EVALUATION OF INTELLIGENT COMPACTION IN ASPHALT PAVEMENT CONSTRUCTION IN TENNESSEE

Submitted to the Tennessee Department of Transportation

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16. Abstract

This study aims to evaluate the applicability of intelligent compaction (IC) in construction of soil subgrade, granular bases and/or subbases, and asphalt layers in flexible pavements in Tennessee.

In the research project, a total of six asphalt mixture projects and one soil project were constructed to evaluate the IC technologies for various construction materials.

Three different in-situ testing methods were employed to evaluate the correlation between in-situ soil properties and IC measurement values (ICMVs), and four in-situ and laboratory testing methods were used to evaluate the relationship between physical and mechanical properties of asphalt mixtures and ICMVs. Simple linear and multiple linear regression analyses were performed to develop correlations between ICMVs and various in-situ test results. Geostatistical semivariogram analysis was performed on spatially referenced ICMVs to evaluate the spatial uniformity of IC measurements and to quantify the compaction quality of soil and asphalt mixtures. Based on the results of the laboratory and on-site tests, the following conclusions can be summarized:

1) For soil compaction, water content of soil had a significant effect on compaction meter value (CMV). A strong and stable linear relationship could be identified between CMV and deflection of soil layer when water content of soil was consistent.
2) A theoretical analysis showed that the difference in 1/CMV at the same point could filter out the effect of underlying support on CMV and reflect the increase in compaction degree of the newly placed asphalt resurfacing layer. Based on the core test results, it is recommended that three IC parameters be selected for evaluating the compaction quality of the resurfacing project: the difference in 1/CMV, the starting surface temperature of compaction (>110℃), and the total number of passes (>2 passes).
3) Two specific cycle costs were developed based on costs for construction of a pavement and savings from improved compaction uniformity over the pavement lifecycle. The benefit-cost analysis indicated a nearly 50% reduction in construction costs for all four projects using IC. The increased service life resulting from using IC was determined based on increased compaction uniformity, which led to a significant annual cost saving for all projects.

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EXECUTIVE SUMMARY

Intelligent compaction (IC) or continuous compaction control (CCC) technologies with global position system (GPS) documentation offer 100 percent coverage information with real-time data visualization of compaction data, which is a significant improvement over traditional quality acceptance (QA) procedures involving tests at discrete point locations. The goals of this project include: (1) Evaluate the IC technologies for compaction of embankment subgrade soils, aggregate base, and asphalt paving materials; (2) Assess the feasibility of utilizing the IC measured values for quality acceptance (QA) purpose. To achieve these goals, a total of six asphalt mixture projects and one soil project were constructed to evaluate the IC technologies for various construction materials. All asphalt mixture projects were performed in Tennessee, and the soil project was finished in China to provide additional data for soil compaction.

1. Chapter 5 summarized the detailed information of six asphalt mixture projects and one soil project.

2. Chapter 6 evaluated the relationships between the compaction meter value (CMV) of soil layer and in-situ point measurement values.

3. Chapter 7 examined the relationships between the CMV of asphalt layer and other factors. The relationships between CMV and other roller operation parameters such as roller frequency, speed and amplitude were examined first, then the effects of asphalt temperature and underlying support on CMV were analyzed further by utilizing the original Witczak model and the multilayer pavement analysis software BISAR.

4. Chapter 8 collected and evaluated IC parameters from six resurfacing projects in Tennessee for estimating the asphalt compaction quality. Field cores were taken from these projects and tested in the laboratory for density, permeability, and indirect tensile (IDT) strength. The lab results were correlated to different IC parameters. Based upon the results of correlation analyses, it is recommended that three IC parameters be used for evaluating the compaction quality of resurfacing project.

5. Chapter 9 employed a geostatistical analysis to evaluate the construction quality of asphalt pavement through the IC technology. Compared with the conventional

quality control method, the geostatistical analysis is capable of evaluating the spatial uniformity and identifying the weak locations during compaction.

6. Chapter 10 employed the asphalt vibratory compactor to simulate the IC compaction of asphalt mixture in the field.

7. Chapter 11 performed a benefit-cost analysis to systematically evaluate the economic benefits of IC. Two specific cycle costs were developed based on costs for construction of a pavement and savings from improved compaction uniformity over the pavement lifecycle.

Based on the work and analyses conducted, the research team reached the following conclusions:

1. For soil compaction, water content of soil had a significant effect on CMV value. A strong and stable linear relationship could be identified between CMV and deflection of soil layer when water content of soil was consistent.
2. For asphalt compaction, a significant linear relationship could be established between CMV and roller vibration amplitude among the operation parameters for Project 3, which provides a basis for adjusting CMV value. The adjustment of CMV and the rising RMV value could help identify the over-compacted area and the optimum pass number.
3. A theoretical analysis showed that the difference in 1/CMV at the same point could filter out the effect of underlying support on CMV and reflect the increase in compaction degree of the newly placed asphalt resurfacing layer.
4. There was a strong correlation between the difference in 1/CMV and core density for Project 3, while no correlation could be found between the original CMV and core density. However, no similar correlation could be identified for asphalt base layer (Project 6), mainly due to the properties of the asphalt layer and the stiffness of the underlying layers. The fact that CMV is sensitive to road modulus indicates that it is not suitable for evaluating the compaction level of asphalt base layer.
5. For asphalt core tests, the density of the cores from all projects showed a strong correlation with the permeability and IDT strength; whereas, the permeability of core samples correlated well with IDT strength for most projects. As the most important factor among IC parameters for asphalt compaction, the starting surface temperature of the breakdown roller had a stable and significant relationship with core densities for all projects.
6. Based on the core test results, it is recommended that three IC parameters be selected for evaluating the compaction quality of the resurfacing project: the difference in 1/CMV, the starting surface temperature of compaction (>110℃), and

the total number of passes (>2 passes).

7. Compared with conventional quality control methods, the geostatistical analysis was capable of evaluating the spatial uniformity and identifying the weak locations during compaction. On the other hand, the measurement depth of IC measurement value (ICMV) and its relationship with the roller operation parameters should be taken into account during the analysis. In general, the geostatistical method can serve as a supportive tool for evaluating the compaction quality with IC technology.
8. The IC compaction of asphalt mixture was simulated by utilizing AVC in the laboratory. A one-dimension accelerometer was employed to monitor the oscillation properties of the vibrator, based on which the response-derived stiffness properties of asphalt mixture were characterized. The stiffness properties of asphalt mixture during compaction were evaluated with response-derived measurement values (CMV, resonant meter value, and compaction control value), and linear relationships could be found between air voids and measurement values.
9. Benefit-cost analysis was performed to systematically evaluate the economic benefits of IC. Two specific cycle costs were developed based on costs for construction of a pavement and savings from improved compaction uniformity over the pavement lifecycle. The benefit-cost analysis indicated a nearly 50% reduction in construction costs for all four projects using IC. The increased service life resulting from using IC was determined based on increased compaction uniformity, which led to a significant annual cost saving for all projects.

The following were recommended for future studies:

1. To improve the understanding of IC for asphalt compaction application, more future studies are needed. For example, the plastic deformation of compaction layer should be considered when performing a theoretical analysis on CMV value. Also, further research should be performed to define a more appropriate index of IC to reveal the compaction level of asphalt layer in the future.
2. It is also recommended that IC data be incorporated into a pavement management system (PMS) so that long term benefits of IC technology may be identified in the future.

CHAPTER 1 INTRODUCTION

1.1 Problem Statement

The Intelligent Compaction (IC) is a relatively new technology for compacting asphalt, base, subbase, and subgrade materials that is gaining attention in the highway construction in the U.S. IC technology consists of an advanced dynamic evaluation device (accelerometer) positioned at the or in roller drums to measure the response of the underlying materials to the compaction forces being applied by the drum. The accelerometer readings are then analyzed by an onboard computer that takes the readings and evaluates the compaction levels and uniformity of the materials. The rollers are also equipped with global positioning system (GPS) technology that establishes the roller drum locations and displays data on the operation and measurements of the rollers in real time. A color-coded display assists the roller operator in achieving the needed coverage over the full pavement area in both daytime and nighttime conditions.

Intelligent compaction (IC) or continuous compaction control (CCC) technologies with global position system (GPS) documentation offer 100 percent coverage information with realtime data visualization of compaction data, which is a significant improvement over traditional quality acceptance (QA) procedures involving tests at discrete point locations.

The objective of the research project was to evaluate the applicability of intelligent compaction in construction of soil grade, granular bases and/or subbases, and asphalt layers in flexible pavements in Tennessee. Potential field project and candidate IC rollers were identified. The selected field projects were constructed using the latest intelligent compaction technology. At the same time, in-situ testing was conducted to verify the construction quality. Economic analyses were also included to show the financial benefits of intelligent compaction.

The study can benefit the TDOT in the following aspects:

(1). Improved compaction quality of asphalt pavements;

(2). Increased quality control and reduced maintenance cost;

(3). Money-savings;

(4). Safer operations.

1.2 Objectives

The objectives of the research project are to:

(1) Evaluate the IC technologies for compaction of embankment subgrade soils, aggregate base, and asphalt paving materials;

(2) Assess the feasibility of utilizing the IC measured values for quality acceptance (QA) purpose.

1.3 Scope of Study

The scope of the research work includes:

- To complete a synthesis of literature review and state DOT survey on the application of IC technologies to compaction of embankment subgrade soils, aggregate base, and asphalt paving materials;
- To perform intelligent compaction and field testing on embankment subgrade soils, aggregate base, and asphalt paving materials;
- To develop correlations between IC measurements and results from field testing;
- To assess the feasibility of using IC technologies for QA purpose based on the findings from the study.

CHAPTER 2 LITERATURE REVIEW

2.1 Background

Compaction is one of the key processes to acquire a desired long-term pavement performance, especially for asphalt mixture layers, including hot-mix asphalt (HMA) and warm-mix asphalt (WMA). Conventional compaction methods have reasonably worked to achieve the target compaction level of asphalt materials over the past decades. However, there are some long existing and nontrivial shortcomings. Typically, in a conventional compaction method, a static or vibratory load is applied via a roller to the material being compacted for a certain number of passes. A uniform roller pattern covering an entire lane is not an easy task to achieve because rollers are manually controlled by roller operators, and even a uniform fixed number of passes cannot guarantee a uniform compaction level, since many other critical factors, such as changes in the underlying support, or non-uniform temperatures in asphalt mixtures, also affect the degree of compaction. Unfortunately, these critical factors are invisible to roller operators sitting on a conventional roller compactor. Therefore, no timely adjustment can be made to avoid potential over-compaction or under-compaction issue.

For asphalt pavements, achieving a uniform and desired density has been adopted as the criteria for quality control (QC) and assurance (QA) over the years (AASHTO, 2001). Typically, some spot tests, either core-drilling or nuclear gauge test (NG), are performed at limited points after compaction and the results are extended to determine whether the entire asphalt layer has achieved the required compaction level. According to how the density is acquired using these conventional methods, some long existing shortcomings are inevitable. The spot tests are only performed at limited locations. As mentioned above, the changes in the critical factors affecting compaction may compromise the representativeness of these spot tests performed at limited locations. Also, merely using asphalt layer density as the acceptance criterion does not ensure the required stiffness and/or strength of the particular layer, or the quality of entire pavement structure to withstand traffic loading. Further, even if the point tests could identify the unsatisfied area successfully, a remedial action is either impossible or expensive since the compaction procedure is already over.

During the past decade, a new compaction technology – Intelligent Compaction (IC) – has gained attention in the asphalt paving industry. According to the Federal Highway Administration (FHWA), IC refers to "*an improved compaction process using rollers equipped with an integrated measurement system that consists of a highly accurate GPS, accelerometers, onboard computer reporting system, and infrared thermometers for HMA feedback control. By integrating measurement, documentation, and control systems, the use of IC rollers allows for real-time monitoring and corrections in the compaction process. IC rollers also maintain a continuous record of color-coded plots that indicate the number of roller passes, compaction level, temperature measurements, and the precise location of the roller drum*. *(Nieves, 2013)*" With the IC technique, some critical factors affecting compaction, such as roller passes and temperature, are made visible to roller operators in real time with color-coded displays. Therefore, there is a potential to improve the quality and uniformity of asphalt layers by making a timely adjustment to the compaction pattern. The IC technology may also overcome the shortcomings of conventional QC/QA methods in two aspects: First, it can 100% cover the compacted area compared to limited test spots in conventional compaction; second, the IC measurement value (ICMV) can serve as an index to evaluate the stiffness or strength of pavement to a certain depth (M. Mooney & Adam, 2007; M. A. Mooney, 2010).

2.2 History of IC

IC technology, also referred to as Continuous Compaction Control (CCC), was initiated in

Europe for soil compaction in the 1970s and has been used since then (M. A. Mooney, 2010). Now, a growing number of IC rollers are also used in asphalt compaction. In 1974, Dr. Heinz Thurner of the Swedish Highway Administration performed the initial field studies on roller-integrated measurement with a Dynapac vibratory roller instrumented with an accelerometer. The test results revealed that the ratio of the amplitude of the first harmonic to the amplitude of the excitation frequency in the frequency domain could be correlated to the degree of compaction and the stiffness of the soil. In 1978, Dr. Thurner and his partner Åke Sandström developed the Compactometer and the compaction meter value (CMV), which was introduced to the technical community at the First International Conference on Compaction held in Paris, France, in 1980 (Forssblad, 1980; Heinz Thurner & Sandström, 1980). Dynapac was the first company to offer the CMV-based Compactometer in 1980, followed by many other roller manufacturers (e.g., Ammann, Caterpillar, and Ingersoll Rand). Bomag introduced the Omega value in 1982, which provided a continuous measure of compaction energy and served as the only alternative to CMV during that time. The Omega value was replaced by a vibration modulus E_{vib} later to provide a measure of dynamic soil stiffness (Kröber, Floss, & Wallrath, 2001). Ammann introduced a similar soil stiffness parameter ks in 1999 (Roland Anderegg & Kaufmann, 2004). In 2004, Sakai introduced the compaction control value (CCV) and used the harmonic content from the measured drum vibration to estimate the compaction degree, which was similar to CMV (Scherocman, Rakowski, & Uchiyama, 2007).

The term of IC has further evolved in recent years by introducing the servo-controlled eccentric excitation. The purpose of that is to improve roller performance and compaction by adjusting the vibratory force amplitude and/or frequency automatically. The Ammann/Case, Bomag, and Dynapac IC rollers can decrease the vertical vibration force or the eccentric force amplitude automatically when undesirable operating conditions are detected (e.g., jump mode). However, the effect of this technology is not well quantified.

2.3 Roller-Integrated Measurement Systems

In the IC technology, various measurements are obtained during the compaction process, including GPS roller position, speed, number of roller passes, surface temperature and roller measurement values (MVs). A detailed evaluation about the compaction level and uniformity can be obtained based on the collected data.

The major manufacturers of IC equipment incorporate various equipment-dependent roller measurement values (MVs) to reflect the compaction state of the underlying materials. Table 2.1 summarizes some common MVs from the major equipment manufacturers.

