.052
Mix Type (*)	3	0.023
Layer-Aggregate Size	3	0.131
Hauling Time	2	0.246

(*): independent variables are significant if $p\text{-value} \leq 0.05$

Table 5.2: General Linear Model Results Moderate Thermal Segregation
 $25^{\circ}\text{F} < \Delta T < 50^{\circ}\text{F} / R^2 = 82.70\%$

Independent Variables	Degrees of Freedom D.F	p-value
Paver Speed	1	0.769
Paver Stop	1	0.599
MTD/MRD (*)	3	0.002
Mix Type	3	0.210
Layer-Aggregate Size	3	0.143
Hauling Time	2	0.859
Time of Day	1	0.816

(*): independent variables are significant if $p\text{-value} \leq 0.05$

Table 5.3: General Linear Model Results Severe Thermal Segregation
 $\Delta T > 50^\circ\text{F} / R^2 = 93.40\%$

Independent Variables	Degrees of Freedom D.F	p-value
Paver Speed	1	0.296
Paver Stop	1	0.567
MTD/MRD (*)	3	0.000
Mix Type	3	0.114
Layer-Aggregate Size (*)	3	0.039
Hauling Time	2	0.205
Time of Day	1	0.684

(): independent variables are significant if p-value ≤ 0.05*

Based on the results, MTD/MRD's constitute the most significant parameter in the development of thermally segregated areas. For all the levels of segregation, the p-value associated to this parameter resulted to be equal or less than 0.05. Additionally, the mix type and layer-aggregate size reported p-values lower than 0.05 for the “no thermal segregation” level and the “severe thermal segregation” level, respectively. From the analyses, it was also found that the relationship between the different variables and the levels of thermal segregation was better explained for the “severe thermal segregation” level analysis ($R^2 = 93.40\%$ and $S = 12.2146$).

Material Transferring Devices (MTD's) / Material Remixing Devices (MRD's)

During the evaluation of the 28 paving projects, it was possible to identify three types of transfer/remixing devices. Sixteen projects were executed with a Roadtec SB-2500 remixing device (commonly called “Shuttle buggy” Figure 5.1) whereas 9 projects considered the use of an Ingersoll Rand Blaw Knox mix transferring device (model MC-330 Figure 5.2). The other type of remixing device was the Weiler 1250A (Figure 5.3) which was utilized only in two projects. Only one project did not take into account the use of this kind of devices.



Figure 5.1: Roadtec SB-2500 Shuttlebuggy MRD



Figure 5.2: Blaw-Knox MC-330 MTD



Figure 5.3: Weiler 1250A MRD

Tukey-multiple comparisons were executed by grouping the identified remixing/transferring devices in four different categories: no MTD/MRD, Roadtec SB-2500 (remixing device), Weiler 1250A (remixing device) and Blaw-Knox MC-330 (transferring device). This was performed in order to check if the remixing operations provided by the Roadtec SB-2500 and the Weiler 1250A had a positive effect in the reduction of the levels of severe temperature segregation when they were compared with the other two categories. For this purpose, the means of the four groups were statistically compared in terms of percentage of severe thermal segregation. Tables 5.4 and 5.5 show the results.

Table 5.4: Mean Comparison Tukey Simultaneous Tests Results

Transfer/Remixing Device (MRD/MTD)	Mean/% Severe Thermal Segregation
No MTD/MRD	69.3
Roadtec SB-2500 MRD	22.4
Blaw-Knox MC-330 MTV	84.1
Weiler 1250A MRD	11.7

Table 5.5: Tukey Simultaneous Tests 95% Confidence Level for MTD/MRD's Severe Thermal Segregation / Mean Comparison

Transfer/Remixing Device (MTD/MRD)	No MTD/MRD	Roadtec SB-2500 MRD	Blaw-Knox MC-330 MTV	Weiler 1250A MRD
No MTD/MRD		NS	NS	SD
Roadtec SB-2500 MRD	NS		SD	NS
Blaw-Knox MC-330 MTV	NS	SD		SD
Weiler 1250A MRD	SD	NS	SD	

SD: significantly different, NS: not significantly different

Based on the results, the effects of both remixing devices (Roadtec SB-2500 and Weiler 1250A) were statistically different from the material transferring devices (Blaw-Knox MC-330) in the formation of thermally segregated areas. The high level of severe thermal segregation reported in table 5.4 (84.1%) for the Blaw-Knox MC-330 MTD demonstrates the negative effects of these devices with regard to the remixing devices. Most of the paving projects where a transferring device was used had higher percentages of moderate and severe thermal segregation than those projects where a remixing device was utilized, which coincides with the multiple comparisons results.

The Blaw-Knox MC-330 MTD is characterized by a hopper that receives the asphalt mix from the hauling units. One particular characteristic of this hopper is that it has wings at the sides where the mix accumulates after the entire load has been discharged. When the hauling unit starts dumping the material in the MTD hopper, part of the material is transported to the paver hopper by the conveyor belt. However, the rest of the material flows towards the sides where the hopper wings are. This volume of material usually remains at this location for several minutes. During this period of time, the mix starts cooling down and then, it is transported to the paver hopper with an inappropriate compaction temperature. Figure 5.4 demonstrates mix temperature differentials at the MTD hopper wings that exceed 80°F. Additionally, Figure 5.5 shows the presence of cold material in the conveyor belt due to several stops of the belt (when the paver hopper is full of material or delays in the mix delivery, etc) and the constant exposure of the material to the environmental conditions. This fact becomes more critical when the paving operations are executed under low environmental temperatures and discontinuous paving operations.

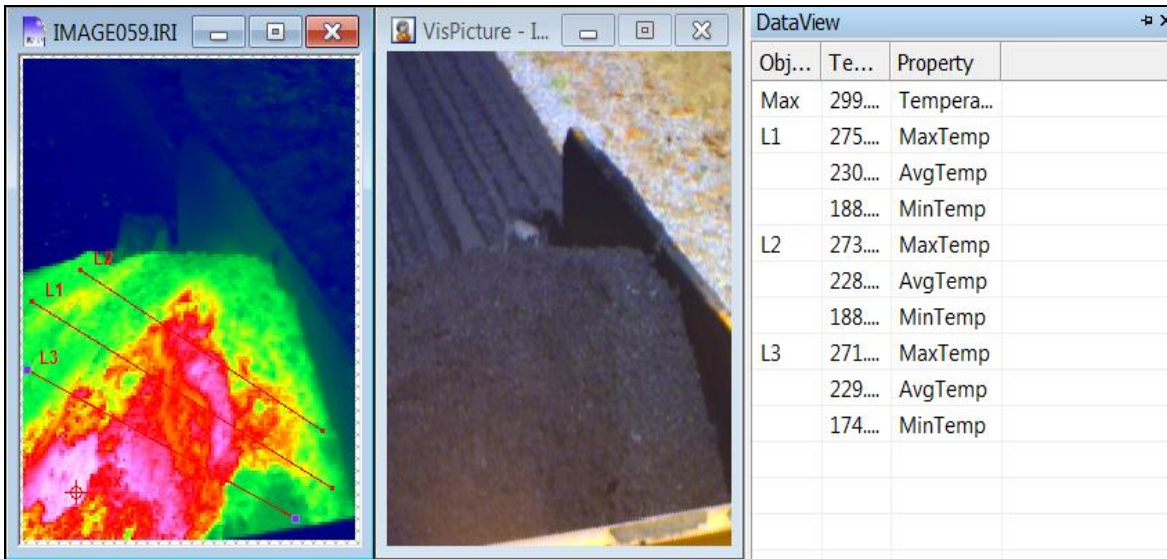


Figure 5.4: Thermal Image Showing Temperature of Mix Accumulated at the MTV Hopper Wings, Project SR-41 (Conceh County)

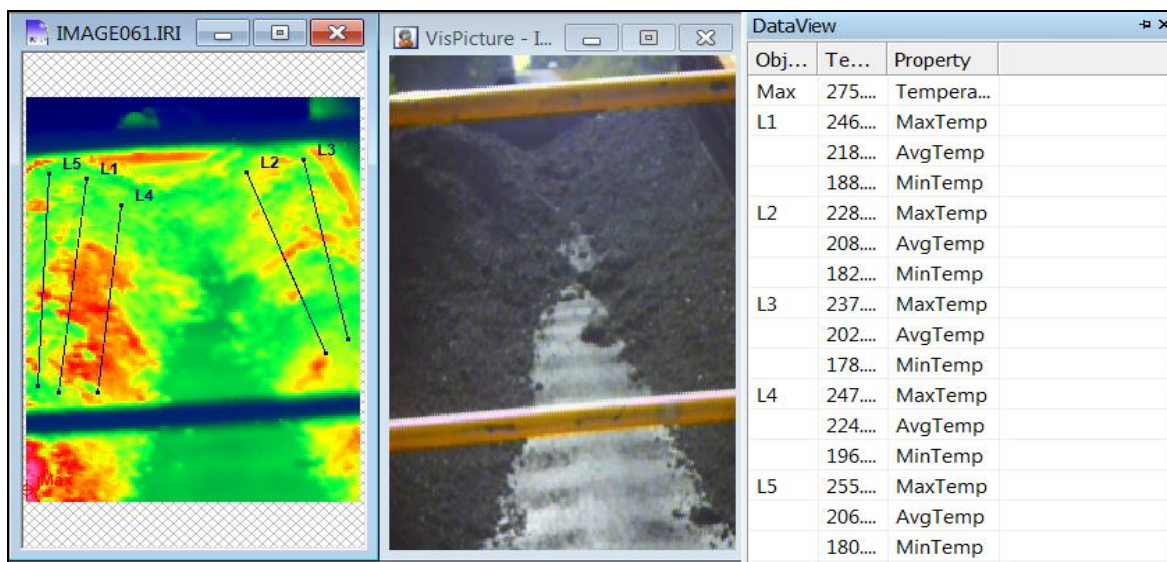


Figure 5.5: Thermal Image Showing Temperature of Mix Accumulated in Blaw Knox MTD Conveyor Belt, Project SR-41 (Conceh County)

The remixing process provided by the internal augers of the Roadtec SB-2500 MRD and the Weiler 1250A MRD reduces significantly temperature differentials, even with long haul times (8). Additionally, the conveyor belts of this equipment are protected by a metallic cover that avoids the exposure of the mix to adverse environmental conditions. In contrast with the Blaw-Knox MC-330 MTD, the hopper of the both MRD's does not have wings and their conveyor belts are wider which allows the movement of a

higher volume of material from the hopper to the internal compartments of the devices. This avoids any accumulation of material at the hopper that could be exposed to the environment.

The figures 5.6 and 5.7 show typical temperature profiles obtained from a paving project where a Blaw-Knox MC-330 MTD and a Roadtec SB-2500 MRD was used, respectively.

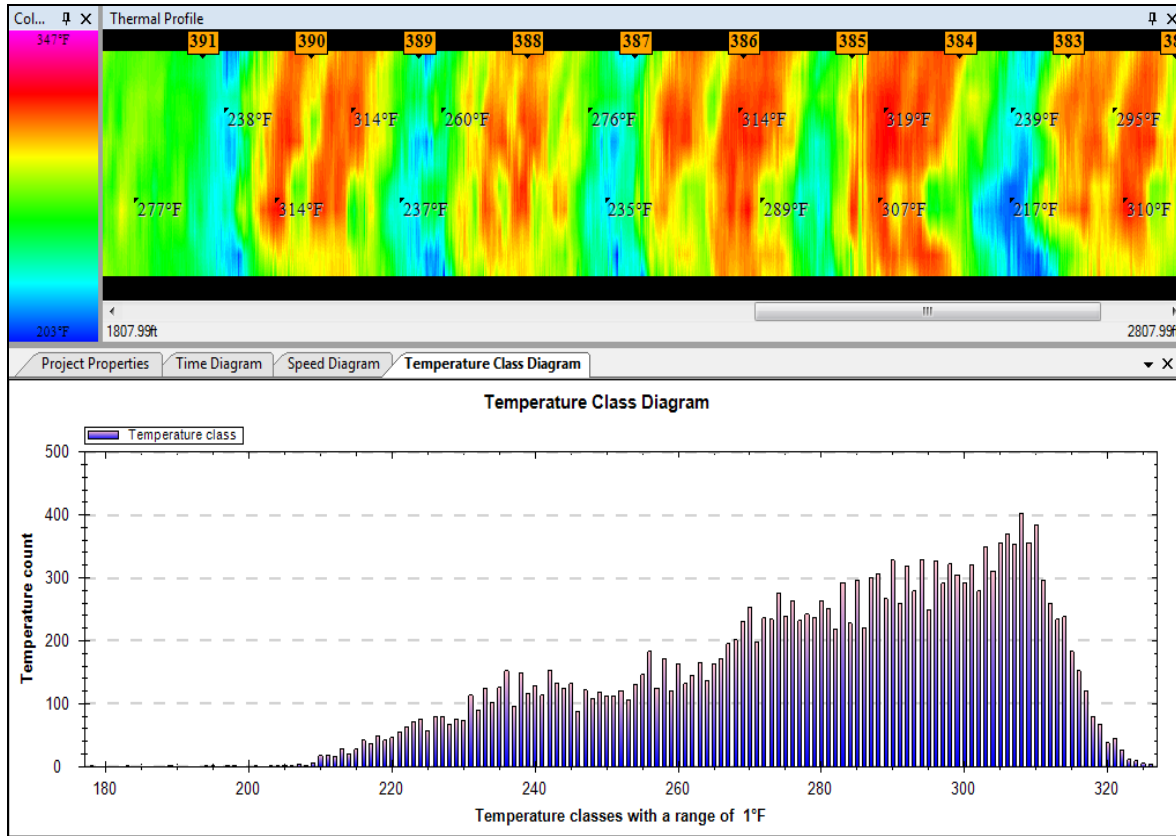


Figure 5.6: Thermal Profile SR-41 Conecuh County Blaw-Knox MC-330 MTD Ambient Temperature 60°F

Table 5.6: Thermal Profile SR-41 Conecuh County

Mean Temperature Behind the Screed (°F)	282
Median Temperature Behind the Screed (°F)	278
Standard Deviation σ (°F)	21.92
Minimum Temperature (°F)	213
Maximum Temperature (°F)	327
Target Compaction Temperature (°F)	290

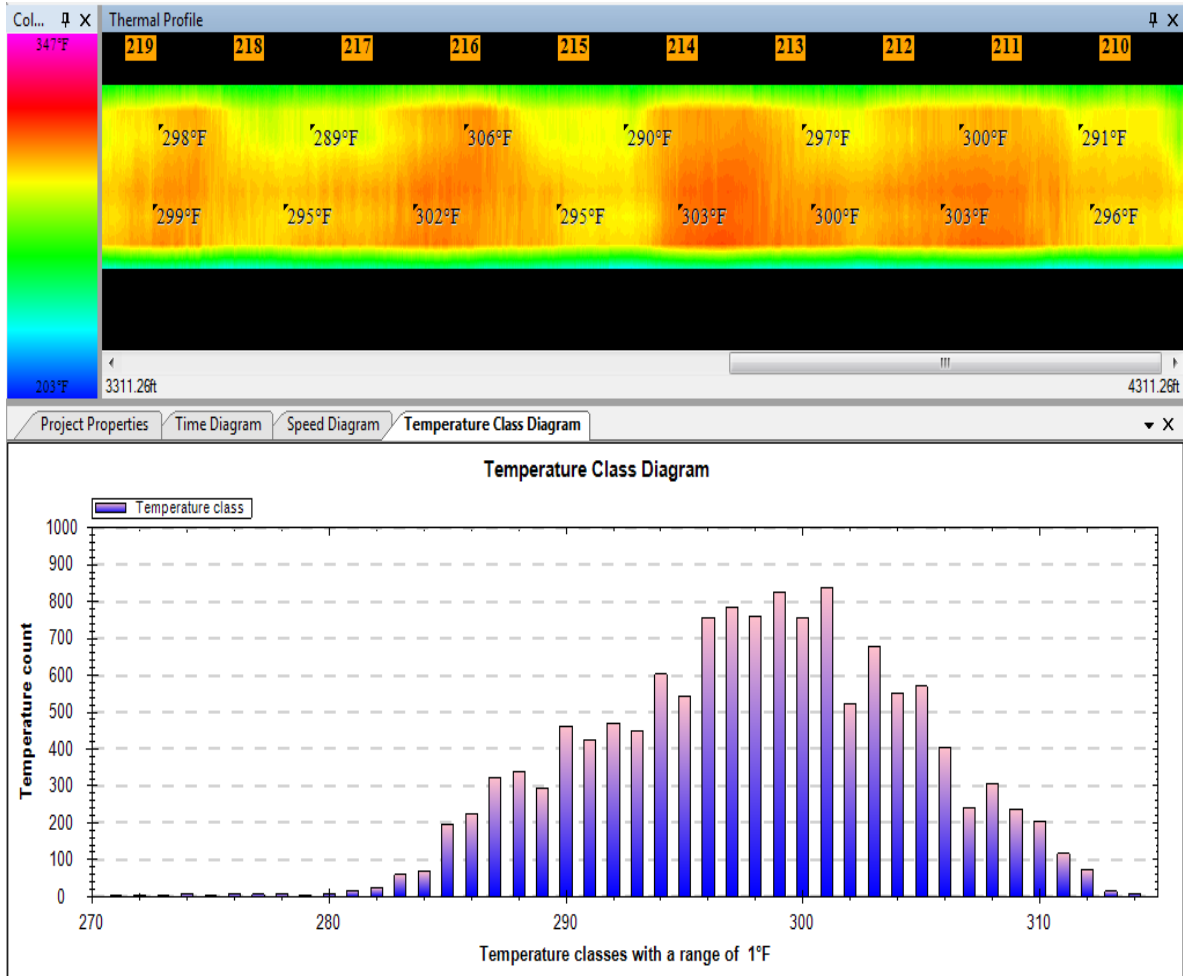


Figure 5.7: Thermal Profile SR-144 St. Claire County Roadtec SB-2500 MTD / Ambient Temperature 53°F

