Quality Control/Quality Assurance Testing for Joint Density and Segregation of Asphalt Mixtures

Final Report
April 2013

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16. Abstract

Longitudinal joint quality control/assurance is essential to the successful performance of asphalt pavements and it has received considerable amount of attention in recent years. The purpose of the study is to evaluate the level of compaction at the longitudinal joint and determine the effect of segregation on the longitudinal joint performance.

Five paving projects with the use of traditional butt joint, infrared joint heater, edge restraint by milling and modified butt joint with the hot pinch longitudinal joint construction techniques were selected in this study. For each project, field density and permeability tests were made and cores from the pavement were obtained for in-lab permeability, air void and indirect tensile strength. Asphalt content and gradations were also obtained to determine the joint segregation. In general, this study finds that the minimum required joint density should be around 90.0% of the theoretical maximum density based on the AASHTO T166 method. The restrained-edge by milling and butt joint with the infrared heat treatment construction methods both create the joint density higher than this 90.0% limit. Traditional butt joint exhibits lower density and higher permeability than the criterion. In addition, all of the projects appear to have segregation at the longitudinal joint except for the edge-restraint by milling method.

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Principal Investigator
R. Christopher Williams
Professor, Civil, Construction, and Environmental Engineering
Iowa State University

Co-Principal Investigator
Hosin "David" Lee
Professor, Civil and Environmental Engineering
University of Iowa

Research Assistants
Can Chen, Iowa State University
Taha Ahmed, University of Iowa

Authors
R. Christopher Williams, Can Chen, Taha Ahmed, and Hosin "David" Lee

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A report from
Institute for Transportation
Iowa State University
2711 South Loop Drive, Suite 4700
Ames, IA 50010-8664
Phone: 515-294-8103 Fax: 515-294-0467
www.intrans.iastate.edu

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

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EXECUTIVE SUMMARY

Longitudinal joint quality control/assurance is essential to the successful performance of asphalt pavement and it has received considerable amount of attention in recent years. The purpose of the study is to evaluate the level of compaction at the longitudinal joint and determine the effect of segregation on the longitudinal joint performance.

Five paving projects are selected for sampling and evaluation in Iowa with each one representing a typical longitudinal joint construction technique. The first two joint construction methods use the tradition butt joint placed with hot mix asphalt (HMA) and warm mix asphalt (WMA). Another three construction methods paved with HMA are the butt joint with an infrared heat treatment, edge restraint by milling method and a modified butt joint with the first pass of rolling 6 inches away from the joint (hot pinch). For each project, joint quality is compared with regard to the “center” of the pavement mat (2 ft right of the joint). Field densities using a PaveTracker 2701 non-nuclear gauge and permeability using an NCAT Permeameter were made. Cores at both the longitudinal joint and 2 ft right of joint were obtained for subsequent lab permeability, AASHTO T166 and AASHTO T331 density, and indirect tensile (IDT) strength testing. Asphalt content and gradations were also obtained by ignition oven method to determine the joint segregation.

In general, this study finds that the minimum required joint density should be 90.0% of theoretical maximum density based on the AASHTO T166 method. The restrained-edge by milling and butt joint with infrared heat treatment construction methods all create joint density values higher than the proposed 90.0% limit. The traditional butt joint paved in both HMA and WMA exhibits lower density and higher permeability than the aforementioned limit. In addition, all of the projects appear to have segregation at the longitudinal joint except for the one using the edge-restraint by milling method. Based on various mix design and joint construction methods, the joints show differences in asphalt content and types of segregation (via gradation) as compared to the job mix formula. Results of this study indicate that lower density of the longitudinal joint is a combination of segregation (gradation), asphalt content variation and insufficient density.

CHAPTER 1 INTRODUCTION

1.1 Problem Statement

Longitudinal joint quality control/assurance is essential to the successful performance of asphalt pavements and it has received a considerable amount of attention in recent years. Poor joint construction can lead to a location where water can easily penetrate the pavement layer and result in an increased potential for moisture damage in the pavement, leading to distresses such as raveling and stripping. Many state agencies are moving toward the implementation of a longitudinal joint specification. According to the information provided at the Research in Progress database on the Transportation Research Board (TRB) Website (TRB, 2010), current available longitudinal joint research projects in the U.S. include *The Evaluation of Longitudinal Joint Density* conducted by the Colorado DOT, *Quality Assurance/Quality Control Testing for Joint Density and Segregation of Asphalt Mixtures* by the Iowa DOT, *The Improved Longitudinal Joint Construction* sponsored by the Kentucky Transportation Cabinet and *The HMA Longitudinal Joint Deterioration Investigation* supported by the National Center for Freight & Infrastructure Research and Education. In one word, assessment of longitudinal joint construction quality can be beneficial to improving the performance of the joint and has drawn a significant amount of research attention in recent years.

The Iowa Department of Transportation (IDOT) currently does not have a test method or specification for identifying segregation and quality control/quality assurance for longitudinal joint density. A number of specific questions are to be answered in the study:

1. What are the best methods for constructing longitudinal joints in Iowa?
2. Is permeability of longitudinal joints related to the joints' performance?
3. If permeability is related to longitudinal joint performance, what are the appropriate quality assurance criteria?
4. What types of tests can be used to detect the segregation on an asphalt mat and longitudinal joint?
5. Does segregation have a great effect on the longitudinal joint performance?

1.2 Objectives

The main purpose of this study is to obtain necessary field and laboratory test data to evaluate the level of compaction at the longitudinal joint and determine the effect of segregation on the longitudinal joint performance.

1.3 Report Organization

The report consists of six chapters including the introductory one as the first. The second chapter provides a comprehensive literature review consisting of longitudinal joint construction methods, pavement density and permeability research work and HMA segregation detection methods. The experimental procedure and testing methods are described in the third chapter. The fourth chapter provides the data collected as part of the research as well as a detailed statistical analysis. The field performance of the longitudinal joints selected for testing and analysis are described in the chapter five. Finally, the sixth chapter outlines the findings, conclusions and makes recommendations.

CHAPTER 2 LITERATURE REVIEW

2.1 Longitudinal Joint Construction Techniques

A longitudinal joint is the interface between two adjacent and parallel asphalt pavement mats. Several types of longitudinal joint construction techniques are commonly used in Iowa. These include the butt joint, notched wedge joint, modified butt joint with pinching, joint with heat treatment, and joint with edge restraint.

The traditional method for constructing a longitudinal joint in Iowa is the butt joint (see Figure 1). The challenge of the butt joint is to achieve adequate density on the unconfined edge of the cold lane. This is because at the time of its compaction, there is no lateral confinement to compact against the cold lane, therefore, the unconfined edge is able to move laterally when the downward compaction force is applied and not attain the desired density. Pinching the butt joint by adding extra material for compaction near to the joint is a way to achieve better butt joint density. Kandal et al. (2002) reported that rolling from the hot lane (6 inches) away from the joint during the first pass can provide better butt joint confinement. They found that this technique would push the material between the roller and joint towards the joint during the initial roller pass, which crowds and pinches the mix at the butt joint area and produces a higher density (see Figure 2). However, this method may make the longitudinal joint appear slightly humped as shown in Figure 2 (right). Researchers in Canada reported that warm mix asphalt (WMA) may produce a tighter butt joint than hot mix asphalt (HMA) as the temperature differential for continuous paving is reduced (Hughes et al., 2009). The heat loss associated with WMA is less, which makes it more versatile during various weather conditions. However, they also remarked that although the WMA is very workable, it has a stiffer makeup than the corresponding HMA and thus held the mix together to reduce gradation segregation.

Temperature is always considered as a key component in longitudinal joint construction. It is generally believed that higher temperatures can help increase compaction of the material at the joint and improve the bond between the cold mat and the hot mat. The basic premise of the joint heat treatment is that after the cold lane is placed, the joint area can be pre-heated just prior to placement of the hot lane, make the constituent asphalt binder in the cold lane more viscous and stickier. Daniel (2006) reported that the infrared heat can penetrate the existing pavement and heat the mixture within 25 to 50 mm of joint up to the temperatures of about 60^{o}C during the initial compaction by the first roller. The temperature would drop down to about 50^{o}C when the finishing roller passes. Results of field trials in Kentucky, Tennessee and New Hampshire have all reported that the use of a joint heater can effectively reduce permeability/increase density, increase the indirect tensile (IDT) strength of the asphalt mixtures and provide a smooth joint (Fleckenstein, et.al. 2002, Huang and Shu, 2010, Daniel, 2006).

The notched wedge joint was originally developed in Michigan and has been gradually considered as a good option for longitudinal joint construction. As shown in Figure 3, an extended joint taper placed on the first paved lane can help reduce joint air voids and the notches should be at least as deep as the nominal maximum aggregate size (NMAS) of the mix and the taper is usually spread out over about 0.3 m (1 ft.). The hot lane should overlap the cold lane notch by about 12.5 to 25 mm (0.5 to 1 inch) to ensure enough material at the notch for adequate compaction. Buchanan (2000) compared the notched-wedge joint technique with the conventional butt joint technique in Colorado, Indiana, Alabama, Wisconsin, and Maryland. The evaluation consisted of comparing the in-place densities obtained through pavement cores at five locations across the longitudinal joint of the pavement: at the centerline and at 150 mm (6 in.) and 450 mm (18 in.) on either side of the centerline. The results of the study indicate that the notched-wedge joint can be successfully used to increase the in-place density at the longitudinal joint. Some decrease in the in-place density was observed at the 150-mm (6-in.) location in the hot lane when the notched-wedge joint was used. However, some construction-related problems for the notched wedge joint were observed and pointed out by Fleckenstein et.al. (2002). These problems include maintaining the upper notch during compaction, raveling on the lower portion of the wedge and aggregate pickup by the small wedge roller. Bulging of the notch was also observed in some cases. It appears that the wedge is restraining the mix from pushing sideways during compaction and is the cause of the bulging.

A longitudinal joint construction technique using the milling operation to form edge restraints for both the cold lane and hot lane was applied in Iowa. In this method, one old lane is milled and the adjacent traffic lane can make a natural vertical edge face for the first new paving lane during compaction. After the first paving lane becomes cold, the adjacent traffic lane would be milled and the first paving lane can serve as the edge restraint for the second paving lane. Since the confinement can be formed during both the paving passes of the cold and hot lanes, it is believed that this technique would result in a better joint performance.

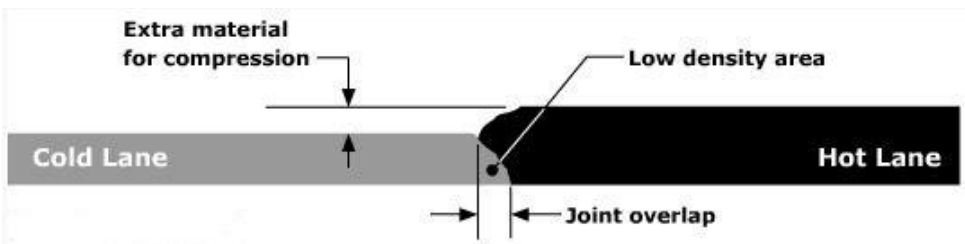


Figure 1. Butt longitudinal joint schematic (WSDOT 2012)

(1) (2)

Figure 2. Butt joint construction with hot pinch (pictured on US 61)

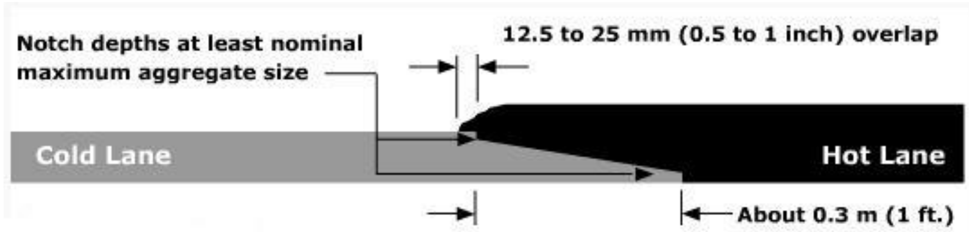


Figure 3. Notched wedge joint schematic (WSDOT 2012)

2.2 Permeability Measurement of HMA

HMA is a porous medium consisting of graded aggregates bound with asphalt binder plus a certain amount of air voids. In pavement construction, it is important that the asphalt mixture be adequately compacted in-place. As air void content increases (or density decreases) in a mixture, permeability would increase. High permeability/air void content would result in an increased potential for moisture damage in pavement, such as raveling and stripping. Zube (1962) performed studies to correlate air void content and permeability in dense-graded mixes and concluded that asphalt mixtures become permeable to water at air void content of approximately 8 percent. He also concluded that above this percentage, the permeability would rapidly increase since the air voids would become interconnected and allow water to easily penetrate into the pavement. Cooley et al. (1999), and Mallick and Daniel (2006) have shown that fine-graded mixtures are less permeable than coarse-graded mixtures. Also, when comparing two HMA mixtures with the same Nominal Maximum Aggregate Size (NMAS), the one with gradations that pass below the maximum density line (MDL) is prone to being more permeable than mixtures having gradations that pass above the MDL. They conclude that larger NMAS and gradations with higher coarse aggregate contents lead to a greater potential for higher levels of permeability. In both instances, less fine aggregate is available to fill the void space between the larger aggregate particles. This would result in larger individual air voids and thus a higher potential for interconnected air voids and permeability value. The permeameter Cooley et al. used in their study is now referred to as the NCAT Permeameter and is shown in Figure 4.

In addition to air void content, effective air void content also referred to as porosity has been gradually used as an indicator to predict the permeability of HMA. It is defined as the percentage of water permeable voids in the compacted HMA mixture. The CoreLok device, which uses the concept of vacuum sealing, can be used for the measurement of porosity. First, a HMA specimen is vacuum-sealed inside a bag and a sealed density of the specimen, and ρ_1 is determined. Then the sealed bag (with specimen) is opened under water and ρ_2, is determined. ρ_2 is called the apparent, or maximum density of the specimen. The density ρ_2 includes the volume caused by inaccessible air voids. Equation 1 is used for the calculation of the porosity (InstroTek Inc., 2011).

$$\text{Porosity} = \left(\frac{\rho_2 - \rho_1}{\rho_2} \right) \times 100 \tag{1}$$

Mohammad et al. (2005) and Kanitpong et al. (2001) both reported that the porosity as measured with the CoreLok vacuum equipment has a better correlation with the measured permeability than with the measured air void content approximation. However, in a another separate study conducted by Kanitpong et al. (2003), the idea that the use of porosity as a better predictor of permeability than the total air void content was not confirmed. Kanitpong et al. (2003) explained that the different conclusions in the two studies could be due to the variability in the degree of saturation of the samples in the two studies. Therefore, more research to prove that porosity can give an accurate estimate of HMA permeability would be needed in the future.

In recent years, several apparatuses have been developed to measure the permeability value of an HMA mixture and among which the NCAT Field Permeameter and the Karol-Warner in-lab permeameter are the most popular ones. Both of them are used as falling head devices to record the drop in water level in a standpipe over a given time interval. Plumber's putty is used as a sealant for NCAT permeameter field testing while the Karol-Warner permeameter uses air pressure exerted onto a rubber membrane to seal the flow paths along the sides of a test HMA mixture sample. The sealing failure problems for the NCAT Field Permeameter were identified by some researchers (Cooley and Brown, 2000; Cross and Bhusal, 2009). Because of the rough surface texture of the HMA, it is difficult to completely seal the bottom of the NCAT Permeameter with the surface of the pavement. Even if the NCAT Permeameter is tightly sealed with the pavement, a weight is recommended to place onto the base plate to resist the uplift of the NCAT Permeameter when water is introduced into the standpipe (see Figure 4).

Figure 4. NCAT field permeameter

In the literature, the permeability values measured by the Karol-Warner Permeameter are always reported to be smaller than that measured by the NCAT Permeameter. Three contributing reasons are identified and summarized as follows. The first one is the effectiveness of the sealing as mentioned above. Another one is the sawing effect of the specimen as noted by Maupin (2001). This is because the sawing process used to separate layers of the core samples can smear asphalt over the voids, close water passages, and reduce the measured in-lab permeability value. Finally, the Karol-Warner Permeameter does not allow for the horizontal flow in the HMA permeability measurement could be contributing for a lower measured permeability value. Mallick et al., (2003) also observed that coarser mixtures with thicker lifts are prone to have larger horizontal flow, whereas finer mixes with thinner lifts tend to have more vertical flow. Therefore, the Karol-Warner Permeameter may underestimate the permeability values for coarse and thick mixtures. Although there are some reported drawbacks to the NCAT and Karol-Warner permeameters, they are still considered as the most appropriate and promising devices for field and in-lab permeability test respectively, since they are readily available commercially and are simpler in the operations. Utilizing the devices, Williams et al. (2010) developed a test method and a criterion for ensuring optimal pavement density and permeability for the Missouri DOT. The permeability criterion is determined based upon the percent within limit (PWL) of pavement air voids in the quality assurance/quality control process as shown in Figure 5 and Figure 6. The upper specification criteria for using the NCAT Permeameter and Karol-Warner Permeameter are 1560 x 10^{-5} cm/s and 530 x10^{-5} cm/s, respectively and the lower criteria are 0 cm/s for both of them.