Table 2.1 Roller Measurement or Machine Values, after (Mallick & El-Korchi, 2013; White & Thompson, 2008)

Manufacturer	**Machine value**	**Name**	**Description/ Notes**
Ammann, Case	k_s	Stiffness	Drum-soil contact force/vertical drum displacement
Bomag	E_{vib}	Vibration modulus	Drum-soil contact force/vertical drum displacement [MN/m²]
Caterpillar, Dynapac, Hamm, Volvo	CMV	Compaction Meter Value	In the frequency domain, ratio of vertical drum acceleration amplitude at fundamental (operating) vibration frequency and first harmonic. Depends on roller dimensions, (i.e., drum diameter and weight) and roller operation parameters (e.g., frequency, amplitude, speed). Dimensionless
	MDP	Machine Drive Power,	Empirical relationship between mechanical performance of the roller to the properties of the compacted soil. Dimensionless
Sakai	CCV	Compaction Control Value	In the frequency domain, algebraic relationship of multiple vertical drum vibration amplitudes includes fundamental frequency and multiple harmonics

As shown in Table 2.1, different vendors use different types of ICMV with the same purpose of evaluating the level of compaction. These measurements are either based on vibration frequency analysis or mechanical modeling. An IC roller usually has an accelerometer mounted on its roller drum, which enables the IC roller to record the machine–ground interaction data and simultaneously calculate the ICMV value as an index relating to the material stiffness. In this research, the vibratory-based Compaction Meter Value (CMV) was adopted as the main roller measurement value. Developed by Geodynamik, CMV is a dimensionless compaction parameter that depends on roller dimensions (i.e., drum diameter and weight) and roller operation parameters (e.g., frequency, amplitude, and speed) and is determined using the dynamic roller response (Å. Sandström, 1994). Since the drum of a vibrating roller subjects the soil or pavement to periodic blows, which is similar to a dynamic plate load test, a reasonable attempt is using the blows from the cylindrical drum as a load test of soil or pavement. After evaluating this idea in numerous field tests (A. Sandström & Pettersson, 2004), the researchers found that there existed a significant relationship between the ratio of the amplitude of the first harmonic to the amplitude of the fundamental frequency of the roller drum and the compaction level achieved. It was also found that the force amplitude 'F' of the blows is proportionate to the first harmonic of the vertical acceleration of the drum, and the displacement 's' during the blow can be expressed by the amplitude of the double integral of the fundamental acceleration component. Therefore, a "cylinder deformation modulus" Ec can be expressed as the ratio of the force to the corresponding displacement as follows:

$$E_c = c_1 \times \frac{F}{S} = C_2 \times \omega^2 \times \frac{A_{2\Omega}}{A_{\Omega}} \tag{2-1}$$

where

$A_{2\Omega}$=acceleration amplitude of the first harmonic component of the vibration

A_{Ω} =acceleration amplitude of the fundamental component of the vibration

ω = fundamental angular frequency of the vibration

c_1, c_2 =constants

Based on that, the CMV is calculated as follows:

$$CMV = C \times \frac{A_{2\Omega}}{A_{\Omega}} \tag{2-2}$$

where C=constant (i.e. 300). Though the actual CMV varies for different rollers, a standardized roller with a same setting can be used to assess the stiffness of the compaction surface similar to an Falling Weight Deflectometer (FWD) equipment or a plate-bearing test (Heinz Thurner & Sandstrom, 2000). The roller also measures the resonant meter value (RMV), which provides an indication of the drum behavior. RMV is calculated as follows:

$$RMV = C \times \frac{A_{0.5\Omega}}{A_{\Omega}} \qquad (2\text{-}3)$$

where $A_{0.5\Omega}$= subharmonic acceleration amplitude caused by jumping. Previous experimental and numerical investigations (Adam & Kopf, 2004a) on roller-soil interaction identified five different drum behavior modes: continuous contact, partial uplift, double jump, rocking motion, and chaotic motion. These modes depend on the nonlinearity of soil behavior under vibration and roller operational settings (i.e., amplitude, frequency, and speed). Theoretically, RMV=0 indicates that the drum is in a continuous contact. When RMV>0, the drum enters a double jump mode or even rocking and chaotic modes. These previous researches reveal that drum behavior affects the CMV measurements (Adam & Kopf, 2004a, 2004b; Brandl & Adam, 1997). Therefore, the CMV measurements must be interpreted in conjunction with the RMV measurements. Since the chaotic motion is harmful to the machines and dangerous for the operator, in current practice, IC roller compactors can use automatic feedback control of the centrifugal force to prevent this motion (Roland Anderegg & Kaufmann, 2004).

As a relative stiffness index, the Sakai CCV is similar to CMV, which is also determined from the measured acceleration data but based on more harmonic frequency components. The concept of the CCV is that the roller drum starts to enter into a “jumping” motion as the ground stiffness increases, resulting in vibration accelerations at various frequency components. CCV is defined as follows:

$$CCV = 100\,\frac{A_{0.5\Omega}+A_{1.5}+A_{2\Omega}+A_{2.5\Omega}+A_{3\Omega}}{A_{\Omega}+A_{0.5\Omega}} \qquad (2\text{-}4)$$

Where, $A_{3\Omega}$ is the acceleration amplitude of the second harmonic component of vibration;

$A_{1.5\Omega}$ and $A_{2.5\Omega}$ are the acceleration amplitude of the 1.5 and 2.5 fundamental frequency of vibration. The concepts of CMV and CCV are illustrated in Figure 2. It should be noted that all the manufacturers employ proprietary data recording and filtering methods using proprietary software.

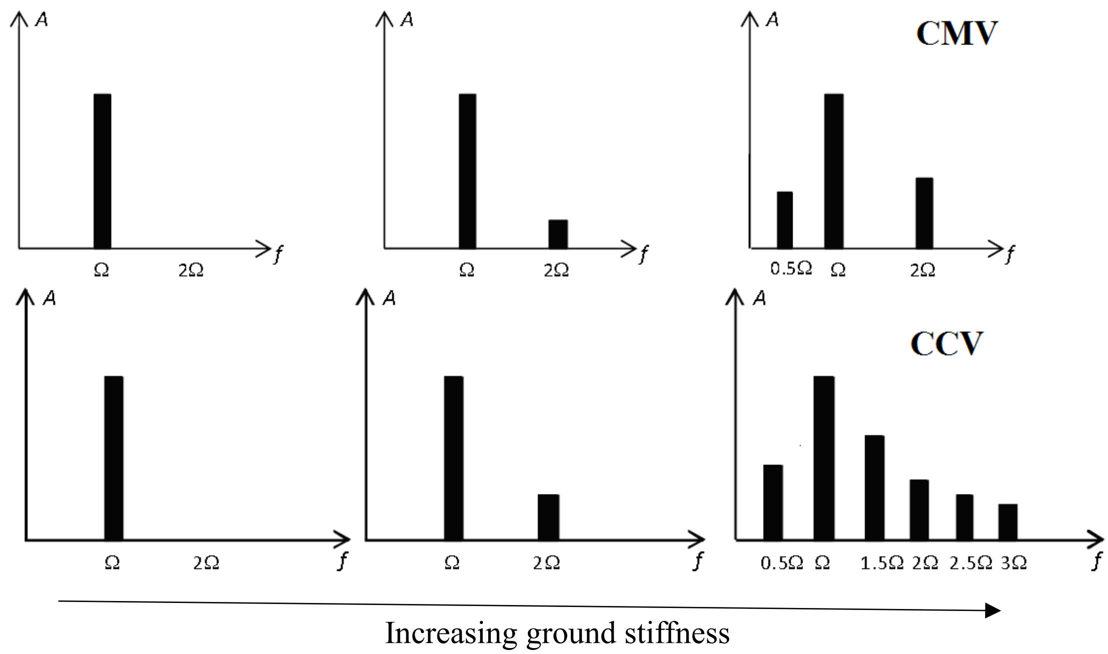


Figure 2.1. The concepts of CMV and CCV

The relationship between CMV, soil modulus, and eccentric force amplitude was investigated by Adam & Kopf (Adam & Kopf, 2004a) as shown in Figure 2.2. Their study revealed how the various operational modes are influenced by eccentric force amplitude and soil modulus. Their research also demonstrated the sensitivity of roller MVs to soil modulus within each operational mode. In Figure 2.3, the CMV increases linearly with soil modulus during partial uplift, but it is insensitive to soil modulus during continuous contact (CMV below approximately 10). These previous researches reveal that the drum behavior affects the CMV measurements (Adam & Kopf, 2004a, 2004b; Brandl & Adam, 1997). Therefore, the CMV measurements must be interpreted in conjunction with the RMV measurements.

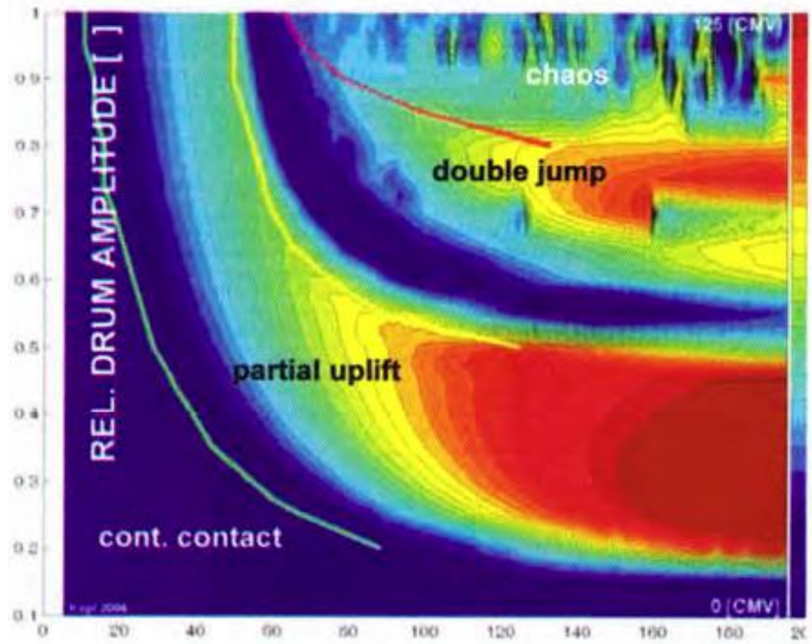


Figure 2.2. CMV depending on soil stiffness and amplitude (from Adam & Kopf 2004)

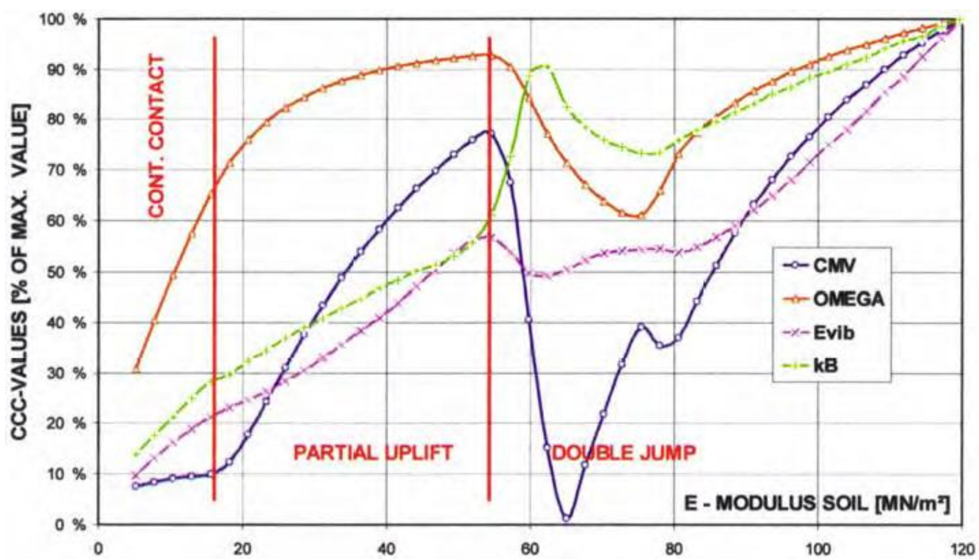


Figure 2.3. Sensitivity of roller MVs to soil modulus (from Adam & Kopf 2004)

2.4 Correlation Studies

Several studies have been performed since the beginning of IC technology to relate ICMVs to spot-test measurements such as density, static plate load test (PLT) modulus and light weight deflectometer (LWD) modulus. For soil compaction, some linear correlations have been found between the ICMVs and various types of soil in-situ point measurement values (Point-MV). Kröber et al. found good correlations from calibration test strips between ICMV and PLT initial and reload modulus with $R^2 > 0.9$ in 2001 (Kröber et al., 2001). Based on average measurements over the length of a test strip (20 m), White et al also found good correlations between ICMVs and Point-MVs ($R^2 = 0.87$ for density and 0.96 for dynamic cone penetrometer) in 2007 (White, Vennapusa, & Thompson, 2007). In 2010, Mooney concluded that simple linear correlations between ICMVs and compaction layer Point-MVs were appropriate for a compaction layer with relatively homogenous underlying layer (M. A. Mooney, 2010).

While applied successfully on soil compaction for many years (Roland Anderegg & Kaufmann, 2004; R Anderegg, von Felten, & Kaufmann, 2006; Å. Sandström, 1994; A. Sandström & Pettersson, 2004), IC is still a relatively immature technology for asphalt mixture compaction. Though some researches showed strong correlations between ICMV and in-situ spot test results of asphalt for few particular projects (Commuri, Mai, & Zaman, 2011; FHWA, 2011; HF Thurner, 2001), strong linear relationships between ICMVs and Point-MVs of asphalt are generally rare to find, especially for density measurements (Hu, Huang, Shu, & Woods, 2017; Hu, Jia, Huang, & Park, 2017; Hu, Shu, Jia, & Huang, 2017; White et al., 2008). There are many possible reasons for the poor correlation between ICMV and asphalt density. In addition to the variation of underlying layer stiffness, the effect of asphalt temperature on ICMV is one major reason while nuclear gauge/core measurements are independent of the temperature. various ICMV definitions from different roller manufacturers also impede the standardization and implementation of IC technology on the asphalt compaction.

In this project, the features of asphalt mixture resurfacing projects can further complicate the situation: mapping existing support materials using IC rollers prior to subsequent asphalt mixture paving can identify weak locations, but usually it is unfeasible for resurfacing projects due to the consideration of potential damage both to the milled original surface and the roller. For some resurfacing projects, all rollers are in a vibration off mode, and no ICMV data can be displayed as an index for the roller operator.

2.5 Existing IC Specifications

Some countries, including Austria, Germany, and Sweden, have introduced and applied the specifications of applying IC technology on soil compaction for many years. In the United States, Minnesota implemented the soil IC specifications in 2007 first followed by many other states.

The German specifications for earthwork QC/QA using IC were introduced in 1994 which apply to subgrade and embankment soils. IC can be specified in two ways in Germany. First, IC can be implemented through initial calibration of ICMVs to Plate Load Test (PLT) modulus or density and subsequent use of the correlation during QA. A second approach uses IC to identify weak areas for spot testing via PLT, LWD, or density methods. The Austrian specifications also allow two different approaches for IC. The first approach is similar to the German specification. An alternative approach, recommended for small sites or where calibration cannot be performed, involves compaction with roller-integrated measurement until the mean ICMV increases by no more than 5%. Acceptance is then based on static PLT or LWD (dynamic PLT) modulus at the weakest area. The Swedish specifications permit the use of IC to identify weak spots for PLT. In Sweden, conventional QA of base and subbase layers is based solely on a certain number of PLTs performed within each 5,000 m^2 control area. When using IC, the number of PLTs can be reduced since the IC can identify the weakest areas. The Mn/DOT specification requires construction of control strips to determine the IC target value (IC-TV) for each type and/or source of soil. During general production operations, the acceptance for the finished segments is that at least 90% of the MVs are at least 90% of the moisture-corrected IC-TV, and all of the MVs must be at least 80% of the moisture-corrected IC-TV. The contractor must recompact until all areas meet these acceptance criteria.

As mentioned in chapter 2.4, asphalt compaction is more complex than soil compaction. Asphalt is temperature-dependent and the possibility of damaging under vibration compaction for the thin layer, requiring a different roller configuration. The varied approaches of several roller manufacturers also need to be considered when developing specifications for asphalt pavement materials. For these reasons, IC concepts and specifications for asphalt materials were developed later than those for soils and aggregates.

In 2013, FHWA drew up generic IC specifications for asphalt materials based on the experience of IC pooled fund and state projects, and encouraged DOTs to modify them to meet specific state specifications. In generic IC specifications, FHWA recommended test section evaluations to verify the mixture volumetric value and determine a compaction curve of the asphalt mixtures in relationship to number of roller passes and to the stiffness of mixture while meeting the xxDOT in-place compaction requirements. Also, FHWA defined the target IC-MV as the point when the increase in the IC-MV of the material between passes is less than 5 percent on the compaction curve. As an IC construction operations criteria, FHWA recommended that a minimum coverage of 90% of the individual construction area shall meet or exceed the optimal number of roller passes and 70% of the individual construction area shall meet or exceed target IC-MV values determined from the test section.

Since then, many DOTs have developed their asphalt IC specifications similar to the FHWA generic specifications. Some modifications are basically focused on the terms of payments and compaction acceptance. California DOT uses intelligent compaction rollers for documenting the following values of asphalt mixture compaction:

1. Number of roller passes
2. Asphalt mixture temperature for first coverage of breakdown compaction
3. Asphalt mixture temperature at the completion of intermediate compaction

California DOT regulates not to collect intelligent compaction measurement values for stiffness when the compacted asphalt mixture layer is less than 0.15 foot. When asphalt mixture thickness is 0.15 foot or greater, intelligent compaction rollers provide additional real time quality control for asphalt mixture density based on intelligent compaction measurement value correlated to the specified asphalt mixture target density. California DOT also specifies that the number of roller passes, asphalt mixture temperature and intelligent compaction measurement values are report only and are not used for compaction acceptance.

As shown in Table 2.2, DC DOT uses the Project Percent Coverage (PPC) to control the partial payments, which is defined as the percent of required daily compaction area where the minimum required cumulative measurement pass count is achieved.

Table 2.2 DC DOT PARTIAL PAYMENTS

Payment Schedule		
Item	Criteria	% of Lump Sum Paid
A	Certification of the IC Supervisor(s), Onsite IC Support and Operators of the Instrumented Rollers	10% payment
B	Instrumented Roller(s) Approved for Use	10% payment
C	Measurement Pass Completion	
	Project Percent Coverage (PPC) ≥ 80%	80% payment
	Project Percent Coverage (PPC) < 80%	% Payment = 0.8×PPC+24

The payment schedule of MNDOT is similar to the DC DOT's method, except that MNDOT adopts the 70% PPC to divide the different payments. MNDOT also evaluate thermal profile segments for the purpose of quality management. Using paver mounted infrared temperature equipment, the segment temperature differential is determined by masking out paver stops and then cutting the lowest 1% and the highest 1.5% of the readings for each 150-foot segment. Basing on the degree of temperature differential, monetary price adjustments are made as shown in Table 2.3.