Table 5.7: Thermal Profile SR-144 St. Claire County

Mean Temperature Behind the Screed (°F)	296
Median Temperature Behind the Screed (°F)	295
Standard Deviation σ (°F)	7.68
Minimum Temperature (°F)	268
Maximum Temperature (°F)	314
Target Compaction Temperature (°F)	300

According to Figures 5.6 and 5.7, the typical thermal profile of the Blaw-Knox MC-330 presents a higher temperature variation with regard to the Roadtec SB-2500 MRD profile. This variation can be explained by comparing the standard deviations and the range of temperatures measured by the PAVE-IR infrared bar. Table 5.6 demonstrates that the variation of temperatures caused by the use of the Blaw-

Knox MC-330 MTD ($\sigma=21.92^{\circ}\text{F}$) is nearly three times higher than the variation measured for the Roadtec SB-2500C ($\sigma=7.68^{\circ}\text{F}$). The wide range of temperatures obtained from the Blaw-Knox MC-330 MTD (114°F) profile supports the aforementioned idea. The temperature class diagrams show a better distribution of the temperature for the Roadtec SB-2500 MRD and at the same time, a narrower range of mix temperatures even though the ambient temperature was cooler. It is important to mention that the SR-41 project (Blaw-Knox MC-330 MTD) reported 95% severe thermal segregation whereas the project SR-144 project (Roadtec SB-2500 MRD) presented 42% no thermal segregation, 58% moderate thermal segregation and no severe segregation. As expected, the unique project that reported 100% severe thermal segregation was executed without considering the use of any transferring/remixing device (8). At this project, the evaluated layer corresponded to a lower binder course placed over a permeable asphalt treated base (PATB). The ALDOT specifications do not allow the use of MTD/MRD's on PATB (Section 410.03a Hauling and Remixing Equipment) due to the heavy loads that characterize these devices. These excessive loads can distort the PATB layer resulting in an uneven surface on which to pave. For this reason, this project did not use a remixing device.

These results clearly demonstrate the effectiveness of remixing devices in minimizing temperature differentials and therefore, for reducing potential compaction problems associated with a loss of mix consistency. The results of this analysis agreed with the results obtained by Gilbert (6).

Layer/Mix Maximum Aggregate Size (MAS)

The type of layer and the maximum aggregate size (MAS) of the mix was the second most significant parameter in the development of high mix temperature differentials. During the field data collection, it was possible to evaluate upper and lower binder courses as well as surface courses characterized by different MAS. The evaluated lower and upper binder courses were characterized by 1in and $\frac{3}{4}$ in MAS whereas the wearing courses were designed for $\frac{3}{4}$ in, $\frac{3}{8}$ in and $\frac{1}{2}$ in MAS.

In order to examine the influence of binder/surface courses and their corresponding MAS in mix thermal segregation, Tukey multiple comparisons were performed by grouping both parameters in four

different categories: binder courses 1in MAS (3 projects), binder/surface courses 3/4in MAS (14 projects), surface courses 3/8in MAS (9 projects) and surface courses 1/2in MAS (2 projects). The means of each group were compared in terms of percentages of severe thermal segregation. Tables 5.8 and 5.9 show the results of the multiple comparisons at a 95% confidence level.

Table 5.8: Mean Percentage of Severe Thermal Segregation / Type of layer-MAS

Type of layer / MAS	Mean / % Severe Thermal Segregation
Binder Course / 1in	78.3
Binder Course / Surface Course 3/4in	42.0
Surface Course 1/2in	38.5
Surface Course 3/8in	28.7

Table 5.9: Tukey Simultaneous Tests 95% Confidence Level for Layer / Maximum Aggregate Size Severe Thermal Segregation Level

Layer / Mix Maximum Aggregate Size	Binder Course / 1in	Binder Course / Surface Course 3/4in	Surface Course 3/8in	Surface Course 1/2in
Binder Course / 1in		SD	SD	SD
Binder Course / Surface Course 3/4in	SD		NS	NS
Surface Course 3/8in	SD	NS		NS
Surface Course 1/2in	SD	NS	NS	

SD: significantly different, NS: not significantly different

According to Table 5.9, the binder courses with 1in MAS were statistically different than other MAS mixtures for potential to have severe thermal segregation. Table 5.8 also shows their negative effects based on the high levels of severe thermal segregation reported for these mixtures. As expected, the 1/2in surface course had the lowest levels of severe thermal segregation. These results may be due to a faster cooling rate for coarser mixes compared to finer mixes and therefore, more likely to have high temperature differentials, as shown in previous studies (6). However, the MAS of the mixture is also related to the layer thickness. Usually, binder courses are thicker than surface courses and therefore, they usually cool down slower than surface courses because they can retain the heat for more time. The results show that there is no statistical difference between the different layers. It is important to mention that the

PAVE-IR infrared bar measures surface temperature and not the temperatures throughout the layer thickness. Additionally, the PAVE-IR bar was set up so that it could measure the temperature of the mat immediately after the material was laid down, and not at the moment when the first rolling pass was applied. At some projects, the period of time between these two stages was more than 10min and the capability of the layer for retaining the heat became a key factor on the compaction process. The aforementioned aspects were not considered in the statistical analysis and thus, the statistical difference was more focused in the MAS rather than the type of layer. Basically, the levels of temperature segregation reported by the infrared bar included the effects of the mix MAS although they did not include the effects of the type of layer.

Mix Type

The mix type was the third most significant parameter on temperature segregation. Four types of mix were evaluated during the field evaluation. Twenty-one paving projects were constructed with dense-graded hot-mix asphalt (HMA) whereas 3 projects considered the use of stone-matrix asphalt (SMA). Warm-mix asphalt (WMA) and open-graded friction course (OGFC) binder/surface courses were examined in 3 and 1 projects, respectively. Mix type was found to be significant for the “no thermal segregation level” at a confidence level of 95%. Based on this, Tukey multiple comparisons were applied for analyzing which type of mix was more significant in the formation of cold areas. The table 5.10 shows the results of the multiple comparisons.

**Table 5.10: Tukey Simultaneous Tests 95% Confidence Level for Mix Type
No Thermal Segregation Level**

Mix Type	SMA	WMA	HMA	OGFC
Stone-Matrix Asphalt (SMA)		SD	NS	NS
Warm-Mix Asphalt (WMA)	SD		NS	NS
Hot-Mix Asphalt (HMA)	NS	NS		NS
Open-Graded Friction Courses (OGFC)	NS	NS	NS	

The results indicate that all types of mix have the same effect in the formation of high temperature differentials except for the WMA and the SMA which reported a statistical difference between them. It was expected that OGFC mixes were more susceptible to high temperature differentials (statistically different) than HMA and WMA due to open graded nature that characterize these mixes. On the other hand, WMA mixes were expected to be less susceptible than the rest of the mixes (statistically different) due to the effect of the additives for maintaining workability for longer periods of time. However, the statistical analysis did not reflect the expected results.

Another approach for analyzing the effect of each type of mix on thermal segregation consisted of examining the distribution of the mat temperatures along the different paving sections. This was performed by analyzing thermal profiles for each type of mix from paving projects executed under similar conditions (material transfer/remixing device). The Figures 5.8, 5.9, 5.10 and 5.11 show typical thermal profiles for SMA, WMA, HMA and OGFC mixes, respectively.

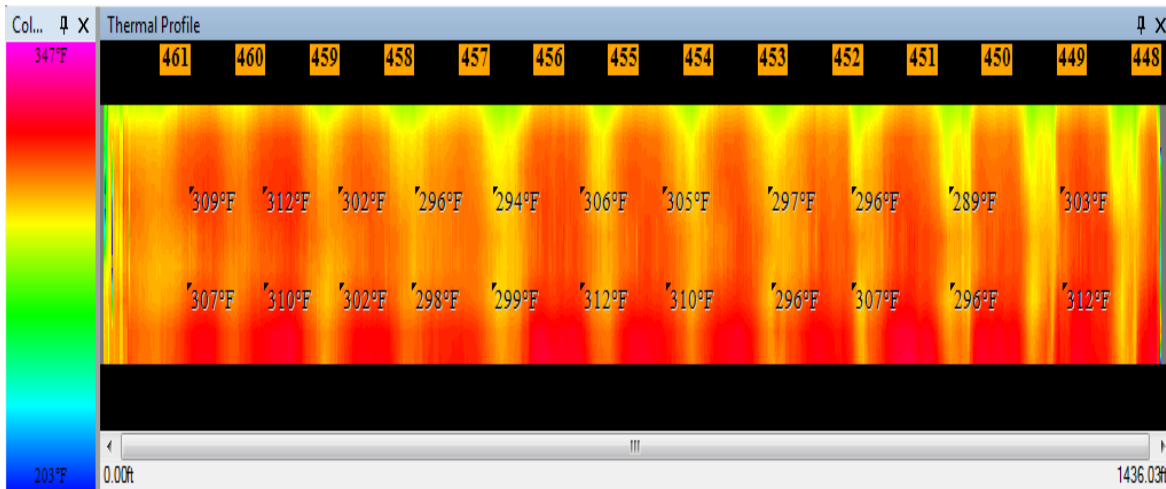


Figure 5.8: Stone-Matrix Asphalt (SMA) Thermal Profile I-65 Shelby County, Nighttime Paving Project, Roadtec SB-2500D, 1” MAS Upper Binder Course, Hauling Time 10min

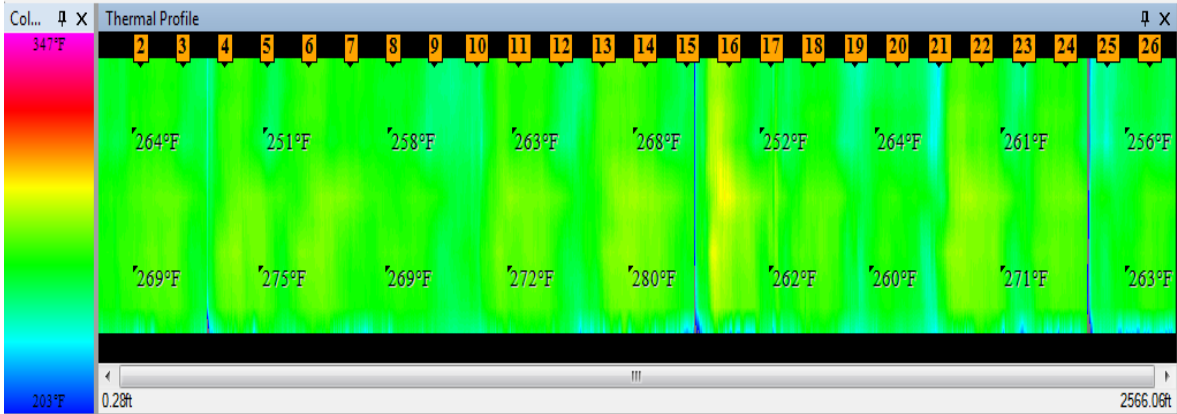


Figure 5.9: Warm-Mix Asphalt (WMA) Thermal Profile SR-165 Russell County, Daytime Paving Project, Roadtec SB-2500C, 3/4" MAS Upper Binder Course, Hauling Time 45min

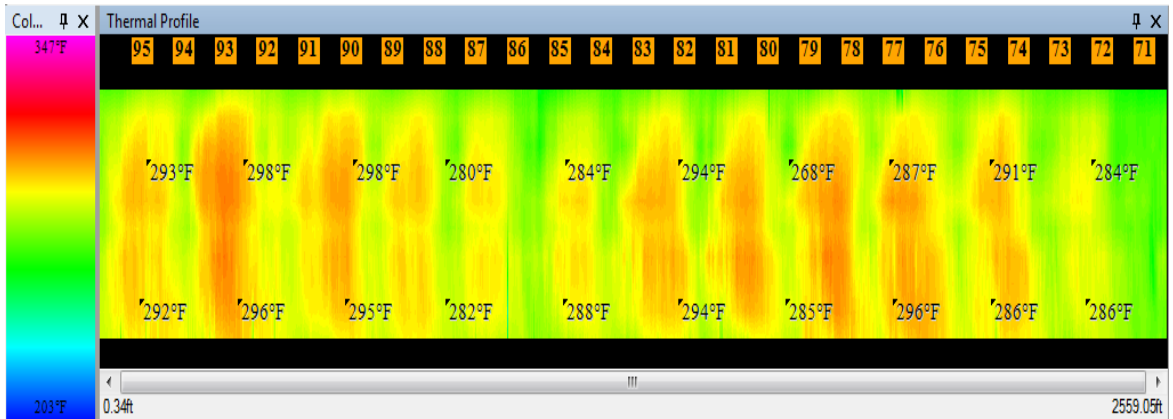


Figure 5.10: Hot-Mix Asphalt (HMA) Thermal Profile SR-184 Colbert County, Daytime Paving Project, Roadtec SB-2500C, 1/2" MAS Surface Course, Hauling Time 45min

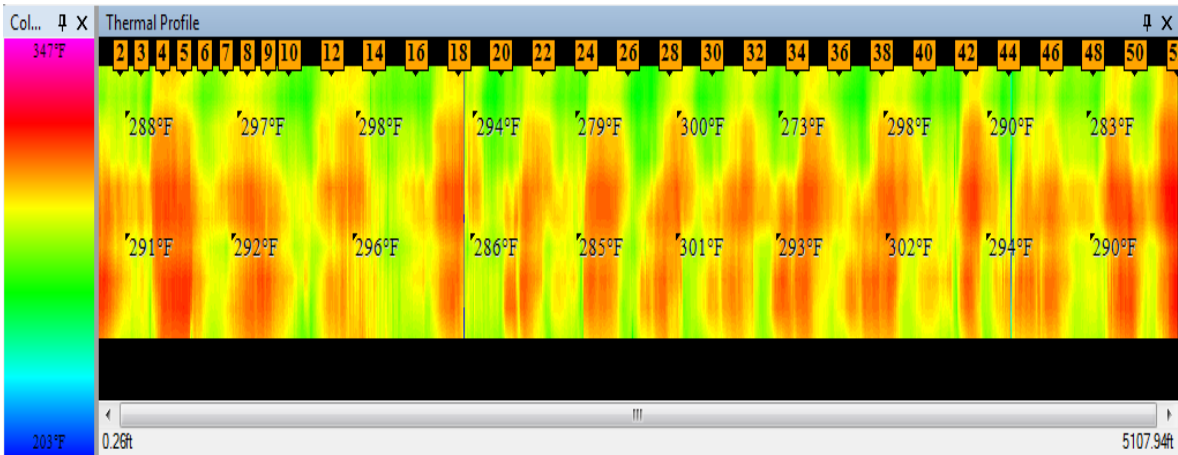


Figure 5.11: Open-Graded Friction Courses (OGFC) Thermal Profile I-65 Shelby County, Nighttime Paving Project, Roadtec SB-2500D, 3/4" MAS Surface Course, Hauling Time 20-30min

The table 5.11 shows the results reported by the Pave Project Manager software.

Table 5.11: Information of Thermal Profiles for Different Type of Mixes

Project	SMA	WMA	HMA	OGFC
Mean Temperature Behind the Screed (°F)	305	263	288	290
Median Temperature Behind the Screed (°F)	306	264	289	292
Standard Deviation (°F)	8.18	8.47	8.06	12.38
Target Compaction Temperature (°F)	325	260	300	325

According to table 5.11, the variation (standard deviation) of mat temperatures were very similar for the different mixes except for the OGFC. As aforementioned, the OGFC is characterized by an open gradation that can accelerate the loss of temperature of the mix prior to its compaction. Although SMA mixes are produced at similar temperatures as the OGFC, the amount of air voids in the mix is not as high as OGFC's. Therefore, the loss of temperature is not as critical as OGFC's. Regarding WMA thermal profile (Figure 5.9), the mean temperature of the evaluated section was similar to the target compaction temperature. Additionally, the low standard deviation obtained for this section demonstrates that the use of WMA can minimize the lack of uniformity of the compaction temperature. The HMA profile had the lowest temperature variation but the mean temperature was 12°F below the target temperature. This is considered a small difference for paving projects executed under good conditions, but could lead to problems under more adverse paving conditions.