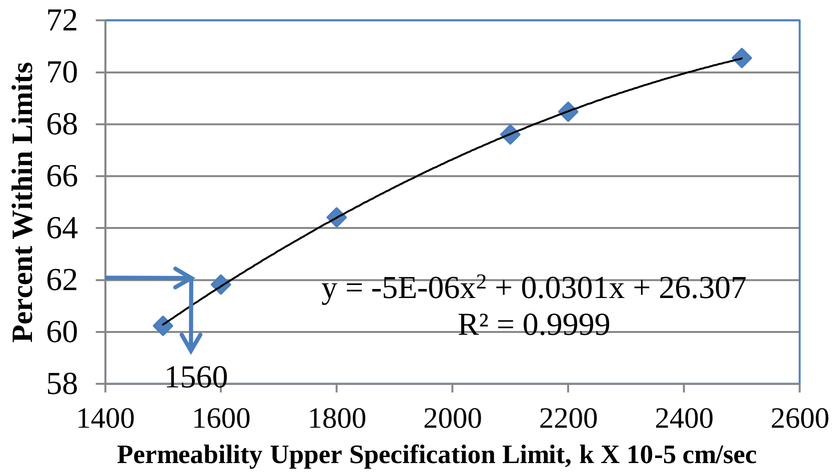


Figure 5. Influence of permeability upper specification limit for the NCAT Permeameter on PWL (Williams et al. 2011)

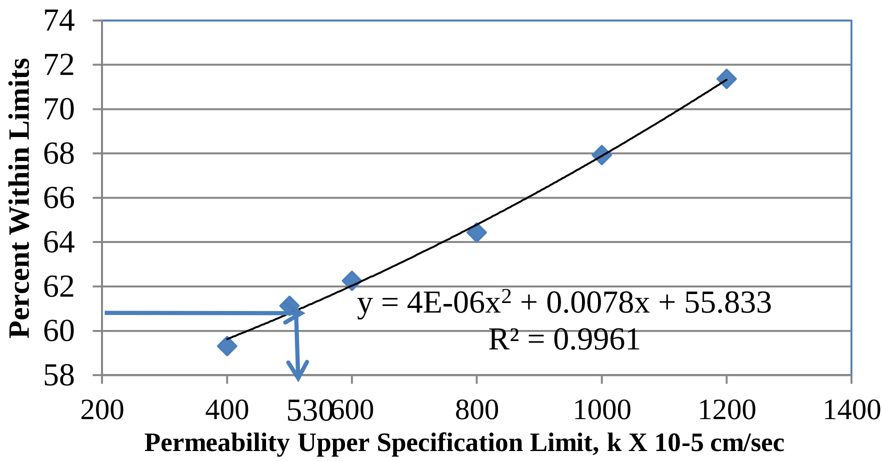


Figure 6. Influence of permeability upper specification limit for the Karol-Warner Permeameter on PWL (Williams et al. 2011)

2.3 Density Measurement of HMA

The bulk specific gravity (G_{mb}) is essentially the density of a compacted HMA sample. A major concern in the HMA industry is the proper measurement of the density of compacted HMA samples, since it is the basis for the volumetric calculations used during HMA mix design, quality control and quality acceptance processes. Therefore, correct and accurate density determination should also be a vital step in the quality acceptance/quality control testing for longitudinal joint construction in this study. There are basically two ways to determine the density of HMA pavement: a destructive core extraction method with subsequent lab testing and non-destructive method including using nuclear and non-nuclear gauges.

Several methods are generally used to determine the density of a HMA core sample. These include the saturated surface dry (SSD) method, paraffin-coated method and Corelok$^{@}$ system method. The SSD method is used for testing is AASHTO T-166 method or water displacement method and is the most commonly-used method to determine the bulk specific gravity of compacted HMA samples according to the AASHTO T-166 procedures for its quick, easy and insensitive operation. The following expression is used to compute the bulk specific gravity using the AASHTO T-166 method:

$$G_{mb}\,(SSD) = \frac{A}{B-C} \qquad (1)$$

where A = mass of the dry specimen in air,

B = mass of the saturated surface dry specimen in air, and

C = mass of the specimen in water.

However, vulnerability to water penetration into the sample and subsequent drainage prior to SSD mass determination can be a critical problem for this method. AASHTO T-166 (AASHTO, 2007) also states, “This method should not be used with samples that contain open or interconnecting voids and absorb more than 2% of water by volume. If the sample absorption exceeds this limit, then AASHTO T-275 (Paraffin-coated method) is recommended.” Unfortunately, the AASHTO T-275 method used for sealing of compacted asphalt samples can have poor repeatability, high sensitivity to operator involvement and training. Furthermore, there are currently no specifications for sealing 150 mm diameter samples. Consequently, few agencies use this method. Another method using the CoreLok device has been employed by many researchers and transportation agencies in recent years to replace the paraffin-coated method. AASHTO T-331 “Standard Test Method for Bulk Specific Gravity and Density of Compacted Bituminous Mixtures Using Automatic Vacuum Sealing Method” has been approved and outlines the G_{mb} determination procedure with the CoreLok device (AASHTO T-331, 2007). The following equation is used for the calculation of the bulk specific gravity of the sample by CoreLok$^{@}$.

$$G_{mb}\ \text{(CoreLok)} = \frac{A}{\left\{B - E - \left(\frac{B-A}{F_T}\right)\right\}} \tag{2}$$

where A= mass of dry sample in air, (g),

B = mass of dry, sealed sample, (g),

E = mass of sealed sample underwater, (g), and

F_T = apparent specific gravity of plastic sealing material.

Comparing with the CoreLok method with the paraffin-coated method, the process of sample preparation by CoreLok@ requires very little operator involvement and minimizes operator sensitivity. A lot of research work has been conducted to evaluate the common and different points between the SSD method and the CoreLok method (Buchanan, 2000, White and Buchanan, 2004) and it is generally believed that the CoreLok procedure can determine the G_{mb} more accurately than the conventional AASHTO T-166 method for coarse-graded mixes. More specifically, the CoreLok@ should be utilized for mixes passing below the restricted zone with water absorption of above 0.4. Figure 7 shows one of the research findings provided by White and Buchanan (2004). It can be seen from the figure that both fine and coarse-graded samples have excellent relationships between CoreLok and AASHTO T-166 method and the CoreLok procedure yields consistently lower G_{mb} values (high air voids). In addition, it is observed that coarse-graded mixes would have a slightly larger difference than fine-graded mixes, which agrees with other research results.

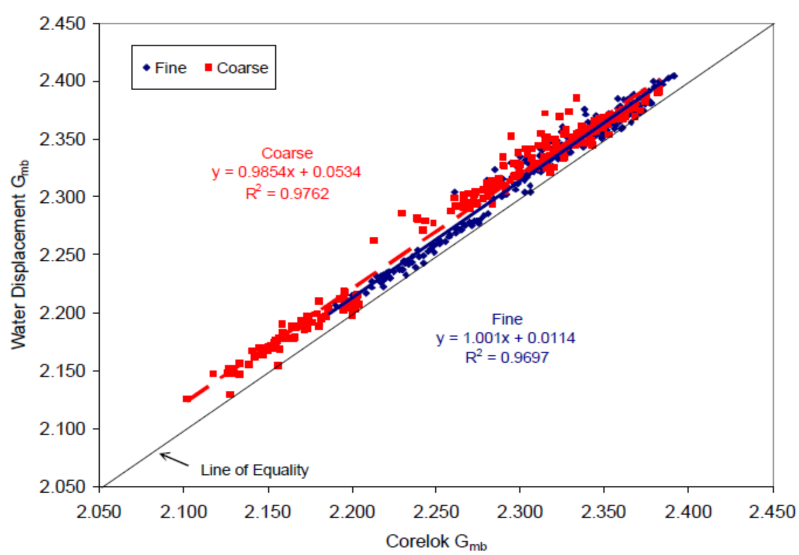


Figure 7. Comparison of the CoreLok method with the water displacement (SSD) method (White and Buchanan 2004)

The non-destructive methods for measuring in-place density of asphalt pavement involve the use of nuclear and non-nuclear density gauges. Kabassi et al., (2011) conducted a literature review covering many important research findings on the subject about the effectiveness of the field application of nuclear and non-nuclear gauges. Their general observation is that the difference in surface texture can cause large variations in nuclear gauge measurements. However, this appears to have no impact on the Pavement Quality Indicator (PQI) non-nuclear gauge. Williams and Hall (2008) evaluated the effects of gauge model, temperature, gauge orientation and the present of sand using the PaveTracker and PQI non-nuclear gauges. They found that gauge orientation, moisture, sand and debris can significantly affect the reading of the two types of gauges.

2.4 Segregation Measurement of HMA

Segregation is a significant asphalt pavement deficiency that can cause poor performance. Segregation can occur at a number of different steps in the asphalt mixture production and placement process. These steps include the mixture design (e.g. gradation selection), aggregate stockpiling, plant production, asphalt mixture storage, truck loading, transport, and laydown (Brock, 1986). Brock (1986) stated that the most important factor related to segregation is properly designing the mix. He also pointed out that a gap graded mixture with low asphalt content cannot be produced without segregation. Kennedy et al. (1986) indicated that asphalt content and gradation are the two mix design factors which significantly affect the tendency for segregation. Mixtures with a large maximum aggregate size, coarse grading or gap grading have a greater tendency to segregate than do finer or well graded mixes. To construct a sound longitudinal joint, mitigation of segregation is important. As stated by AASHTO (1997), the longitudinal joint area has a higher probability of being segregated. This commonly occurs from the augers not being run at sufficient speeds on the paver, allowing the coarse aggregates to roll to the outside of the mat. In addition, in order to avoid joint segregation during the paving process the auger and tunnel should be extended within 12 to 18 inches of the end gate so the material can be carried, and not pushed out to the joint. Traditionally, several testing methods have been generally used to detect and measure the segregation of asphalt mixtures and they can also be mainly divided into two categories: non-destructive and destructive methods.

Permeability and nuclear/non-nuclear density tests have been used experimentally to confirm segregation existence. Laboratory-compacted asphalt mixture slabs prepared with different levels and locations of segregation were tested using the nuclear moisture/density gauge and an air permeameter by Williams, et al. (1996). They found that the nuclear moisture/density gauge is capable of accurately measuring both asphalt content and density in a dry pavement condition. They also found that the air permeability tests identify greater sensitivity to surface segregation but not blind segregation. In addition, the air permeameter is only successful in detecting coarse segregation but not fine segregation. This is mainly because the permeability test depends more on the interconnected nature of void volume rather than simply the percent of voids. Fine, dense-graded mixtures would have sufficiently low permeability that even when moderately segregated, there is little to no statistical difference in permeability measurements. Larsen and Henault (2006) used density profiles obtained from a PaveTracker non-nuclear density gauge to quantify the level of segregation in Connecticut. However, they found that the spatial variation in density alone from the non-nuclear density gauge cannot identify the existence of segregation.

Core extraction and testing include measuring changes in asphalt content gradation and density are the most commonly used destructive methods. A decrease in asphalt content with an increase in coarseness is the single constant factor reported in all of the research on segregation measurement (Cross and Brown, 1993; Williams, et al., 1996). In addition, changes in coarser aggregate gradation fractions were also commonly used to measure segregation. Cross and Brown (1993) concluded that a variation in the percent passing the No. 4 sieve greater than 8 to 10 percent can easily lead to segregation and raveling, but no specific criteria was provided in their work. They also identified that when a mixture becomes coarser because of segregation as measured by a change in percent passing the No.4 sieve, the measured asphalt content decreases. Several other studies have related the segregation conditions with the mixture mechanical performance characteristics. Cross et al. (1997) found an increase of 5 percent in coarseness, measured as a change in the percent retained on the No.4 sieve, corresponded to about an 11 percent decrease in tensile strength. These measurements were also strongly correlated with air voids. This suggests that any correlation between tensile strength measurements and pavement performance should include both a measure of the degree of segregation and air voids. Williams et al. (1996) compared the mix performance with five different levels of segregation using a laboratory wheel tracking device similar to the Hamburg Steel Wheel Tester, the PURwheel tracking device (PTD). Rutting/stripping test using the PTD indicates that segregated mix demonstrates significant decrease in performance. The very fine and very coarse segregated mixes exhibit approximately a 70 percent decrease in the number of wheel passes to achieve the same level of rutting comparing with the control mix (no segregation).

CHAPTER 3 TEST PLAN AND PROCEDURE

3.1 Site Selection and Description

Five projects are selected for sampling and evaluation in this study. Each project represents a typical longitudinal joint construction technique as shown in Table 1. The route numbers for the five projects are designated as the project names for simplicity in this study. The five different types of longitudinal joint construction methods are the traditional butt joint placed with HMA and WMA, butt joint with infrared heat treatment, joint edge restraint by milling, and modified butt joint with the first pass of roller offset 6 in. away from the joint (pinching).

Table 1. Project list and longitudinal joint type

	Longitudinal Joint Type				
	Butt Joint (HMA)	Butt Joint (WMA)	Butt joint with infrared heater	Butt joint with pinching	Edge restraint by milling
Project Name	US 6	IA 148	IA 13	US 61	I-35

The following are brief site reports for each project when the longitudinal joint construction was evaluated.

Site 1: Project US 6

The project is located at US 6 highway from the east junction of US 151 to the west city line Tiffin in Iowa County. The lifted thickness for the surface course was 1.5 in. The mix type studied is a 3M Surface 1/2 L-4 (HMA). There had been no recent rainfall report before the time of field construction. The air temperature was between 80 to 94^{o}F and the mat temperature was between 310 to 330^{o}F from 10 am to 5 pm.

Site 2: Project IA 148

The project is located on IA 148 highway from IA 92 N. to west of junction of IA 83 in the city of Anita in Cass County. The lifted thickness for the surface course was 1.5 in. The mix type studied is a 1M Surface 1/2 Type A (WMA). A water injection method was used to produce WMA with 1.8% water filling rate. There had been no recent rainfall report before the time of field construction. The air temperature was between 80 to 92^{o}F and the mat temperature was between 220 to 240^{o}F from 10 am to 5 pm.

Site 3: Project IA 13

The project is located on IA 13 highway from 3/4 mi. north of County Home Road to 1 mile north Central City in Linn County. The lifted thickness for the surface course was 1.5 in. The mix type is a 3M Surface 1/2 L-4 (HMA). The weather was cloudy during construction. The air temperature was between 80 to 86^{o}F and the mat temperature was between 280 to 300^{o}F from 10 am to 5 pm.

Site 4: Project I-35

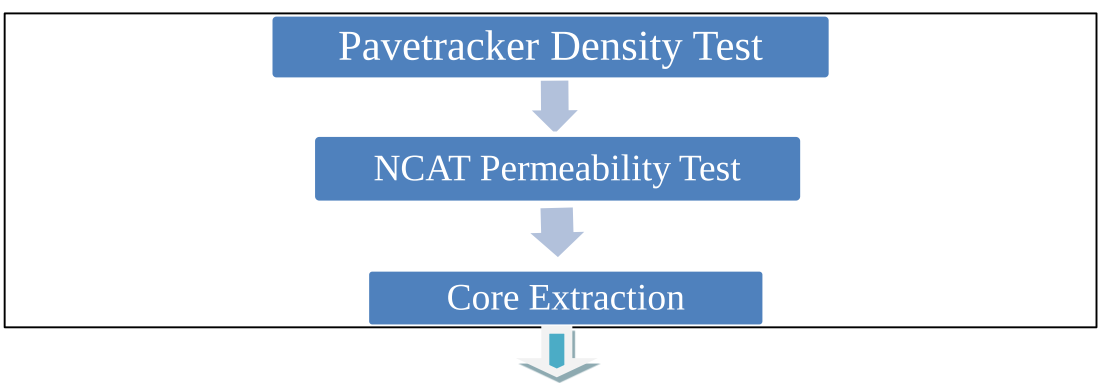
The project is located at I-35 highway from just north of US 34 to the Warren County line in Clarke County. The lifted thickness for the surface course was 2 in. The mix type is a 30M Surface 1/2 L-2 (HMA). There had been reported rain during truck transportation of the mix. The air temperature was between 62 to 72°F and the mat temperature was between 260 to 270°F from 10 am to 5 pm.

Site 5: Project US 61

The project is located at US 61 highway from just north of County Road X-38 to just north of 180th Street in Lee County. The lifted thickness for the surface course was 2 in. The mix type is a 3M Surface 1/2 L-4 (HMA). No rain was reported during construction. The air temperature was between 75 to 87°F and the mat temperature was between 260 to 270°F from 10 am to 5 pm.

3.2 Test Plan

The test plan contains two parts: field testing and laboratory testing as shown in Figure 8. Field testing and sampling consisted of obtaining pavement density by the PaveTracker non-nuclear gauge, field permeability measurements using the NCAT Permeameter and extracting pavement cores from 6 random locations for each project. For each test location, field tests were done on the pavement longitudinal joint and the center of the hot lane (about 2 ft right of longitudinal joint). Therefore, this results in testing a total number of 12 field locations and corresponding 12 core extractions from each project for a paving day (about 3 to 5 miles/day). Before the field testing, dry ice was used to cool the test locations. Field density measurements using a PaveTracker non-nuclear gauge can be greatly affected by water; therefore, they are performed first at each location, please refer to Figure 9 (a). Once the PaveTracker density measurements were completed, NCAT permeability tests were made at the same location. After the pavement surface was cooled with dry ice, core samples were taken at the same places where the field density and permeability tests were performed. The core sample sizes are from 4 to 6 inches in diameter and the thickness equal to the lift thickness of the surface course. Finally, these cores were transported to the Bituminous Materials Laboratory at Iowa State University for further testing.