Table 2.3 Temperature differential and monetary adjustment

Segment Temperature Differential		**Monetary Adjustment**
Range	**Category**	**Adjustment per 150-ft Segment**
< =25.0°F	Good	$20 incentive
> 25.1 and < = 50°F	Moderate	No pay adjustment
> 50 °F	Severe	$20 disincentive

Since IC is equipment-based technology, the specifications must be flexible enough to deal with the varied capabilities of IC rollers and properties of compacted materials, and new specifications should be developed in order to take full advantage of IC's benefits.

CHAPTER 3 EXPERIMENTAL TESTING METHODS

Field spot tests and laboratory are both essential to provide correlation to IC measurements. Various IC specifications have recommended specific in-situ spot tests for verification tests and obtaining IC target values. The in-situ and laboratory tests for soil and asphalt mixtures are described in the following sections.

3.1 In-situ Testing Methods for Soils

Three different in-situ testing methods were employed to evaluate the in-situ soil physical and mechanical properties:

• Portable Soil Moisture Meters,

• Core cutter method,

• Falling Weight Deflectometer (FWD).

Core Cutter Method

A cylindrical core cutter is a seamless steel tube. For determination of the dry density of the soil, the cutter is pressed into the soil mass so that it is filled with the soil. The cutter filled with the soil is lifted. The mass of the soil in the cutter is determined. The dry density is obtained as:

$$\rho = \frac{\gamma}{1+w} = \frac{(M\ /V)}{1+w} \qquad (3\text{-}1)$$

Where M= mass of the wet soil in the cutter

V= internal volume of the cutter

w= water content.

Falling Weight Deflectometer

FWD testing was performed by applying one seating drop using a nominal applied contact stress of about 390 kPa followed by three test drops each at a nominal applied contact stress of about 390 kPa, 590 kPa and 800 kPa. The actual applied force was recorded using a load cell. A composite modulus value was calculated using measured deflection at the center of the plate using Eq. 3-2.

$$E = \frac{(1-\eta^2)\sigma_0 r}{d_0} \times F \qquad (3\text{-}2)$$

where E = elastic modulus (MPa),

d_0 = measured settlement (mm),

η = Poisson's ratio,

σ_0 = applied stress (MPa),

r = radius of the plate(mm),

F = shape factor depending on stress distribution

(a) (b) (c)

Figure 3.1. In-situ testing methods for soil used on the project, (a) Portable Soil Moisture Meters, (b) Core cutter method, (c) FWD

3.2 In-situ and Laboratory Testing Methods for Asphalt Mixtures

Several different in-situ and laboratory tests were employed to evaluate the physical and mechanical properties of asphalt mixtures as shown in Figure 3.2.

- Calibrated nuclear gauge (NG) to determine NG density of asphalt mixtures,
- Cores test to determine the bulk density of asphalt mixtures in the laboratory,
- Constant-head permeability test at laboratory,
- Indirect Tensile Strength (IDT) at laboratory.

Nuclear Density Gauge (NG)

The nuclear density gauge (NG) was used to measure the densities of asphalt mixtures. The nuclear density gauge measures the in-place material density based on the gamma radiation. NG usually contain a small gamma source (about 10 mCi) such as Cesium-137 on the end of a retractable rod.

The device consists of a hander, a retractable rod, the frame, a shielding, a source, and a Geiger-Mueller detector. The source emits gamma rays that interact with electrons in the asphalt pavement through absorption, Compton scattering, and the photoelectric effect. The detector (situated in the gauge opposite from the handle) counts gamma rays that reach it from the source. Then, the received number of gamma rays by the detector is correlated to the density of asphalt mixtures.

Permeability Test

The constant-head permeability test method was used to test the permeability of cores. In this test, water is forced by a known constant pressure through a core specimen of known dimensions and the rate of flow is determined (Aiban & Znidarčić, 1989). The permeability "K", is calculated from the following formula:

$$K = Q/iAt \qquad (3\text{-}3)$$

Where Q = quantity of water discharged

i = the hydraulic gradient = H/L

where: H = head of water

L = height of sample

A = cross-sectional area of specimen

t = elapsed time

Indirect Tensile Strength (IDT)

Evaluating the tensile strength of asphalt concrete is important since tension is developed at the bottom of the asphalt layer under traffic loads. IDT test is applied by using two curved loading strips moving with a rate of deformation of 51 mm/min. (2 in./min.). Tensile stresses are developed in the horizontal direction, and when these stresses reach the tensile strength, the specimen fails in tension along the vertical diameter (Mamlouk, Zaniewski, & Peng, 2006). The test was performed under ambient temperature. The indirect tensile strength is calculated as:

$$\sigma_t = \frac{2P}{\pi t D} \qquad (3\text{-}4)$$

Where σ_t= tensile strength, MPa (psi)

P = load at failure, N (lb)

t = thickness of specimen, mm (in.)

D = diameter of specimen, mm (in.)

(a)

(b)

(c)

(d)

Figure 3.2. In-situ and laboratory testing methods for asphalt mixtures (a) Calibrated nuclear gauge (NG), (b) Corelock system for cores test, (c) Constant-head permeability test, (d) Indirect Tensile Strength (IDT)

CHAPTER 4 DATA ANALYSIS METHODS

Simple linear and multiple linear regression analyses was performed to develop correlations between ICMVs and various in-situ test results. Geostatistical semivariogram analysis was performed on spatially referenced ICMVs to evaluate the spatial uniformity of the IC measurements and to quantify the compaction quality of soil and asphalt mixtures. The original Witczak model and the multilayer pavement analysis software BISAR were employed to calculate the displacements of the newly placed asphalt layer and underlying layers under a vibratory roller. A brief overview of these analysis methods and tools is provided below.

4.1 Regression Analysis

Simple linear and multiple linear regression analyses between ICMVs and in-situ test results were developed by spatially pairing the data using Veta software and JMP software. Veta is a software tool for intelligent construction data management (ICDM). Veta's functionality includes viewing and analyzing intelligent compaction (IC) and paver-mounted thermal profile data. One can overlay the data onto a map of the site, perform various statistical analyses, and create reports. JMP is a computer program for statistics developed by the JMP business unit of SAS Institute.

The simple linear analysis was performed by considering point test results as "true" independent variables and ICMVs as dependent variables using the models shown in Equation 4.1,

$$\text{ICMV} = b_0 + b_1 * Point\ test \qquad (4\text{-}1)$$

where b_0 = intercept, b_1 = regression parameter.

However, the relationship between ICMV and point test results may be complicated, and more additional factors should be considered such as the moisture content of soil or the temperature of asphalt. Under such situation, multiple linear regression analysis was applied to better understand the influence factors for the compaction. In multiple linear regression, it attempts to model the relationship between two or more independent variables and a response variable by fitting a linear equation to observed data. Every value of the independent variable x is associated with a value of the dependent variable y.

$$Y = b_0 + b_1 * x_1 + b_2 * x_2 + \cdots . + b_n * x_n \tag{4-2}$$

where b_0 = intercept, b_1, $b_2 \ldots b_n$ = regression parameter

Statistical significance of the independent variable was assessed based on p values. The selected criteria for identifying the significance of a parameter included: p-value < 0.05 = significant. The best fit model is determined based on the strength of the regression relationships assessed by the coefficient of determination (i.e., R^2) values.

4.2 Geostatistical Analysis

Achieving a uniform and desirable density has been adopted as the criterion for the asphalt pavement compaction over the years. Univariate statistics are typically used to describe the uniformity of the compacted asphalt nowadays. However, univariate statistics are incapable of addressing the issue of spatial uniformity (Gringarten & Deutsch, 2001; Yarus & Chambers, 2006). Two datasets with identical means and variances can have distinct spatial characteristics, therefore, it is necessary to combine the method of geostatistical analysis to better quantify the spatial uniformity, to improve the process control and to identify the poorly compacted locations during the asphalt compaction.

A fundamental assumption of geostatistics is the existence of spatial autocorrelation (Olea, 2006), which can be simply described as the phenomenon that in the vicinity of large values there are other large values, while small values may close to other small values. Although constant material inputs are generally used in pavement design, the engineering properties of asphalt mixtures can vary significantly in the spatial direction. Geostatistical analysis tools including semivariogram model are useful for evaluating the spatial variation and the actual performance of asphalt layer (Gringarten & Deutsch, 2001). An increasing number of researches have been conducted to understand the effect of spatial variability on the actual performance of pavement structures (Thompson & White, 2007; Vennapusa, White, & Morris, 2009). However, detailed information with accurate location is essential for the geostatistical analysis, and the conventional point-wise measurements for asphalt compaction are difficult to meet the requirements (AASHTO, 2001).

IC can provide the real-time spatially referenced compaction measurements with 100% coverage, which is a radical change from the conventional spot density measurements of the asphalt layer. The machine-ground interactions are evaluated by sensors such as accelerometer or torque sensor, and recorded as ICMV with a default data mesh size around 1.0m*0.15m (White et al., 2008). The spatially referenced ICMV data offer the opportunity to perform the geostatistical analysis on the asphalt layer.

Unlike univariate statistics, geostatistics is a branch of statistics focusing on spatial datasets which can characterize and quantify spatial variability. The semivariogram is a common tool used in geostatistical studies to describe spatial relationships in many earth science applications. It is defined as one-half of the average squared differences between data values that are separated at a certain distance (Isaaks & Srivastava, 1989). If this value is calculated repeatedly for as many different values of distance as the sample data support, a semivariogram plot can be obtained as shown in Fig. 1 (Vennapusa et al., 2009). The mathematical expression to achieve the experimental semivariogram $\gamma(h)$ is as follows:

$$\gamma(h) = \frac{1}{2n(h)} \sum_{i=1}^{n(h)} [z(x_i + h) - z(x_i)]^2 \qquad (4\text{-}3)$$

Where h= lag distance; $z(x_i)$=measurement taken at location xi; n(h)=number of data pairs for lag distance h of a specific lag area (Olea, 2006). The distances between pairs at which the semivariogram is calculated are called lags. For instance, lags may be calculated for samples that are 10 m apart, then samples that are 20 m apart, then 30 m, etc. In this case the distance between lags is 10 m.

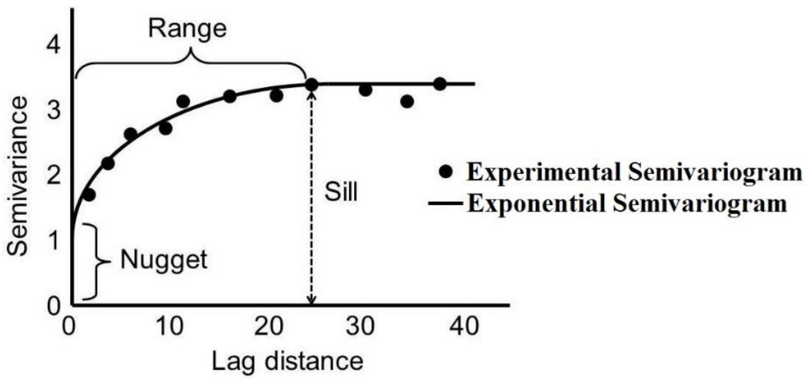


Figure 4.1. Typical sample semivariogram

Three main parameters of a semivariogram plot as shown in Figure 4.1 are *Range*, *Sill* and *Nugget* respectively. As introduced before, a fundamental assumption of geostatistics is the existence of spatial autocorrelation. Therefore, as the separation distance increases, the corresponding semivariogram value usually also increases. However, at certain distance, the semivariogram no longer increases and reaches a plateau. The distance for the semivariogram reaching the plateau is called the *Range*. Sample locations separated by distances farther apart than the *Range* are not spatially autocorrelated, whereas locations closer than the *Range* are autocorrelated, therefore, longer range values indicate better spatial continuity. As for the *Sill*, it is defined as the plateau that the semivariogram reaches at the range. The sill for a semivariogram is approximately equal to the variance of the data, measuring how far a set of data are spread out from its mean. Theoretically, the value of semivariogram is equal to zero at h=0; however, variability in very short scale may result in a significant dissimilarity between sample values separated by extremely short distances. The *Nugget* is applied to describe a discontinuity at the origin of the semivariogram caused by this phenomenon.

To give an algebraic formula for the relationship between semivariogram values at specified distance, a theoretical model (the exponential semivariogram curve in Figure 1) is usually fitted to the experimental values. Some commonly models to fit an experimental semivariogram include linear, spherical, exponential and Gaussian models. Among these models, the exponential model fits well with most of the experimental semivariograms (Vennapusa et al., 2009). In this study, the exponential model is utilized to fit the experimental semivariograms.

If the data values are not stationary and show a systematic trend, this trend need to be removed prior to modeling a semivariogram (Vennapusa et al., 2009). To increase the data's univariate normality which be required by many interpolation and simulation methods, the data can be converted to normal scores (Olea, 2006). After the transformation, the data will have a normal distribution with a mean of zero and a variance of one. However, this operation is optional, and the meaning of the semivariogram parameters may be difficult to interpret after the transformation. In this study, both the semivariograms before and after the transformation were analyzed and compared to find a preferable model to evaluate the asphalt compaction.

Geostatistics can also be used to predict a value at unsampled locations based on values at sampled data. Kriging is a stochastic interpolation procedure to create "smoothed" contour maps of ICMV, which can be used to analyze the nonuniformity and compare the maps. In this research, Veta software and ArcGIS software will be utilized to perform the geostatistical analysis. ArcGIS is a geographic information system (GIS) for working with maps and geographic information. It is used for: creating and using maps; compiling geographic data; analyzing mapped information; sharing and discovering geographic information; using maps and geographic information in a range of applications; and managing geographic information in a database.

4.3 The Witczak Model and BISAR Software

The Witczak dynamic modulus predictive model and the multilayer pavement analysis software BISAR were utilized in this study to evaluate the effects of underlying layers on CMV. BISAR was developed by Shell Company to calculate the stresses and strains in roads. The Witczak model was developed to estimate the dynamic modulus of asphalt mixture from simpler material properties and volumetric (Andrei, Witczak, & Mirza, 1999). In this model, the dynamic modulus at a given loading frequency and temperature depends on a number of factors such as the viscosity of the asphalt, the effective asphalt content and the percentage of air voids, which can be expressed as follows:

$$\log|E^*| = -1.249937 + 0.02923\rho_{200} - 0.001767(\rho_{200})^2 - 0.002841\rho_4 - 0.05809V_a - 0.082208\frac{V_{beff}}{V_{beff}+V_a} + \frac{3.871977 - 0.0021\rho_4 + 0.003958\rho_{38} - 0.000017(\rho_{38})^2 + 0.00547\rho_{34}}{1+\exp(-0.603313 - 0.313351\log(f) - 0.393532\log(\eta))}$$

(4-4)

Where

E^* = dynamic modulus, 10^5 psi (68.9 MPa);

η = asphalt viscosity, 10^6 Poise;

f = loading frequency, Hz;

V_a = air void content, %;

V_{beff} = effective binder content, % by volume;

ρ_{34} = cumulative % retained on the 19-mm sieve;

ρ_{38} = cumulative % retained on the 9.5-mm sieve;

ρ_4 = cumulative % retained on the 4.76-mm sieve;

ρ_{200} = % passing the 0.075-mm sieve.

CHAPTER 5 IC COMPACTION PROJECTS

This chapter summarizes all the projects of soils and asphalt mixtures compacted using IC technology and the statistical results using Veta.

5.1 Projects Description

Project 1. CNM194 Crockett Co. resurfacing project

Location: SR20 (US412) from Dyer County line (L.M. 0.0) to Lyons/Birmingham Rd. (L.M.8.39)

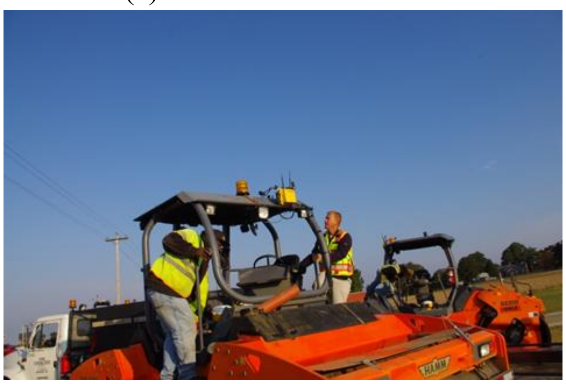
Length and width : 33.56 total lane miles; 4-lane divided; No milling

Asphalt mixture type: PG70-22 411TLD

Time: October 2013

In this project, all IC equipment was supplied by Trimble Company. The GPS base station had been set up before the construction day with a covering scope around 2.5 miles in radius. There were two HAMM HD 120 double drum rollers for the compaction, equipped with GPS receiver and antenna, HCQ GPS navigator, and two temperature sensors. It should be noted that only one roller, which was responsible for the breakdown compaction, installed an accelerometer attached on the front drum. The second roller did not equip the accelerometer. When the compaction began, the HCQ GPS navigator showed the location, the compaction pass, and temperature using different color. The IC data will be transferred to the company website every five minutes, so the compaction data can be checked on the website based on the google map, which include compaction pass, temperature, and stiffness.