It was also possible to analyze the effect of each type of mix on temperature segregation by comparing their variances and pooled standard deviations. In contrast with the previous analysis, this would allow to verify the effect of each type of mix considering the effect of the other factors included in the GLM. The results are shown in Table 5.12.

Table 5.12: Type of Mixes Pooled Standard Deviations

Type of Mix	Pooled Variance	Pooled Standard Deviation
HMA	201.92	14.21
SMA	117.31	10.83
WMA	97.87	9.89
OGFC (*)	153.27	12.38

(*): only one paving project

Based on the results, HMA had the highest variation in temperature as a consequence of the use of Blaw-Knox MC-330 MTD in 9 of the 21 HMA paving projects. In the other hand, WMA and SMA projects showed the lowest standard deviations mainly due to the utilization of remixing operations in all of the projects where these mixes were placed and the WMA low mixing temperatures that help the mix to cool down more slowly than the rest of the mixes. This analysis also indicates that the effect of the type of mix on thermally segregated areas is not as significant as other variables like MTV/MRD's and the mix MAS.

Finally, a Bartlett's test was performed to compare the variances obtained for each type of mixes at a confident level of 95%. This procedure was chosen for this study due to its sensitivity to departures from normality and because it allows comparing populations with different number of observations. In this study, the number of temperature readings varied within a range of 4,000 to 110,000 observations among the different paving projects. Based on the Bartlett's statistic χ_0^2 , all the variances were statistically different. This demonstrates the benefits of WMA for reducing temperature differentials but also, the advantages of using remixing devices for all type of mixes.

Hauling Time and Paver Speed

At some projects, it was possible to take infrared pictures of the truck beds for measuring the difference in temperatures between the surface material and the material closer to the core of the load, at different hauling times. The figures 5.12, 5.13 and 5.14 show infrared images for the following hauling time categories: hauling times less or equal than 20min (4 paving projects), hauling times between 21min and 40min (10 paving projects), and finally hauling times greater than 40min (14 paving projects). These categories were arbitrarily selected based on project data. All of the 28 paving projects used tarpaulin covers over the truck beds.

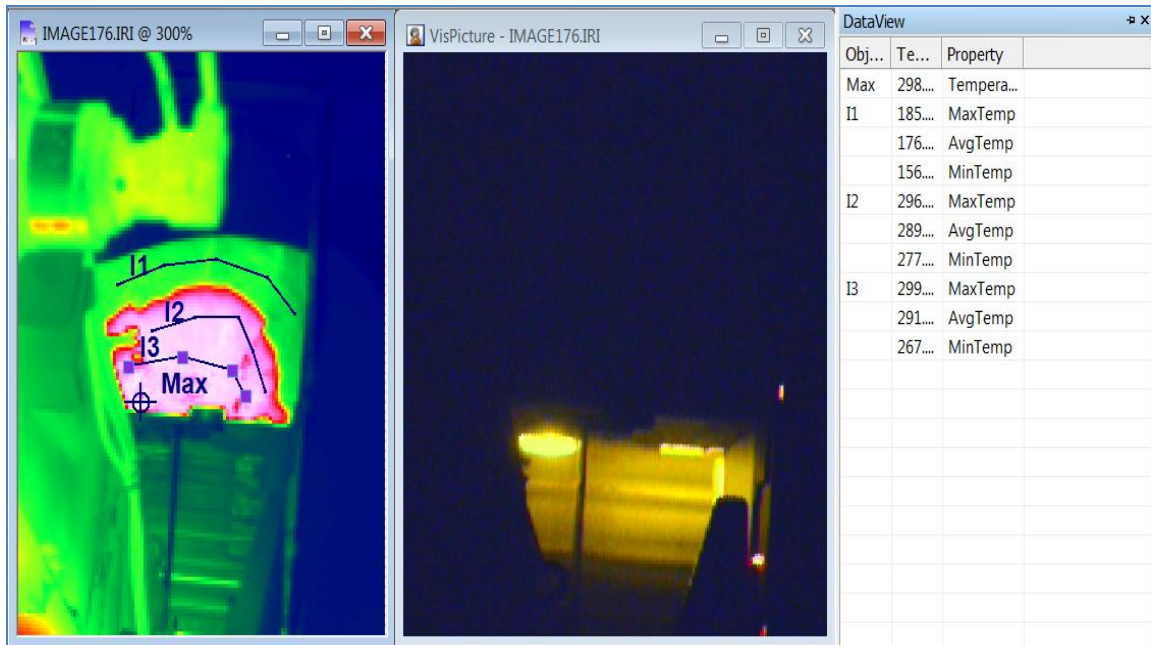


Figure 5.12: Hauling Time 15min, Ambient Temperature 88°F Stone-Matrix Asphalt (SMA), Compaction Temperature 310°F Infrared Image I-85 Montgomery County, Nighttime Paving Project, 1/2” MAS Surface Course

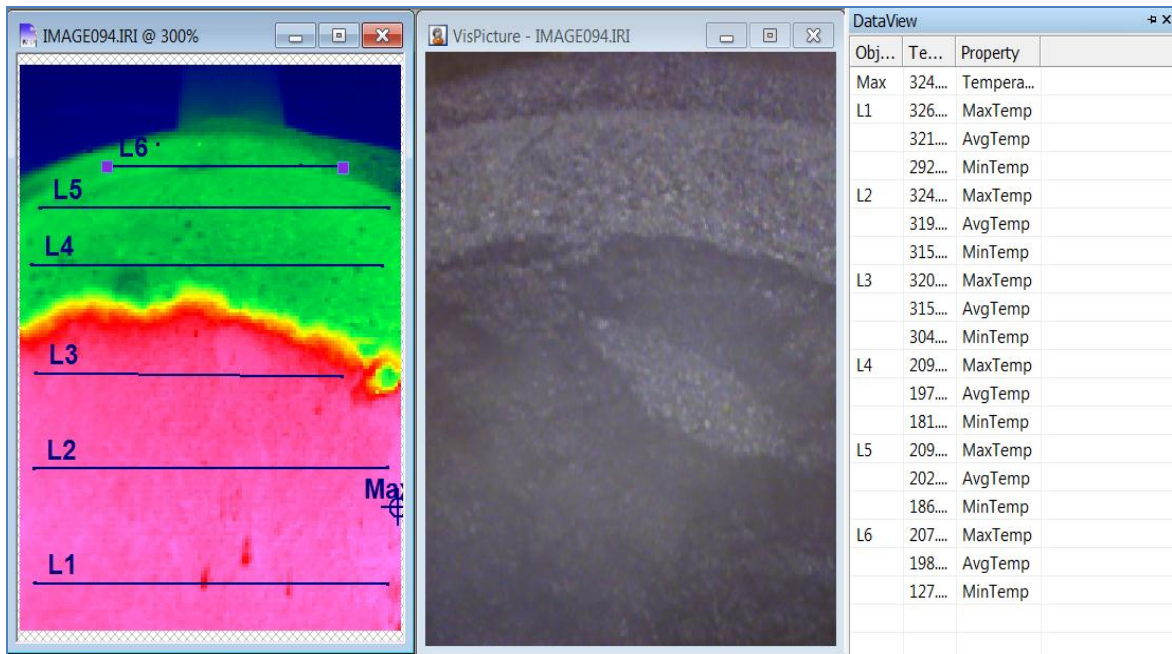


Figure 5.13: Hauling Time 30min, Ambient Temperature 88°F Hot-Mix Asphalt (HMA), Compaction Temperature 300°F Infrared Image SR-25 Bibb County, Daytime Paving Project, 1” MAS Upper Binder Course

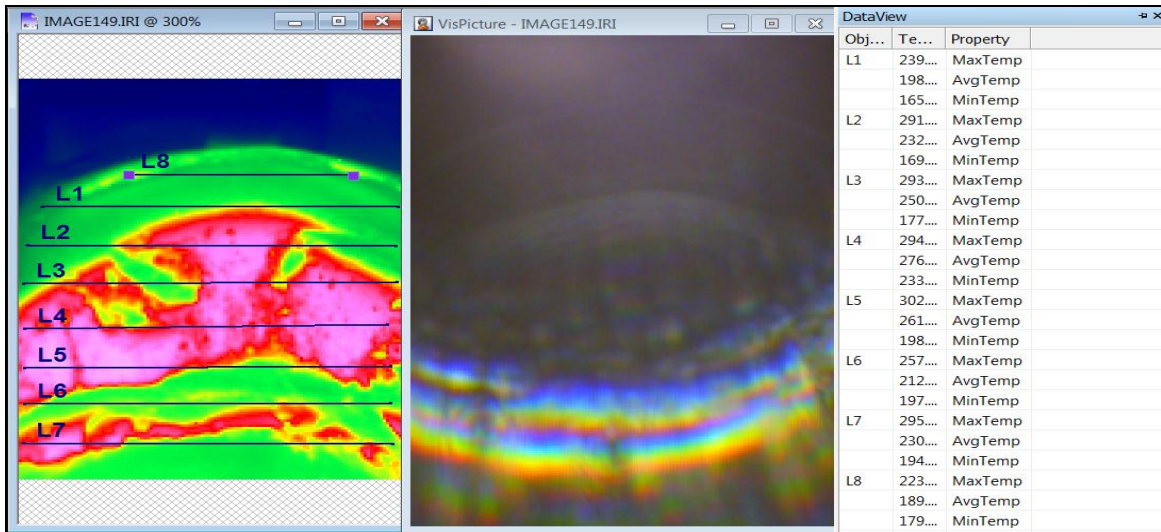


Figure 5.14: Hauling Time 60min, Ambient Temperature 86°F Hot-Mix Asphalt (HMA), Compaction Temperature 300°F, Infrared Image Airport Boulevard, Mobile County, Daytime Paving Project, 1/2" MAS Surface Course

Based on the average temperatures reported by the RAZ-IR analysis software, it is evident that the material closer to the surface rapidly loses temperature due to its constant exposure to ambient temperatures compared to the material near to the core of the load. Even with tarpaulin covers, the material tended to lose temperature significantly (up to 197°F). Table 5.13 presents the temperature differentials in the truck bed for the aforementioned projects.

Table 5.13: Mix Temperature Differentials in the Truck Bed

Project	Hauling Time (min)	Temperature Differential (°F)*	Paver Speed (ft/min)
I-85	15	115	18.38
SR-25	30	124	15.38
Airport Boulevard	60	123	55.43

(*): difference between maximum average temperature and minimum average temperature

It was expected that the temperature differentials for the 60min hauling time exceeded the 15min and 30min hauling times temperature differentials. However, the results do not reflect this tendency, specifically for the 60min hauling time. It is important to mention that the hauling time is usually defined as the time that a hauling unit spends from the production plant until the construction site, once it is loaded with asphalt mix. In most of the evaluated projects, the hauling units had to wait extra 5 to 15min before dumping the material in the MRD/MTD/paver hopper, depending on the speed of the paving

operations. The table 5.13 shows that the Airport Boulevard project was done at a very high speed with regard to the other projects. This indicates that despite the long hauling time, the higher speed of the paving operation minimized further loss of temperature in the truck. The other two projects were executed at lower speed. This increased the waiting time of the hauling units until the discharging operations as well as temperature differentials at the truck bed. This demonstrates how the paver speed usually governs this type of project.

The figure 5.15 presents the percentages of thermal segregation obtained for the three hauling time periods established for the analysis, based on the results obtained for the 28 paving projects. The figure also shows the levels of segregation obtained for both Roadtec SB-2500MRD and Blaw-Knox MC-330 MTD at each hauling time period in order to examine the influence of these devices for minimizing the effect of long hauling times.

Based on results shown in figure 5.15, comparing the percentages of severe thermal segregation from both devices for hauling times greater than 40min, it can be observed that these percentages are significantly higher for the Blaw-Knox MC-330 MTD than those obtained for Roadtec SB-2500 MRD (difference of 65% of severe thermal segregation). On the other hand, the percentages of “no thermal segregation” were higher for those projects where a Roadtec SB-2500 MRD was used and the hauling times were greater than 40min. This indicates that the remixing devices such as the Roadtec SB-2500 can reduce significantly the formation of high temperature differentials, even at hauling times greater than 40min. Transferring devices such as the Blaw-Knox MC-330 resulted in the formation of more cold spots and its effect can considerably increase as the hauling times increase. From Figure 5.15, it can be concluded that even hauling time less than 20min can produce high levels of severe thermal segregation if a remixing device is not considered.

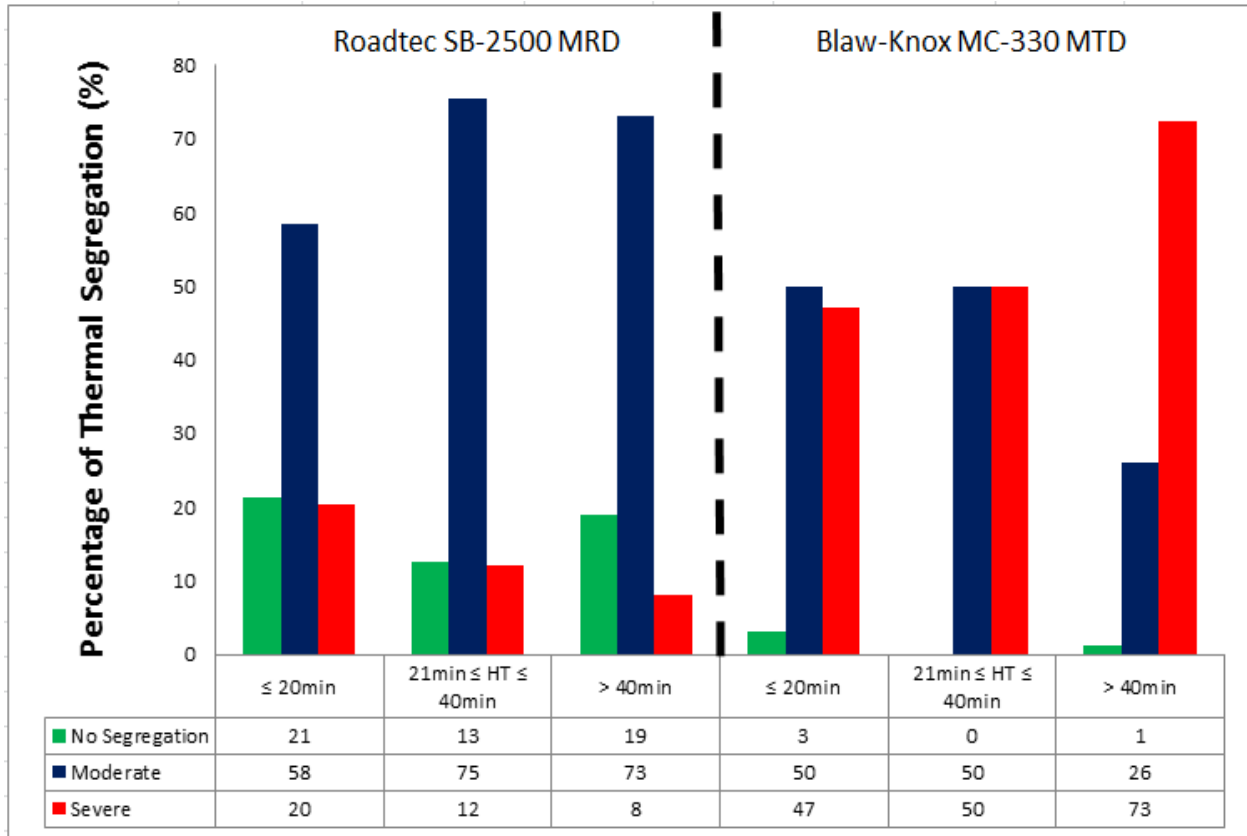


Figure 5.15: Relationship between Levels of Thermal Segregation and Hauling Times for Roadtec SB-2500 MRD and Blaw-Knox MC-330 MTD

Based on the results, the effect of hauling time on the development of thermally segregated areas can be significantly reduced as long as remixing devices are used in the paving process.

Interruption of Paving Operations: Paver Stops

Interruption of the paving operations is commonly caused by problems with the production plant, delays associated to the hauling operations or issues related to the construction equipment. During the evaluation of the paving project SR-278 in Calhoun County, it was possible to identify a 45min paver stop that produced a temperature differential of almost 100°F as shown in figure 5.16.