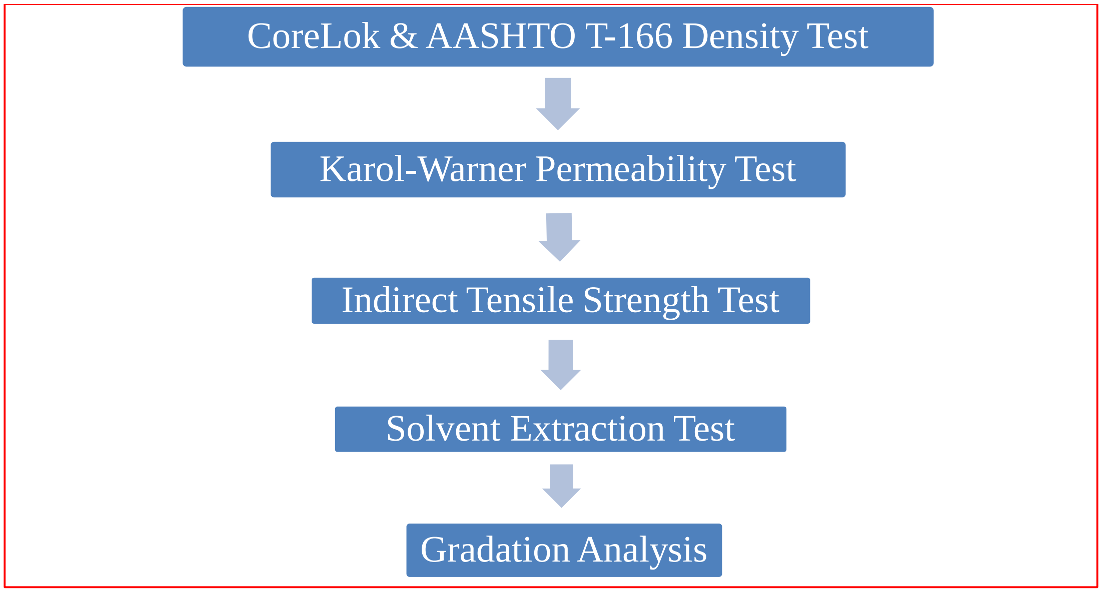


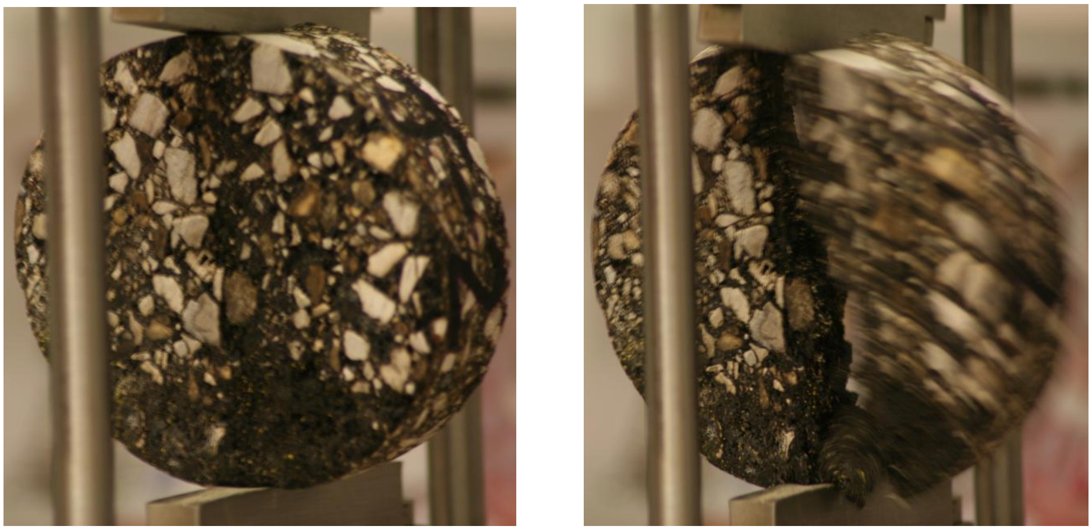
Figure 8. Test plan sequence and procedures

(a) PaveTracker

(b) Coring

(c)CoreLok

(d) Karol-Warner Permeameter



(e) Indirect Tensile Strength before and after testing

Figure 9. Test methods

The field cores taken from the field were trimmed to remove foreign materials and tack coats on the bottom of the cores. The following tests were performed on each field core sample: 1) voids analysis, 2) in-lab permeability, 3) indirect tensile strength, 4) determination of asphalt content and gradation. The void analysis includes the bulk specific gravity test in accordance with AASHTO-T166 and the AASHTO T-331 method by the CoreLok system as shown in Figure 9 (c). The effective air void content/porosity was also obtained by the CoreLok system. After the air void testing, a Karol-Warner permeameter was used for the in-lab permeability test and the test device is depicted in Figure 9 (d). Further, field cores were then subjected to the indirect tensile strength (IDT) test following ASTM D-6931 standard shown in Figure 9 (e). The field cores were monotonically loaded to failure along the vertical diametric axis in the IDT test at a constant rate of 50 mm/min. The joint core samples were loaded along the direction of the longitudinal joint so that failure could occur along the joint and the IDT strength at the joint can be obtained. The broken core samples were collected, retained and used to determine the asphalt content by the ignition method according to AASHTO T-308. Finally, a washed sieve analysis was performed on the materials remaining from the ignition test in accordance with AASHTO T-30.

CHAPTER 4 TEST RESULTS AND STATISTICAL ANALYSIS

4.1 Introduction

This chapter is divided into three sections. The first section summarizes and analyses the results using the field and laboratory density test methods and determining the corresponding air voids. Evaluations and summaries of the field and laboratory permeability tests and IDT test results are provided in the second section. In the third section, evaluation on the effects of segregation on mix properties and performance is conducted based upon the information provided through the testing of the field core samples.

4.2 Summary and Evaluation of Density and Air Void Determination Methods

The three methods for determining or estimating the density of the pavement or bulk specific gravity (G_{mb}) of the core samples and the corresponding air voids are the PaveTracker Density, and the G_{mb} via the CoreLok (AASHTO T331) and the SSD method (AASHTO T166).

Table 2 through Table 6 summarize the results for each project using the three different methods. Comparisons of the mean air void values using AASHTO T-166 and CoreLok methods for all of the five projects are shown in Figure 10 and Figure 11. As can be seen, both methods demonstrate the ability to detect the differences in density on the longitudinal joint and 2 ft right of the joint. By visual inspection, Project I-35 with the edge restraint by milling technique forms the joint with the lowest air voids/highest density while the projects constructed by the traditional butt joint for both HMA and WMA give the highest air voids and lowest density. Figure 12 further compares the air voids of sample cores obtained by the CoreLok and AASHTO T-166 methods for all of the projects using scatter plots. Samples on both pavement mat and joint show good relationships between CoreLok and AASHTO T-166 methods (R^2 = 0.85 and 0.93, respectively). In addition, a line of equity is drawn and all of the points are slightly below the line of equality which means that the CoreLok procedure yields consistently lower G_{mb} values (high air voids), which agrees with the aforementioned research findings discussed in the literature review. Finally, a scatter plot between the PaveTracker non-nuclear gauge and the air voids determined by the CoreLok method for all projects is presented in Figure 13. Clearly, the data shows a substantial scatter in the plot and results in a very poor correlation (R^2 = 0.29 and 0.38, for the pavement mat and joint, respectively). A line of equality is drawn and data far above the 45^0 line represents that the PaveTracker nonnuclear gauge tends to overestimate the air void results. As discussed in the literature review, pavement temperature and moisture can both influence the reading of this type of density gauge. Further, the research team also finds that the smoothness of the pavement can also greatly affect the gauge measurement. Longitudinal joints can be rough due to the segregation and improper compaction. The rough and unsmooth texture of the joint surface can make the field density gauge placed on it without fully touching the pavement and this would easily result in an inaccurate measurement (see Figure 2 on the left).

In order to evaluate whether these methods used above have significant differences or not, the all pairs Tukey-Kramer HSD method is performed for multiple comparisons. The porosity values obtained by the CoreLok are also included in the comparisons. The advantage of the porosity measurement is that the method relies on the apparent maximum specific gravity of the test sample instead of the theoretical maximum specific gravity values of loose mix that does not always represent the samples with fine and coarse segregation. These plots show that the porosity, AASHTO T-166, CoreLok methods and PaveTracker density gauge provide similar results on the joint and mat, respectively for only IA 13 and US 61 projects. For the other projects, the PaveTracker density gauge gives significantly higher values. It is also found that the porosity values are slightly lower than the CoreLok air voids for Project I-35 and are very close to the CoreLok for the other four projects.

Table 2. Summary of air void results for the US 6 project

	CoreLok		AASHTO		PaveTracker	
Test Location	2 ft right of Joint	On Joint	2 ft right of Joint	On Joint	2 ft right of Joint	On Joint
1	5.4	15.0	5.6	13.0	9.1	20.8
2	7.2	13.6	6.3	11.2	9.5	19.8
3	7.4	8.4	5.8	7.4	11.4	15.0
4	5.4	13.8	5.5	10.8	9.3	14.5
5	6.3	9.8	6.5	8.6	11.9	14.7
6	6.2	11.4	5.9	10.3	9.8	13.6
Mean	6.3	12.0	5.9	10.2	10.2	16.4
Std. Dev.	0.9	2.6	0.4	2.0	1.2	3.1

Table 3. Summary of air voids test results for the IA 148 project

	CoreLok		AASHTO		PaveTracker	
Test Location	2 ft right of Joint	On Joint	2 ft right of Joint	On Joint	2 ft right of Joint	On Joint
1	8.9	12.7	8.5	11.4	10	12.4
2	6.8	12.0	6.6	10.5	7.5	13.0
3	6.5	11.3	6.2	10.1	7.1	12.9
4	9.0	12.3	9.0	10.2	9.6	15.5
5	10.6	12.3	8.9	9.9	10.4	13.0
6	9.9	11.9	8.6	10.5	10.3	13.5
Mean	8.6	12.1	8.0	10.4	9.1	13.4
Std. Dev.	1.6	0.5	1.2	0.5	1.5	1.1

Table 4. Summary of air voids test results for the I-35 project

	CoreLok		AASHTO		PaveTracker	
Test Location	2 ft right of Joint	On Joint	2 ft right of Joint	On Joint	2 ft right of Joint	On Joint
1	8.8	8.4	7.3	7.8	13.1	12.3
2	7.3	7.4	6.0	6.9	10.5	12.0
3	7.7	7.4	6.6	6.1	10.9	14.2
4	8.7	8.1	6.8	6.6	12.5	11.3
5	8.5	9.2	7.5	7.6	14.6	12.0
6	9.1	7.7	7.9	6.8	10.4	10.9
Mean	8.4	8.0	7.0	7.0	12.0	12.1
Std. Dev.	0.7	0.7	0.7	0.6	1.7	1.1

Table 5. Summary of air voids test results for the IA 13 project

	CoreLok		AASHTO		PaveTracker	
Test Location	2 ft right of Joint	On Joint	2 ft right of Joint	On Joint	2 ft right of Joint	On Joint
1	6.0	10.8	5.4	10.0	5.7	9.6
2	5.5	11.8	5.1	10.5	5.2	12.7
3	9.0	10.3	8.2	9.9	11.4	9.8
4	9.3	9.1	7.8	8.7	7.5	8.2
5	8.6	8.9	8.5	8.2	12.6	11.3
6	8.5	9.0	7.7	7.8	10.2	13.2
Mean	7.8	10.0	7.1	9.2	8.7	10.8
Std. Dev.	1.6	1.2	1.5	1.1	3.1	1.9

Table 6. Summary of air voids test results for the US 61 project

	CoreLok		AASHTO		PaveTracker	
Test Location	2 ft right of Joint	On Joint	2 ft right of Joint	On Joint	2 ft right of Joint	On Joint
1	6.0	13.1	5.8	10.9	2.0	4.6
2	7.3	12.8	6.8	11.3	3.4	21.7
3	5.5	8.6	5.6	8.2	4.9	4.8
4	5.7	10.6	5.7	9.4	8.0	5.1
5	5.9	11.2	5.6	10.1	2.8	10.2
6	8.0	12.3	7.4	11.2	2.7	10.2
Mean	6.1	11.4	6.2	10.2	4.0	9.5
Std. Dev.	1.0	1.7	0.8	1.2	2.2	6.6

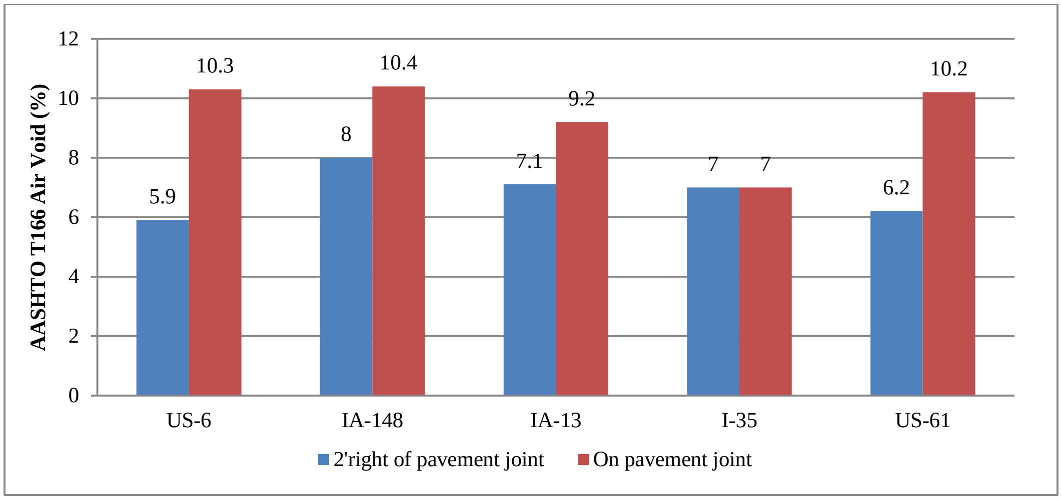


Figure 10. Comparison of mean air voids values for all projects using the AASHTO T-166 method

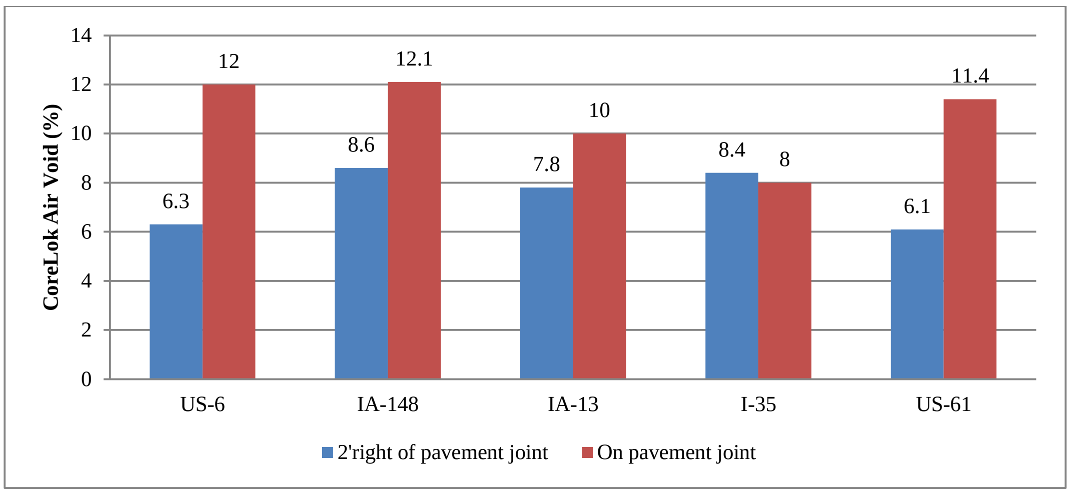


Figure 11. Comparison of mean air voids values for all projects using the CoreLok method

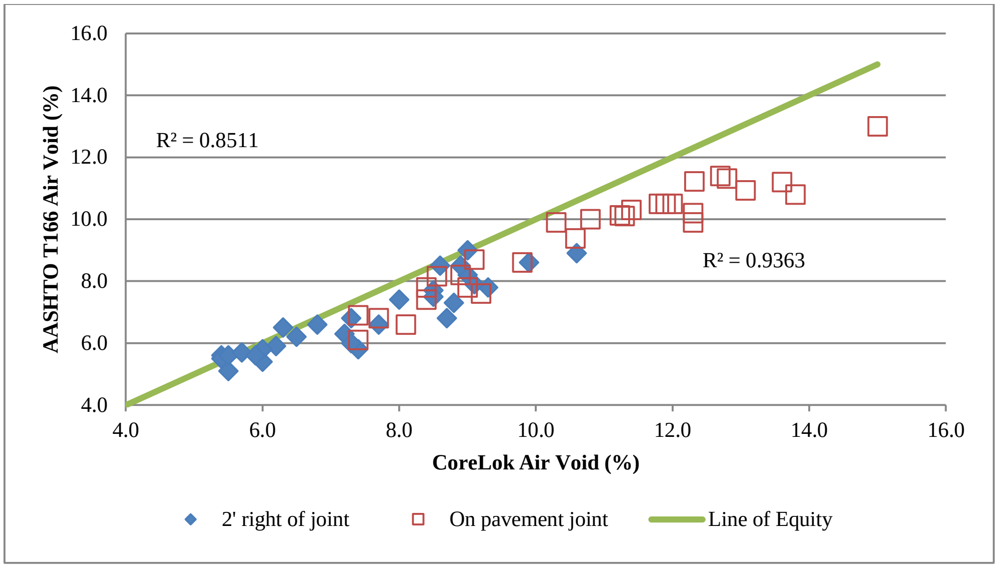


Figure 12. Comparison of AASHTO T-166 air voids and CoreLok air voids for all projects

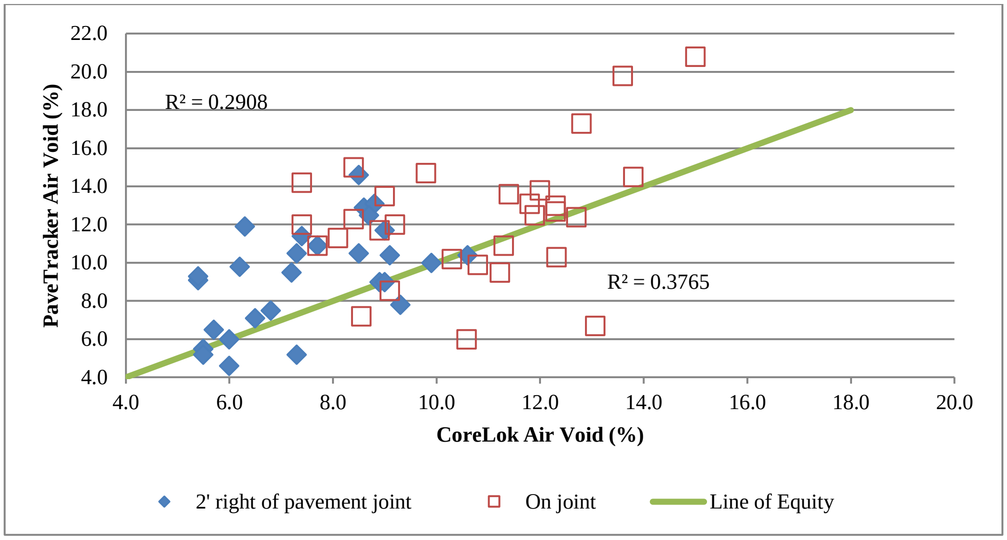


Figure 13. Comparison of PaveTracker air voids and CoreLok air voids for all projects