(a) GPS base station (b) temperature sensor



(c) GPS receiver and antenna (d) GPS navigator

Figure 5.1. Project CNM194

Project 2. CNM278 Lincoln Co. resurfacing project

Location: SR15 (US64) from Giles County line (L.M.0.0) to SR244 (L.M.6.56)

Length and width : 25.5 lane miles; 4-lane divided; Milled

Asphalt mixture type: 1-1/4” PG64-22 411D

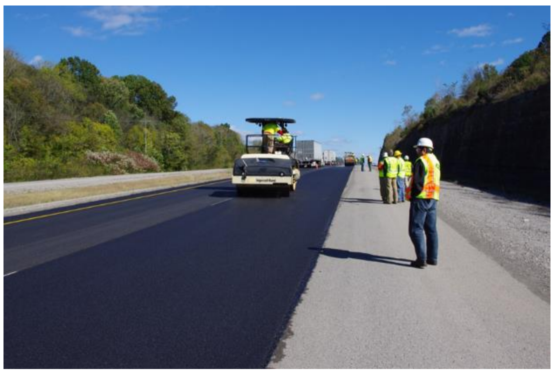
Time: October 2013

In this project, all IC equipment was supplied by Trimble Company. The GPS base station was set up above the hill next to the road. There were one HAMM HD 120 double drum roller for the breakdown compaction, and one INGERSOLL RAND double drum roller for the intermediate compaction. The standard equipment for these rollers includes a GPS receiver and antenna, a HCQ GPS navigator, and temperature sensors.

Only the breakdown roller equipped with an accelerometer.

(a) GPS base station

(b) The breakdown roller



(c) The intermediate roller

(d) GPS navigator

Figure 5.2. Project CNM 278

Project 3. CNM201 Hamilton Co. resurfacing project

Location: S.R. 58 beginning north of Eller Road (L.M. 6.43) and extending to Lakewood Acres Drive (L.M. 9.20)

Length and width : 12.95 lane miles; Milled

Asphalt mixture type: 1-1/4” PG70-22 411D

Time: October 2013

In this project, all IC equipment was supplied by Trimble Company. The GPS base station was set up on the medial strip of the road. There was one SAKAI double drum roller for the breakdown compaction and one CAT double drum roller for the intermediate compaction. The standard equipment for these rollers includes a GPS receiver and antenna, a HCQ GPS navigator, and temperature sensors. Only the breakdown roller had an accelerometer.

(a) GPS base station

(b) The breakdown roller

(c) The intermediate roller

(d) GPS navigator

Figure 5.3. Project CNM201

Project 4. CNM165 Knox Co. resurfacing project

Location: SR331 from McCamey Rd (L.M.0.29) to Pleasant Grove Rd. (L.M. 10.76)

Length and width : 10.47 miles; 2 lanes

Asphalt mixture type: 1-1/4” PG70-22 411D mix from L.M.0.29 to L.M.8.31

0.8” PG70-22 411TLD mix from L.M.8.31 to L.M. 10.76

Time: October 2013

In this project, all IC equipment was supplied by Trimble Company. Two GPS base stations were set up on along the road. There were one HAMM double drum roller for the breakdown compaction and another HAMM double drum roller for the intermediate compaction. The standard equipment for these rollers includes a GPS receiver and antenna, a HCQ GPS navigator, and temperature sensors. Only the breakdown roller equipped with an accelerometer.

Figure 5.4. Project CNM165

Project 5. CNN 006 Greene Co. resurfacing project

Location: U.S. 11E (S.R. 34) from S.R. 348 (L.M. 7.73) to the S.R. 70 ramp (L.M. 13.85).

Length and width : 6.120 miles

Asphalt mixture type: 1-1/4" PG70-22 411D

Time: May 2014

In this project, all IC equipment was supplied by Hamm Company. There was no GPS base station necessary for the Hamm roller, because the GPS receiver on the roller had enough accuracy for the project. There were one HAMM HD 120 double drum roller for the breakdown compaction, and one HAMM HD 90 double drum roller for the finish compaction. The standard equipment for these rollers is GPS receiver and antenna, HCQ

GPS navigator, temperature sensors, accelerometer, and control panel. Unlike the Trimble equipment, the position of HAMM's HCQ GPS navigator is above the head of operator, and HAMM also has a control panel which shows the same data on the navigator. No available IC data were obtained from this project.

(a) The breakdown roller

(b) The intermediate roller

(c) GPS navigator

(d) Control panel

Figure 5.5. Project CNN 006

Project 6. CNN137 Davidson Co. new pavement

Location: Construction of the Traffic Incident Management (TIM) training site facility in Nashville

Time: August 2014

A new pavement project was constructed in Davidson County, which included subbase, base and asphalt layer as shown in Figure 5.6.

In this project, all IC equipment was supplied by Trimble Company. The GPS base station was set up on the site. A CAT single drum roller was used for the subbase and base compaction. One INGERSOLL RAND double drum roller and one BOMAG double drum roller were used for the asphalt mixture compaction.

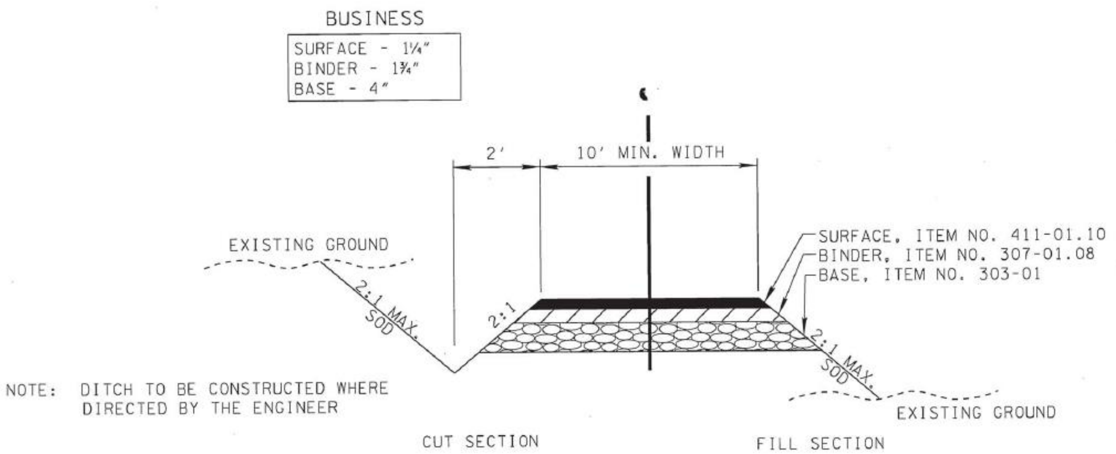


Figure 5.6. Typical section

(a) Roller for soil compaction (b) Roller for asphalt mixture compaction

Figure 5.7. Project CNN137

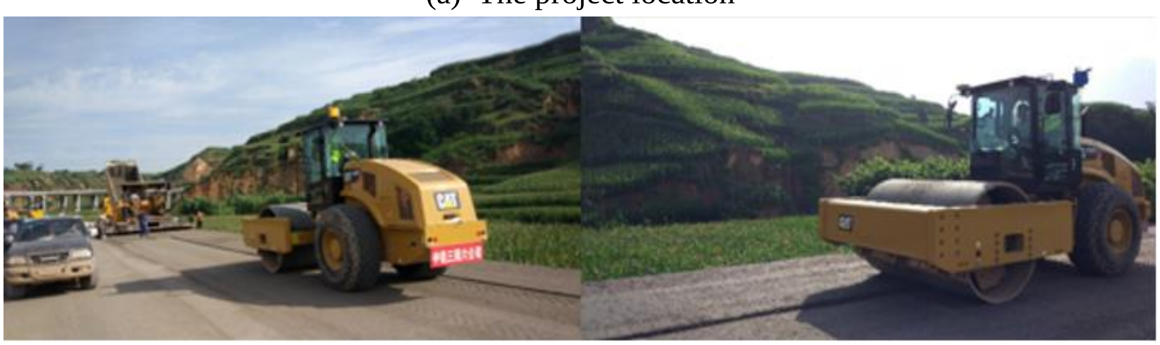
Project 7. Soil compaction project in Shanxi, China

Location: Lvliang, Shanxi province, China

Time: August 2016

A new pavement including base and subbase layer was constructed in Davidson County in 2014; however, no enough in-situ test data were obtained in this project to perform the regression analysis. To evaluate the IC technology for soil compaction, a soil compaction project was constructed in China. In this project, a CAT single drum roller equipped with the same Trimble IC equipment as TDOT projects was adopted for the soil compaction, and three different in-situ testing methods, including soil moisture, core cutter method, and FWD, were employed in this project to evaluate the relationship between ICMV and in-situ tests.

(a) The project location



(b) The roller

Figure 5.8. Soil compaction project

5.2 Statistical analyses using Veta

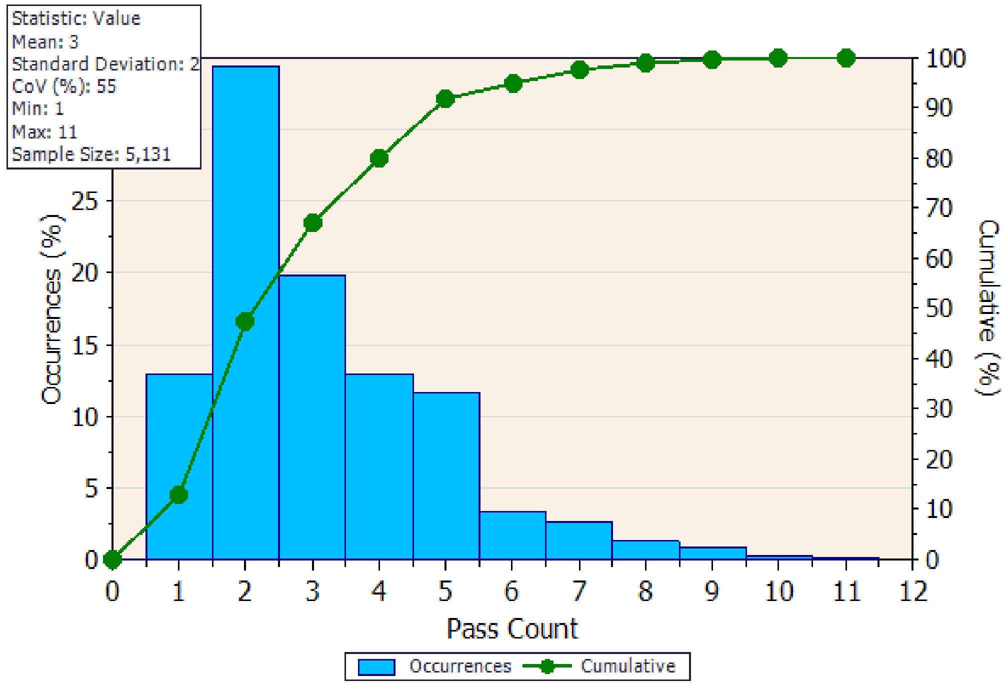
During the compaction, the IC data were uploaded from the roller to the IC provider's website in real time, and both the contractor and the agency can access and analyze the IC data after the compaction. To analyze the IC data, a software naming Veta was utilized in this study. Veta is a software tool for intelligent construction data management (ICDM). Veta's functionality includes viewing and analyzing IC and paver-mounted thermal profile data. One can overlay the data onto a map of the site, perform various statistical analyses, and create reports using Veta.

In Veta, the analysis results of the overall data include Final Coverage, Individual Passes, and All Passes data. One can select a variable to display distribution statistics, a histogram of the data, and correlation (if associated spot tests are available). The ICMV results include a compaction curve if at least 3 passes were selected. The main overall results are as follows:

1.Distribution - Final Coverage

In Veta, the Final Coverage data is shown when select a variable from the Overall section of the menu. On the Distribution tab a table displays statistics for that variable, including mean, standard deviation, coefficient of variation, minimum, maximum, and sample size. If a specification target is enabled for that variable, the table includes the target status (Passed/Failed) and % of target achieved. All Passes results can also include a Distribution analysis.

Histograms show the occurrence frequency of each dataset, based on the center value of each range. Vertical bars show the occurrences of each value. A line shows the cumulative frequency distribution, plotting the percent of data at or below each value. Figure 5.9 illustrates the pass count and CMV distribution of final coverage for Project 1 (CNM194 Crockett Co. resurfacing project) on October 14, 2013.



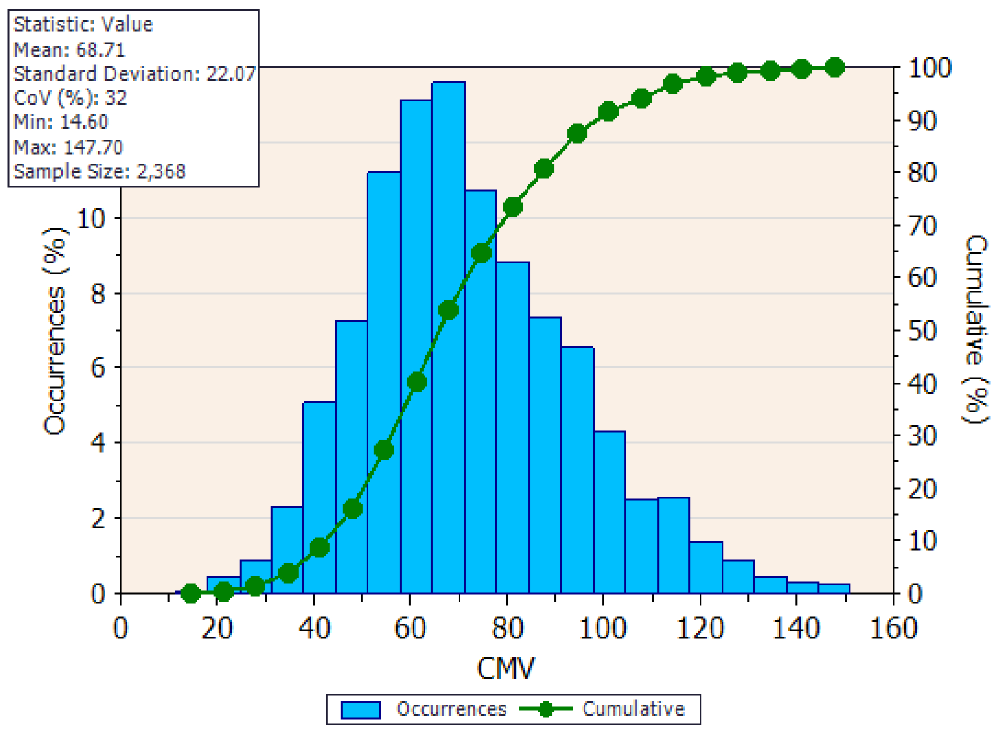


Figure 5.9. The pass count and CMV distribution

2. Correlation - Individual Passes

In Veta, if spot test data was entered for a pass included in the analysis, correlation is calculated. The Correlation tab shows the linear regression between ICMV and in-situ spot tests. ICMV within a circular area (at a radius defined in the Setup) around an in-situ spot test location are extracted for the correlation analysis. The correlation analysis report can be used to determine target value of ICMV. All Passes results can also include a Correlation analysis when associated spot test data exists. Figure 5.10 shows the correlation results between CMV and core density of Project 1.

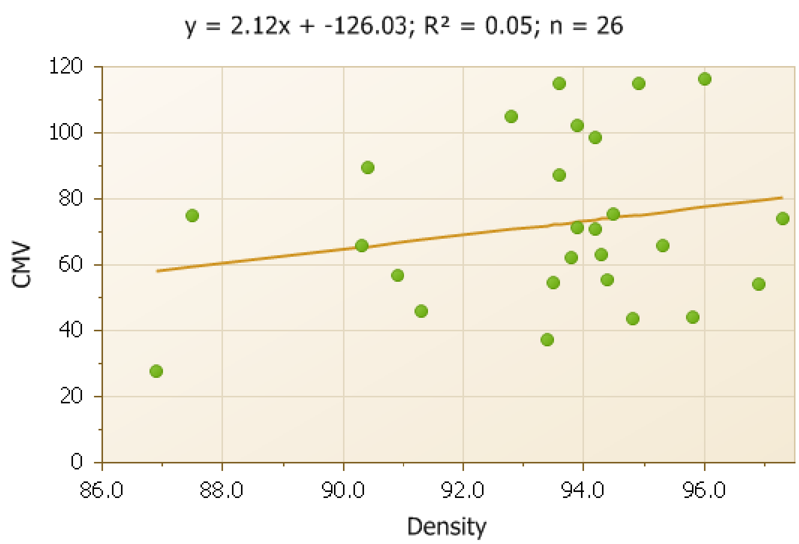


Figure 5.10. Correlation analysis

3. Compaction Curve - All Passes

If Maximum Pass is 3 or greater on the Setup screen, All Passes data is generated. For ICMV there is a Compaction Curve tab in Veta. Compaction curve is a plot of mean ICMV versus roller passes. It can be used to determine optimal roller passes based on test strip data. Figure 5.11 shows the compaction curve of the test strip of Project 3 (CNM201 Hamilton Co. resurfacing project).