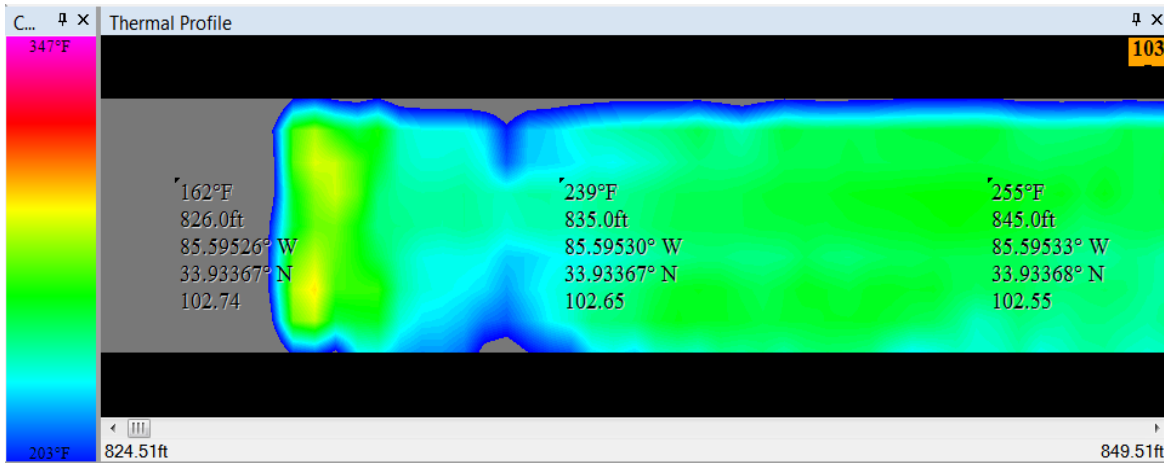


Figure 5.16: 45min Paver Stop, SR-278 Project in Calhoun County

The same project was visited a month after the temperature readings were performed. During the extraction of the cores, it was possible to identify signs of raveling and aggregate segregation at the paver stop zone caused by the lack of workability of the mix during the compaction. During a paver stop, the mix that remains at the sides of the paver screed is exposed to the environment causing an accelerated loss of temperature. This issue can be more critical when paving operations are executed in adverse weather conditions. In this case, a good practice would be keeping the screed heaters turned on during the stop, checking the temperature of the material at the screed prior to its compaction or in the worst of the cases, the removal of the cold material and construction of a transverse joint.

It was expected that the paver stops identified by the PAVE-IR infrared bar would be significant in the formation of cold areas. However, the times considered in the statistical analysis were accumulated times. These accumulated times not only considered paver stops due to delays in the paving operations, they also included paver stops where no material was in the screed (for example when the paver had to be moved to other construction joint due to presence of a bridge or due to a change of lanes to be paved and then, it had to wait for the first load). Thus, the paver stop times varied significantly depending on the conditions of the project. This might affect the results of the statistical analysis with regard to the level of significance of this specific factor.

Time of Day / Ambient Temperature

Based on the GLM results, the time of day was not significant in the development of temperature differentials. The effects of daytime paving versus nighttime paving were also included in this analysis. It was expected that nighttime projects were going to be more susceptible to thermal segregation than daytime projects due to the lower ambient temperatures.

The GLM showed that the time of day was the least significant parameter of all the evaluated variables in the development of high temperature differentials. It is important to mention that four of the five nighttime projects reported ambient temperatures over 70°F and they were executed with a remixing device. The results for these projects reported low to moderate levels of temperature segregation. On the other hand, the SR-144 daytime project located in Saint Claire County was performed under an ambient temperature of 53°F (8:00am) and it included a remixing device. In this case, the PAVE-IR infrared bar reported 0% of severe thermal segregation. The similar results obtained from those projects demonstrate how times of day are not controlling variables in regard to thermal differentials when they are compared to remixing devices.

The figures 5.17 and 5.18 present infrared images from the SR-41 and SR-10 projects respectively. It is evident that the material in the truck bed corresponding to the SR-41 lost more temperature than the material from the SR-10 project due to the lowest ambient temperatures reported that day. For the SR-41 project, the PAVE-IR infrared bar reported 95% severe thermal segregation whereas the SR-10 project registered 70% severe thermal segregation. Both projects were performed under similar conditions except for the ambient temperatures and the type of haul units. First, the ambient temperature of SR-41 was 60°F while the ambient temperature of SR-10 ranged from 73° to 85°F. A second exception was that a portion of the haul trucks on SR-10 were live-bottom trucks. This was the only project where live-bottom trucks were used so it was uncertain if truck types had a significant effect. It is expected that if more projects had been evaluated during the early spring and late fall seasons, the ambient temperatures may have had a greater effect on thermal segregation levels.

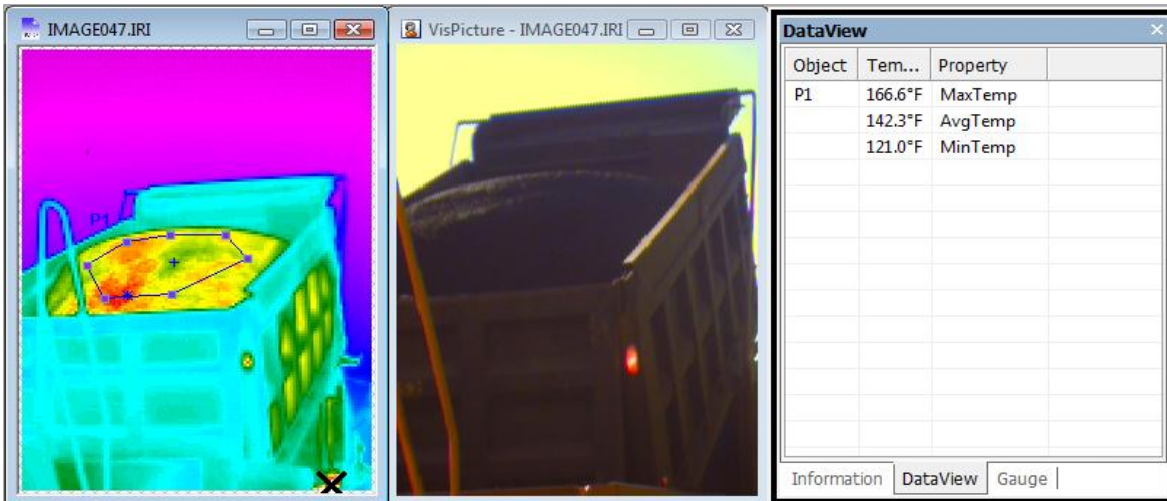


Figure 5.17: Material at Truck Bed SR-41 Conecuh County Ambient Temperature 60°F Hauling Time 90min, Blaw-Knox MC-330 MTD

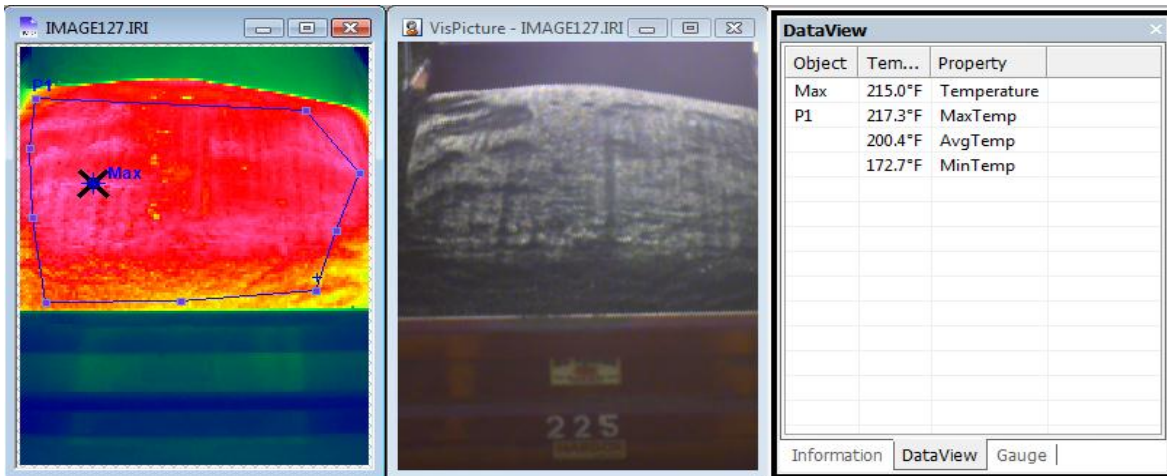


Figure 5.18: Material at Truck Bed SR-10 Wilcox County Ambient Temperature 85°F Hauling Time 75min, Blaw-Knox MC-330 MTD

Evaluation per Division

An overall comparison among the nine ALDOT Divisions was performed to identify which Divisions were more prone to thermal segregation. Figure 5.19 presents the average levels of thermal segregation reported by the PAVE-IR infrared bar for each Division.

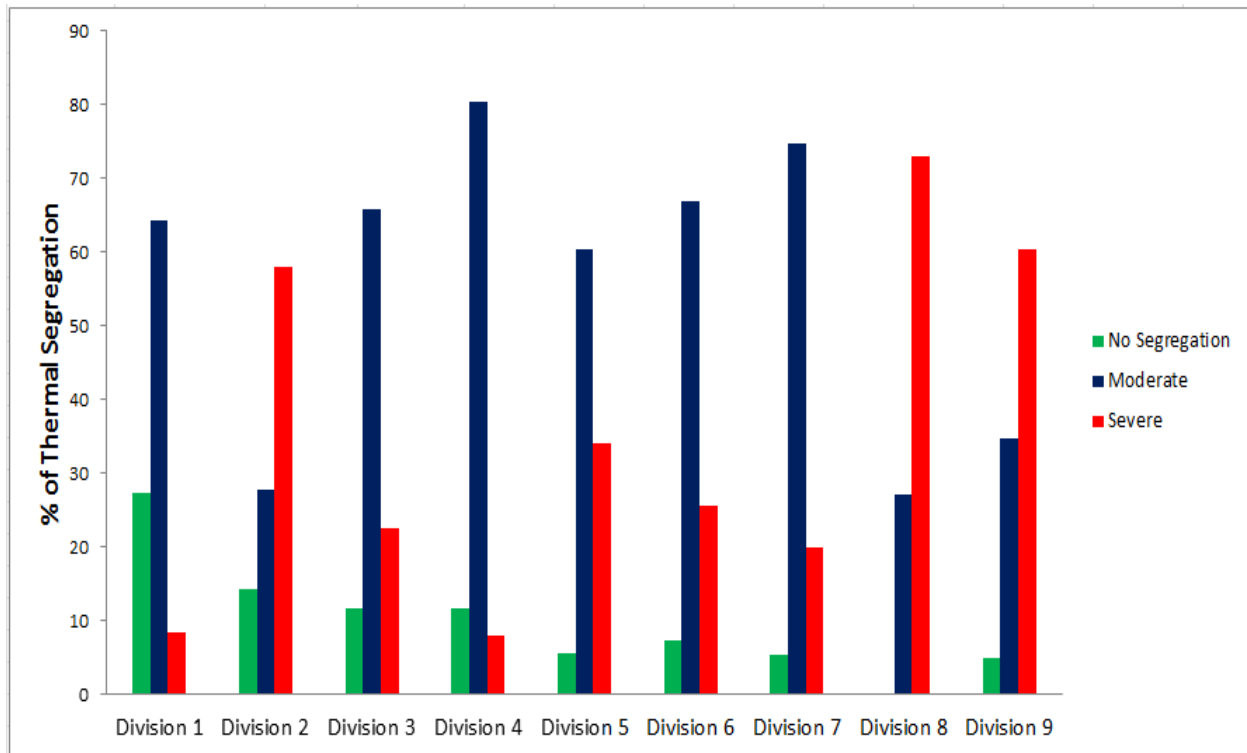


Figure 5.19: Levels of Thermal Segregation for each ALDOT Division

According to the Figure 5.19, projects in 8th Division had more cases of severe segregation. This was due to the three projects in the 8th division lacking a remixing device. The three projects used a Blaw-Knox MC-330 material transferring device which was previously shown to be the most significant factor in the development of high temperature differentials. In contrast, Division 4 had the lowest levels of severe thermal segregation due to the use of remixing processes in two of the three evaluated projects. At the same time, Division 4 had more projects with moderate levels of thermal segregation since one of the projects did not utilize MTD's or MRD's. Overall, projects in Division 1 seemed to minimize high temperature differentials. This was concluded since the percentages of no segregation in this division were the highest and the severe segregation levels were one of the smallest. Still, the difference was more a function of the type of transfer/remixing device used than actual construction practices.

ALDOT specifications give the contractor the option of remixing devices, such as Roadtec SB-2500 and Weiler. Nine of the 28 projects evaluated in the study did not use remixing devices during the paving operations. Some interviewed paving crews and foremen stated that material transferring devices were

commonly used due to their lower cost relative to material remixing devices. Additionally, they questioned the use of remixing devices rather than transferring devices since the loss of mix temperature prior to compaction was inevitable and that its use did not bring many advantages. This study has demonstrated that remixing devices significantly reduce the formation of cold spots.

Recommendations for Current ALDOT Specification

The construction requirements for asphalt pavements specified in Section 410.03 (a).3 “Hauling and Remixing Equipment” of the ALDOT Standard Specifications for Highway Construction 2012 (40) include the aspects related to material transfer/remixing devices.

The first recommendation is related to the equipment previously approved by ALDOT as remixing devices able to accomplish remixing operations. These devices are listed as follows.

- Roadtec Shuttebuggy
- Blaw-Knox MC-330/Twin Pug Tub
- Weiler E1250/E2850
- Terex Cedarapids CR 662 RM

In the projects evaluated in this study, only the first three devices were used. The results of this project demonstrated that using material transfer vehicles such as the Blaw-Knox MC-330 resulted in frequent high temperature differentials. Therefore, it is recommended that the Blaw-Knox MC-330 be omitted from the above list until improvements can be made that can be demonstrated to significantly reduce temperature differentials.

It was not possible to evaluate the Terex Cedarapids device since it was not used at any of the evaluated projects. This device and others not on the above list should be demonstrated to significantly reduce temperature differentials before they are accepted for use.

It is also recommended that when paving under low ambient temperatures (less than 60°F), the use of remixing devices be mandatory.

Agencies like the Texas Department of Transportation (TxDOT) have included in its specification the evaluation of thermal profiles obtained from infrared thermometers, infrared cameras or the PAVE-IR infrared bar, as part of the quality control and quality assurance performed by contractors and agencies, respectively. In this case, it is recommended that ALDOT implement in its specification, the equipment (PAVE-IR infrared bar), the levels of thermal segregation allowed (for example, “no thermal segregation”, “moderate thermal segregation” and “severe thermal segregation”) and the corrective actions that must be taken in order to monitor and to minimize the effect of high temperature differentials on mat density. The results have demonstrated that the use of the PAVE-IR infrared bar system has facilitated the evaluation and the analysis of the mat temperature during the paving operations. Additionally, several features like the GPS have allowed detecting the location of high temperature differentials and thus, it has been possible to make a further and detailed evaluation of in-place densities at these areas. Additionally, the levels of thermal segregation used during the execution of this project have been found to be appropriate for categorizing the temperature differentials obtained from thermal profiles.

The SR-247 paving project was the only project that did not use of a remixing/transferring device due to the prohibition of these devices for paving over a PATB in the state of Alabama. As a consequence of this, 100% severe thermal segregation was measured during the field evaluation. Several construction practices can be proposed in order to allow the use of remixing devices in this specific case and that at the same time, do not compromise the functionality of the PATB. For example, the lane next to the construction lane can be used to move the trucks and the MRD while it transports the mix to the paver laterally with the conveyor belt. This might be a good practice for new pavement construction. However, actions like this will depend on the construction site conditions. Based on this, the ALDOT should leave the decision of using a MRD to the project engineer instead of totally prohibiting the use of these devices over PATB. Thus, the actual specification should be more flexible and give to the engineer the option of make a management decision that can benefit the functionality of the PATB that, at the same time does not affect the structural integrity of the upper layer.

Finally, the formulation of economic incentives for the evaluation of thermal profiles can be implemented as part of contractor's quality control during paving operations. For example, an incentive can be offered to contractors for using the PAVE-IR infrared bar for the evaluation and control of thermally segregated areas. If the evaluation meets the requirements, the incentive would be earned by the contractor. Otherwise, the contractor would have to take the corrective actions or assume associated costs in the form of pay reductions.

The following best practices are recommended to minimize thermally segregated areas during paving operations.

- Measure the mix temperature once the mix comes out from the production plant to the hauling units in order to check its compliance with job mix formula mixing temperature
- Use of tarpaulin covers over the truck beds to protect the mix from cooling or from adverse conditions (from the production plant until the material is discharged into the paver or MRD hopper)
- Check the mechanical conditions of the paving and compaction equipment (MRD, pavers, rollers, tack coat trucks) prior the execution of the paving operations in order to avoid delays caused by problems with the equipment.
- Develop a paving plan that has determined the number of haul trucks needed to maintain a continuous operation based on anticipated production.
- Maintain a constant paving rate which can minimize wait time for haul trucks to discharge the material in the MRD hopper.
- Avoid the use of material transferring vehicles (MTD's) that does not provide remixing operations.
- Avoid the use of paving equipment (paver or MTD/MRD) that have hoppers with wings at the sides where the material can accumulate and loses temperature.