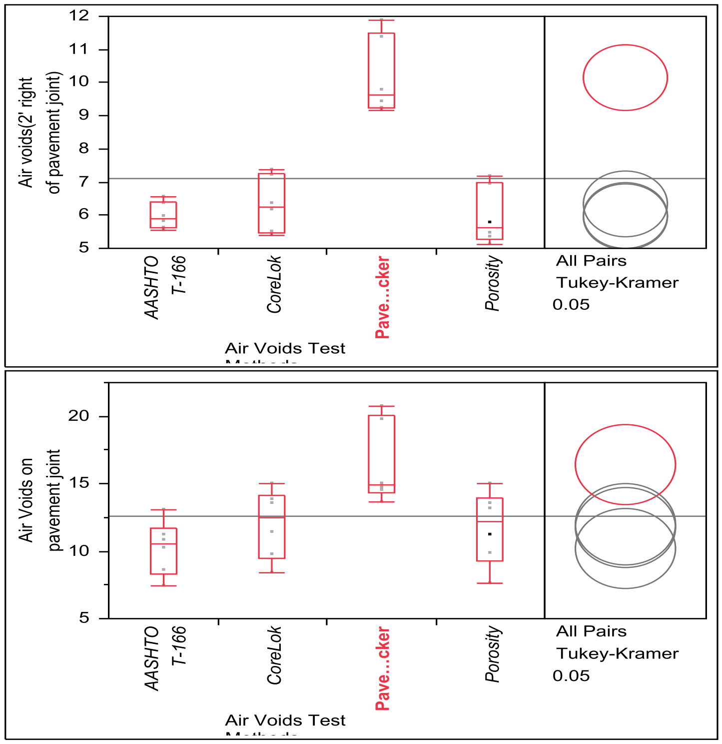


Figure 14. Comparison of air voids test methods by the HSD test for the US 6 project (methods with the same color are not significantly different)

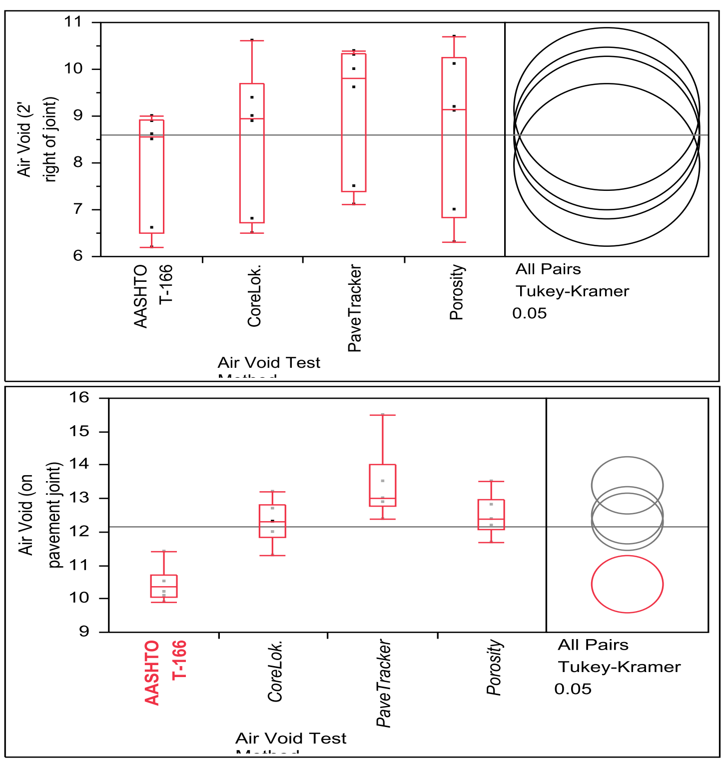


Figure 15. Comparison of air voids test methods by the HSD test for the IA 148 project

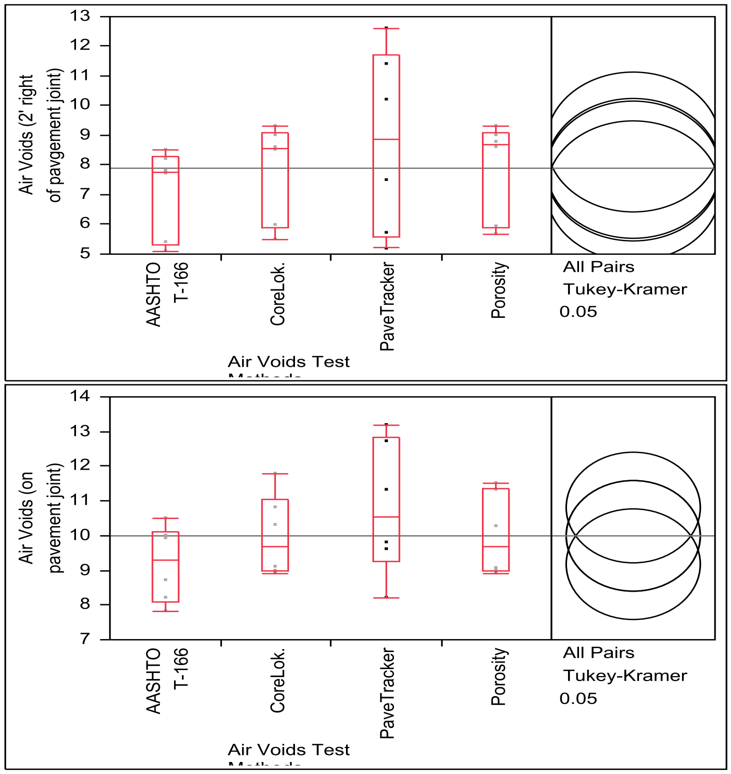


Figure 16. Comparison of air voids test methods by the HSD test for the IA 13 project

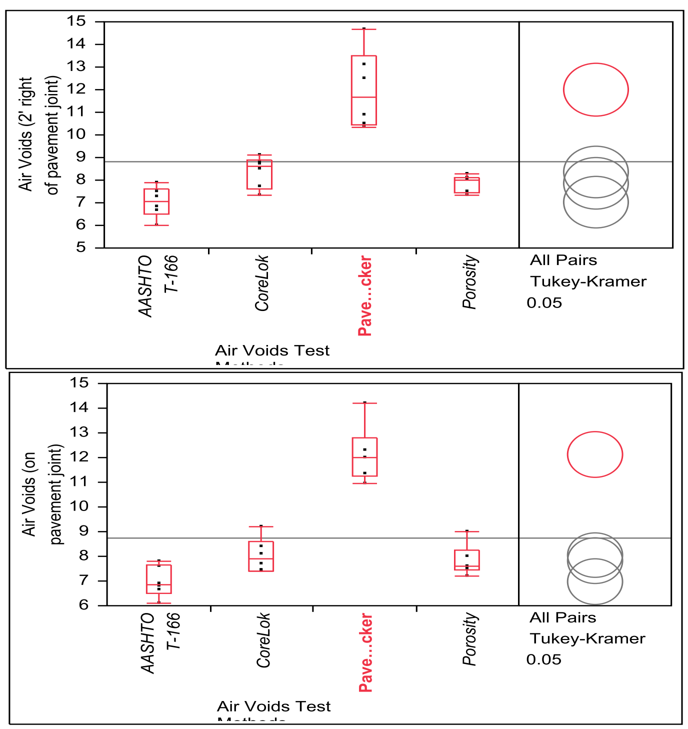


Figure 17. Comparison of air voids test methods by the HSD test for the I-35 project

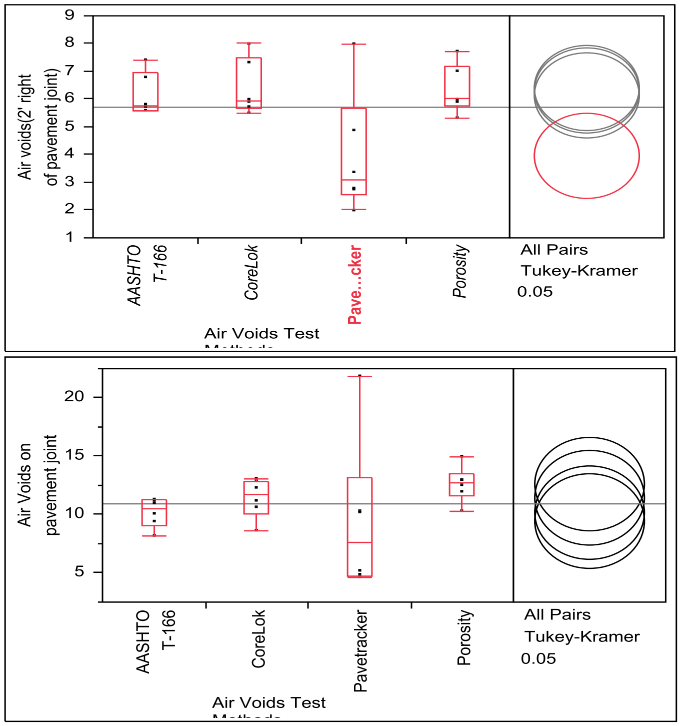


Figure 18. Comparison of air voids test methods by the HSD test for the US 61 project

4.3 Summary and Evaluation of Permeability Test and IDT Test Results

Results of the NCAT and Karol-Warner (K-W) permeability testing for each project are shown in Table 7 through Table 11. Comparisons of the mean permeability values for the five projects are shown in Figure 19 and Figure 20. From Figure 19, it is clearly seen that the IA 13 and I-35 projects using joint heater treatment and edge restraint by milling techniques give significantly lower permeability values than the US 6 and IA 148 projects that use the traditional butt joint. Applying the modified butt joint as shown on the project US 61 also produces relatively improved permeability results. Comparing Figure 19 and Figure 20, the NCAT Permeameter provides either higher or lower values when compared with the K-W Permeameter for most of the projects. This would be expected since the NCAT Permeameter allows for both vertical and horizontal flow while the K-W permeameter is only limited to vertical flow. Another reason contributing to the larger permeability values could be due to the leakage of NCAT Permeameter. The NCAT Permeameter uses plumbers putty to seal the device to the pavement. For the pavement at low air voids, water can hardly penetrate into the road and it would leak out of the permeameter from the surface sealed area. Thus the test method is operational dependent. Further, the K-W and NCAT permeability values are correlated to the CoreLok air voids as shown in Figure 21 and Figure 22, respectively. The relationship between CoreLok air voids and K-W permeability has a much better coefficient of determination (R^2) than that of CoreLok air voids and NCAT permeability. This tends to indicate that the NCAT permeameter is less reliable than the in-lab K-W permeameter although it is easy to use. Determinations of critical in-place air void and permeability values are presented in Figure 23 through Figure 26. Instead of using the average PWL for all projects tested to determine the permeability criterion, selection of pavement density criteria at which mixes become permeable can be evaluated by using the regression equation/relationship. This is because PWL is not sensitive to the air void variation when the density of longitudinal joints is very low. The critical air voids is considered to be the point at which the two lines tangent to the regression line intersect. At the intersecting point of these two lines, a bisecting line was then drawn from the regression line. The point at which the bisecting line hits the regression line was defined as the critical point for air voids and permeability. Although the method gives different critical air voids for the CoreLok (AASHTO T331) and AASHTO T166 as seen in the figures, it illustrates close critical K-W permeability values, which is around 1.9e-04 cm/s and 1.5e-03 cm/s on the pavement mat and joint, respectively. As discussed previously, the NCAT permeability values show very large scatter after correlated with air void values, which makes critical NCAT permeability values difficult to be identified by the method using the regression line. As shown in the figures, the minimum required joint density should be 90.0% of theoretical maximum density based on the AASHTO T166 method and 88.3% based on CoreLok (AASHTO T331) method. In the same approach, the graphical representation show that the critical air voids on pavement mat is around 92.7% and 91.7 % of theoretical maximum density according to the AASHTO T166 and the CoreLok method, respectively. In addition, as can be seen from the four figures the CoreLok and AASHTO T166 methods have very close R^2 values for the density testing on pavement mat. However, the AASHTO T166 method becomes much less sensitive for the density testing on longitudinal joint and provides more scattered results, where both fine segregation and coarse segregation are also detected on the longitudinal joint in the following section. A summary of the air void and permeability criteria on pavement joint and 2 ft right of the joint is listed in Table 12.

Table 7. Summary of permeability test results for the US 6 project

	K-W permeability value (cm/s)		NCAT permeability value (cm/s)	
Test Location	2 ft right of Joint	On Joint	2 ft right of Joint	On Joint
1	1.00E-06	6.49E-03	1.20E-04	5.55E-03
2	2.06E-04	5.95E-03	2.23E-04	6.43E-03
3	1.24E-04	1.24E-04	1.00E-06	4.21E-03
4	1.00E-06	6.78E-03	7.08E-05	3.39E-03
5	1.00E-06	2.90E-04	3.48E-04	3.54E-03
6	1.10E-04	5.83E-04	2.77E-04	2.14E-03
Mean	7.38E-05	3.37E-03	1.73E-04	4.21E-03
Std. Dev.	8.63E-05	3.34E-03	1.32E-04	1.55E-03

Table 8. Summary of permeability test results for the IA 148 project

	K-W permeability value (cm/s)		NCAT permeability value (cm/s)	
Test Location	2 ft right of Joint	On Joint	2 ft right of Joint	On Joint
1	2.96E-04	2.95E-03	7.48E-04	2.50E-03
2	2.07E-04	2.37E-03	3.07E-04	2.03E-03
3	1.76E-04	3.90E-03	9.71E-04	2.88E-03
4	8.93E-04	2.73E-03	1.39E-03	3.15E-03
5	1.13E-03	3.45E-03	1.60E-03	2.18E-03
6	6.60E-04	3.92E-03	5.93E-04	1.57E-03
Mean	5.60E-04	3.22E-03	9.35E-04	2.39E-03
Std. Dev.	3.97E-04	6.39E-04	4.89E-04	5.79E-04

Table 9. Summary of permeability test results for the IA 13 project

	K-W permeability value (cm/s)		NCAT permeability value (cm/s)	
Test Location	2 ft right of Joint	On Joint	2 ft right of Joint	On Joint
1	4.90E-05	5.57E-04	7.80E-04	1.41E-03
2	4.80E-05	7.54E-04	4.12E-04	6.34E-04
3	4.60E-04	1.91E-04	3.04E-04	7.57E-04
4	3.38E-04	3.19E-05	2.09E-04	2.42E-04
5	1.93E-04	4.79E-05	2.14E-04	3.99E-04
6	1.38E-04	4.01E-04	6.61E-04	4.77E-04
Mean	2.04E-04	3.30E-04	4.30E-04	6.53E-04
Std. Dev.	1.65E-04	2.91E-04	2.40E-04	4.12E-04

Table 10. Summary of permeability test results for the I-35 project

	K-W permeability value (cm/s)		NCAT permeability value (cm/s)	
Test Location	2 ft right of Joint	On Joint	2 ft right of Joint	On Joint
1	2.31E-04	6.40E-04	3.78E-04	3.34E-04
2	1.00E-06	3.29E-04	3.37E-05	3.17E-04
3	5.56E-05	1.20E-04	5.94E-04	2.62E-04
4	8.06E-05	1.94E-04	3.05E-04	2.68E-04
5	2.45E-04	3.47E-04	7.17E-04	6.83E-04
6	3.54E-04	1.92E-04	3.38E-04	3.17E-04
Mean	1.61E-04	3.04E-04	3.94E-04	3.63E-04
Std. Dev.	1.36E-04	1.87E-04	2.39E-04	1.59E-04

Table 11. Summary of permeability test results for the US 61 project

	K-W permeability value (cm/s)		NCAT permeability value (cm/s)	
Test Location	2 ft right of Joint	On Joint	2 ft right of Joint	On Joint
1	7.06E-05	3.50E-03	6.11E-04	5.22E-03
2	1.10E-04	1.19E-03	1.00E-06	2.55E-03
3	1.00E-06	5.04E-04	1.00E-06	8.12E-04
4	1.00E-06	8.40E-04	2.02E-04	1.70E-03
5	7.30E-06	1.47E-03	1.06E-04	1.74E-03
6	2.01E-04	2.31E-03	2.93E-04	1.57E-03
Mean	6.52E-05	1.64E-03	2.02E-04	2.27E-03
Std. Dev.	8.01E-05	1.10E-03	2.30E-04	1.55E-03