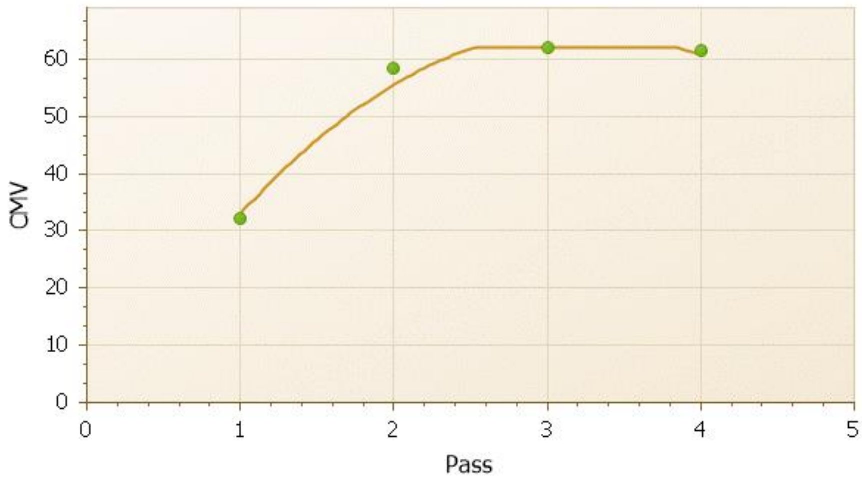


Figure 5.11. The compaction curve of Project 3

4. Semivariogram

The ICMV cells in Veta include a semivariogram analysis. Semivariogram is a metric for uniformity. The Semivariogram tab consists of a table of statistics along with a plot of a theoretical semivariogram model. Fitted parameters include range, sill, vertical scale, and nuggets. Larger range values in combination with lower sill values indicate better uniformity. Figure 5.12 shows the semivariogram of final coverage for Project 1 on October 14, 2013.

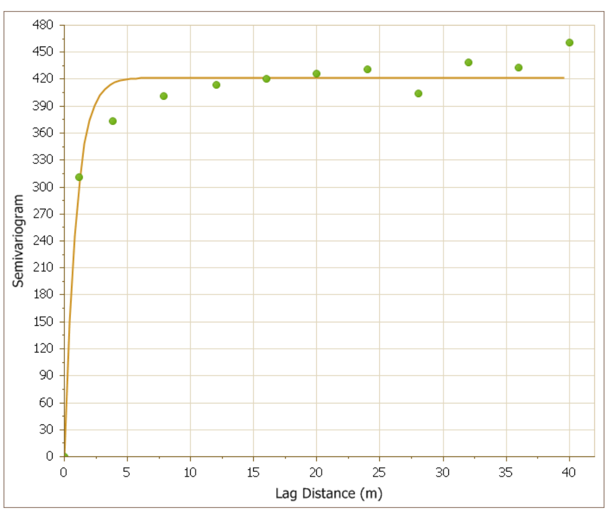


Figure 5.12. The semivariogram of Project 1

The final coverage data of project 1, 2, and 3 were summarized using Veta on a daily basis. The data from other projects were either problematic or too small to be counted on a daily basis. In general, the statistical results of these projects are similar and stable when comparing by date or by project. Table 5.1 to 5.6 show the statistical results of breakdown roller and intermediate roller respectively for each project. As shown in tables, Project 1 had the most stable pass count for both the breakdown roller and the intermediate roller compared to the other projects. Project 2 had the highest surface temperature for both breakdown and intermediate roller, whereas Project 3 had the largest pass count for both rollers when comparing to the other projects.

Project 1

Table 5.1 Statistical results of the breakdown roller (Project 1)

Date	Pass count			Roller Speed (kph)			Temperature (°C)			CMV			Semivariogram	
	μ	σ	CV	μ	σ	CV	μ	σ	CV	μ	σ	CV	Range (m)	Sill
10/14/13	3	2	52	6.1	1.9	31	115.4	16.76	13	68.7	22.08	32	2.8	401.93
10/15/13	3	1	48	6.8	2	29	109.6	18.25	14	61.68	18.8	30	2.4	320.22
10/17/13	3	1	51	6.7	1.9	29	114.2	15.87	12	60.85	20.12	33	2.8	387.08
10/18/13	2	1	52	6.7	1.9	28	118.2	16.26	12	55.62	18.28	33	2.8	308.94
10/21/13	3	1	51	6.9	2.1	30	108.6	18.58	15	63.32	19.8	31	2.4	335.14
10/22/13	3	1	54	6.2	1.9	30	98.6	21.32	18	76.33	26.92	35	3.6	438.67
10/23/13	3	1	52	6.5	2.1	32	99.1	19.5	17	84.17	27.14	32	4.8	495.73
10/24/13	3	1	51	6.7	1.9	28	106.4	17.62	14	70.28	19.29	27	3.6	309.52
10/26/13	3	1	47	6.9	1.9	28	100.1	19.24	16	66.34	17.59	27	3.6	251.17
10/28/13	3	1	46	6.5	2.1	32	102.7	20.33	17	63.79	18.65	29	3.6	280.66
10/29/13	3	1	49	6.7	1.9	29	95.1	18.08	16	63.32	28.61	45	4	512.9
10/30/13	3	1	45	6.7	2	30	85.1	21.08	20	69.12	27.08	39	4.8	608.68
11/1/13	2	1	66	6.7	1.8	27	90.2	17.75	16	63.83	18.78	29	3.6	302.19

Table 5.2 Statistical results of the intermediate roller (Project 1)

Date	Pass count			Roller Speed (kph)			Temperature (°C)		
	μ	σ	CV	μ	σ	CV	μ	σ	CV
10/14/13	2	1	58	5.7	1.9	33	76.8	12.91	14
10/15/13	2	1	55	6.2	1.8	29	75.4	12.4	13
10/17/13	2	1	59	6.3	1.7	27	75.7	10.79	12

10/18/13	2	1	56	6.2	1.7	27	77.7	10.02	10
10/21/13	2	1	59	6	1.7	28	71.3	10.98	12
10/22/13	2	1	59	4.7	1.8	38	63.5	8.43	10
10/23/13	2	1	55	6	1.8	30	65.8	8.14	10
10/24/13	2	1	57	6.3	1.7	27	71.3	11.36	13
10/26/13	2	1	54	5.9	1.8	30	69.3	10.64	12
10/28/13	2	1	61	5.9	1.7	29	75.5	11.34	12
10/29/13	2	1	53	6.3	1.7	27	65.7	10.39	12
10/30/13	2	1	53	6.3	2	31	65.4	10.94	13

Project 2

Table 5.3 Statistical results of the breakdown roller (Project 2)

Date	Pass count			Roller Speed (kph)			Temperature (°C)		
	μ	σ	CV	μ	σ	CV	μ	σ	CV
10/21/13	1	1	40	6.2	2.1	34	126.3	17.05	12
10/22/13	2	1	45	6.9	2.2	32	119.2	13.5	10
10/23/13	3	2	62	6.8	2.2	32	122.7	14.22	10
10/24/13	5	2	44	6.8	2.1	31	123.5	18.91	13
10/30/13	1	1	39	6.4	2.3	36	114.1	15.74	12
11/1/13	3	2	69	6.7	2.3	34	119.3	16.84	12
11/4/13	4	3	68	6.6	2.1	33	118.3	14.69	11
11/5/13	2	1	54	6.8	2.2	33	122.7	13.77	10

Table 5.4 Statistical results of the intermediate roller (Project 2)

Date	Pass count			Roller Speed (kph)			Temperature (°C)		
	μ	σ	CV	μ	σ	CV	μ	σ	CV
10/21/13	7	3	36	5.8	3.2	55	83.5	12.03	12
10/22/13	5	2	45	5.9	1.8	31	82.7	13.14	13
10/23/13	6	2	40	5.7	1.8	32	91.8	14.59	13
10/24/13	6	2	38	6	1.9	32	92.2	16.24	15
10/30/13	6	2	37	5.8	1.8	31	80.5	11.71	12
11/1/13	6	2	37	5.5	2	36	85.3	11.08	11
11/4/13	6	2	40	5.9	1.8	31	81.8	11.21	11
11/5/13	6	2	40	5.8	1.9	32	81.9	11.46	12

Project 3

Table 5.5 Statistical results of the breakdown roller (Project 3)

Date	Pass count			Roller Speed (kph)			Temperature (°C)			CMV			Semivariogram	
	μ	σ	CV	μ	σ	CV	μ	σ	CV	μ	σ	CV	Range (m)	Sill
10/22/13	5	2	34	6.7	2.1	30	100.1	11.7	10	65.31	11.4	17	7.2	100.54
10/23/13	7	2	33	6.1	1.9	31	89.5	13.9	13	60.55	15.43	25	4.4	208.04
10/24/13	5	3	47	6.2	4.9	79	82.2	17.56	18	55.48	20.23	36	5.6	338
10/28/13	6	2	30	5.1	1.5	30	79.4	9.47	10	67.88	11.33	17	3.18	124.95
10/29/13	4	4	87	7.7	3.1	40	92.4	25.08	23	42.22	19.7	47	6.8	61.92
10/30/13	8	4	46	7.5	4.2	55	115.3	16.45	12	42.45	23.97	56	4.4	482.68
10/31/13	5	2	45	7.8	2.5	32	113	17.3	13	47.27	20.59	44	6.8	353.09
11/1/13	6	4	63	7.8	4.6	59	106.7	25.65	21	42.63	18.97	45	4	227.09

Table 5.6 Statistical results of the intermediate roller (Project 3)

Date	Pass count			Roller Speed (kph)			Temperature (°C)		
	μ	σ	CV	μ	σ	CV	μ	σ	CV
10/22/13	7	3	35	7.1	5.6	79	69.1	10.34	12
10/23/13	6	2	34	9.1	6.5	72	66.7	10.06	12
10/24/13	6	2	36	8	5.9	74	75.2	13.33	14
10/28/13	9	3	33	9	9	100	61.2	8.64	11
10/29/13	7	3	44	9.1	7.1	79	69.6	11.71	13
10/30/13	6	3	47	9.7	5.7	58	80.6	13.57	14
10/31/13	6	3	42	9.8	6.5	66	79.4	15.7	16
11/1/13	7	3	44	9.8	6	61	75.9	13.56	14

5.3 Summary

A total of six asphalt mixture projects and one soil project were constructed to evaluate the IC technologies for various materials. All asphalt mixture projects were performed in Tennessee, and the soil project was finished in China to provide additional data for the soil compaction.

To analyze the IC data, a software naming Veta was utilized in this study. Veta is a software tool for IC which functions include viewing and analyzing IC and paver-mounted thermal profile data. One can overlay the data onto a map of the site, perform various statistical analyses, and create reports using Veta.

The final coverage data of project 1, 2, and 3 were summarized using Veta on a daily basis. In general, the statistical results of these projects are similar and stable when comparing by date or by project. Project 1 had the most stable pass count for both the breakdown roller and the intermediate roller compared to the other projects. Project 2 had the highest surface temperature for both breakdown and intermediate roller, whereas Project 3 had the largest pass count for both rollers when comparing to the other projects.

CHAPTER 6 CORRELATION ANALYSIS FOR SOIL COMPACTION

One of the objectives of this research is to evaluate the IC technologies for compaction of embankment subgrade soils, aggregate base, and asphalt paving materials. However, the finished IC projects in Tennessee are mainly focused on the asphalt layer compaction. The only project containing part of soil compaction is Project 6 in Davidson County with no field tests obtained. Therefore, in August 2016, a project using the same IC technology as that for the Tennessee projects was constructed in Shanxi Province, China to evaluate the IC technology for soil compaction.

6.1 Project Description and In-situ Tests

The project contained subgrade soil and aggregate subbase compaction. It should be noted that unlike the aggregate subbase in Tennessee, the aggregate subbase and base in this project were stabilized using cement. Therefore, this study focused on the subgrade soil compaction using IC technology. The test section of the project had a length of around 460 m. Figure 5.8 shows the IC roller for this project. The compaction properties of subgrade soils were analyzed by the standard Proctor test. The subgrade soil compaction test results are shown in Table 6.1 and Figure 6.1.

Table 6.1. Compaction test results of subgrade soil

No	Water content (%)	Dry density (g/cm^3)
1	9.5	1.73
2	11.2	1.77
3	12.7	1.82
4	15	1.76
5	16.8	1.72

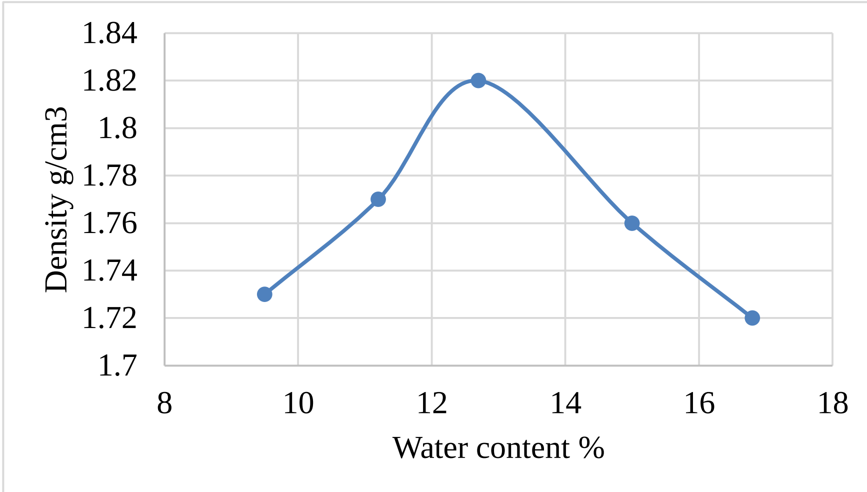


Figure 6.1. Soil compaction curve

According to the soil compaction curve in Figure 6.1, the maximum dry density of subgrade soil is 1.82 g/cm^3, and the optimum moisture content is 12.8% correspondingly. Based on the laboratory compaction test results, the field test section for subgrade soil was divided into four segments with different moisture contents. The test section arrangement is shown in Table 6.2.