- Use effective remixing devices for mixes with maximum aggregate sizes equal or larger than 1.0". Mixes with high asphalt contents and open gradations such as OGFC must consider the use of remixing process as well.
- Even for hauling times lower than 20min, it is recommended to use a device with an effective remixing process.
- Reduce the number of paver stops commonly caused by problems with the production plant, delays associated with the lack of sufficient quantity of haul trucks or erratic placement procedures. the hauling operations or issues related to the construction equipment. This would avoid the presence of transverse cracking due to the formation of cold joints.

Analysis of In-Place Density

Figure 5.20 shows the average in-place densities obtained for each project at the designated hot and cold areas as follows.

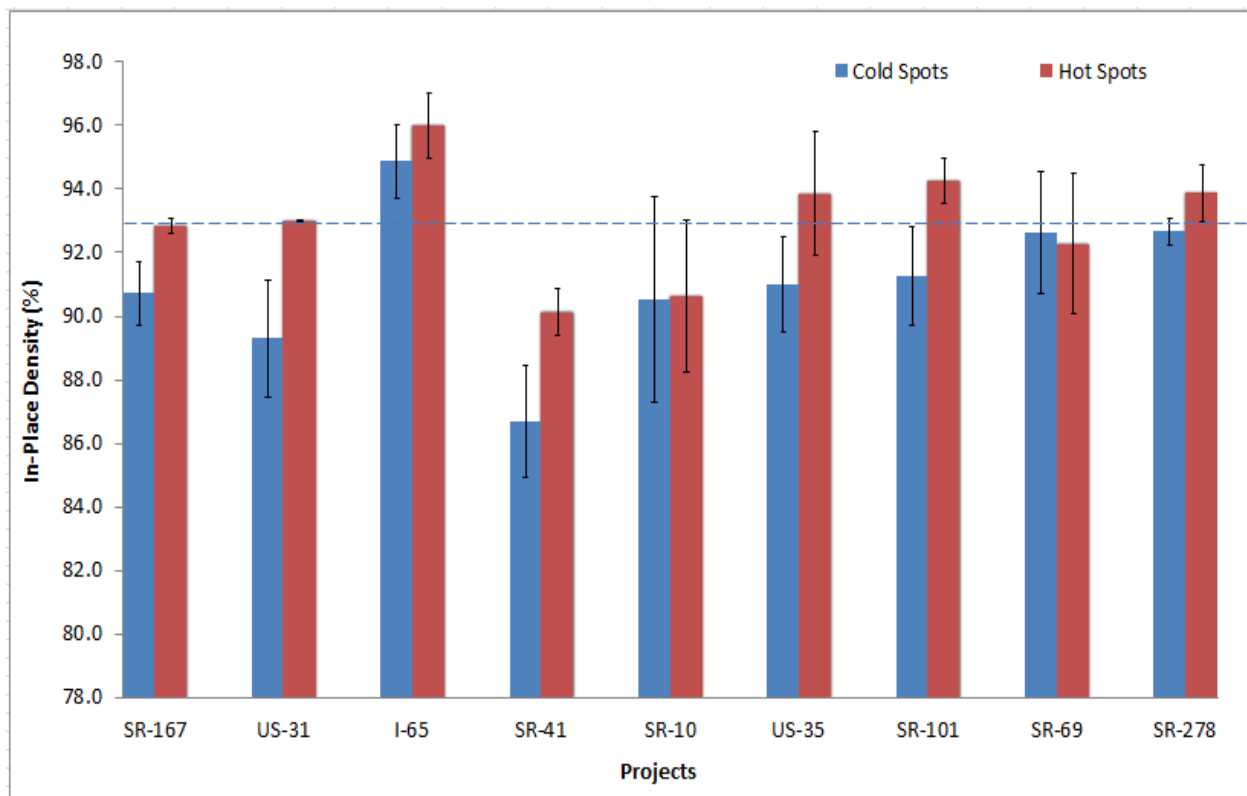


Figure 5.20: Comparison between In-Place Densities (%) Hot Spots versus Cold Spots

According to Figure 5.20, eight of the nine evaluated projects reported lower average in-place densities at the cold areas than those obtained from the hot areas. For seven of these projects, the differences in average densities were more than 1.0% which was considered in this project as a significant difference. The dashed line in the graph denotes the 93.0% in-place density level which corresponds to a common target initial in-place density (40) value for accepting mix compaction processes. The results demonstrate that the average densities at the cold areas were below 93.0% at all the evaluated projects except for the I-65 project. Meanwhile, the hot areas satisfied the in-place density criteria in 6 of the 9 projects. The differences in densities between hot and cold areas are the result of the lack of consistency of the mix prior to its compaction. It is also evident that the cold areas are characterized by a greater variability in the density levels. Six of the 9 projects reported higher standard deviations at the cold areas compared to the hot areas. During the analysis of the thermal profiles, it was possible to identify cold “streaks” or “spots” at the cold areas, which were not distributed uniformly along the width of the paving section. This means that mix temperatures tended to be different at the right, left and “between” wheel-paths where the field cores were extracted. Probably, this difference was reflected in the high standard deviations obtained from the analyses of in-place densities.

Table 5.14 presents the results obtained from the volumetric characterization of the field cores extracted from the 9 paving projects selected for each ALDOT division. Additionally, Table 5.14 includes the p-values obtained from 2-sample t-tests performed for each project for comparing the in-place density of both hot spots and cold spots.

Table 5.14: Core Analyses In-Place Density

ITEM				Volumetric Properties										Statistical Results % Gmm Analysis	
				Cold spots					Hot spots					2-Sample t-test 95% CL	
Project	Division	Type of mix	MAS	Sample ID	Gmb	Gmm	% Gmm	% Gmm AVG	Sample ID	Gmb	Gmm	% Gmm	% Gmm AVG	p-value	Criteria (*)
SR-167	7	HMA	3/4"	SR 167-1	2.251	2.476	90.9	90.7	SR 167-5	2.292	2.476	92.6	92.8	0.027	Different
				SR 167-2	2.219	2.476	89.6		SR 167-6	2.303	2.476	93.0			
				SR 167-3	2.278	2.476	92.0		SR 167-7	2.301	2.476	92.9			
				SR 167-4	2.237	2.476	90.3								
US-31	3	HMA	3/4"	US 31-1	2.267	2.584	87.7	89.3	US 31-4	2.404	2.584	93.0	93.0	0.074	Equal
				US 31-2	2.360	2.584	91.3		US 31-5	2.403	2.584	93.0			
				US 31-3	2.295	2.584	88.8		US 31-6	2.402	2.584	93.0			
I-65	6	SMA	1/2"	I-65-1	2.368	2.461	96.2	94.9	I-65-4	2.361	2.461	95.9	96.0	0.277	Equal
				I-65-2	2.315	2.461	94.1		I-65-5	2.338	2.461	95.0			
				I-65-3	2.322	2.461	94.4		I-65-6	2.389	2.461	97.1			
SR-41	9	HMA	3/4"	SR-41-4	2.330	2.638	88.3	86.7	SR-41-1	2.373	2.638	89.9	90.1	0.089	Equal
				SR-41-5	2.292	2.638	86.9		SR-41-2	2.399	2.638	90.9			
				SR-41-6	2.238	2.638	84.8		SR-41-3	2.361	2.638	89.5			
SR-10	8	HMA	1/2"	SR-10-4	DAMAGED SAMPLE			90.5	SR-10-1	2.172	2.47	87.9	90.6	0.962	Equal
				SR-10-5	2.179	2.47	88.2		SR-10-2	2.258	2.47	91.4			
				SR-10-6	2.292	2.47	92.8		SR-10-3	2.286	2.47	92.6			
US-35	1	WMA	3/4"	US-35-4	2.237	2.494	89.7	91.0	US-35-1	2.285	2.494	91.6	93.9	0.116	Equal
				US-35-5	2.261	2.494	90.7		US-35-2	2.363	2.494	94.7			
				US-35-6	2.311	2.494	92.7		US-35-3	2.374	2.494	95.2			
SR-101	2	HMA	1/2"	SR-101-4	2.403	2.636	91.2	91.3	SR-101-1	2.463	2.636	93.4	94.3	0.026	Different
				SR-101-5	2.394	2.636	90.8		SR-101-2	2.498	2.636	94.8			
				SR-101-6	2.364	2.636	89.7		SR-101-3	2.493	2.636	94.6			
				SR-101-7	2.461	2.636	93.4								
SR-69	5	HMA	3/8"	SR-69-6	2.313	2.447	94.5	92.6	SR-69-1	2.308	2.447	94.3	92.3	0.794	Equal
				SR-69-7	2.234	2.447	91.3		SR-69-2	2.217	2.447	90.6			
				SR-69-8	2.322	2.447	94.9		SR-69-3	2.325	2.447	95.0			
				SR-69-9	2.225	2.447	90.9		SR-69-4	2.216	2.447	90.6			
				SR-69-10	2.239	2.447	91.5		SR-69-5	2.224	2.447	90.9			
SR-278	4	HMA	3/4"	SR-278-4	2.337	2.526	92.5	92.7	SR-278-1	2.358	2.526	93.3	93.9	0.165	Equal
				SR-278-5	2.332	2.526	92.3		SR-278-2	2.397	2.526	94.9			
				SR-278-6	2.353	2.526	93.2		SR-278-3	2.359	2.526	93.4			

(*): means are different if p-value < 0.05

Based on the two sided 2-sample t-test results, only two of the nine evaluated projects reported difference in density between hot and cold spots that were statistically different ($p\text{-value} < 0.05$), at a confidence level of 95%. Two other projects (US-31 and SR-41) had $p\text{-values}$ less than 0.10 which are also considered to have practical differences in density between hot and cold spots. The difference between the average in-place density of hot and cold spots for US-35 WMA project was 2.9% which is also considered a large difference in practical terms, but due to the variation in results among the three samples, the $p\text{-value}$ was 0.116. It is important to remember that statistical analyses increase their analytical power when the number of observations collected is increased. Although the average density levels at hot and cold spots seem to be different from an engineering point of view, the limited number of samples did not provide enough statistical evidence to observe the expected difference.

In addition to the lack of uniformity of mix temperature prior to its compaction, there were other sources of variability for the density results that could also have affected the statistical results. The first source is related to the construction variability that can be associated to the HMA compaction process such as the roller speed, the sequence and number of rollers, the number of roller passes over a specific area and the rolling patterns. An inconsistent compaction process of the mat related to the aforementioned activities can lead to non-uniform in-place densities, especially along the paving width where the field samples for this project were taken.

Another source of variability can be attributed to the location of the cores in the field with the GPS. The degree of accuracy of recreational-grade receivers such as the GPS unit used for this project can be affected by factors such as weather conditions and vegetation cover. The presence of trees along the roadway and cloudy conditions can interfere with satellite signals received by the GPS units, increasing the error with regard to the true location of a specific location (44). During the location of the cores in the field, the GPS unit reported an error of $\pm 9\text{ft}$ with respect to the desired position. In order to minimize this error, the points where the cores were going to be

extracted were selected close to the center of a specific hot or cold area. Overall, the hot and cold areas generally had lengths that exceeded 30ft. The selection of the points at the center of these areas allowed being always within the ± 9 ft error range.

Additionally, several cores extracted from the cold spots of US-35 and SR-41 showed absorption percentages greater than 2.0%, which is the maximum allowed by ALDOT (40) for applying the saturated surface dry method (AASHTO T-166). This factor was not taken into account and densities were determined in accordance to AASHTO T-166. Use of the Corelock method would likely have resulted in lower densities for the cold areas and therefore, in higher differences in density between cold and hot spots for these specific projects (50).

Figure 5.21 shows the relationship found at each evaluated project between the difference in densities and the temperature differentials reported by the PAVE-IR infrared bar as follows.

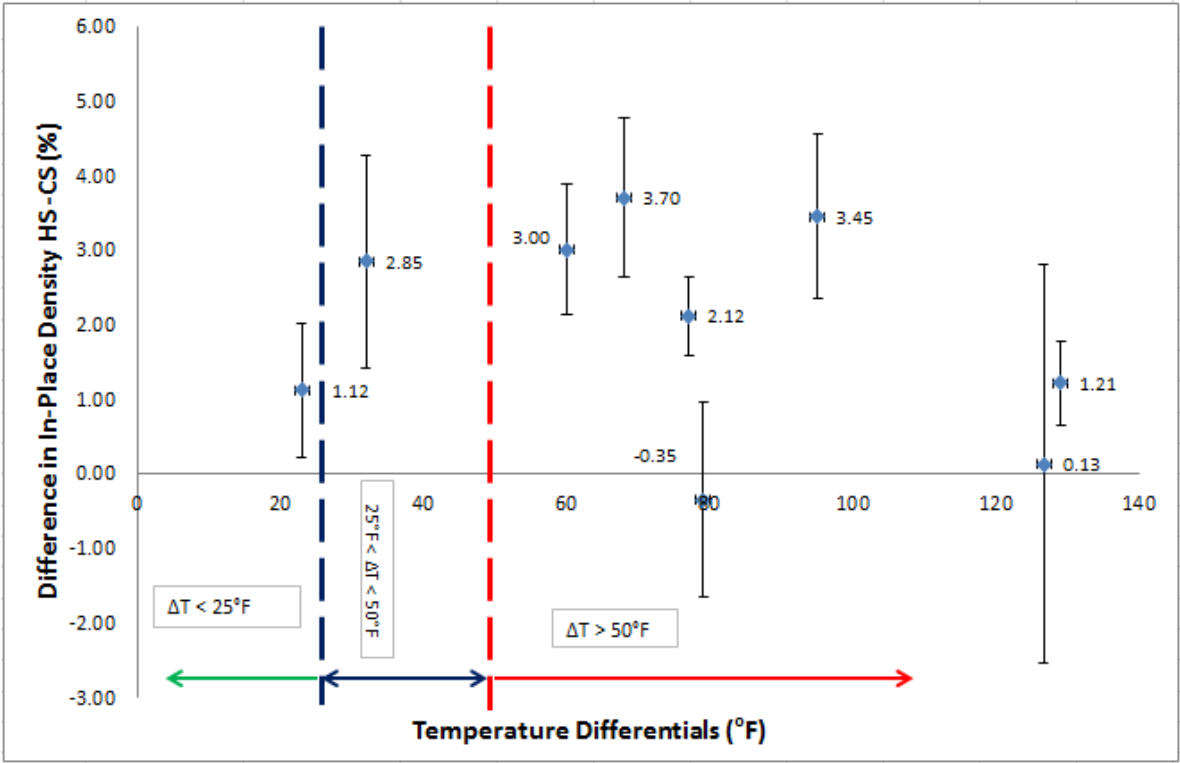


Figure 5.21: Core In-Place Density versus Temperature Differentials for the Nine Projects

It was expected that the difference in densities would increase as temperature differentials increased. However, from Figure 5.21 it can be observed that there is not a clear relationship between both parameters which coincides with Amirkhanian's findings (8) when he compared the contractor's core data with temperature differentials. This can be supported by the low R^2 value reported by this relationship ($R^2=0.099$). Probably, other factors related to the composition of the mix such as the asphalt content, the gradation and the type of aggregate can be more significant than high temperature differentials. Additionally, the differences in compaction techniques among the evaluated projects could also have more influence on in-place densities.

Seven of the 9 sets (projects) of field cores analyzed were taken from locations that reported temperature differentials greater than 50°F (severe thermal segregation). The difference in densities for 5 of these projects was greater than 1.0%. The other 2 projects had differences less than 1.0%. One of these projects was SR-10 where only two samples from the cold area were tested due to damage of the third specimen after the extraction. Probably, the inclusion of this sample in the analysis would have resulted in a greater difference in density. The other project where the difference in density was insignificant was SR-69 in Hale County where the low difference may be attributed to the fine-graded nature of the mix (3/8in, more workable mix at low temperature, finest mix evaluated) and the high speed of the paving operations (66ft/min, highest paver speed among all projects). The results also show that temperature differentials between 25°F and 50°F (moderate thermal segregation) can cause a difference in density higher than 2.0%.

Despite the poor relationship between temperature differentials and differences in density, temperature differentials greater than 50°F can potentially lead to areas of pavement that are more likely to have problems with accelerated aging and susceptibility to moisture damage, especially in surface courses which are directly exposed to air and water contact. The mix composition and the compaction techniques can represent more significant parameters than temperature differentials although there is no evidence of this. Therefore, future investigations about thermal segregation

could include the evaluation of these two parameters to better define their significance with regard to temperature differentials and density.

Chapter 6 : Laboratory Testing Results and Discussion

Bending Beam Fatigue Testing

Table 6.1 presents the number of cycles to failure and initial stiffness from the bending beam fatigue tests on mixtures from each project. Additionally, the table shows the results reported by the regression analyses for the relationship between air void levels and either initial stiffness and fatigue cycles as a consequence of thermally segregated areas.