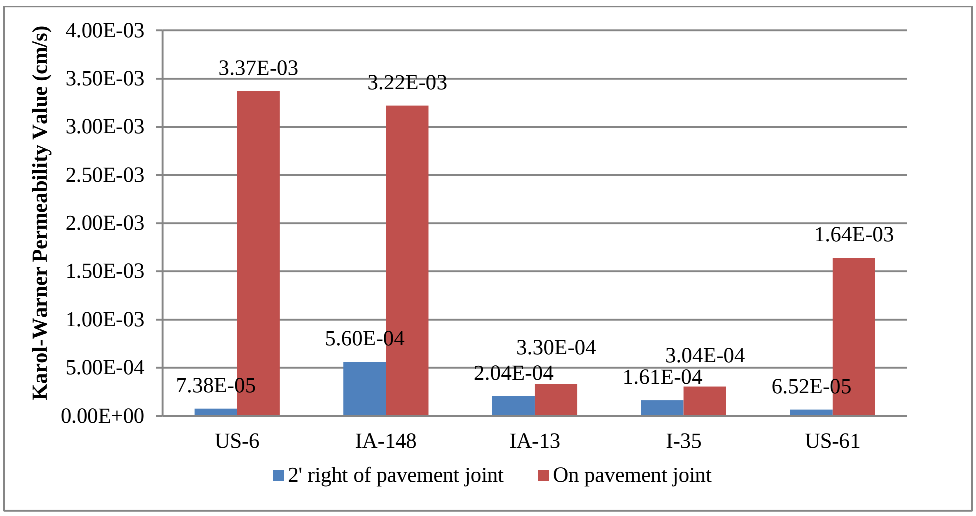


Figure 19. Comparison of mean K-W permeability values for all projects

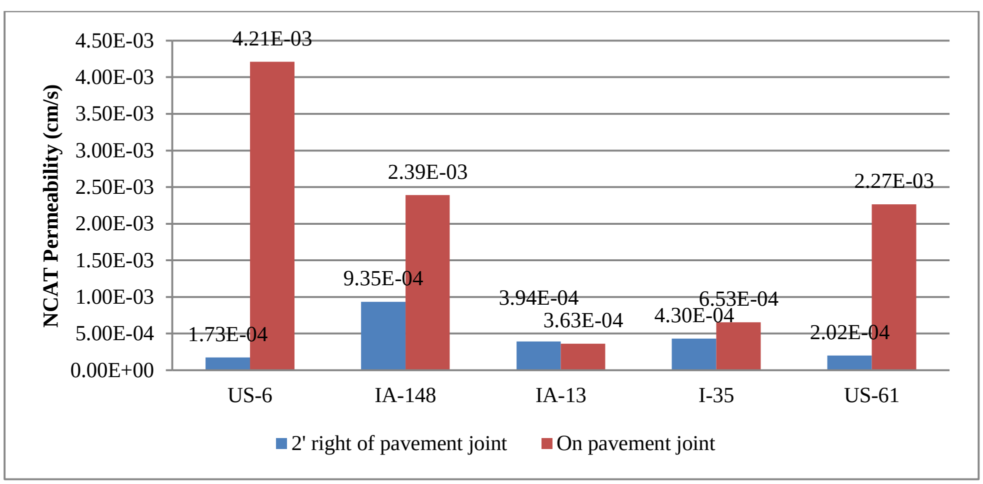


Figure 20. Comparison of mean NCAT permeability values for all projects

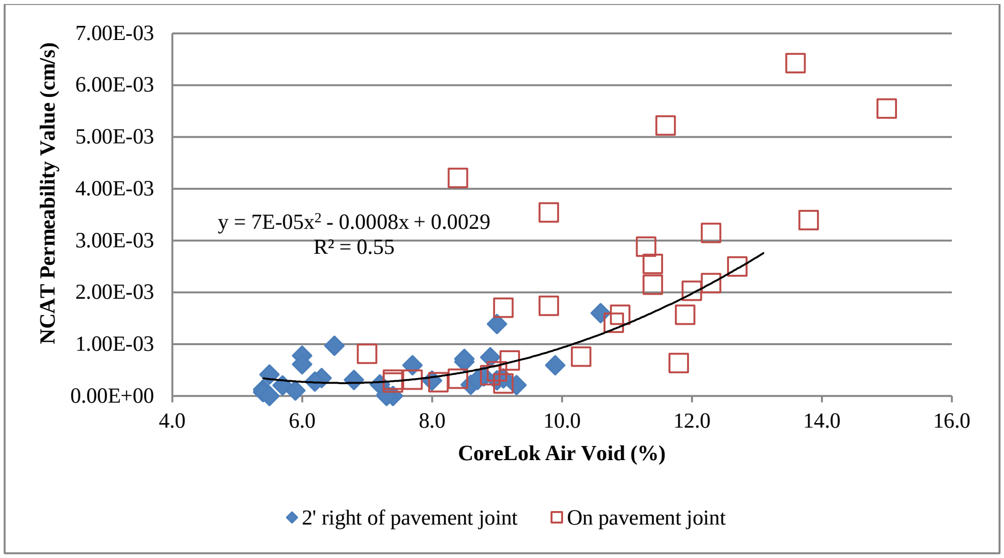


Figure 21. Comparison of CoreLok air voids and NCAT permeability values

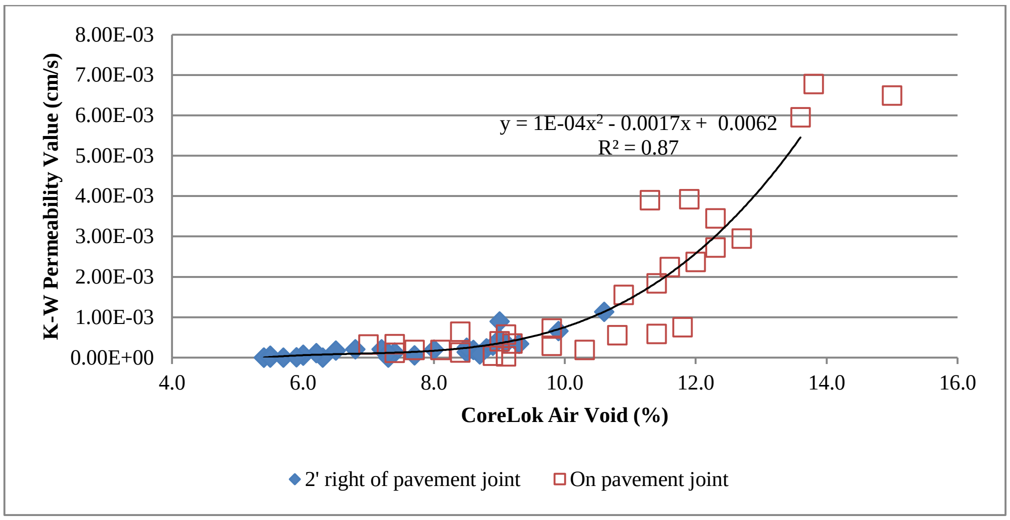


Figure 22. Comparison of CoreLok air voids and K-W permeability values

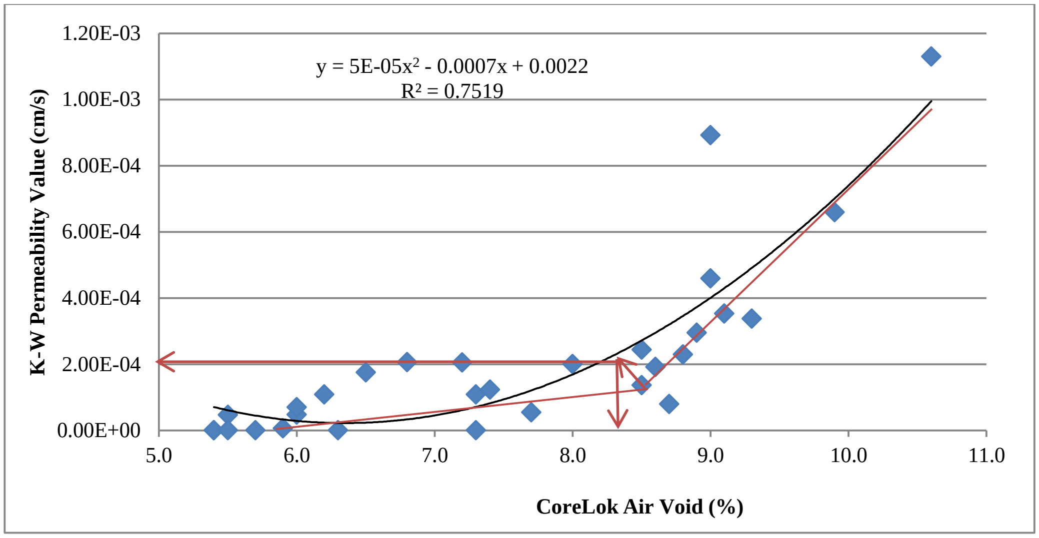


Figure 23. Selection of critical permeability and CoreLok air voids values (2 ft right of the pavement joint)

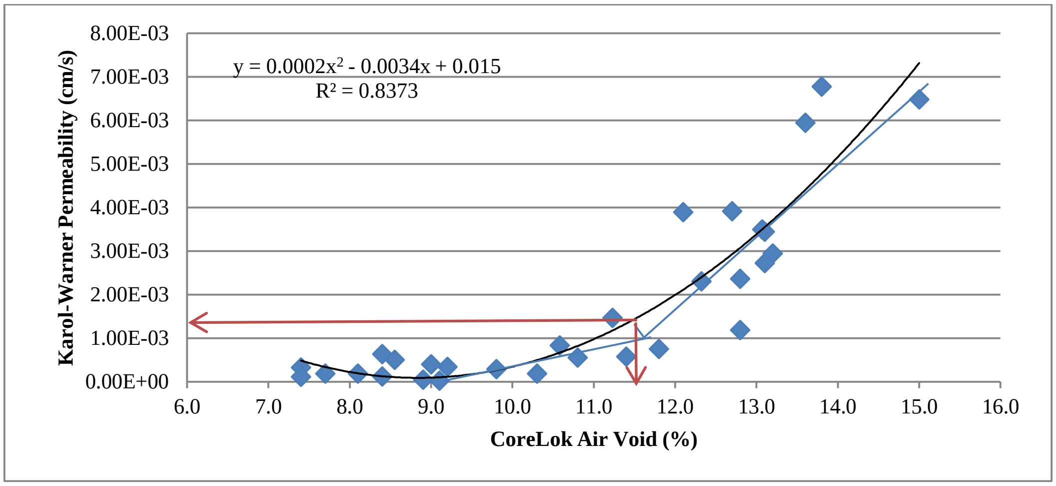


Figure 24. Selection of critical permeability and CoreLok air voids values (on the pavement joint)

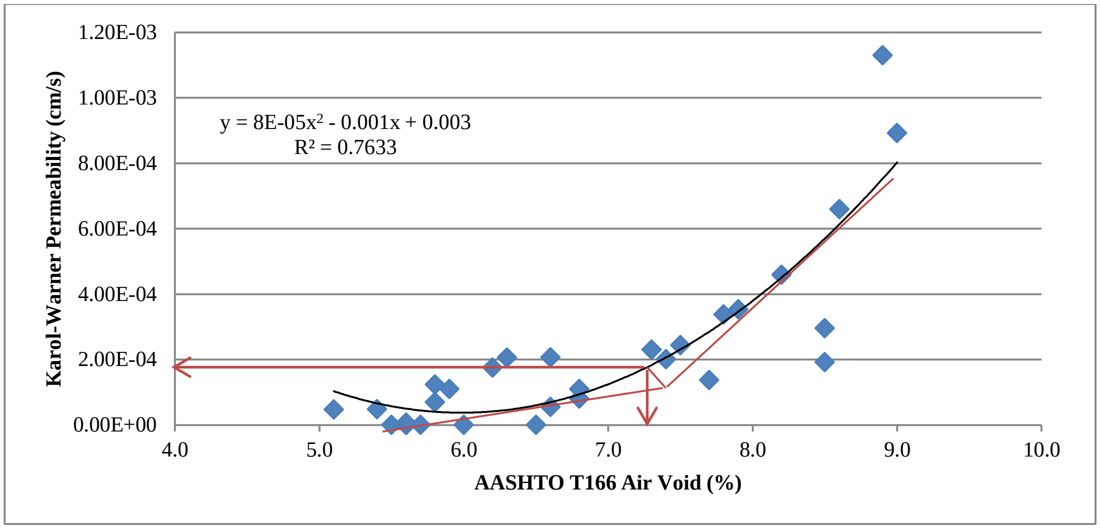


Figure 25. Selection of critical permeability and AASHTO T166 air voids values (2 ft right of the pavement joint)

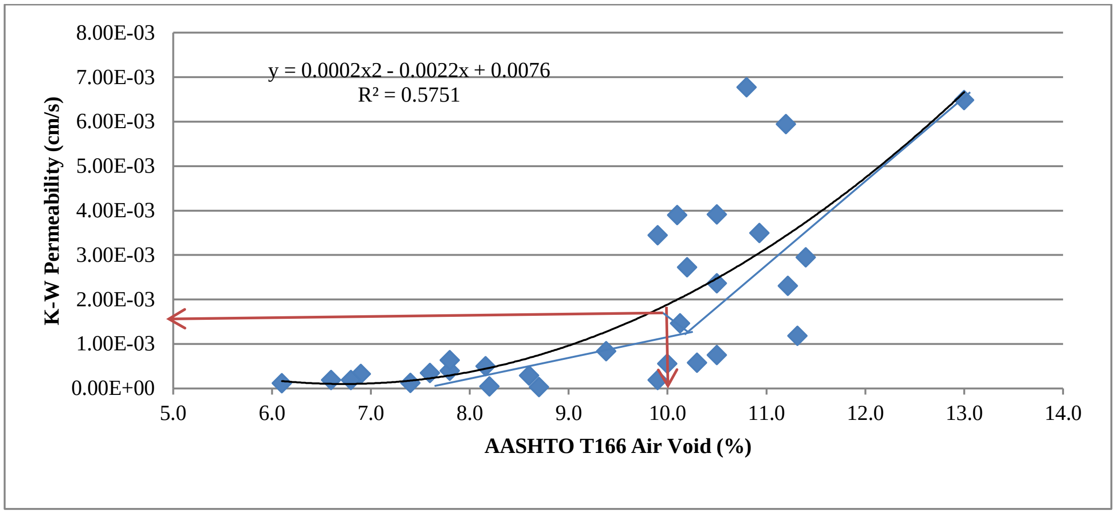


Figure 26. Selection of critical permeability and AASHTO T166 air voids values (on the pavement joint)

Table 12. K-W permeability and air voids criterion summary

	CoreLok air void	AASHTOT166 air void	K-W Permeability (cm/s)
2 ft right of joint	8.3	7.3	1.90E-04
On the joint	11.7	10.0	1.50E-03

Figure 27 through Figure 31 use double axis plots to show the inter-relationships of air voids, IDT strength and permeability value for each project. From the five figures it can be seen that the IDT strength has a linear relationship with the air voids while the permeability value has a non-linear trend with air voids. It is also found that the higher air voids, the higher permeability values and the lower the IDT strength. Figure 32 shows that for all of the projects the mean IDT strength on the pavement mat is higher than that on the pavement joint. The IDT strength ratio of the on the joint and on the mat is also shown in Figure 32. Without any special treatment, the butt joints paved in HMA and WMA (US 6 Project and IA 148 Project) exhibit the lowest ratios. It is recommended that the ratio value should not be lower than 0.8.

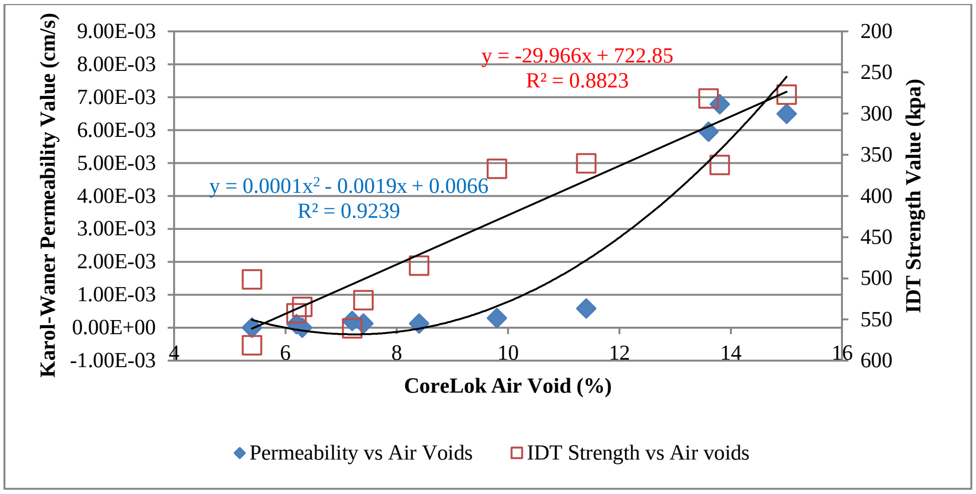


Figure 27. Air voids vs. K-W permeability and IDT strength for the US 6 project

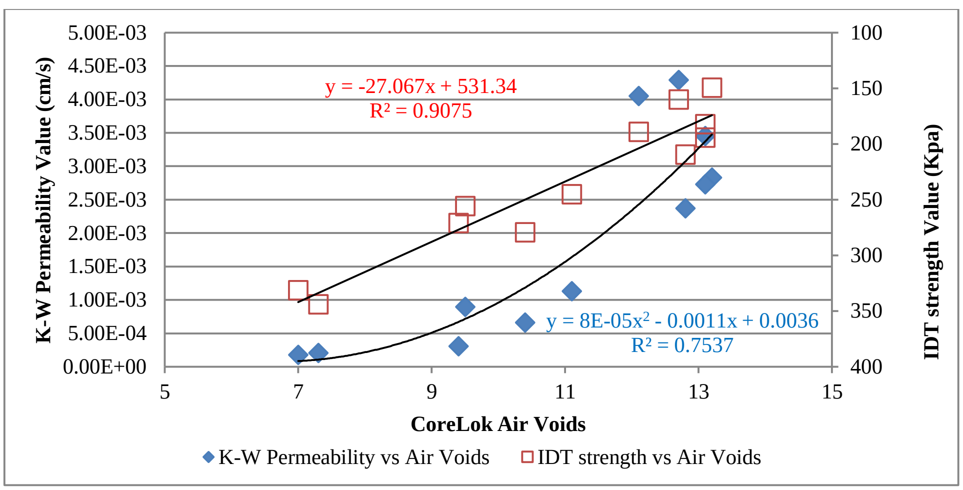


Figure 28. Air voids vs. K-W permeability and IDT strength for the IA 148 project