Table 6.2. The field test arrangement for subgrade soil compaction

Segment No	1	2	3	4
Water content (%)	High	Low	High	Low
Length (m)	150	150	80	80

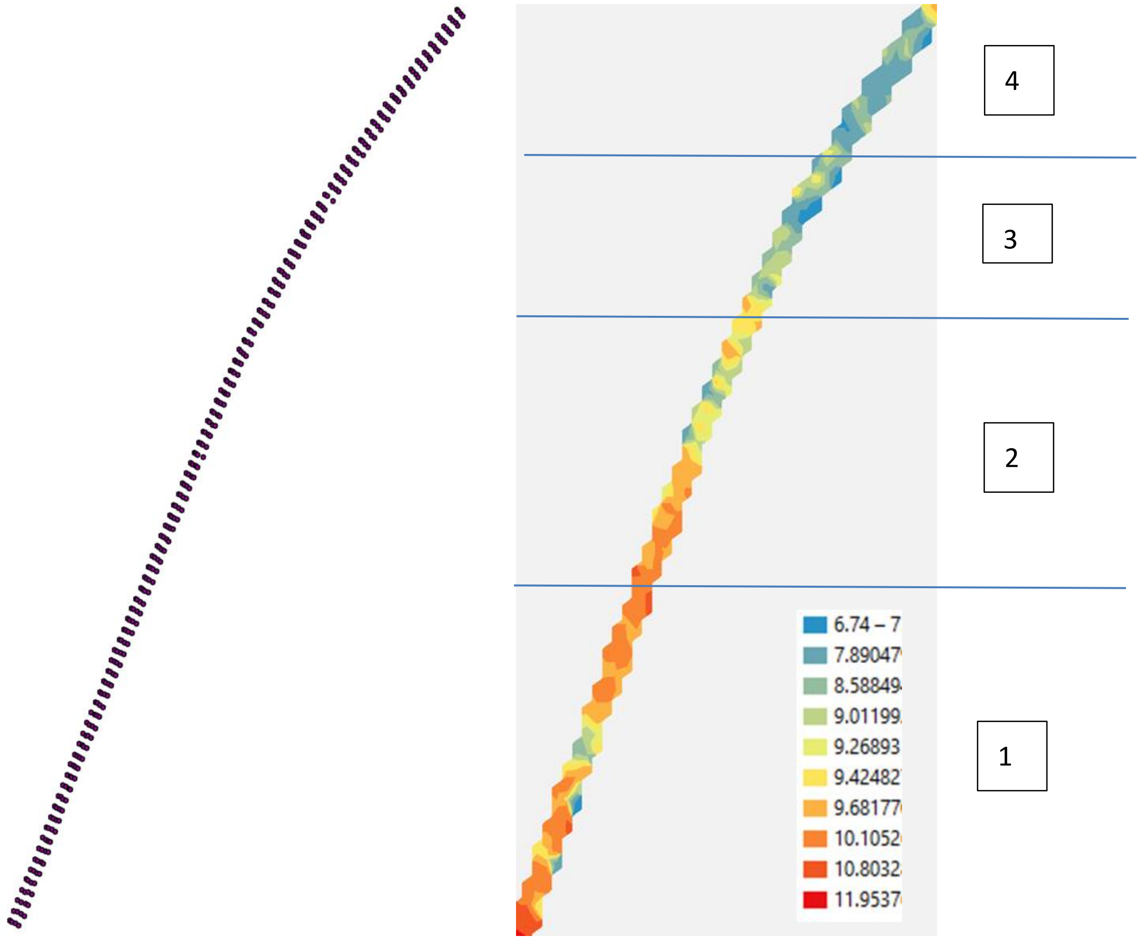
Field tests including density, moisture content, and falling weight deflectometer (FWD) were performed on the subgrade soil layer, and correlation analyses were performed between the IC measurements and the different on-site tests. Before the compaction, the water contents were measured by a Portable Soil Moisture Meter with a frequency of four points on the same cross section every 10 meters. There was a total of 370 points including water content and conductivity. The compaction work began immediately with the end of the test. After the compaction, 15 points were selected for each segment based on the CMV value, and the compaction degree (The dry density) and water content were measured using Core Cutter Method. There was a total of 60 densities data. The deflection values at the same locations were measured later by FWD test. There was a total of 60 deflection data.

6.2 Correlation analyses for soil compaction

Three different in-situ testing methods were employed to test the water content, the dry density, and the deflection of subgrade soil. Regression analyses between CMV and in-situ test results were further performed in this study to evaluate the IC technologies for soil compaction

1. Water content (Before the compaction)

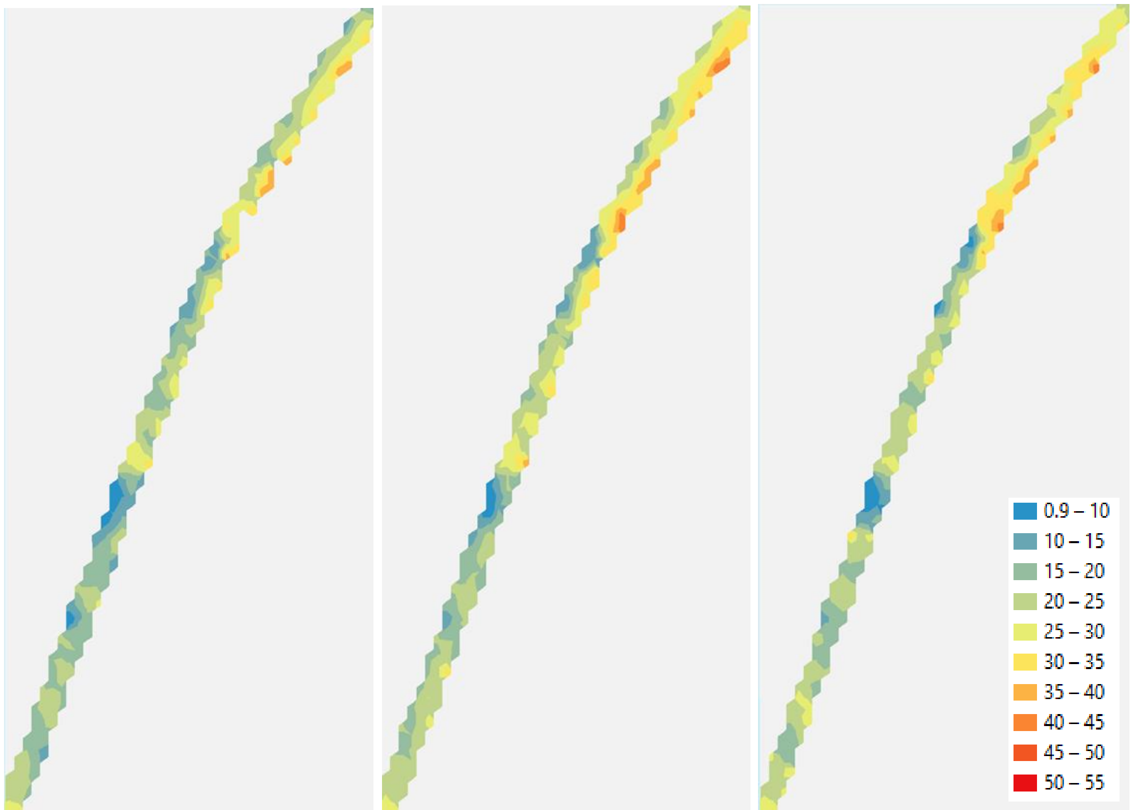
A total of 370 water content tests were performed before the compaction. The distribution of test points is shown in figure 6.2 (a). Kriging is a stochastic interpolation procedure to create “smoothed” contour maps of data. Using ArcGIS, the contour map of water content was produced as shown in figure 6.2 (b). It can be seen from figure 6.2 that the water content gradually decreases from the segment 1 to 4, which is not exactly the same as the original plan. The reason for that is the considerable time required for the test which was started with the segment 1. When the test ended in segment 4, the water content was dropped significantly.



(a) Test points (b) Contour map of water content

Figure 6.2. The water content of soil before the compaction

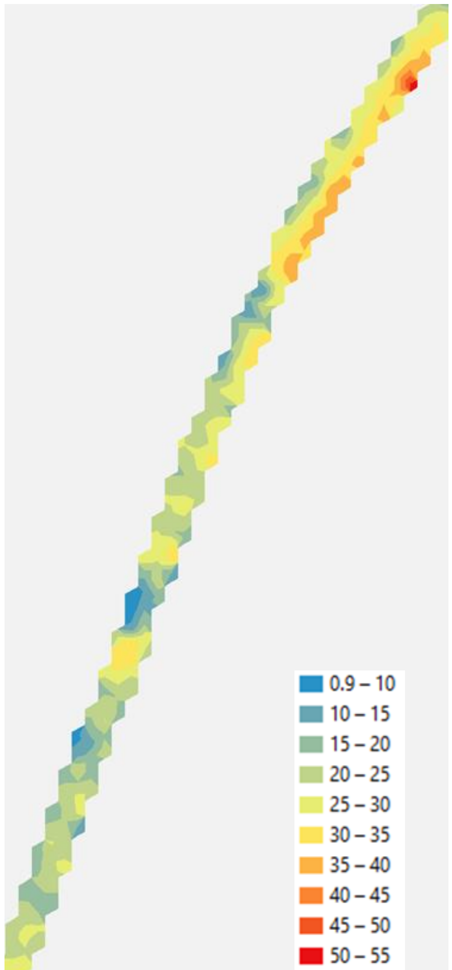
Using the same ArcGIS tool, the kriged contour maps of CMV for different passes during the compaction were produced. As shown in figure 6.3, the segment 1 had a relatively small average CMV value, whereas the segment 3 and 4 maintained a higher average CMV value during the compaction. It can also be observed that with the progress of compaction, the CMV value usually increased for the same location indicating an increase of compaction degree.

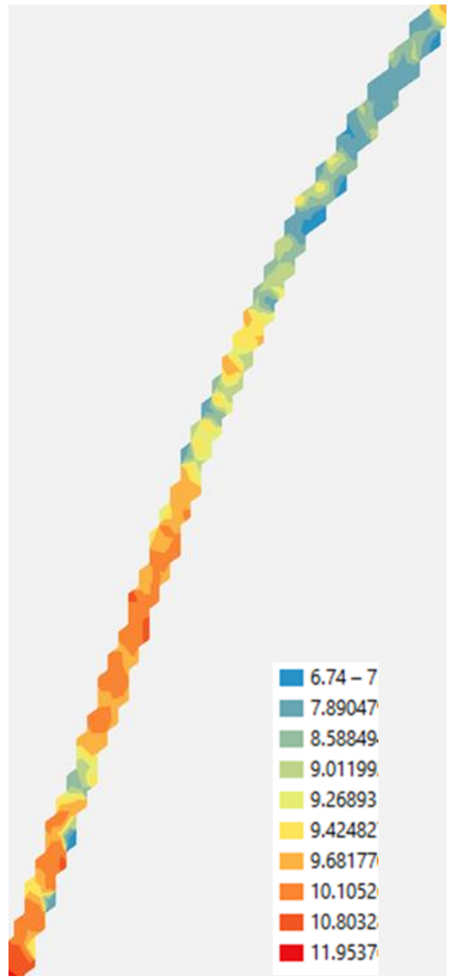


(a) Pass 1 (b) Pass 2 (c) Pass 3

Figure 6.3. The contour maps of CMV

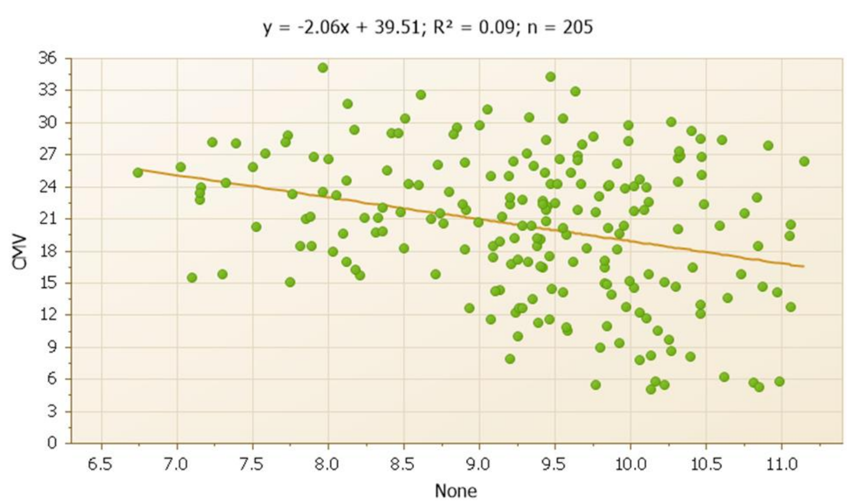
Figure 6.4 compares the contour map of the final coverage of CMV and the water content. It can be observed that areas with high CMV values tend to have low water contents, and vice versa, which means a correlation may exists between CMV and water content during the compaction. Veta software was utilized to perform the correlation analysis between CMV of different pass and water content. The analysis results were shown in figure 6.5. It can be seen form the figure that the water content of soil affects the CMV value, and with the progress of compaction, the R square value increases from 0.09 to 0.17.



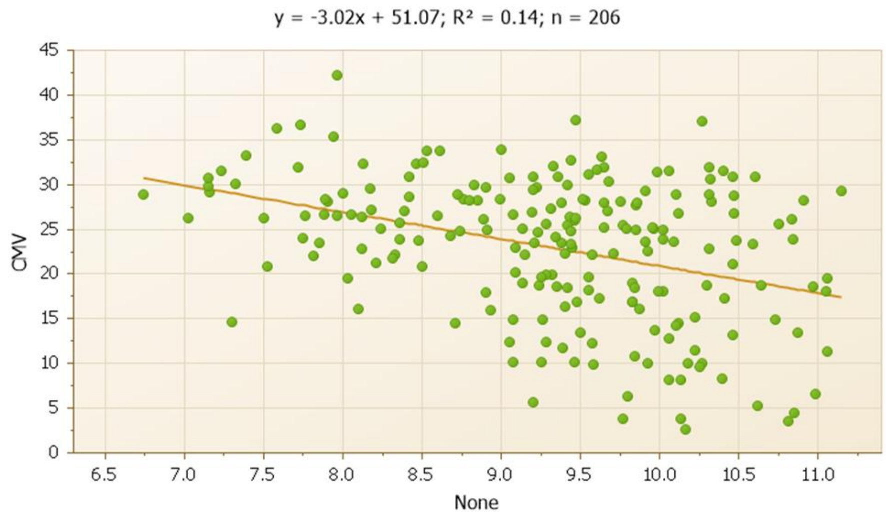


(a) The final coverage of CMV (b) Water content

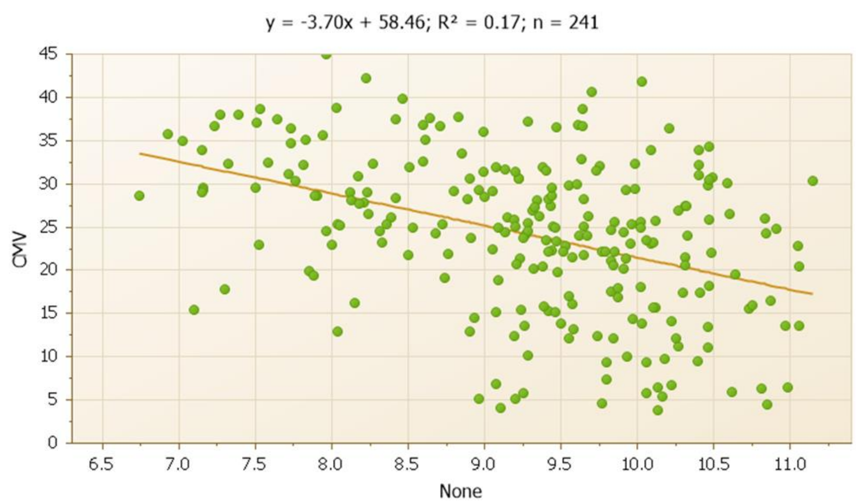
Figure 6.4. A comparison between CMV and water content



(a) Pass 1



(b) Pass 2

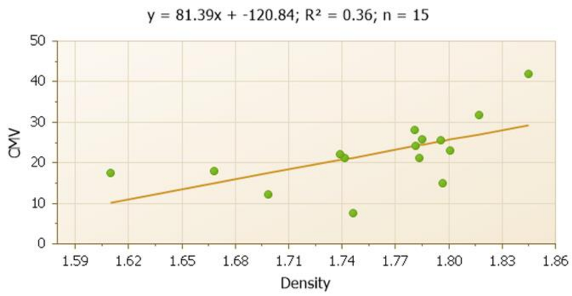


(c) Pass 3

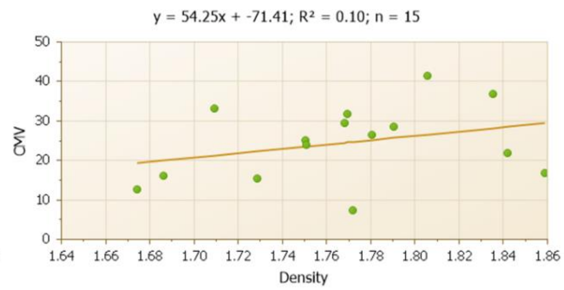
Figure 6.5 Correlations between CMV and water content

2. Dry density

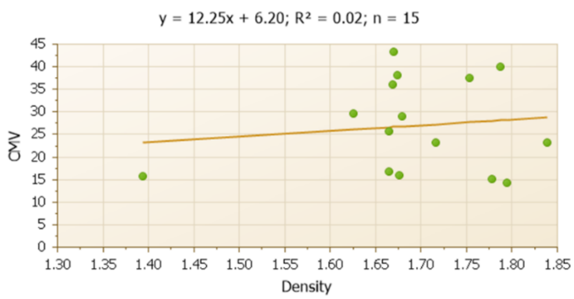
A total of 60 denisty tests were performed after the compaction using core cutter method. Correlation analyses between density and CMV were performed separately according to each segment. The analysis results were shown in figure 6.6.