The results of the regression analyses show a “poor” relationship between mix air voids and the cycles to failure. Only one project (US-35 at 600 $\mu\epsilon$) had a p-value less than 0.05 and an R^2 of 96.3 showed a strong relationship between air void contents and cycles to failure. All other projects had no correlations between air void contents and cycles to failures. These results coincide with other studies (19, 22), which demonstrated that the number of cycles to failure from bending beam fatigue testing is not significantly affected by variations in air voids.

From the results, it was also possible to compare the regression models (slope and intersection simultaneously) obtained by plotting the number of fatigue cycles obtained for both hot and cold areas versus the micro-strains levels (600 μm and 300 μm). This analysis was conducted to determine if the number of fatigue cycles obtained from the specimens compacted at high air void contents (cold areas) were different from those specimens compacted at low air void contents (hot areas). These plots were developed for each project and they are shown from Figure 6.1 to Figure 6.7. The results of the equivalency of the regression models are shown in Table 6.2.

Table 6.1: Beam Fatigue Testing Single Results and Regression Analyses

General Information		Hot Spots					Cold Spots					Statistical Analysis: Regression			
												AV% - In N_f		AV% -Initial Stiffness	
Project	Type of mix	Sample ID	Micro-strain	% Air Voids	No. Cycles N_f	Initial Stiffness (MPa)	Sample ID	Micro-strain	% Air Voids	No. Cycles N_f	Initial Stiffness (MPa)	p-value	R ² (%)	p-value	R ² (%)
SR-167	HMA	SR-167-1	600	7.5	20710	6518	SR-167-7	600	9.1	33490	5489	0.968	0.10	0.016	96.9
		SR-167-2	600	7.2	20920	6736	SR-167-8	600	9.4	14250	5588				
		SR-167-3	300	7.9	960960	6904	SR-167-9	300	9.8	1773280	6340	0.342	43.3	0.055	89.3
		SR-167-4	300	7.4	985450	7437	SR-167-10	300	9.6	1007280	6034				
US-31	HMA	US-31-2	600	7.4	47300	5773	US-31-4	600	11.5	23720	4581	0.956	0.20	0.099	81.1
		US-31-5	600	6.9	22370	7270	US-31-7	600	10.6	43900	4330				
		US-31-6	300	7.8	1202440	7600	US-31-8	300	10.1	1307390	5451	0.813	3.50	0.058	88.8
		US-31-11	300	7.1	2611540	7105	US-31-12	300	10.8	2033950	5211				
I-65	SMA	I-65-2	600	4.6	146920	8175	I-65-4	600	4.9	157540	7214	0.797	4.10	0.618	14.6
		I-65-6	600	4.5	123720	7077	I-65-5	600	5.4	137050	7124				
		I-65-9	300	4.6	1.44E+08	8517	I-65-3	300	4.8	1.07E+08	7841	0.746	6.50	0.199	64.1
		I-65-11	300	4.6	5.87E+07	8549	I-65-8	300	5.3	7.52E+07	7817				
SR-41	HMA	SR-41-1	600	9.3	9050	6887	SR-41-7	600	12.6	17780	4716	0.140	73.9	0.002	99.7
		SR-41-2	600	10.7	6750	6048	SR-41-8	600	13.2	23750	4486				
		SR-41-4	300	10.4	2078820	6650	SR-41-9	300	13.3	1142610	5296	0.658	11.7	0.010	98.0
		SR-41-6	300	9.8	1039880	6945	SR-41-10	300	12.9	1115130	5186				
US-35	WMA	US-35-5	600	6.1	26930	5805	US-35-4	600	9.1	21830	5188	0.019	96.3	0.037	92.6
		US-35-6	600	5.9	29180	6165	US-35-9	600	8.7	21960	5222				
		US-35-7	300	6.9	834320	6693	US-35-10	300	9.8	535030	5631	0.975	0.10	0.032	93.8
		US-35-8	300	6.2	498370	7187	US-35-11	300	9.2	713790	6178				
SR-101	HMA	SR-101-5	600	6.4	47610	6714	SR-101-1	600	9.2	54590	5708	0.992	0.00	0.048	90.7
		SR-101-8	600	5.8	61500	7724	SR-101-4	600	8.9	56910	5540				
		SR-101-2	300	6.1	4392420	8156	SR-101-6	300	8.5	2474780	7703	0.509	24.1	0.190	65.7
		SR-101-7	300	5.8	2452180	9308	SR-101-9	300	8.8	2493130	7794				
SR-278	HMA	SR-278-2	600	5.6	32180	7969	SR-278-1	600	7.2	36060	7158	0.307	48.1	0.072	86.2
		SR-278-6	600	5.5	21040	8523	SR-278-3	600	7.1	32140	6645				
		SR-278-8	300	6.1	2337590	8376	SR-278-4	300	7.2	4131750	7421	0.853	2.20	0.039	92.4
		SR-278-9	300	6.3	2964110	8082	SR-278-7	300	7.1	1006560	7727				

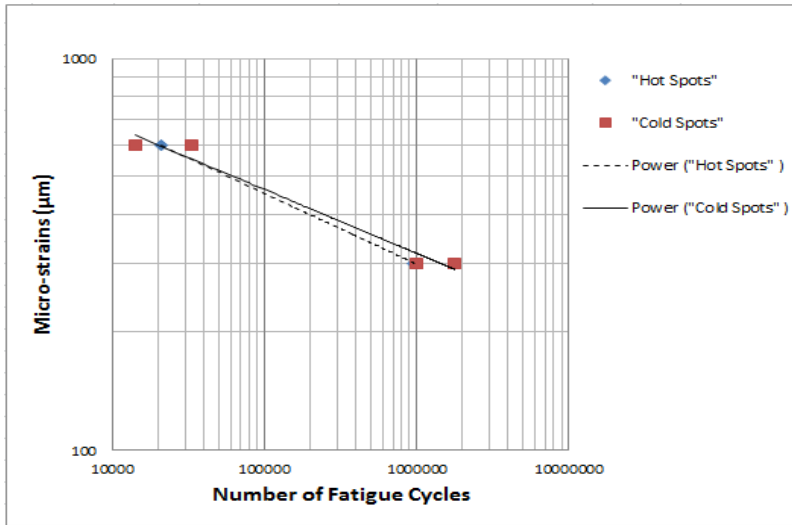


Figure 6.1: SR-167 Micro-strain Levels vs Number of Fatigue Cycles

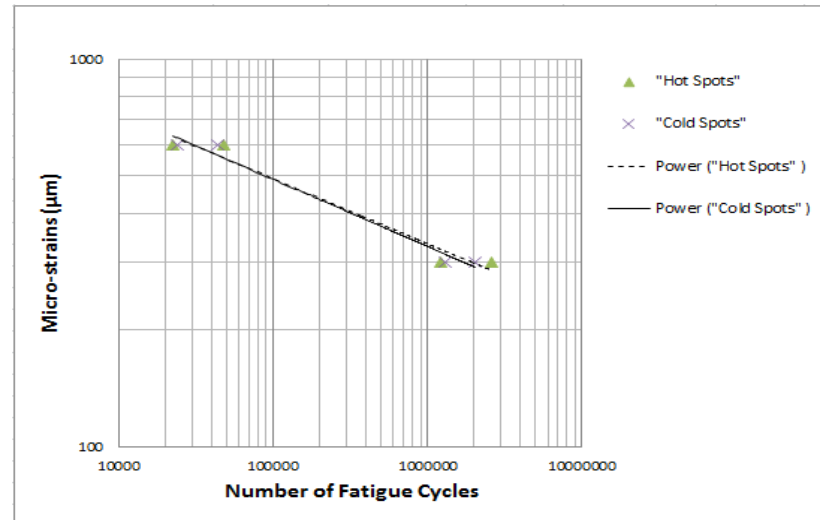


Figure 6.2: US-31 Micro-strain Levels vs Number of Fatigue Cycles

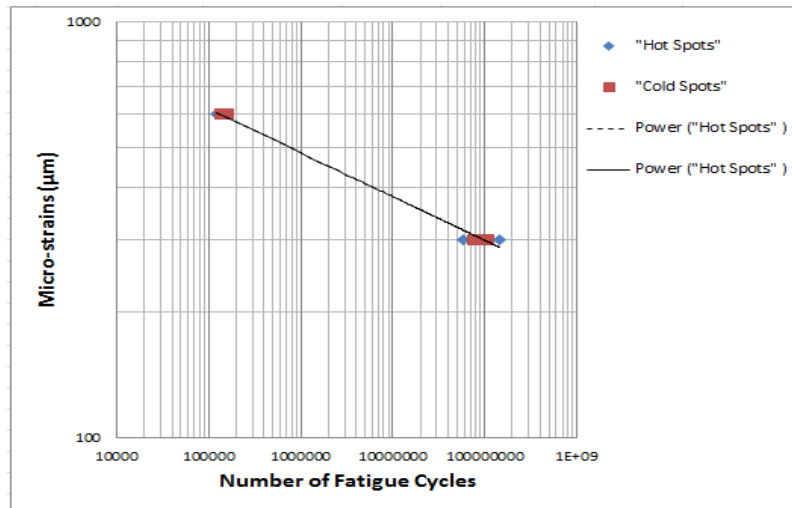


Figure 6.3: I-65 Micro-strain Levels vs Number of Fatigue Cycles

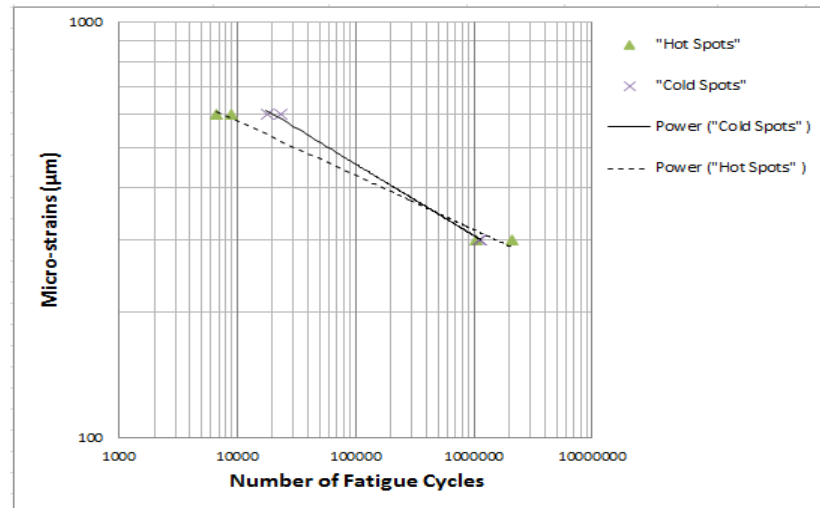


Figure 6.4: SR-41 Micro-strain Levels vs Number of Fatigue Cycles

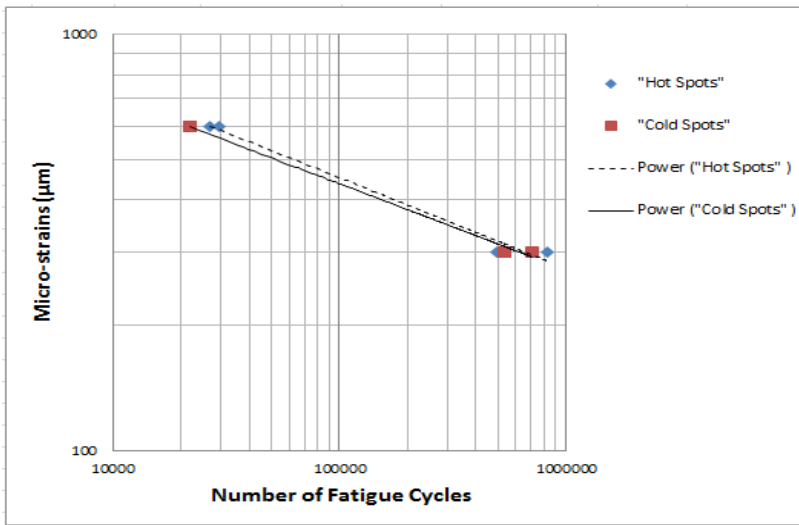


Figure 6.5: US-35 Micro-strain Levels vs Number of Fatigue Cycles

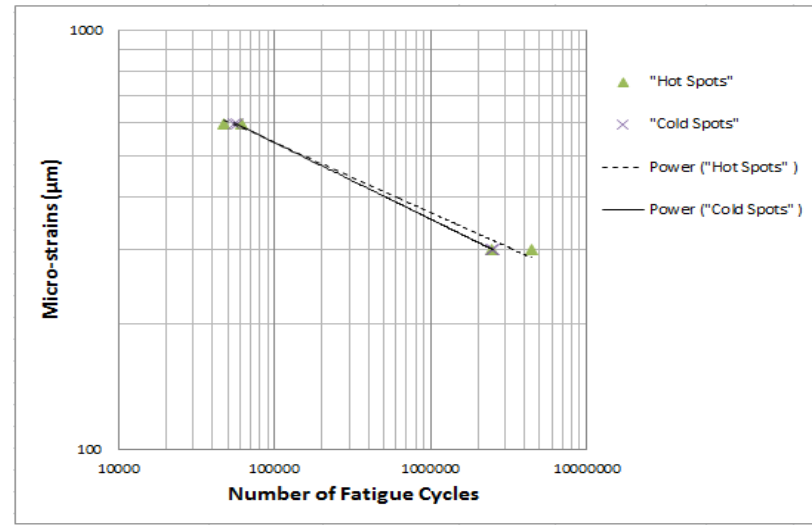


Figure 6.6: SR-101 Micro-strain Levels vs Number of Fatigue Cycles

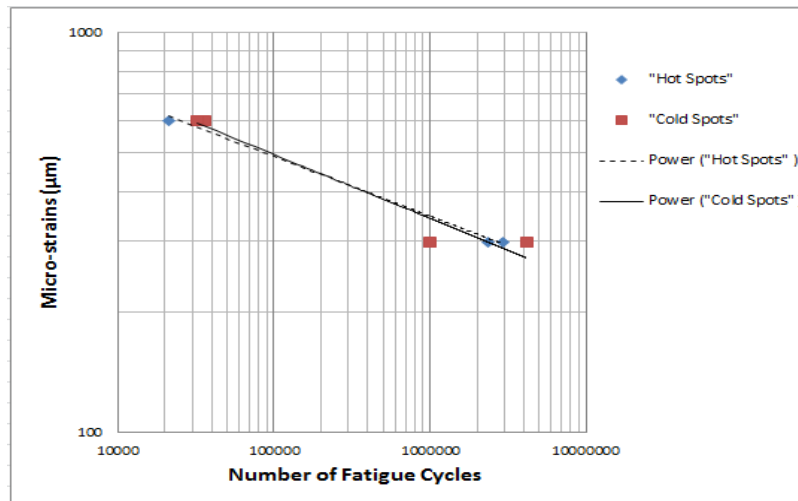


Figure 6.7: SR-278 Micro-strain Levels vs Number of Fatigue Cycles

**Table 6.2: Equivalency of Regression Models Micro-strains – Fatigue Cycles
Hot Areas and Cold Areas**

Project	p-value	Significantly Different (*)
SR-167	0.500	No
US-31	0.577	No
I-65	0.678	No
SR-41	0.570	No
US-35	0.678	No
SR-101	0.534	No
SR-278	0.687	No

(*): regression models are different if $p\text{-value} < 0.05$

According to the results, the regression models for both hot areas and cold areas were not statistically different for all the projects ($p\text{-value} > 0.05$). This means that similar number of fatigue cycles would be obtained at the same applied tensile micro-strain, regardless the difference in air voids between cold spots and hot spots. This further demonstrated the independency between the number of fatigue cycles and the difference in air voids.

The aforementioned results can also indicate that there are more significant parameters that affect fatigue life of mixtures such as asphalt content or aggregate gradation (17). Additionally, it is believed that the high variability associated with fatigue testing contributed to the unexpected results.

The non-uniform composition of the samples can be one of the main factors associated with the variability of the obtained fatigue parameters (45). In this case, the heterogeneity of a sample composition is mainly caused by the size, the shape and the degree of interconnection of the air voids, in addition to the absolute volume of these voids and the aggregate configuration. The presence of one large air void in the specimen can significantly reduce the load-carrying capacity of the sample with regard to presence of smaller air voids randomly distributed in the specimen. This fact can become more critical in larger specimens such as beam specimens with high air void contents.

The initial stiffness was found to have a strong relationship with air voids, with a higher mix stiffness corresponding to lower air void contents (19, 22). The figures 6.8-6.14 show the trend described previously. This indicates that the high air void levels caused by the lack of uniformity of mix temperature prior to its compaction can significantly reduce the stiffness of the asphalt concrete layers which would lead to higher strains under load and consequently a lower fatigue life.

Future research projects should consider the preparation of three or more specimens in order to reinforce the evidence used for demonstrating the relationships among air void contents, initial stiffness and the number of fatigue cycles.