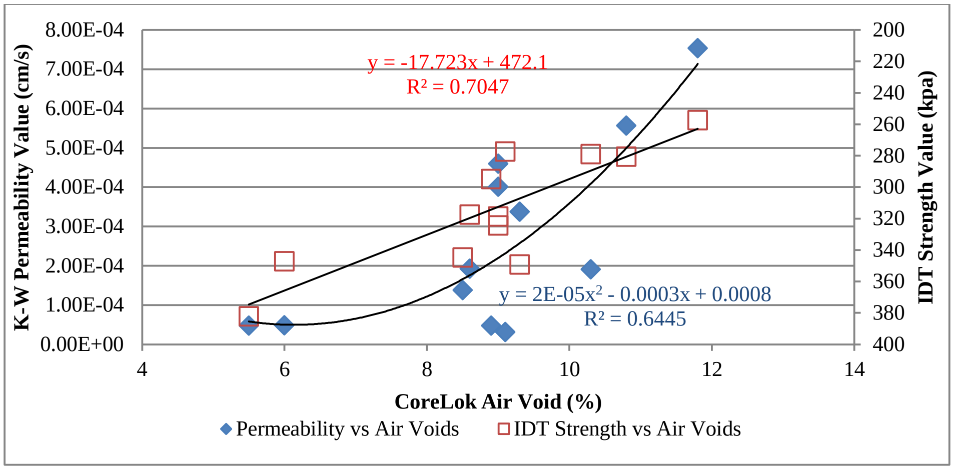


Figure 29. Air voids vs. K-W permeability and IDT strength for the IA 13 project

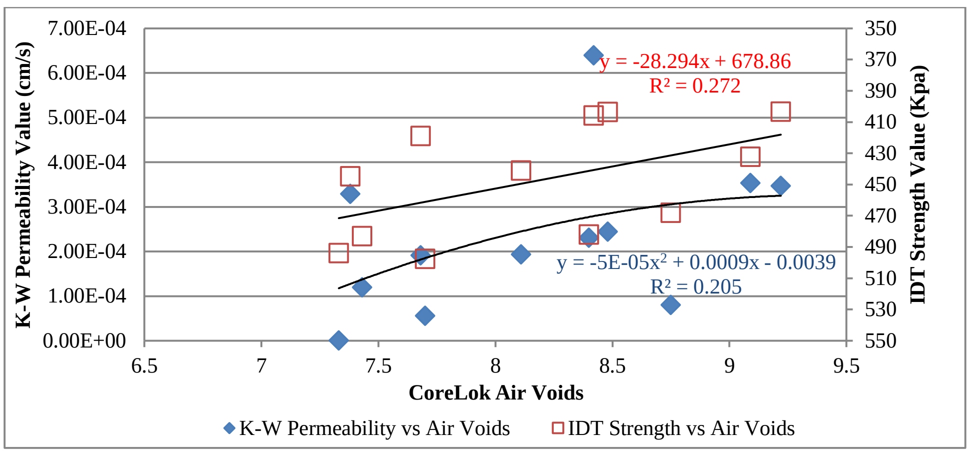


Figure 30. Air voids vs. K-W permeability and IDT strength for the I-35 project

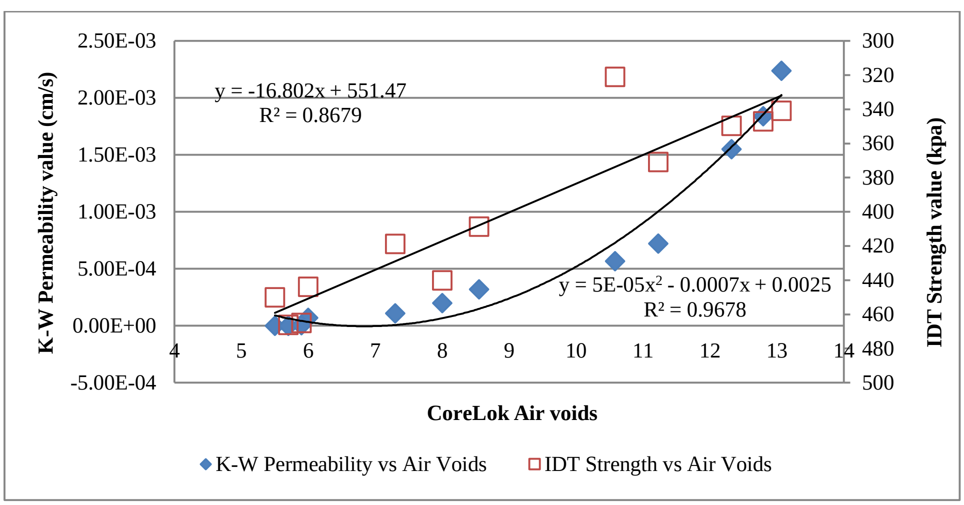


Figure 31. Air voids vs. K-W permeability and IDT strength for the US 61 project

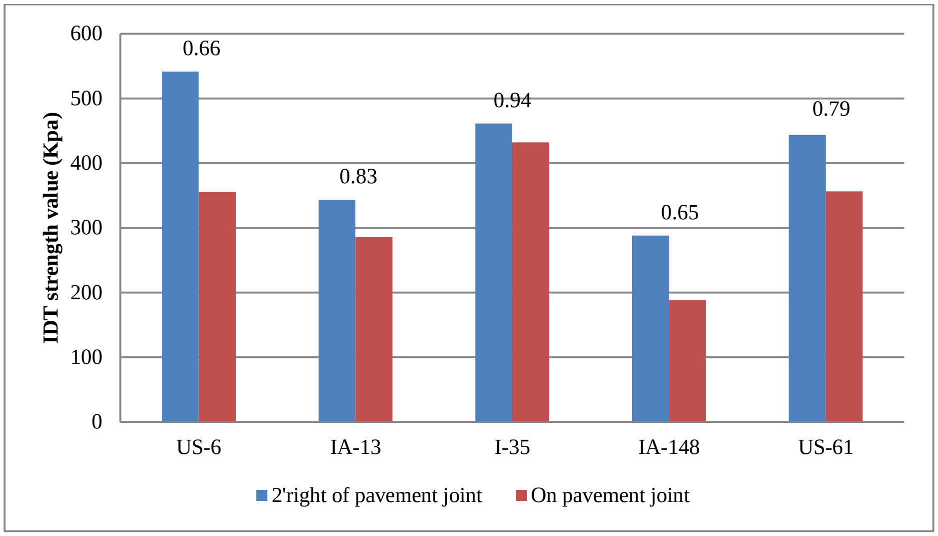


Figure 32. Comparison of mean IDT strength values for all projects

4.4 Core Test Result Summary and Evaluation of Segregation on Performance

Asphalt content and gradation of the field samples were determined according to AASHTO T-308 and AASHTO T-30 procedures, respectively. The asphalt film thickness (AFT) is also calculated, which is thought that the differences in AFT could be an indicator to identify segregation at the pavement joint. The AFT calculation is based on the calculated aggregate surface area and the effective asphalt content. Detailed calculations are shown in the Appendix. The results of the asphalt content, gradation and AFT data for all of the projects are listed in Table 13 through Table 17.

Table 13. Binder content and gradation summary for the US 6 project

	Binder Content (%)	Film Thickness (μm)	Sieve Size									
			¾ in.	½ in.	3/8 in.	#4	#8	#16	#30	#50	#100	#200
JMF	4.7	9.5	100	93	87	64	42	30	21.5	8.4	5.5	3.7
Test Locations: 2 ft right of Pavement Joint												
1	4.01	8.2	100	94.1	85.1	62.0	41.4	27.9	19.9	9.0	5.6	4.4
2	4.41	8.7	100	93.1	88.7	66.9	43.1	30.6	19.6	8.5	5.0	4.3
3	5.05	10.0	100	90.1	81.4	61.3	39.7	28.5	18.6	8.1	4.7	3.8
4	5.19	9.3	100	93.7	87.7	65.7	42.5	30.4	19.8	8.8	5.2	4.3
5	4.91	8.1	100	97.5	92.2	69.2	45.7	32.9	21.6	9.8	6.0	5.1
6	4.66	8.4	100	93.5	87.9	65.6	43.7	31.6	20.7	9.3	5.6	4.6
Test Locations: On Pavement Joint												
1	5.07	9.7	100	94.4	85.9	61.5	39.9	29.0	19.1	8.3	4.9	4.0
2	4.84	10.4	100	90.2	83.6	59.6	38.6	27.8	18.2	7.6	4.3	3.4
3	4.76	9.1	100	91.8	82.9	63.4	42.4	30.1	19.6	8.6	5.2	4.1
4	4.60	10.1	100	92.0	83.9	61.1	39.3	28.0	18.0	7.3	4.2	3.5
5	4.75	9.3	100	88.2	78.3	56.3	37.5	27.6	18.3	8.3	5.2	4.3
6	4.97	9.2	100	97.0	86.0	62.4	41.6	30.1	19.7	8.8	5.4	4.4

Table 14. Binder content and gradation summary for the IA 148 project

	Binder Content (%)	Film Thickness (μm)	Sieve Size									
			¾ in.	½ in.	3/8 in.	#4	#8	#16	#30	#50	#100	#200
JMF	5.3	10.9	100	91	87	64	44	32	18	7.3	4.1	3.5
Test Locations: 2 ft right of Pavement Joint												
1	6.22	11.4	99.2	92.7	88.9	67.9	46.7	34.4	20.2	8.7	4.3	3.1
2	5.86	10.9	100	93.0	87.7	64.3	44.8	33.4	19.9	8.7	4.3	3.5
3	5.75	11.2	100	91.9	85.7	67.6	46.5	33.8	19.5	8.1	4.0	3.2
4	5.52	11.0	100	91.8	90.6	68.9	46.7	33.1	18.4	7.6	4.1	3.3
5	5.37	11.5	100	93.7	87.5	64.7	44.5	32.9	18.8	7.4	3.6	2.9
6	5.45	11.0	100	92.9	87.1	67.2	45.9	33.4	18.5	7.6	4.0	3.3
Test Locations: On Pavement Joint												
1	5.81	10.2	100	94.1	90.9	69.6	46.2	33.6	19.9	8.9	4.9	4.0
2	5.48	10.8	100	93.0	87.0	65.0	43.9	32.0	18.7	8.0	4.3	3.5
3	5.15	12.6	100	88.8	81.4	58.1	39.2	29.7	17.4	7.0	3.1	2.4
4	5.27	10.8	100	93.5	89.4	67.1	44.6	32.6	18.7	7.9	4.1	3.3
5	4.85	10.5	100	89.9	83.5	62.8	41.4	31.2	18.1	7.7	4.1	3.5
6	5.09	10.5	100	95.3	89.9	66.6	44.4	33.3	19.2	8.1	4.1	3.3

Table 15. Binder content and gradation summary for the IA 13 project

	Binder Content (%)	Film Thickness (μm)	Sieve Size									
			¾ in.	½ in.	3/8 in.	#4	#8	#16	#30	#50	#100	#200
JMF	5.24	8.13	100	97	86	64	50	41	30	18	8.8	3.7
Test Locations: 2 ft right of Pavement Joint												
1	6.41	9.3	100	98.4	83.8	63.4	49.2	39.3	28.9	17.2	9.5	3.1
2	6.13	9.1	100	94.4	82.4	62.9	48.2	39.0	28.8	17.4	9.7	2.9
3	6.31	9.7	100	97.9	86.4	63.6	48.0	37.8	27.3	15.9	8.7	3.0
4	6.15	9.6	100	97.8	87.2	63.7	48.4	38.2	27.7	16.1	8.8	2.8
5	6.93	8.9	100	99.1	91.5	70.6	55.0	43.3	31.0	18.1	10.1	3.5
6	6.41	9.0	100	96.5	89.1	64.9	49.7	39.7	29.0	17.5	10.1	3.3
Test Locations: On Pavement Joint												
1	6.13	7.6	100	97.2	88.3	66.2	51.6	42.8	32.8	21.6	13.0	4.3
2	6.54	8.0	100	95.5	83.9	63.6	49.7	41.5	32.0	21.0	12.6	3.9
3	6.25	8.2	100	95.5	85.5	63.3	49.9	40.8	30.9	19.4	11.2	3.8
4	6.38	7.8	100	98.3	88.2	67.0	53.5	43.8	33.1	20.4	11.7	4.2
5	6.29	7.6	100	97.7	86.1	65.8	52.0	42.5	32.3	20.4	12.1	4.5
6	6.19	8.2	100	96.9	87.1	66.4	51.9	41.7	31.1	19.4	10.7	3.7

Table 16. Binder content and gradation summary for the I-35 project

	Binder Content (%)	Film Thickness (μm)	Sieve Size									
			¾ in.	½ in.	3/8 in.	#4	#8	#16	#30	#50	#100	#200
JMF	5.40	10.5	100	91	82	68	50	33	20	10	5.3	4.1
Test Locations: 2 ft right of Pavement Joint												
1	5.69	10.2	100	93.9	86.2	72.1	52.0	34.7	21.8	10.9	5.6	4.1
2	5.49	9.9	100	93.2	82.9	70.8	51.4	34.6	21.8	11.1	5.7	4.2
3	5.58	10.1	100	93.8	85.9	72.1	52.2	34.4	21.5	10.7	5.5	4.1
4	5.51	9.7	100	95.8	88.9	74.1	53.9	36.1	22.8	11.6	5.8	4.2
5	5.61	9.8	100	93.0	85.2	73.5	53.2	35.4	22.1	11.2	5.8	4.3
6	5.65	10.0	100	94.8	86.9	72.3	52.2	34.3	21.4	10.8	5.6	4.3
Test Locations: On Pavement Joint												
1	5.71	9.0	100	93.8	85.9	71.6	52.1	35.4	23.1	12.6	6.9	5.2
2	5.47	9.0	100	92.6	84.2	70.3	51.6	35.2	23.1	12.7	6.8	4.9
3	5.40	9.0	99.3	89.1	81.8	67.7	50.0	34.4	22.6	12.5	6.9	5.0
4	5.80	8.8	100	93.1	86.5	73.2	54.1	37.1	24.3	13.3	7.1	5.2
5	5.62	9.6	100	91.7	83.5	68.9	50.5	34.3	22.3	11.9	6.2	4.5
6	5.52	9.0	100	90.2	83.4	70.3	52.0	35.7	23.4	12.8	6.8	4.9

Table 17. Binder content and gradation summary for the US 61 project

	Binder Content (%)	Film Thickness (μm)	Sieve Size									
			¾ in.	½ in.	3/8 in.	#4	#8	#16	#30	#50	#100	#200
JMF	6.12	11.0	100	97	88	65	46	32	20	8.2	4.5	3.7
Test Locations: 2 ft right of Pavement Joint												
1	6.19	11.1	100	97.4	87.6	64.1	46.4	32.6	20.0	9.0	4.7	3.3
2	6.22	10.1	100	97.0	87.3	64.7	46.9	32.9	19.5	9.5	5.4	4.3
3	6.13	11.4	100	97.4	87.6	63.1	45.4	31.8	19.3	8.0	4.4	3.3
4	6.20	10.3	100	96.5	86.7	66.3	47.7	33.7	20.8	8.9	5.0	4.1
5	6.28	10.2	100	86.4	87.5	67.3	45.5	34.6	20.7	9.1	5.2	4.3
6	6.27	9.7	100	97.4	86.6	66.0	47.4	34.8	21.0	9.3	5.5	4.5
Test Locations: On Pavement Joint												
1	6.74	10.1	100	97.4	88.0	66.6	48.5	34.8	21.2	9.5	5.8	4.5
2	6.90	8.4	100	96.1	89.0	69.7	50.3	36.4	26.2	11.7	7.6	6.2
3	6.95	9.3	100	95.4	86.8	67.7	50.1	36.6	22.8	10.8	6.7	5.2
4	6.62	10.2	100	96.7	85.7	64.2	47.5	34.5	21.1	9.5	5.7	4.3
5	7.04	10.7	100	97.8	88.4	67.9	49.2	35.3	21.0	9.1	5.4	4.1
6	6.50	10.1	100	97.5	89.1	67.6	48.7	34.9	21.0	9.3	5.5	4.3

Combining the gradation data (retained on each sieve), the fineness modulus is also calculated as an overall gradation descriptor. Finally, all of these data listed in the tables above used the JMP software for the analysis of variance (ANOVA) to determine whether there are statistically significant differences between the paired data for on pavement joint and on the mat values. A 95% confidence was used in all cases. If statistically significant differences are evident, plus (+) and minus (-) signs are provided as further descriptors. A (+) sign indicates that the test values on the pavement joint are significantly higher than that on pavement mat, while a (-) sign conveys that the test values on the joint are significantly lower than those on the pavement mat. Gradation results on each sieve are taken as the value retained on each sieve for comparison. Therefore, a positive sign (+) for the gradation change indicates that significantly more aggregates are retained on the respective sieve for the longitudinal joint samples. Based on the results of the analysis shown in Table 18, the following observations are found.

Project US 6 (HMA butt joint): significant differences in fineness modulus, asphalt film thickness and percent passing the #4, #8, #30 and #50 sieves are identified. The differences with (+) positive signs indicate that the longitudinal joint gradation is significantly coarser than the pavement mat. In addition, all permeability and air void measurements are clearly able to detect the lower density and coarse segregation (coarser gradation) at the longitudinal joint.

Project IA 148 (WMA Butt joint): significant differences in the asphalt content and percent passing the #8 and #16 sieves are found. A decrease in asphalt content and the gradation on key sieves are coarser than the pavement mat is a typical pattern for coarse segregation. Also, all permeability and air void measurements are clearly able to detect the lower density and coarse segregation at the longitudinal joint.

Project IA 13 (Infrared Joint heater): significant differences in fineness modulus, asphalt film thickness and percent passing the #16, #30, #50, #100 and #200 sieves are identified. The difference with (-) negative signs reveal that the longitudinal joint gradation is significantly finer than the pavement mat. Fine segregation may help reduce permeability and neither the NCAT Permeameter nor the K-W Permeameter show statistical differences in permeability. However, density and stiffness differences between the longitudinal joint and the pavement mat have been quantitatively identified by the air void and IDT strength values.

Project I-35 (Milling and Filling): significant difference in gradation as identified with the 1/2 in. and #4 sieves are found on the pavement joint (higher amounts), while significantly less aggregates are retained on the #16, #30, #50, #100, #200 sieves. Finally, no statistical difference is found in the overall gradation comparisons. In addition, none of other tests (density, permeability and IDT strength tests) have shown significant differences. This tends to indicate that the longitudinal joint formed by milling and filling has no segregation with close density and stiffness values to that of the pavement mat.

Project US 61 (Modified butt joint -pinching): Higher asphalt content is present at the longitudinal joint by pinching. However, gradation results from the ignition oven test show that significant difference in the fineness modulus and percent passing the #4, #8, #16, #100 and #200 sieves are seen. The difference with (-) positive signs indicate that the joint gradation is significantly finer than those on the pavement mat. In addition, significantly lower density and IDT strength are clearly shown at the longitudinal joint by the ANOVA test.