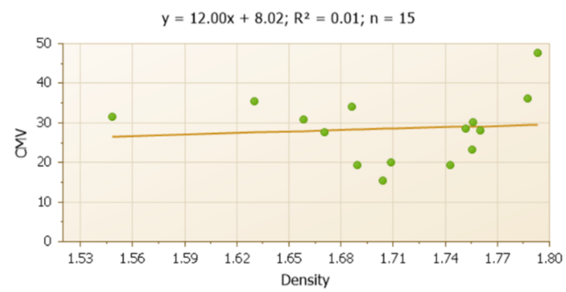
(a) Segment 1



(b) Segment 2



(c) Segment 3



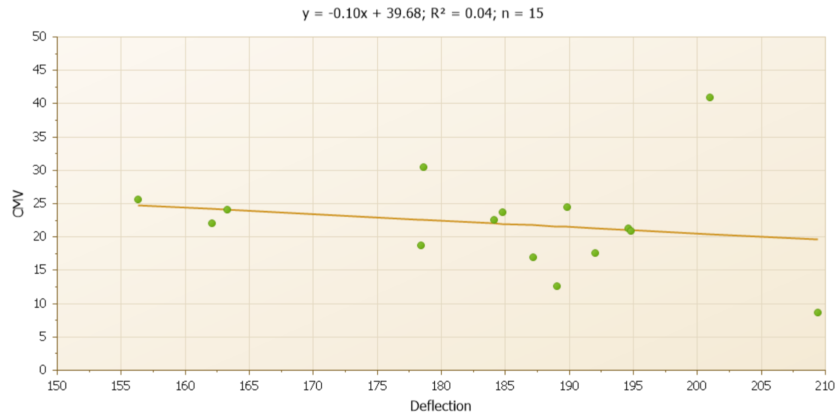
(d) Segment 4

Figure 6.6 Correlations between CMV and density

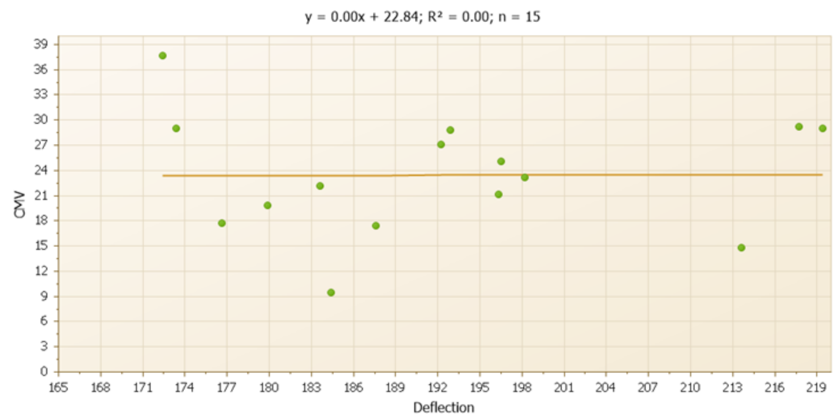
As shown in figure 6.6, the relationship between CMV and density varies among different segments. The segment 1 had the strongest linear relationship with R^2=0.36, whereas no relationship could be found for segment 3 and 4. The multiple linear regression analyses were further performed to include both density and water content of soil. It was found that adding the factor of water content improved the correlation for most segments, with an increase of R^2 from 0.10 to 0.28 for segment 2 and 0.01 to 0.18 for segment 4 respectively.

3. Deflection test

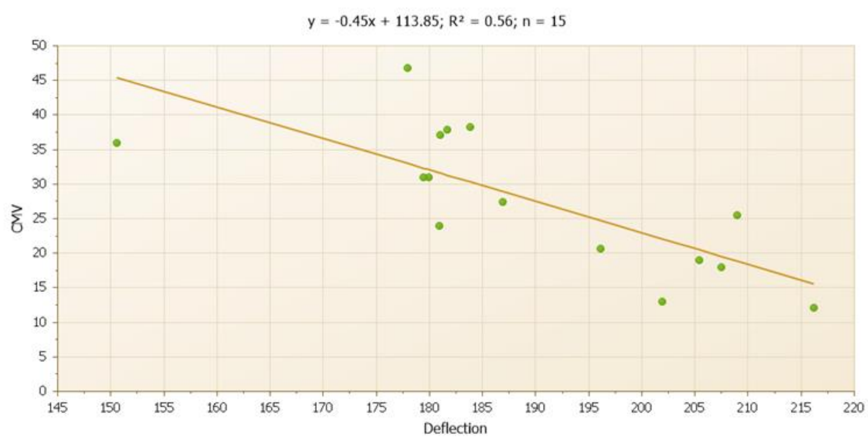
A total of 60 deflection tests were performed later after the density tests had been finished. Correlation analyses between deflection and CMV were performed separately according to each segment. The analysis results were shown in figure 6.7.



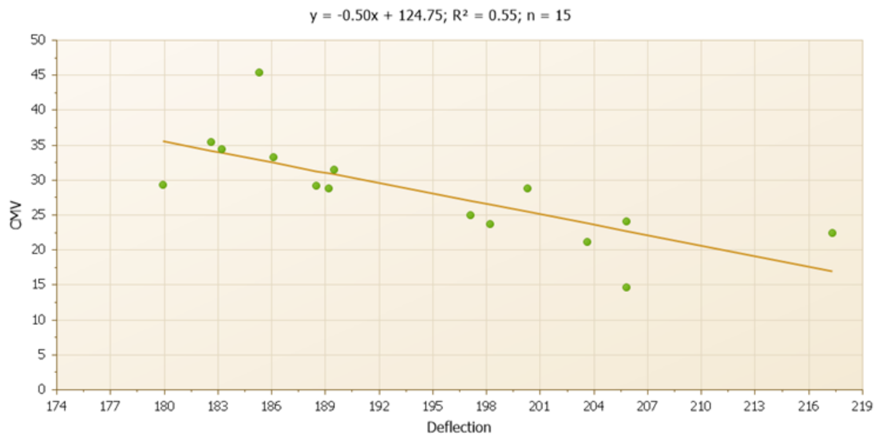
(a) Segment 1



(b) Segment 2



(c) Segment 3



(d) Segment 4

Figure 6.7 Correlations between CMV and deflection

As shown in figure 6.7, the relationship between CMV and deflection varies significantly among segments. There is no relationship between CMV and deflection for segment 1 and 2, whereas a strong linear relationship could be identified for segment 3 and 4 with R^2=0.56 and 0.55 respectively. A reasonable explanation is that when the CMV was recorded, the water contents of segment 1 and 2 were high. However, after the compaction and density test, the water content had dropped significantly during the FWD test. Therefore, the deflection results represented the soil bearing capacity at low water content, whereas the CMV results represented the soil stiffness at high water content. The segment 3 and 4 kept a relatively low and constant water content throughout the whole tests; therefore, yielded a strong linear relationship between CMV and deflection.

6.3 Comments and Summary

For the soil compaction, some linear correlations have been found between ICMV and various types of soil in-situ point measurement values. Kröber et al. (2001) found that the correlations from calibration test strips between ICMV and static plate load test (PLT) initial and reload modulus showed a strong correlation with $R^2 > 0.9$. Based on average measurements over the length of the test strip (20 m), White et al. (2007) defined the correlations between ICMVs and Point densities (R^2 = 0.87). Mooney et al. (2010) concluded that simple linear correlations between ICMVs and point measurement values were possible for a soil layer with a relatively homogenous underlying layer. The practices in this study also support these findings.

In this study, the relationships between the CMV of soil layer and in-situ point measurement values were thoroughly examined. The water content of soil affects the CMV value, and the relationship between CMV and density varies among different segments. A strong and stable linear relationship between CMV and deflection could be identified when the water content of soil was consistent. When using IC roller to compact the subgrade soil, there exists a strong relationship between CMV and deflection, which is hard to find during the asphalt layer compaction. Our findings are consistent with previous researches, and it shows the potential of utilizing IC measurements for QA of soil compaction.

CHAPTER 7 CORRELATION ANALYSIS FOR ASPHALT COMPACTION

According to previous researches, strong linear relationships between ICMVs and Point-MVs of asphalt pavements are rarely found, especially for the density measurements. Practices also support these findings: Six asphalt pavement projects were constructed using IC rollers, and thirty cores were randomly selected and measured for each project after the compaction. There is no direct correlation between ICMVs and core densities for all the projects. Many factors may contribute to the inconsistent correlation between ICMV and pavement density. These include the non-linear behavior of IC roller for certain roller parameters and road stiffness combinations; the effect of pavement temperature on ICMV while NG/core measurements are independent of asphalt temperature; and the measuring depth of ICMV.

In this study, the relationships between the ICMV of asphalt layer and other factors were thoroughly examined. Using the data of Project 3, the relationships between ICMV and other roller operation parameters such as roller frequency, speed and amplitude were examined first, then the effects of asphalt temperature and underlying support on ICMV were analyzed further by utilizing the original Witczak model and the multilayer pavement analysis software BISAR.

7.1 The effects of roller operation parameters

As discussed in Chapter 2, CMV is a dimensionless compaction parameter that partly depends on roller operation parameters such as frequency, amplitude, and speed. Therefore, the relationships between ICMV and other roller operation parameters such as roller frequency, speed and amplitude were examined using Project 3 data.

For the project 3, a high frequency double drum vibratory roller equipped with IC system, SAKAI SW880, was used as the breakdown roller. It was operated at a frequency of 69 Hz and an amplitude of around 0.5 mm throughout the project. The breakdown roller was kept in the vibration mode most of the compaction time to obtain adequate CMV data for different passes. Another vibratory roller, CAT CB54B, was used for intermediate compaction which was also equipped with IC but in a static mode. All IC data for both rollers during construction, including roller passes, temperature and CMV, were recorded and stored during the compaction. It should be noted that no CMV data can be obtained when the roller is in the static mode.

According to a primary examination of IC data, most of the resurfacing areas were compacted for three passes by the breakdown roller in the vibration mode. After that, only part of areas was compacted for an extra pass and most of these passes were in the static mode. Therefore, the amount of CMV data dropped significantly after the third pass. In this study, the relationships were examined not only between CMV and roller speed, frequency and amplitude, but also between CMVs for different passes. Subsequently, only data points with valid CMV value for all the three passes were selected for further analyses. Figure 7.1 shows the distribution of CMV for the first pass of the breakdown roller. Many data points with large CMV values occurred at the boundary of pavement or at the junction of lanes, which is attributed to the fact that part of the drum crossed the range of resurfacing area during the compaction. These abnormal data were also filtered before defining the correlation between CMV and other parameters.

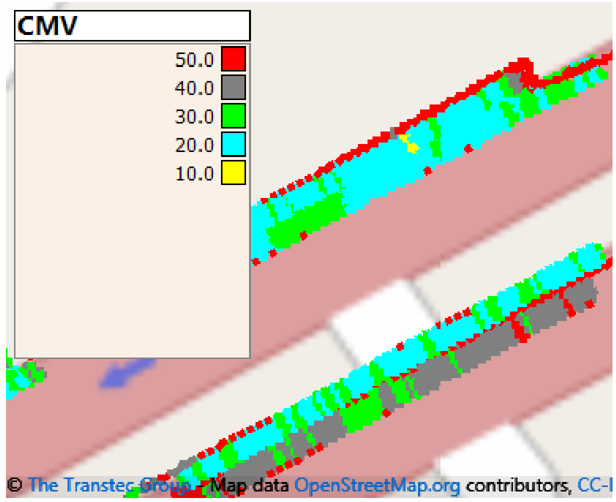


Figure 7.1. Distribution of CMV for the first pass

The data of the first pass of one single day, October 22, 2013, were utilized first to examine the potential relationships between CMV and roller operation parameters including roller speed, RMV, frequency, amplitude, and surface temperature. The results are shown in Figure 7.2. There are not significant linear relationships between CMV and speed, RMV, frequency, and surface temperature. However, there is a significant correlation between CMV and amplitude. These findings are verified by IC data from other passes in the same day and also IC data from other days. In general, the correlation patterns between CMV and amplitude are very consistent for the whole project (R^2 = 0.45~0.65). It should be noted that similar correlation patterns between CMV and amplitude cannot be identified among the rest projects.

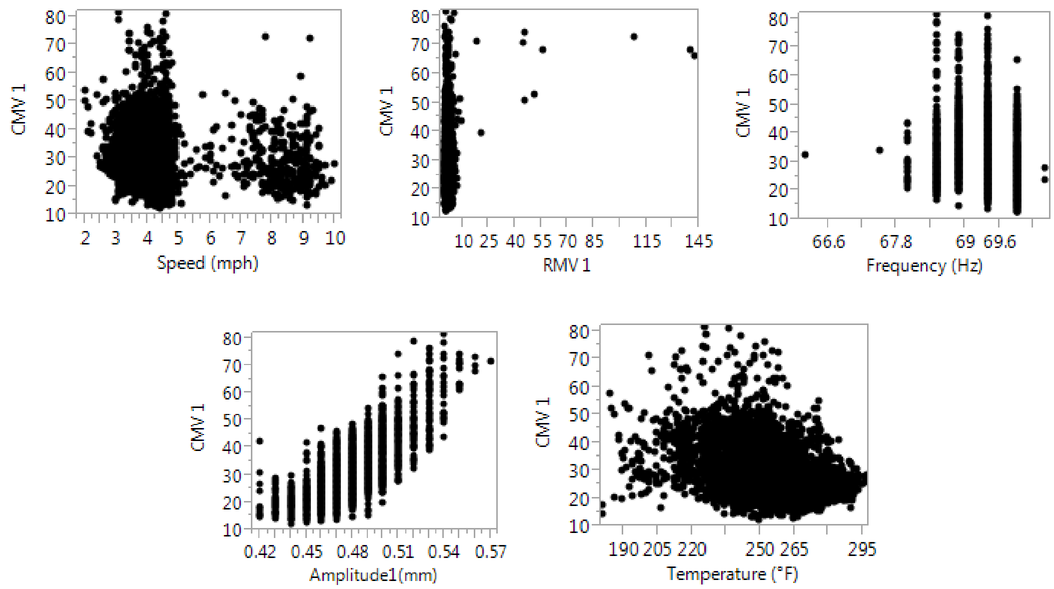
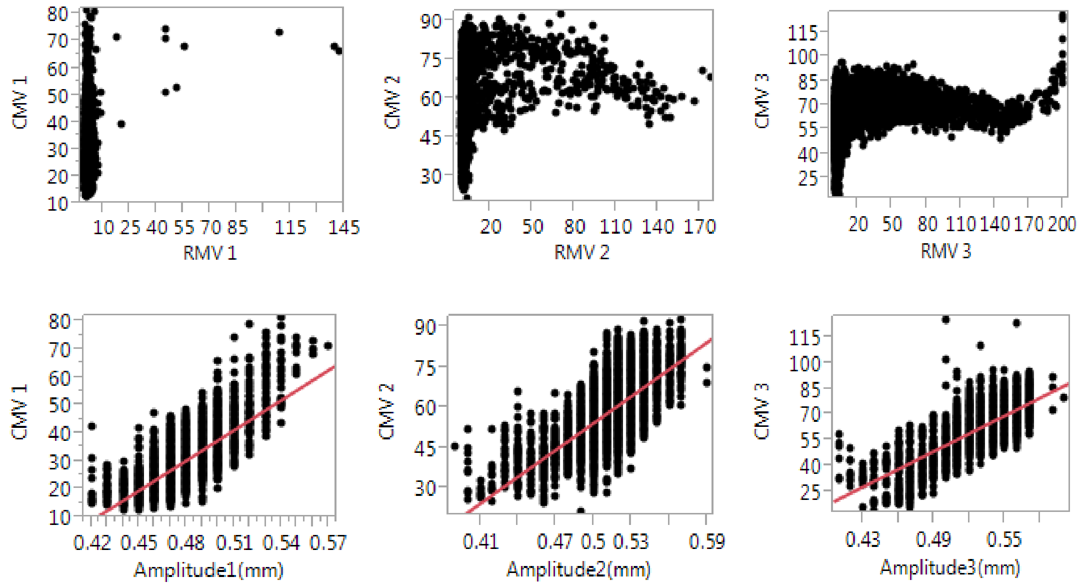


Figure 7.2. Correlations between CMV and other variables

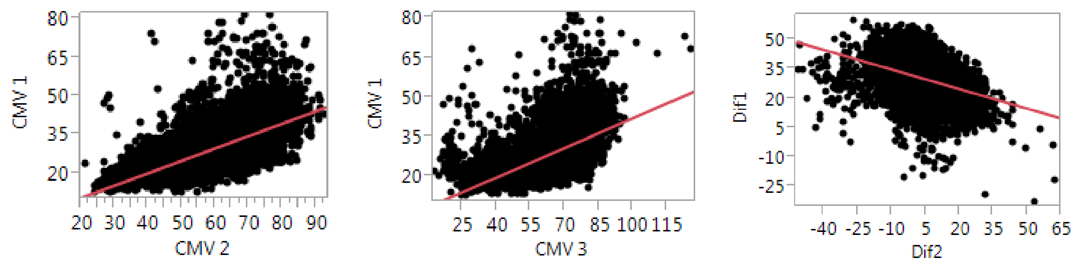
Figure 7.3 shows the relationships between CMV and RMV and amplitude for different passes. Apparently, the relationships between CMV and amplitude almost keep the same for all passes and the relatively large R^2 suggests that the correlation is strong. Sandström (1994) performed a numerical simulation of a vibratory roller on a cohesionless soil. By using the simulation program, he demonstrated that the low amplitude does not put the roller into the double jump mode and the CMV increases monotonously with the stiffness of the soil. In contrast, the high amplitude will force the roller into the double jump mode and the CMV vs. stiffness curve will show a discontinuity. It is important that the roller keeps in the same stable mode, and the same amplitude is used when utilizing CMV for compaction verification. The relatively small amplitude and a consistent frequency of this project offer the possibility of using CMV to evaluate the compaction level. The small RMV value for the first pass in Figure 7.3 also indicates that the drum behavior is in a continuous contact or partial uplift mode for the first pass; however, an increasing number of points have larger RMV for the subsequent passes due to the increasing stiffness of asphalt layer during the compaction.