According to the results, it is evident that the bending beam fatigue testing is insensitive to changes in air voids in terms of measured number of fatigue cycles. Future research projects should consider aging and/or moisture conditioning as part of fatigue testing(22). In terms of mix stiffness, the bending beam fatigue testing was sensitive to air voids. AMPT testing may also be considered for future research for developing dynamic modulus master curves that can be introduced into MEPDG considering the differences in in-place densities caused by high temperature differentials. This would allow predicting the damage over a period of time in terms of fatigue and rutting for both cold areas and hot areas.

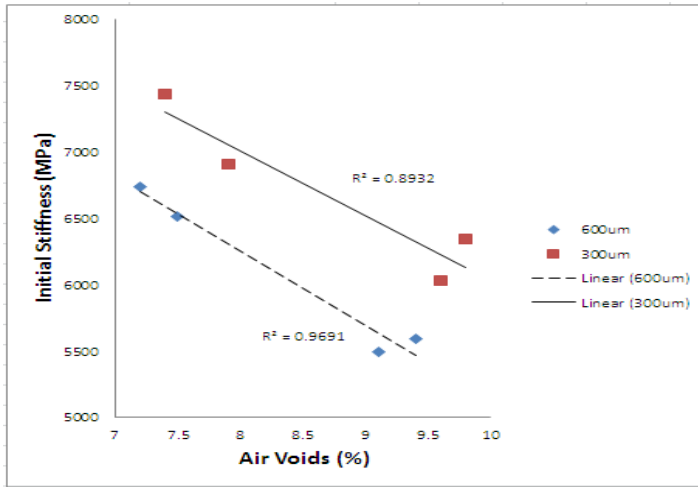


Figure 6.8: SR-167 Initial Stiffness (MPa) – Air Voids (%)

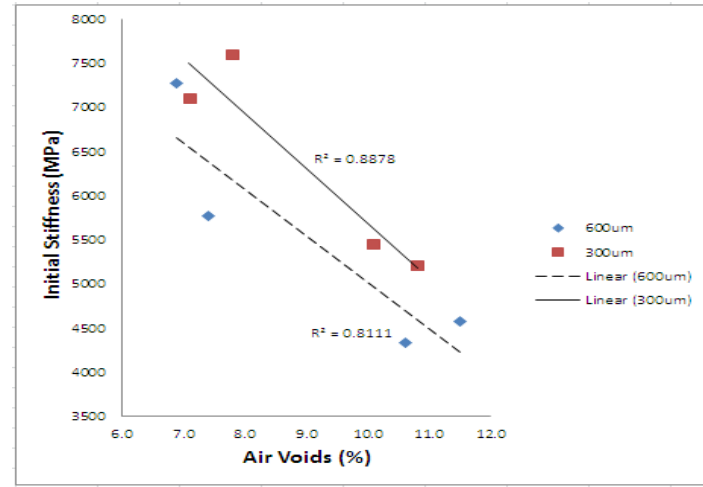


Figure 6.9: US-31 Initial Stiffness (MPa) – Air Voids (%)

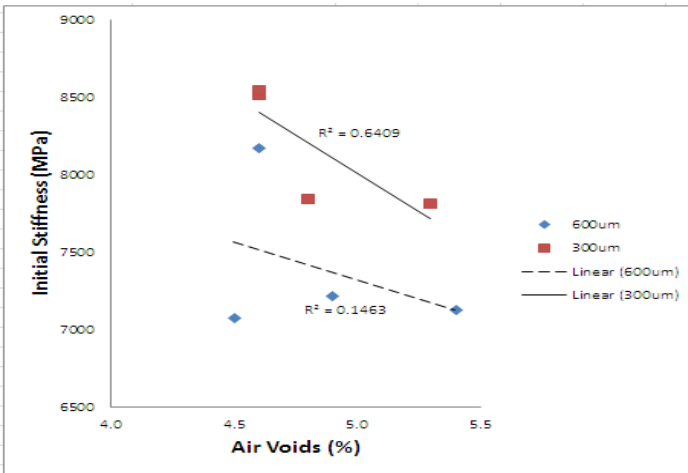


Figure 6.10: I-65 Initial Stiffness (MPa) – Air Voids (%)

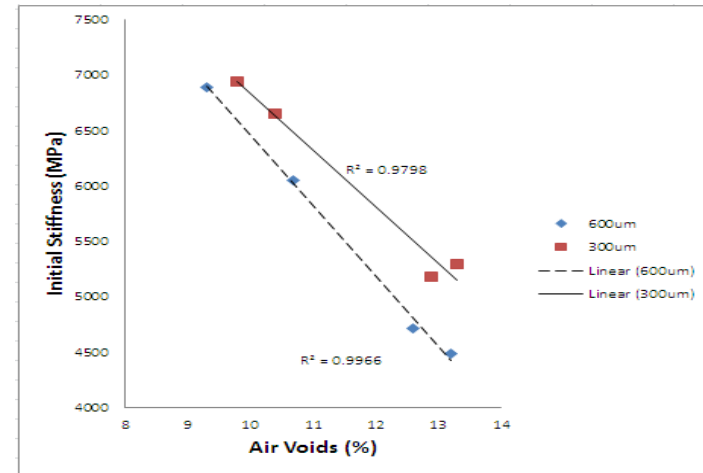


Figure 6.11: SR-41 Initial Stiffness (MPa) – Air Voids (%)

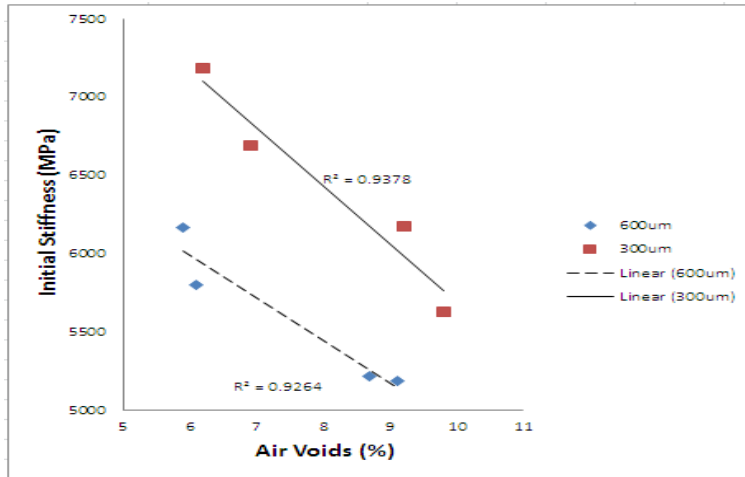


Figure 6.12: US-35 Initial Stiffness (MPa) – Air Voids (%)

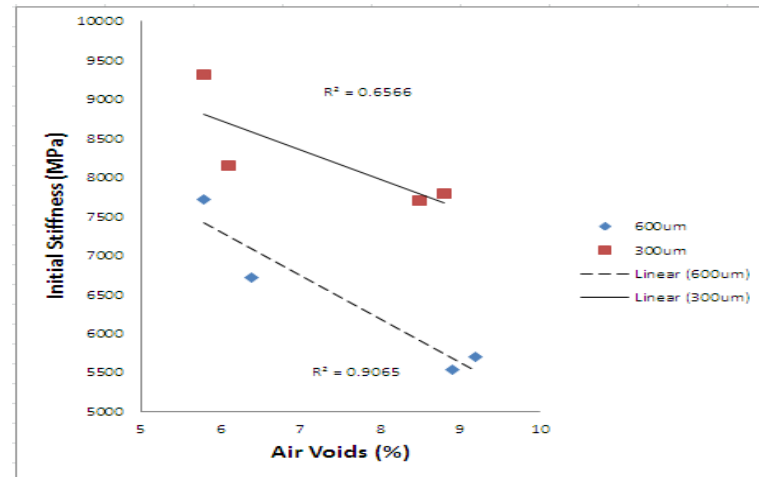


Figure 6.13: SR-101 Initial Stiffness (MPa) – Air Voids (%)

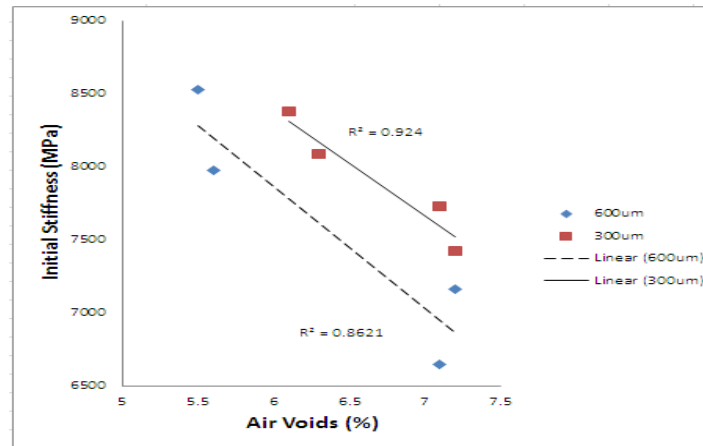


Figure 6.14: SR-278 Initial Stiffness (MPa) – Air Voids (%)

Fracture Energy Testing

Tables 6.3 and figure 6.15 present the fracture energy results obtained from indirect tensile strength testing and calculated by utilizing Matlab 7.10. The table also shows the results of the 2-sample t-test performed for comparison purposes. As previously mentioned in the methodology, it was necessary to take a second set of field cores from the first five evaluated projects. Therefore, an additional 2 sample t-test was performed for comparing in-place densities obtained from the new selected cold and hot areas. The results are presented in Table 6.3.

It was expected that the energy required to fracture a specimen extracted from a hot spot would be greater than the fracture energy for a specimen from a cold spot. Although that was the case in 8 of 9 projects, the statistical analysis indicated that the difference in fracture energy between cold and hot areas was not statistically significant.

From Table 6.3, it can be observed that the magnitudes and differences in density between hot and cold areas for the first five projects decreased with regard to the density levels reported in Table 5.11. The 2 sample t-test results demonstrate that the difference in density between both areas is not significant. The reduction of the differences in density between both areas can be attributed to traffic compaction. It appears that densification from traffic reduced the differences in air voids. Fracture energy results were not significantly different between the cold and hot areas for these specific projects except for one of the nine projects.

Table 6.3: Fracture Energy Testing / Two-Sample t-Test

General Information			Hot Spots					Cold Spots					Statistical Results %Gmm Analysis		Statistical Results FE Analysis	
													2-Sample t-test 95% CL		2-Sample t-test 95% CL	
Project	Type of mix	MAS	Sample ID	%Gmm	FE (psi)	%Gmm AVG	FE AVG (psi)	Sample ID	%Gmm	FE (psi)	%Gmm AVG	FE AVG (psi)	p-value	Criteria (***)	p-value	Criteria (***)
SR-167 (*)	HMA	3/4"	SR-167-4 HS RL	95.3	0.60	94.0	0.75	SR-167-1 CS RL	94.7	0.43	92.9	0.56	0.504	Equal	0.269	Equal
			SR-167-5 HS CL	93.0	1.00			SR-167-2 CS CL	90.7	0.70						
			SR-167-6 HS LL	93.6	0.65			SR-167-3 CS LL	93.4	0.55						
US-31 (*)	HMA	3/4"	US-31-1 HS RL	94.1	1.00	93.1	0.89	US-31-4 CS RL	92.9	0.70	92.4	0.66	0.455	Equal	0.161	Equal
			US-31-2 HS CL	93.2	1.00			US-31-5 CS CL	92.4	0.75						
			US-31-3 HS LL	91.9	0.68			US-31-6 CS LL	92.0	0.55						
I-65 (*)	SMA	1/2"	I-65-4 HS RL	94.7	1.07	95.1	0.93	I-65-1 CS RL	96.0	0.88	96.1	0.89	0.373	Equal	0.686	Equal
			I-65-5 HS CL	94.1	0.76			I-65-2 CS CL	94.8	0.92						
			I-65-6 HS LL	96.4	0.97			I-65-3 CS LL	97.6	0.87						
SR-41 (*)	HMA	3/4"	SR-41-1 HS RL	92.4	0.69	91.9	0.82	SR-41-4 CS RL	91.0	0.38	89.3	0.57	0.100	Equal	0.128	Equal
			SR-41-2 HS CL	91.7	0.82			SR-41-5 CS CL	88.6	0.74						
			SR-41-3 HS LL	91.6	0.96			SR-41-6 CS LL	88.3	0.60						
SR-10 (**)	HMA	1/2"	SR-10-4 HS RL	92.5	1.07	92.6	0.93	SR-10-1 CS RL	93.2	1.48	92.0	0.99	0.473	Equal	0.836	Equal
			SR-10-5 HS CL	93.1	0.83			SR-10-2 CS CL	91.6	0.72						
			SR-10-6 HS LL	92.2	0.89			SR-10-3 CS LL	91.3	0.77						
US-35	WMA	3/4"	US-35-1 HS RL	91.6	0.50	93.8	0.85	US-35-4 CS RL	89.7	0.54	91.0	0.66	0.116	Equal	0.434	Equal
			US-35-2 HS CL	94.7	0.94			US-35-5 CS CL	90.7	0.64						
			US-35-3 HS LL	95.2	1.12			US-35-6 CS LL	92.7	0.80						
SR-101	HMA	1/2"	SR-101-1 HS RL	93.4	1.23	94.3	1.13	SR-101-4 CS RL1	91.2	0.70	91.3	0.80	0.026	Different	0.014	Different
			SR-101-2 HS CL	94.8	1.11			SR-101-5 CS RL2	90.8	0.92						
			SR-101-3 HS LL	94.6	1.05			SR-101-6 CS CL	89.7	0.68						
								SR-101-7 CS LL	93.3	0.91						
SR-69	HMA	3/8"	SR-69-1 HS RL	94.1	0.71	92.1	0.69	SR-69-6 CS RL	94.4	0.69	92.5	0.59	0.794	Equal	0.210	Equal
			SR-69-2 HS CL	90.4	0.68			SR-69-7 CS CL	91.1	0.61						
			SR-69-3 HS LL	94.9	0.90			SR-69-8 CS LL	94.7	0.58						
			SR-69-4 HS CLB	90.4	0.52			SR-69-9 CS CLB	90.8	0.41						
			SR-69-5 HS CLA	90.7	0.66			SR-69-10 CS CLA	91.4	0.65						
SR-278	HMA	3/4"	SR-278-1 HS RL	93.3	0.82	93.9	0.90	SR-278-4 PS RL	92.0	0.73	92.8	0.77	0.165	Equal	0.142	Equal
			SR-278-2 HS CL	94.9	0.98			SR-278-5 PS CL	93.4	0.79						
			SR-278-3 HS LL	93.4	0.90			SR-278-6 PS LL	93.1	0.80						

(*): 2nd set of cores age between 11 and 12 months.

(**): 2nd set of cores age of 5 months

(***): means are different if p-value < 0.05

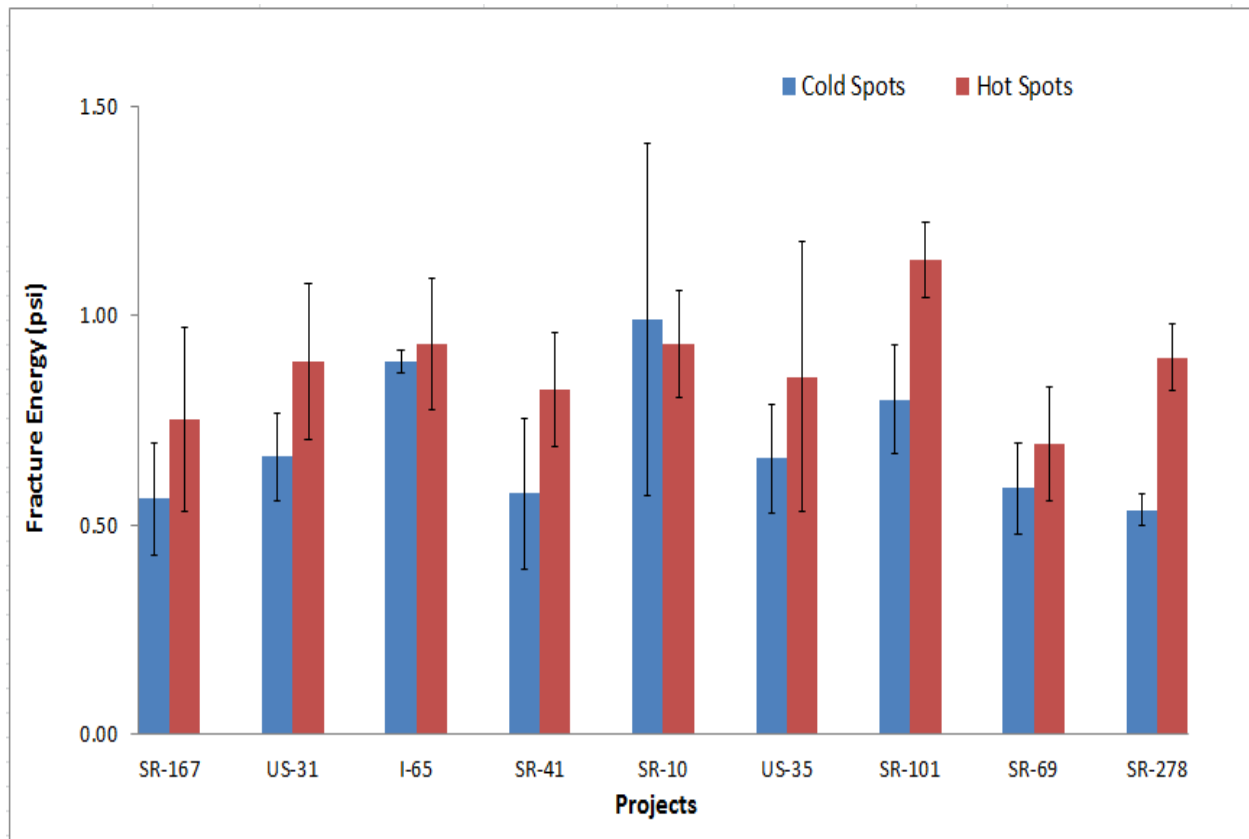


Figure 6.15: Comparison between Field Cores Fracture Energy Hot Spots vs Cold Spots

Figure 6.16 presents the plot of differences in fracture energy versus mat temperature differentials. No relationship was evident between temperature differentials and fracture energy despite that some of the projects showed differences in density greater than 1.0%. Perhaps, other factors such as mixture properties or the compaction methods used in the projects have more influence on the cracking resistance than high temperature differentials. For future research projects, it is recommended to evaluate the influence of the compaction patterns and the mix properties to examine other possible significant factors.