Table 18. Summary of one-way ANOVA results for all projects

	US 6	IA 13	I-35	IA 148	US 61
	Joint vs. Mat	Joint vs. Mat	Joint vs. Mat	Joint vs. Mat	Joint vs. Mat
NCAT Permeability	Significant (+)			Significant (+)	Significant (+)
K-W Permeability	Significant (+)			Significant (+)	Significant (+)
CoreLok Air Voids	Significant (+)	Significant (+)		Significant (+)	Significant (+)
AASHTO T166 Air Voids	Significant (+)	Significant (+)		Significant (+)	Significant (+)
Porosity	Significant (+)	Significant (+)		Significant (+)	Significant (+)
PaveTracker	Significant (+)			Significant (+)	Significant (+)
IDT strength	Significant (-)	Significant (-)		Significant (-)	
Asphalt Content				Significant (-)	Significant (+)
Film thickness	Significant (+)	Significant (-)	Significant (-)		
% pass 1/2 in. change			Significant (+)		
% pass 3/8 in. change					
% pass #4 deviation	Significant (+)		Significant (+)		Significant (-)
% pass #8 change	Significant (+)			Significant (+)	Significant (-)
% pass #16 change		Significant (-)	Significant (-)	Significant (+)	Significant (-)
% pass #30 change	Significant (+)	Significant (-)	Significant (-)		
% pass #50 change	Significant (+)	Significant (-)	Significant (-)		
% pass #100 change		Significant (-)	Significant (-)		Significant (-)
% pass # 200 deviation		Significant (-)	Significant (-)		Significant (-)
Fineness Modulus	Significant (+)	Significant (-)			Significant (-)

From the one-way ANOVA results it can be seen that only the I-35 project among the five projects appears to have no segregation or slight segregation at the longitudinal joint. Based on various mix design and joint construction methods, the joints for the other four projects show differences in asphalt content and types of segregation as compared with the corresponding job mix formula. In order to capture the level of segregation difference on the pavement mat and on the joint, the key sieves are defined as follows: 1) the selected sieve should be closest to the 50/50 passing, 2) the percent passing on the sieve should also has significant difference between pavement mat and joint. As can be seen from Table 18, the No. 8 sieve is considered as the indicator sieve for the project US 6 and IA 148, No. 16 sieve is used for the IA 13 project, No. 4 sieve is used for the I-35 project, and No. 8 sieve is selected for the US 61 project. The relationship between the gradation deviation on the indicator sieve and the change in asphalt film thickness for all five projects are shown from Figure 33 through Figure 37. In order to examine the effect of segregation on the longitudinal joint density performance, the relationship between the gradation changes on the identified key sieve and the CoreLok air voids are displayed in Figure 38 through Figure 42. The goodness of fit (R^2) for the relationship between the air void and gradation deviation on the indicator sieve may reflect out whether segregation can greatly affects the longitudinal joint density or not. As can be seen, the R^2 values for projects US 6, IA 13 and US 61 are around 0.4 to 0.5 showing that some correlation does exists between density variations and segregation. However, the relatively low R^2 correlation may also indicates that although segregation can affect the longitudinal joint performance, it may not be the only factor attributed to the lower density achieved at the joints. Spatial variations in density for the longitudinal joint can be the result of lack of roller compaction, which is impossible be controlled during field experimental test. The R^2 for the IA 148 project and I-35 project are poor. This could be mainly because the longitudinal joint density decrease on IA 148 project is more related to the deficiency in asphalt content and the preliminary investigation has shown that the I-35 project appears to have no segregation. In general, the trend between air voids and segregation shows that the air voids content increases on both coarse segregation and fine segregation and the coarse segregation shows a higher rate of change comparing with fine segregated joints.

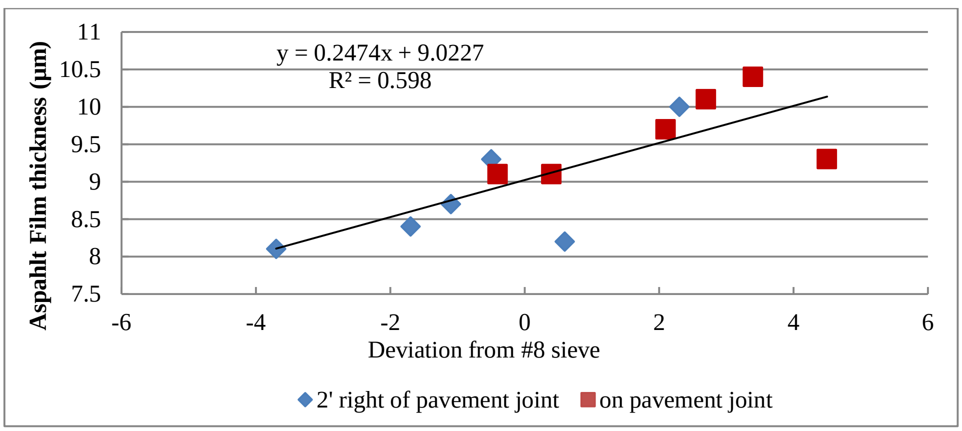


Figure 33. AFT vs. gradation changes for the US 6 project on the No. 8 sieve

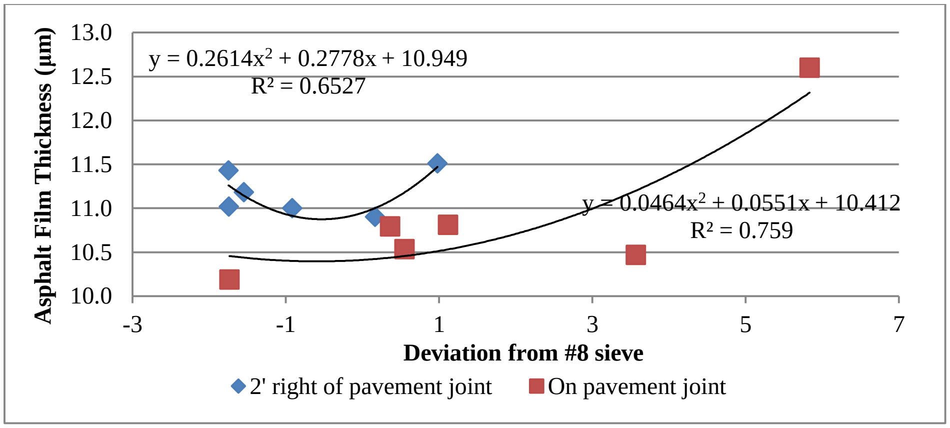


Figure 34. AFT vs. gradation changes for the IA 148 project on the No. 8 sieve

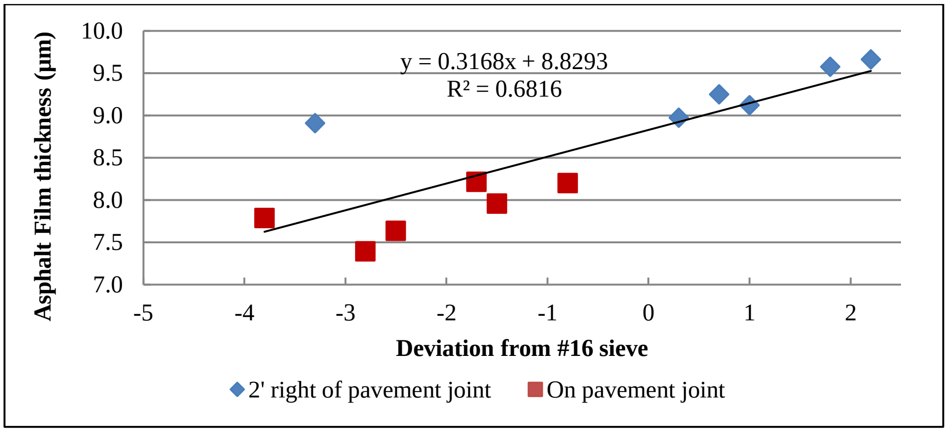


Figure 35. AFT vs. gradation changes for the IA 13 project on the No. 16 sieve

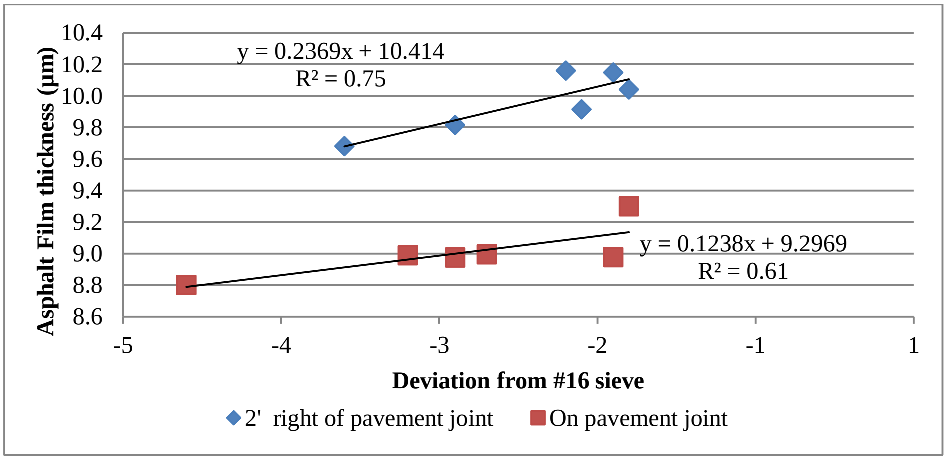


Figure 36. AFT vs. gradation changes for the I-35 project on the No. 16 sieve

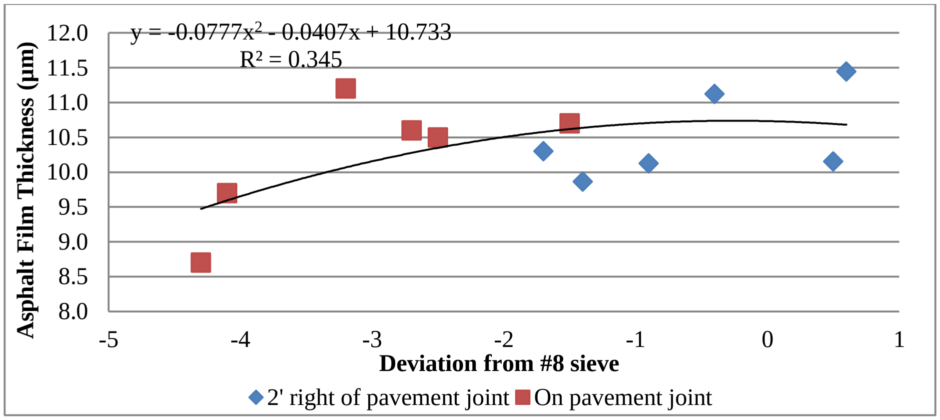


Figure 37. AFT vs. gradation changes for the US 61 project on the No. 8 sieve

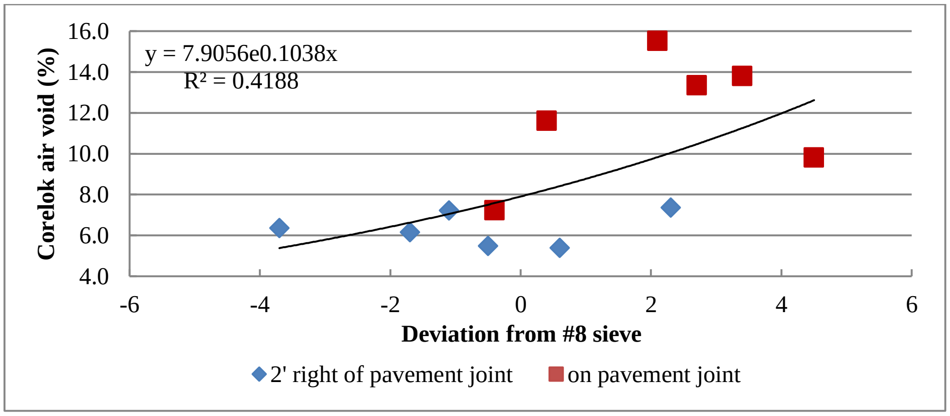


Figure 38. Air void vs. gradation changes for the US 6 project on the No.8 sieve

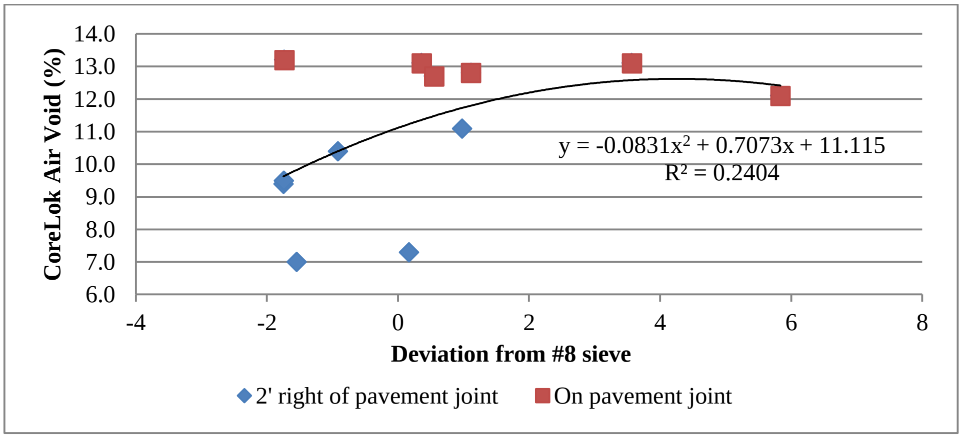


Figure 39. Air voids vs. gradation changes for the IA 148 project on the No.8 sieve

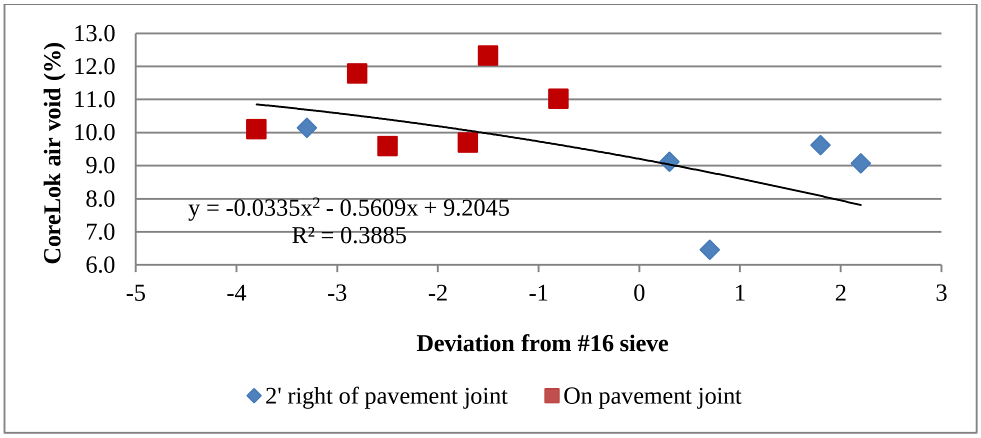


Figure 40. Air voids vs. gradation changes for the IA 13 project on the No.16 sieve

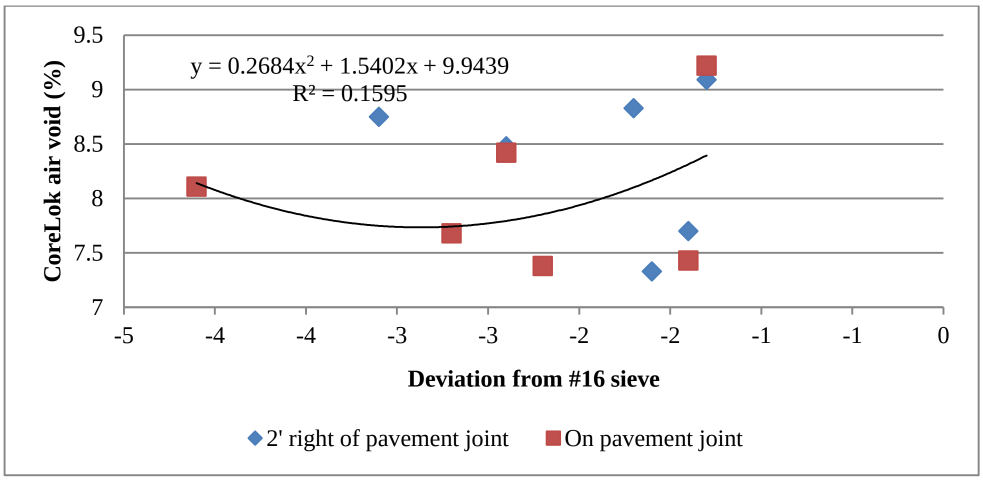


Figure 41. Air voids vs. gradation changes for the I-35 project on the No.16 sieve

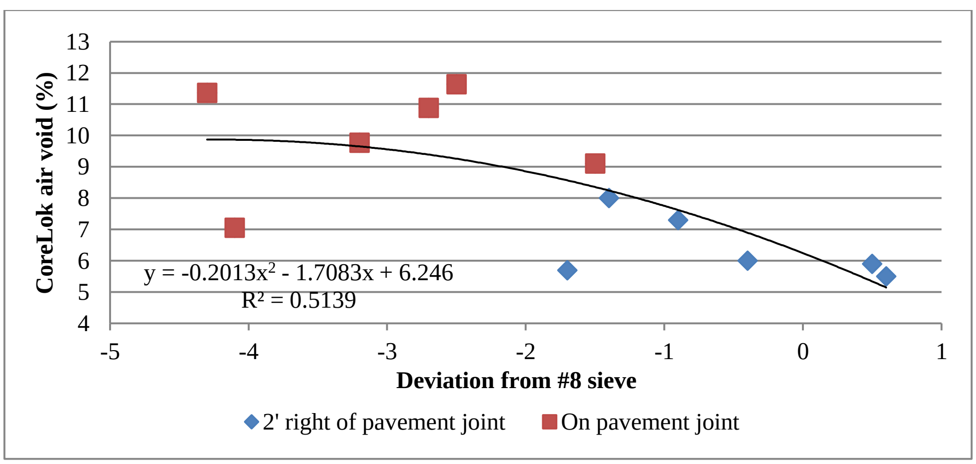


Figure 42. Air voids vs. gradation changes for the US 61 project on the No.8 sieve

CHAPTER 5 LONGITUDINAL JOINT FIELD PERFORMANCE

The field performance of the longitudinal joints selected via random station locations associated with the segregation aspect of the project was done by the University of Iowa in the 2012 Summer. The field performance assessment was done using a visual survey approach and developing a relative ranking performance. The condition survey was conducted according to ASTM D6433-09 “Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys.” Field surveys were done on only the projects constructed in the 2011 construction season which were US 6, IA 13 and I-35 projects. The IA 148 and US 61 projects were paved in 2012 and thus there was an inadequate amount of time for performance to develop. However, surface wave tests were performed on the IA 148 and US 61 projects.