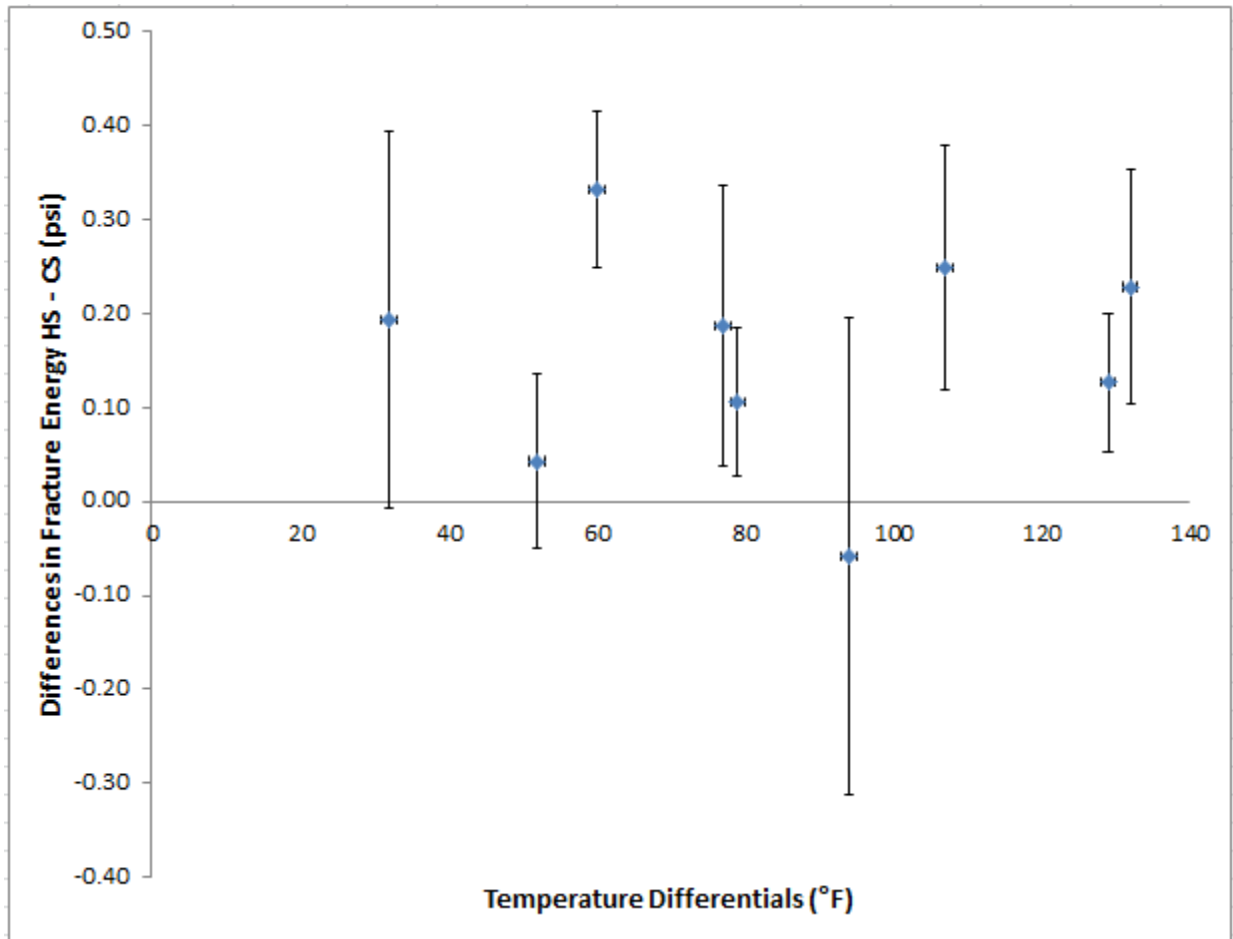


Figure 6.16: Relationship between Differences in Fracture Energy and Temperature Differentials

Another source of variability could be GPS positioning error associated with locating the hot spots and cold spots for coring.

The heterogeneous nature of the field pavement samples could also contribute to the variability of fracture energy results (45). As aforementioned, the presence of one large air void in the specimen can produce a weak area that can significantly reduce the load-carrying capacity of the sample.

It is important to note that the small number of samples were not sufficient to establish relationships between density and fracture energy. Therefore, it is recommended to obtain more samples from the field in order to support the findings of future studies related to thermal segregation.

Chapter 7 : Conclusions

- The results of the statistical analysis on the collected field data indicates that the use of material remixing devices such as the Roadtec SB-2500 MRD and the Weiler 1250A MRD, can reduce significantly the presence of high temperature differentials in the mat prior to its compaction.
- Material transferring devices did not improve temperature of the occurrence of thermally segregated areas. The absence of a remixing system, accumulation of mix in hopper wings and the opened-air conveyor may have contributed to unsatisfactory results with this type of machine.
- Mixes with maximum aggregate sizes of 1in were more susceptible to temperature segregation than the rest of the mixes. Coarser mixes may cool down faster than finer mixes and therefore, their effects are more notorious in the development of high temperature differentials.
- The results showed that warm-mix asphalt helps to maintain mix temperature uniformity prior to its compaction, based on the standard deviations reported by the MOBA PAVE-IR infrared bar.
- Remixing devices can reduce excessive temperature variations in mixes highly susceptible to cool down quickly such as HMA, SMA and OGFC. HMA mixes reported the highest variations in temperatures due to the lack of remixing actions provided by the Blaw-Knox MC-330 MTD.
- The effects of hauling time on thermal segregation can be significantly reduced as long as a remixing process can be incorporated into the paving processes. Even haul times of less than 20min can result in high levels of severe thermal segregation if a remixing device is not considered.
- The results of this study indicate that the time of day was not significant in the development of high temperature differentials. Despite this, it is believed that nighttime paving projects are more susceptible to thermal segregation than daytime projects due to the continuous reduction of ambient temperature during the night and the rapid temperature loss of the paved layers.

- The ALDOT MRD specification to eliminate thermal segregation is not effective due to trucking delays and equipment that does not adequately remix hot and cold materials. The MRD specification should be revised and requirements for a continuous operation should be enforced.
- The MOBA Pave-IR infrared bar is a powerful and efficient tool for determining the presence of high temperature differentials over the mat prior to its compaction. Although the infrared camera was also helpful for the identification of thermally segregated areas in the field, the MOBA Pave-IR infrared bar can provide an accurate location of the cold and hot spots for future analyses, which was considered a great advantage over infrared cameras.
- The use of the MOBA Pave-IR infrared bar software facilitated the analysis of the thermal profiles obtained from the field. With this tool, it was possible to determine the levels of thermal segregation, the exact location of cold spots and hot spots and the distribution of mix temperature along the evaluated pavement section. Equipment such as infrared cameras and infrared guns do not offer these analytical capabilities which are very helpful for this kind of studies.
- The free manipulation of devices such as infrared cameras allows users to identify potential causes of high temperature differentials. For example, infrared images of the mix at the truck bed, the MTD hopper or conveyor belt were taken easily from different angles with the infrared camera used in this study.
- Based on the evaluated projects, it was found that the average in-place density obtained from the cold spots was 1.0% lower than average density obtained from the hot spots in 7 of the 9 divisions, indicating a negative effect of thermal segregation on mat in-place densities.
- The results of the statistical analyses show that there is no significant difference between cold and hot spots in terms of in-place density. However, it is believed that the amount of collected samples did not provide enough statistical evidence of the aforementioned difference, in addition to the different sources of variability involved in this study.

- Although no statistical relationship was evident between temperature differentials and in-place density; it was found that temperature differentials greater than 50°F can cause excessive differences in density.
- The bending beam fatigue test results were not significantly affected by the variations in air voids used in this study. The lack of significance was probably due to the large variability associated with the beam fatigue test.
- The mix initial stiffness was affected by air voids, with higher mix stiffness corresponding to lower air void contents. Therefore cold spots would result in higher strains under any given load which would lead to more fatigue cracking in the field.
- Despite that the fracture energy of the hot spots were slightly greater than cold spots in 8 of the 9 projects, the statistical analyses did not show a significant difference between hot and cold spots.
- Based on the field samples obtained from the 5 revisited projects, the densification produced by traffic reduced the effects of high temperature differentials on in-place densities, after several months of service. This probably influenced the fracture energy results.
- The results indicate that the effect of high temperature differentials in the fatigue life of asphalt concrete layers is probably not as significant as mix properties.

Chapter 8 : Recommendations

- This study demonstrated that the MOBA Pave-IR infrared bar and infrared cameras can be powerful tools for the identification of poor paving practices. Contractors should consider the use of these tools to help them improve their paving operations in order to achieve better in-place densities, better smoothness, and better overall quality pavements.
- Performance monitoring of the projects should be followed for several years of service under real traffic and environmental conditions. If differences in performance are noted, then testing of cores could help assess the influence of aging and moisture damage.
- Future research projects related to thermal segregation should evaluate the effect of other factors such as compaction patterns, mixture composition, and seasonal paving on temperature differentials. Future research projects on thermal segregation may also better evaluate the influence of WMA and OGFC on thermal segregation and performance of these mix types.
- It is recommended to obtain a greater amount of samples from projects in order to increase the statistical power of the experiments.
- Future research projects should consider the use of laboratory compaction methods that are influenced by mix temperature to better assess the effects of temperature on performance test results.
- The effect of moisture damage and/or aging on fatigue could be evaluated by testing unconditioned and conditioned beam samples.
- The dynamic modulus test should also be considered so that the influence of the experimental factors could be introduced into MEPDG analyses to predict the effects on pavement performance

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Appendix A

Route	Division	MRD/MTV	Mix Type	Layer / MAS	Thickness (in)	Hauling Time (min)	Paver Speed (ft/min)	Paver Stop (min)	Time of Day	No TS	Moderate TS	Severe TS
SR 167	7	Blaw Knox MC-330	Dense-graded HMA	Surface/Binder Course 3/4"	1.50	21 min ≤ HT ≤ 40min	22.27	0.00	Day	0	40	60
SR 69	1	Roadtec SB-2500C	Dense-graded HMA	Surface/Binder Course 3/4"	1.25	21 min ≤ HT ≤ 40min	29.86	27.00	Day	5	70	25
US 31	3	Roadtec SB-2500B	Dense-graded HMA	Surface/Binder Course 3/4"	1.75	21 min ≤ HT ≤ 40min	19.23	66.00	Day	5	95	0
SR 165	4	Roadtec SB-2500C	WMA	Surface/Binder Course 3/4"	1.25	> 40min	18.14	33.00	Day	27	67	6
I-65	6	Roadtec SB-2500B	SMA	Surface Course 1/2"	1.50	21 min ≤ HT ≤ 40min	22.90	3.00	Night	19	69	12
SR 27	7	Roadtec SB-2500C	Dense-graded HMA	Surface/Binder Course 3/4"	1.50	21 min ≤ HT ≤ 40min	41.13	0.00	Day	4	96	0
SR 41	9	Blaw Knox MC-330	Dense-graded HMA	Surface/Binder Course 3/4"	1.50	> 40min	41.83	7.00	Day	0	5	95
SR 59	9	Roadtec SB2500C	Dense-graded HMA	Surface/Binder Course 3/4"	1.50	> 40min	21.41	41.00	Night	15	74	11
US 80	6	Blaw Knox MC-330	Dense-graded HMA	Surface/Binder Course 3/4"	1.65	≤ 20min	37.43	4.00	Day	3	50	47
SR 136	8	Blaw Knox MC-330	Dense-graded HMA	Surface Course 1/2"	1.50	> 40min	21.33	121.00	Day	0	30	70
SR 144	3	Roadtec SB2500C	Dense-graded HMA	Surface Course 1/2"	1.50	21 min ≤ HT ≤ 40min	20.56	10.00	Day	42	58	0
US 72	1	Roadtec SB2500D	Dense-graded HMA	Surface/Binder Course 3/4"	1.25	21 min ≤ HT ≤ 40min	56.31	0.00	Day	13	87	0
SR 60	5	Roadtec SB2500D	Dense-graded HMA	Surface Course 3/8"	1.00	> 40min	26.46	0.00	Day	9	78	13
SR 137	7	Roadtec SB2500C	WMA	Surface/Binder Course 3/4"	1.50	> 40min	24.55	11.51	Day	12	88	0
I-65	3	Roadtec SB2500D	SMA	Binder Course 1"	2.50	≤ 20min	19.52	10.00	Night	0	57	43
SR 10	8	Blaw Knox MC-330	Dense-graded HMA	Surface Course 1/2"	1.25	> 40min	23.88	14.00	Day	0	21	79
SR 184	2	Roadtec SB2500	Dense-graded HMA	Surface Course 1/2"	1.50	> 40min	32.66	8.00	Day	43	57	0
Airport Blvd	9	Blaw Knox MC-330	Dense-graded HMA	Surface Course 1/2"	1.25	> 40min	55.43	0.00	Day	0	25	75
US 35	1	Roadtec SB2500D	WMA	Surface/Binder Course 3/4"	3.00	≤ 20min	26.19	25.00	Day	64	36	0
SR 101	2	Blaw Knox MC-330	Dense-graded HMA	Surface Course 1/2"	1.25	21 min ≤ HT ≤ 40min	28.02	0.00	Day	0	26	74
SR 10	8	Blaw Knox MC-330	Dense-graded HMA	Surface Course 1/2"	1.25	> 40min	30.94	13.00	Day	0	30	70
SR 247	2	No MTD/MRD used	Dense-graded HMA	Binder Course 1"	2.75	> 40min	17.20	61.00	Day	0	0	100
SR 275	4	Weiler 1250A	Dense-graded HMA	Surface/Binder Course 3/4"	2.00	> 40min	28.98	17.00	Day	0	100	0
SR 69	5	Blaw Knox MC-330	Dense-graded HMA	Surface Course 3/8"	0.88	> 40min	66.62	13.00	Day	8	46	46
I-85	6	Roadtec SB2500C	SMA	Surface Course 1/2"	1.50	≤ 20min	18.38	10.00	Night	0	82	18
SR 278	4	Roadtec SB2500C	Dense-graded HMA	Surface/Binder Course 3/4"	1.50	> 40min	25.58	45.00	Day	8	74	18
SR 25	5	Weiler 1250A	Dense-graded HMA	Binder Course 1"	2.75	21 min ≤ HT ≤ 40min	15.38	0.00	Day	0	57	43
I-65	3	Roadtec SB2500D	OGFC	Surface/Binder Course 3/4"	0.88	21 min ≤ HT ≤ 40min	33.30	21.00	Night	0	53	47

Appendix B



Evaluation of Hot-Mix Asphalt Thermal Segregation / Field Worksheet

Table 1. General Information				
Date:		Project:		
Route:		Contractor:		
County:		City:		State:
Weather:		Ambient Temperature (°F):		

Table 2. Construction Equipment and HMA Characteristics				
Paver Type:		Transfer Dev.:		Truck types:
Paving Width (ft):		Start point:		Truck covers:
Layer Thickness (in):		Binder:		Mix type (by MAS):
Paving length (ft):		Job Mix Formula Temperature (°F):		
Hauling time (min):		Hauling distance (miles):		
Existence Surface Temperature (°F):				

Table 3. Temperature field measurements				
Lecture	Station (ft)	Temperature gun (°F)		
		Reading 1	Reading 2	Reading 3
1				
2				
3				
4				
5				
6				
7				
8				
9				
10				
11				
12				
13				
14				
15				
16				
17				
18				
19				