US 6 Project

Although no signs of cracks have occurred yet, slight raveling and visibly large air voids along the longitudinal joint have been seen in the US 6 project. Figure 43 shows a picture of the surveyed longitudinal joint performance.

Figure 43. Longitudinal joint performance for the US 6 project

IA 13 Project

The northbound direction was surveyed on the IA 13 project, where the field tests and core extraction were performed. The IA 13 route shows a transverse crack every 20 feet starting from the project’s start to the end point (approximately 2 miles). Additionally, there is a longitudinal crack along with the shoulder separating the shoulder from the main traffic lanes, which shows a lack of adhesion between the shoulder and the main road joint. For the longitudinal crack between the two lanes, there is a minor longitudinal crack in the joint, which is very narrow and not deep. Figure 44 shows the pictures of the surveyed sections on the IA 13 traffic lanes and the longitudinal joint.

Figure 44. Longitudinal joint performance for the IA 13 project

I-35 Project

The southbound direction was surveyed on the I-35 project, the same as where the field tests and core extraction were performed. The longitudinal joint on the I-35 project exhibits the best performance with no cracking. Slight bleeding/flushing was found in the wheel paths of the I-35 pavement mat (see Figure 45). This could be due to the combination of relatively higher asphalt film thickness and higher volume of traffic on I-35 as it is a 30million ESAL design.

Figure 45. Longitudinal joint performance for the I-35 project

Relative Performance Ranking of Projects

After the field performance assessment, a relative longitudinal joint performance ranking is developed and shown in Table 19. Premature joint failures are a result of a combination of low density, high permeability, segregation and lack of joint adhesion. In order to evaluate which factor is the most important one, the performance test results on joint samples are also ranked into three different levels and compared with the longitudinal joint field performance. In Table 19, the IDT strength ranking shows the best correlation with the actual field performance and density is considered to be the second best fit. This suggests that the joint IDT strength as a measure of the joint's ability to resist an applied stress without tensile failure appears to be the most direct and accurate method to evaluate the performance of longitudinal joints.

Table 19. Longitudinal joint performance ranking

	Joint Condition Ranking	Permeability	IDT strength	Segregation	Air Void/Density
IA 13: butt joint	B	C	**B**	B	C
US 6: Joint Heater	C	A	**C**	B	B
I-35:Edge restraint by milling	A	A	**A**	A	A

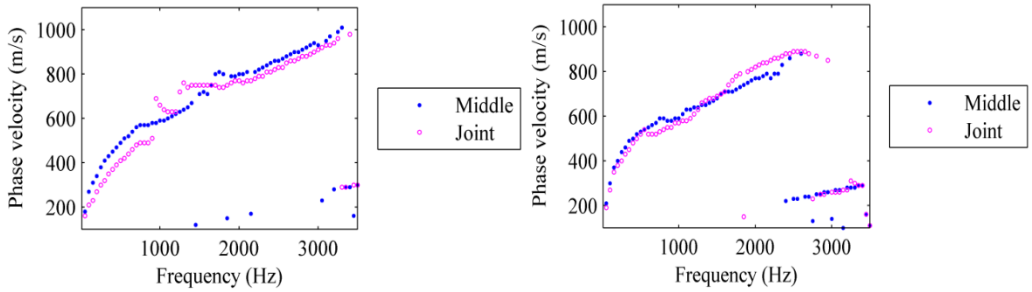
* A = Good; B = Fair; C = Poor.

Surface Wave Testing

Surface wave testing was performed on the IA 148 and US 61 projects as they were constructed in the 2012 construction season and an insufficient amount of time for subsequent field performance condition surveys was not available.

IA 148 Project

Surface wave testing was performed on project IA 148 after the field density and permeability tests were done. The surface wave testing on new asphalt pavements on IA 148 was carried out using a moving-source one-receiver (MSOR) with a 0.2-meter impact offset and 0.1-meter impact spacing. A sampling interval of 0.0122 milliseconds was used. The experimental dispersion images are shown in Figure 47. The first and second testing stations were at the same locations where other field tests and sample coring were done. As shown in the figure, the dispersion image obtained in the middle has a slightly higher phase velocity than the image obtained on the longitudinal joint due to the stiffness differences. This tends to indicate that the top layer in the middle is stiffer than that at the joint. Figure 46b shows the close dispersion images and close profiles between the middle and the joint. This result indicates that the surface wave cannot distinguish the stiffness difference on the joint and 2 ft right of the joint (middle of the pavement mat). This could be due to the lower stiffness of WMA (water injection method) which makes the surface wave testing not very sensitive for testing depending upon the test frequency. Examination of phase velocities below about 1500hz shows the joint density is lower than the middle portion of the mat.

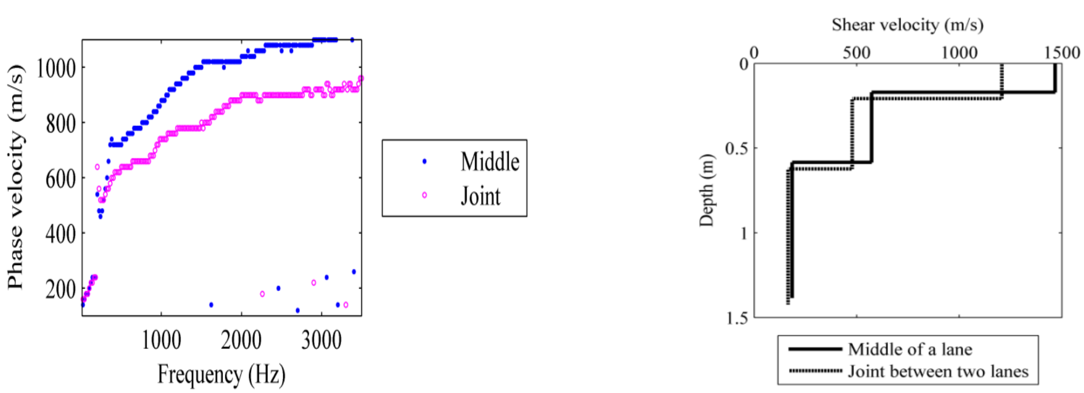


(a) Station 1 (b) Station 2

Figure 46. Experimental dispersion images on the IA 148 project

US 61 Project

Surface wave testing was performed on project US 61 after the field density and permeability tests were done. The surface wave testing on US 61 was carried out using a moving-source one-receiver (MSOR) with a 0.2-meter impact offset and 0.1-meter impact spacing. A sampling interval of 0.0122 milliseconds was used. The experimental dispersion images are shown in Figure 47. As shown in Figure 47a, the dispersion image obtained on the pavement mat has a much higher phase velocity than that obtained in longitudinal joint between two lanes due to the stiffness difference between the middle and the joint. This tends to indicate that the top layer in the middle is much stiffer than that in the joint. Figure 47b shows the corresponding inverted profiles in terms of each pavement layer. As can be seen on the top layer, the shear wave velocity at the center of the pavement mat is much higher than that at the joint. With the increase of thickness/depth, the wave shear velocities gets closer, this indicates that the differences in stiffness between the longitudinal joint and pavement mat may only happen on the top layer.



(a) Frequency vs. phase velocity (b) Shear velocity vs. depth

Figure 47. Experimental dispersion images on the US 61 project

CHAPTER 6 FINDINGS AND RECOMMENDATIONS

Premature longitudinal joint failures are a result of a combination of low density, high permeability, segregation and lack of joint adhesion. Five paving projects were selected for sampling and evaluation in Iowa with each one representing a typical longitudinal joint construction technique. The first two joint construction methods used the traditional butt joint with hot mix asphalt and warm mix asphalt. Another three construction methods paved with HMA are the butt joint with an infrared heat treatment, edge restraint by milling method and a modified butt joint with the first pass of rolling 6 inches away from the joint (hot pinch). For each project, joint quality is compared with regard to the center of the pavement mat (2 ft right of joint). Field densities using a PaveTracker 2701 non-nuclear gauge and permeability test by an NCAT Permeameter were made. Cores on both the longitudinal joint and 2 ft right of the joint were obtained for subsequent lab permeability, AASHTO T166 and AASHTO T331 density and IDT strength testing. Asphalt content and gradations were also obtained after using an ignition oven to burn the organic materials to determine the joint segregation. Beneficial findings are summarized as follows:

- The CoreLok method (AASHTO T-331) in general yields lower density values and thus higher air void values than AASHTO T-166. Greater differences in the density results are seen for the samples at the longitudinal joint.

- The PaveTracker does not have a strong relationship to neither AASHTO T166 nor the CoreLok methods for measuring density.

- The porosity measurement by a CoreLok is recommended for use in longitudinal joint quality control. Firstly, it gives very stable values and compares well with AASHTO T-166 and AASHTO T-331. Secondly, the method measures the apparent maximum specific gravity of the test sample instead of the theoretical maximum specific gravity values of loose mix that does not always represent the samples with fine and coarse segregation.

- It is recommended that the minimum required longitudinal joint density should be 90.0% and 88.3% of theoretical maximum density based on the AASHTO T166 and CoreLok (AASHTO T331) methods, respectively.

- A corresponding Karol-Warner in-lab permeability criteria identified according to the minimum required longitudinal joint density is 1.50e-03 cm/s.

- The NCAT Permeameter is an easy tool to use in the field, but it requires care to obtain a proper seal. No permeability criteria is determined due to its’ poor relationship with in-place air voids.

- The restrained-edge by milling, butt joint with the infrared heat treatment and the modified butt joint with hot pinch construction methods all create the joint density values improved results than the recommended density requirement and in-lab permeability criterion.

- The traditional butt joint paved in both HMA and WMA exhibits lower density and higher permeability than the criterion. The IDT strength ratio (IDT strength on joint divided by that on the pavement mat) is also found to be around 0.6.

- All of the projects appear to have segregation at the longitudinal joint except for the one using the edge-restraint by milling method on I-35. Based on various mix design and joint construction methods, the joints show quite different changes in asphalt content and types of segregation as compared with the job mix formula. Results of this study indicate that the lower density of longitudinal joints is a combination of gradation segregation, significant asphalt content variation and a lack of field compaction.

- A seismic wave testing method appears to be a promising way for field longitudinal joint quality control.

REFERENCES

AASHTO (1997) "Segregation Causes and Cures for Hot Mix Asphalt," American Association of State Highway and Transpiration Officials.

AASHTO T-166 (2007), "Standard Method of Test for Bulk Specific Gravity of Compacted Hot Mix Asphalt Using Saturated Surface-Dry Specimens, AASHTO Designation: T-166" *Adopted by the American Association of State and Highway Transportation Officials.*

AASHTO T-331 (2007), "Bulk Specific Gravity and Density of Compacted Asphalt Mixtures Using Automatic Vacuum Sealing Method" *Adopted by the American Association of State and Highway Transportation Officials*.

Buchanan, M.S. (2000), "Evaluation of Notched-Wedge Longitudinal Joint Construction" *Transportation Research Record 1712: 50-57.*

Buchanan, M. S. (2000), "An Evaluation of Selected Methods for Measuring the Bulk Specific Gravity of Compacted Hot Mix Asphalt (HMA) Mixes" *Vol. 69, Association of Asphalt Paving Technologist, pp. 608–634.*

Brock, J.D., (1986), "Segregation in Asphalt Mixtures" *Association of Asphalt Paving Technologists" Vol. 55, pp.269-276.*

Cross, A.S. and Brown, E.R. (1993), "Effect of Segregation on Performance of Hot-Mix Asphalt" *Transportation Research Record 1417: 117-126.*

Cooley, L.A., Brown, E.R. (2000). "Selection and Evaluation of Field Permeability Device for Asphalt Pavements" *Transportation Research Record 1723: 73-82.*

Cooley, L.A., Brown, E.R., and Maghsoodloo, S. (1999). "Developing Critical Field Permeability and Pavement Density Values for Coarse-Graded Superpave Pavements" *Transportation Research Record 1761: 41-49.*

Daniel, J.S. (2006), "Use of an Infrared Joint Heater to Improve Longitudinal Joint Performance in Hot Mix Asphalt Pavements" *Journal of Performance of Constructed Facilities, 20(2), pp. 167 – 175.*

Fleckenstein, L.J., Allen, D. and Schultz, D. (2002), "Compaction at the Longitudinal Construction Joint in Asphalt Pavements" *Kentucky Research Center, Report No.KYSPR-00-208.*

Huang, B.S., and Shu, X. (2010), "Evaluation of Longitudinal Joints of HMA Pavements in Tennessee" *Tennessee Department of Transportation, Project No. RES1304.*

Hughes, T., Davidson, J.K. and Cormier, A. (2009), "Performance of Warm Mix Technology in the Province of New Brunswick" *Proceedings of 54th Conference, Canadian Technical Asphalt Association.*

InstroTeck Inc. (2011), "CoreLok$^{@}$ Operator's Guide," Retrieved from www.instrotek.com.

Kennedy, T.W., Tahmoressi, M., Holmgreen, R.J., Jr., Anagnos,J.N., (1986), "Segregation of Asphalt Mixtures – Causes Identification and Cures" *Center for Transportation Research, Austin, TX, FHWA/TX-87/67+366-1F.*

Kabassi, K., Im, H., Bode, T., Zhuang, Z., and Cho, Y., (2011), "Non-nuclear Method for HMA Density Measurements" *47th ASC Annual International Conference*.

Kanitpong, K., Bahia, H. U., and Benson, C. H., (2001), "Hydraulic Conductivity of Laboratory Compacted Asphalt Mixtures" *Transportation Research Record: 1767: 25-32.*

Kanitpong, K., Bahia, H.U., Benson, C.H., and Wang, X.D., (2003), "Measuring and Predicting Hydraulic Conductivity of Compacted Asphalt Mixtures in the Laboratory," *Presented at 82nd Annual Meeting of the Transportation Research Board.*

Kandhal, P.S., Ramirez, T.L. and Ingram, P.M. (2002), "Evaluation of Eight Longitudinal Joint Construction Techniques for Asphalt Pavements in Pennsylvania," NCAT Report No. 02-03.

Larsen, D.A. and Henault, J.W. (2006)"Quantifying Segregation in HMA Pavements Using Non-nuclear Density Devices: Data Collection Report for Connecticut" *Connecticut Department of Transportation*, *Report No. CT-2238-F-06-2*.

Mallick, R.B., Cooley, L.A., Teto, M.R., Bradbury, R.L. and Peabody, D. (2003), "An evaluation of factors affecting permeability of superpave designed pavements," *NCAT Report 03-02*.

Mallick, R.B. and Daniel, J.S. (2006), "Development and evaluation of a field permeameter as a longitudinal joint quality indicator," *The International Journal of Pavement Engineering 7(1): 11-21*.

Mohammad, L. N., Herath, A., Wu, Z., and Cooper, S. (2005), "A Comparative Study of Factors Influencing the Permeability of Hot Mix Asphalt Mixtures," *Association of Asphalt Paving Technologists, Vol. 74E, pp. 1-25*.

Maupin, G.W., Jr. (2001), "Asphalt Permeability Testing Specimen Preparation and Testing Variability," Transportation Research Board," *Transportation Research Record 1767: 83–91*.

Transportation Research Board Website (2010). "Research in Progress" Retrieved from http://rip.trb.org/search/search.aspx?f1=k%3A%3AKeywords+%28Title%2C+Abstract%2C+or+Index+Terms%29&sc=xx%3A%3AAll+Categories&t1=longitudinal+joint

Williams, R.C., Duncan, G., and White, T.D. (1996). "Hot Mix Asphalt Segregation: Measurement and Effect" *Transportation Research Record* 1543: 97-105.

Williams, R.C., Raouf, M.A. and Schroer, J. (2010). "Alternative Test Methods for Measuring Permeability of Asphalt Mixes" *Transportation Research Record* 2594.

Williams, S.G., and Hall, K.D. (2008). "Critical Factors Affecting Field Determination of Hot-Mix Asphalt Density Using Non-nuclear Devices" *Transportation Research Record* 2081: 150-157

Washington State Department of Transportation (WSDOT) (2010). "Factors Effecting HMA Permeability" Retrieved from http://pavementinteractive.org/index.php?title=WSDOT:Factors_Effecting_HMA_Permeability

White, T.D., and Buchanan, M.S., (2004) "Effect of End-Point Compaction on Superpave Hot Mix Asphalt Mix Designs" *Mississippi Transportation Research Center, Report No. FHWA/MS-DOT-RD-03-145.*

Zube, E. (1962). "Compaction Studies of Asphalt Concrete Pavement as related to the Water Permeability Test" Publication 62-22. Department of Public Works. Division of Highways, California.

APPENDIX CALCULATION OF ASPHALT FILM THICKNESS (IOWA DOT METHOD)

The surface area factors for various sieve sizes are shown in the table below. The surface area (SA) is found by taking the % passing times the Surface Area Coefficient. The Surface Area for the material above the #4 sieve is a constant 0.0041. The total surface area is found by adding all of the individual surface area values.

Sieve	1 1/2	1	3/4	1/2	3/8	#4	#8	#16	#30	#50	#100	#200
Gradation	100	100	100	93	86.5	63	41.5	30	21.5	8.4	5.5	4
Surface Area Coefficient						0.0041	0.0082	0.0164	0.0287	0.0614	0.1229	0.3277

The asphalt film thickness is calculated using the following formulas:

Step 1: Calculation of aggregate effective specific gravity (G_{se})

$$G_{se} = \frac{100 - P_b}{100 / G_{mm} - P_b / G_b}$$

where, G_b is the specific gravity of the binder (provided by the asphalt binder supplier), P_b is the actual asphalt content, and G_{mm} is the maximum specific gravity of the mix.

Step 2: Calculation of percent binder absorbed (P_{ba})

$$P_{ba} = 100\left(\frac{G_{se} - G_{sb}}{G_{se}G_{sb}}\right)G_b$$

where, G_{sb} is the bulk specific gravity of the aggregate (from mix design information).

Step 3: Calculation of effective binder content (P_{be})

$$P_{be} = P_b - \left(\frac{P_{ba} \times P_s}{100}\right)$$

where, P_s is the percent aggregates.

Step 4: Calculation of film thickness (FT)

$$FT = \frac{P_{be}}{SA} \times 10$$