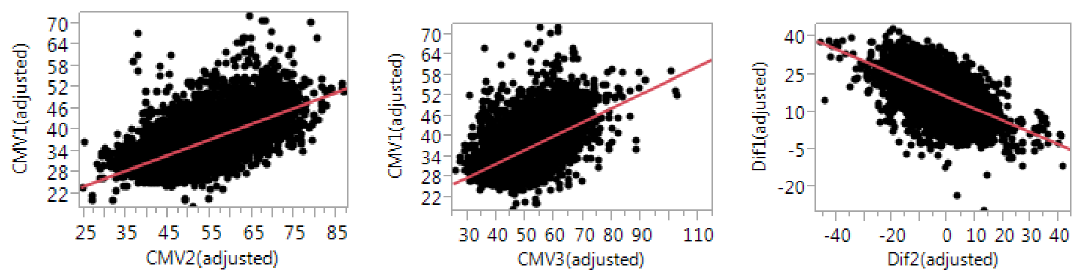
(a) Pass 1 (b) Pass 2 (c) Pass 3

Figure 7.3. Relationships between CMV vs. RMV and amplitude for different passes

Since it is important to keep the same amplitude when evaluating compaction quality, the average value of coefficients of linear fit results between CMV and amplitude is used to adjust the CMV value for all roller passes. The relationships between the CMVs for different passes and the relationship between the CMV differences of the same points are analyzed for both raw data and adjusted data (CMV1, 2 and 3=CMV value of pass1, 2 and 3; Dif1=the difference between CMV1 and CMV2, Dif2=the difference between CMV2 and CMV3). According to Figure 7.4, linear relationships could be found between the CMVs of different passes for both raw data and adjusted data; however, the adjusted data offers relatively stronger and more consistent results than the raw data. The positive relationships among different CMVs reveal that the total stiffness of pavement to a certain depth will increase after continuous rolling. An inverse correlation can be found between Dif1 and Dif2. It may suggest that when the CMV value of certain point rises significantly from pass 1 to pass 2, there will be no further increase on the CMV value for the subsequent pass, and vice versa, so at the end of compaction a more uniform distribution of CMV will be achieved.



(a) Raw data



(b) Adjusted data

Figure 7.4. Relationship between CMVs for different passes

The adjustment of CMV can also help to identify the situation of over compaction. When a vibratory roller passes over pavement materials that have been already adequately compacted, it can perceptively or imperceptibly bounce on the pavement surface and actually cause a reduction in mixture density, but this reduction of density is hard to perceive in real-time using the traditional spot tests. In figure 7.3, the number of points with large RMV value increases significantly in the third pass, which means that the roller was in a double jump mode or even a chaotic mode and the vibratory compaction actually damaged the quality of these areas. This phenomenon can also be tracked and evaluated by the compaction curve. The compaction curve fits the mean CMV versus roller pass number by the two times polynomial model, so the optimum roller pass number can be identified (Xu, Chang, & Gallivan, 2012). Figure 7.5

shows the compaction curves for both the original and adjusted CMV. If the original CMV mean value is used, the mean CMV still increases at the third pass, suggesting that the third pass still helps to increase the density of resurfacing asphalt layer. However, a more accurate compaction curve obtained from the adjusted CMV suggests that two passes for this resurfacing compaction are enough. Considering a temperature drop from the second pass to the third pass will also result in an increase on asphalt mixture stiffness, the third pass definitely causes a reduction in mixture density.

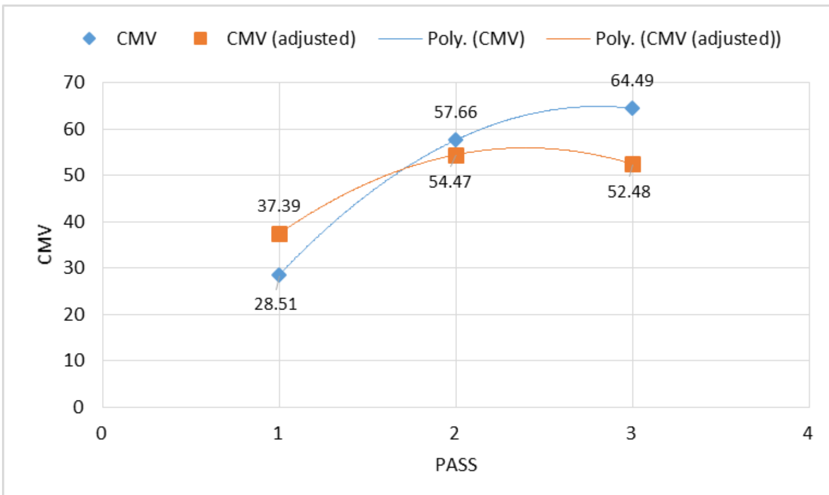


Figure 7.5. The compaction curve

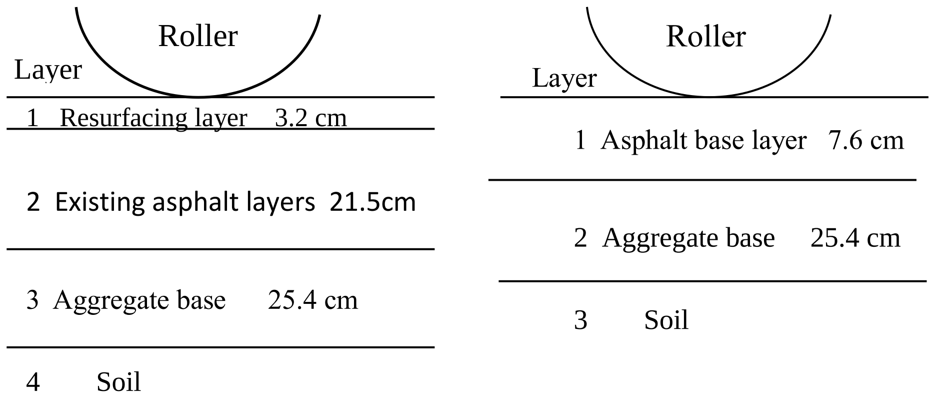
7.2 The effects of mixture temperature and underlying support

In addition to the effect of machine settings, the CMV measurement is also influenced by the conditions of underlying layers and mixture temperatures. During compaction, a vibration roller puts a parabolic load on a rectangular area of asphalt surface, and a "cylinder deformation modulus" Ec can be recorded as CMV which is an integral value to a certain depth (HF Thurner, 2001). If all moduli of underlying layers are known, by using some analytical stress calculation methods such as "substitute height method", theoretically the CMV measurement can be related to the modulus of surface layer (Vennapusa et al., 2009). As mentioned before, the effect of machine settings can be eliminated using the corrected CMV value, and the effect of temperature can be monitored using the IC technology and considered in the asphalt mixture material model (Wiser, 2010). However, the accurate moduli of underlying layers are hard to acquire, and a uniform underlying support rarely occurs in real projects.

To evaluate the effects of mixture temperature and underlying support, two asphalt pavement projects were selected for the research, which included one resurfacing project (Project 3), and one new asphalt base layer project (Project 6).

The resurfacing project (Project 3) using the IC technology was performed in Hamilton County, Tennessee in October 2013. A 3.2 cm thick overlay was applied on an existing asphalt pavement road in accordance with the TDOT specifications. Figure 7.6 (a) illustrates the layer structures of this resurfacing project. This type of asphalt pavement resurfacing has been successfully applied on resurfacing program by TDOT for many years.

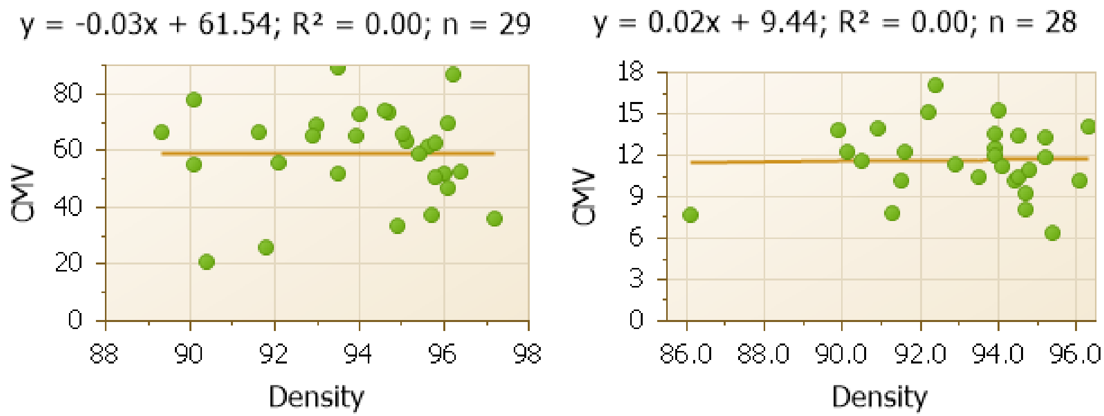
The new asphalt base layer (Project 6) using the IC technology was performed in Davidson County, Tennessee in August 2014. An asphalt base layer of 7.6 cm thickness was applied on an aggregate base layer in accordance with the TDOT specification. Figure 7.6 (b) illustrates the layer structures of this project. A high frequency double drum vibratory roller equipped with IC system, Ingersoll Rand DD-110HF, was served as the breakdown roller. The roller was operated at a frequency of 56 Hz and at an amplitude of approximately 0.43 mm. The breakdown roller was kept in vibration mode most of the compaction time. Another vibratory roller, BOMAG 278AD, was used for intermediate rolling in static mode.



(a) Project 3 (b) Project 6

Figure 7.6. Road structure cross sections

As shown in Figure 7.6, these two projects represent two typical situations for asphalt pavement compaction. One is compacting a thin asphalt overlay on thick existing asphalt layer, and the other is compacting a thick asphalt layer on aggregate base. Thirty cores were randomly selected after the whole compaction for each project and the lab densities (G_{mm}%) were measured (using saturated surface dry method or the CoreLok method according to the specific situation of cores). There is no direct correlation between the Final Coverage CMVs and core densities for these two projects as shown in Figure 7.7. To explore any possible correlations, the nearest IC data for each core location were selected (Each IC data point represents a default data mesh size of 1.0m*0.15m for these two projects), and the CMVs for the first pass and the last pass of the breakdown roller were used to perform the correlation analyses with the core densities again separately. Still no significant relationships could be found. Factors such as asphalt pavement temperature and underlying support need to be considered before the CMV data can be practically applied to guide the compaction level of asphalt pavement.



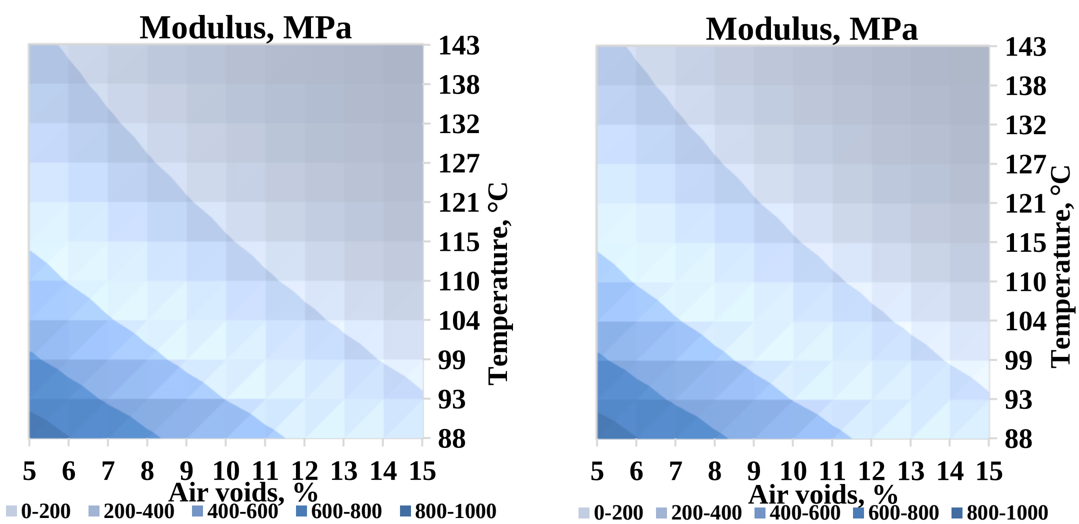
(a) Project 1 (b) Project 2

Figure 7.7. Relationships between CMVs and core densities

In order to utilize CMV as an indicator for evaluating the compaction quality of asphalt pavement, two issues have to be properly solved. One is the different nature of measured properties: while the CMV measures the stiffness of pavement as a mechanical property, the density is a physical nature of asphalt mixtures. The other issue is that the CMV measurement is influenced by multiple factors including the machine settings (i.e. amplitude, frequency), conditions of underlying layers and asphalt pavement temperatures. During compaction, a vibration roller puts a dynamic parabolic load on a rectangular area of asphalt pavement surface, and a “cylinder deformation modulus” Ec can be obtained as an integral modulus to a certain depth utilizing the IC technology (HF Thurner, 2001). For layered pavement structures, if moduli of elasticity for all layers are known, some analytical calculation methods such as “substitute height method” make it possible to calculate the stress and strain behavior in the pavement, and in turn the expected measured Ec can be calculated (Roland Anderegg & Kaufmann, 2004). Therefore, the CMV measurement can be related to the modulus of the surface layer theoretically, if all moduli of the underlying layers are known. Further, there are many material models that relate the dynamic modulus of asphalt mixture to parameters such as temperature, asphalt content, and air voids content (Wiser, 2010). Therefore, for the first question, even if the measured properties are different, there is still a possibility to correlate CMV with asphalt pavement density (air voids content).

However, this relationship is hard to find due to the second issue. The CMV measurement is influenced by multiple factors including the machine settings, conditions of underlying layers and asphalt pavement temperatures. Keeping the roller parameters constant can control the effect of machine settings, and the effect of temperature can be monitored using the IC technology and considered in the asphalt material model; however, the effect of underlying support requires careful consideration because the variation of underlying layers is almost unknown for most projects.

To evaluate the effects of underlying layers on CMV, the influence depth of CMV measurement needs to be clarified first. Previous studies gave inconclusive results regarding the measurement depth of a vibration roller. For 10 ton rollers, Floss et al. (1991) found the approximate measurement depth is 0.6 to 1.0 m, whereas Brandl and Adam recommended a measurement depth of 0.6 to 0.8 m in 2000 (M. A. Mooney et al., 2010). Anderegg and Kaufmann (2004) stated in 2004 that an IC roller can be assessed at a depth of 1,000 times the amplitude, that is, for a 0.5 mm of amplitude in this project, approximately 50 cm of depth is measured. To simplify the problem, both projects in this research adopted a measurement depth of 50 cm when calculating the deformations under the roller. During pavement compaction, the density of new asphalt layer increases with the increase of passes. At the same time, the pavement temperature drops. All these factors affect the modulus of the asphalt layer. Using the job mix formula and roller parameters of each project, the dynamic modulus of new asphalt mixture layer was calculated for the possible range of air voids and temperature during compaction utilizing the Witczak model. The range of temperature was from the recorded data of IC, and the range of air voids was determined based on the nuclear gauge (NG) test results during compaction. As shown in Figure 7.8, the dynamic modulus of the resurfacing asphalt layer of project 3 varied between 60 and 900 MPa from the beginning to the end of compaction, whereas the dynamic modulus of base asphalt layer of project 6 varied between 10 and 700 MPa for the same air voids and temperature range. As for the old underlying asphalt layer, the temperature of asphalt layer was close to the ambient temperature, and the air voids were approximately 4% after a long service life. Using a typical design formula of Tennessee and available records, the dynamic modulus of the underlying old asphalt layer in project 3 varied between 20,000 and 50,000 MPa. Since the entire project was milled prior to paving, the underlying asphalt aging is not considered (Farrar, Harnsberger, Thomas, & Wiser, 2006). The variation of resilient modulus for the aggregate base and soil are set as between 100 and 200 MPa, and 50 and 100 MPa respectively based upon the available data of projects.



(a) Project 3 (b) Project 6

Figure 7.8. Distribution of asphalt mixture modulus during compaction

Four locations with various underlying layer modulus combinations were chosen for each project, and for each location, the dynamic modulus of new asphalt layer will increase according to Figure 7.8 along with the passes of vibratory roller. CMV actually measures the displacement under a dynamic compaction load; however, numerous published literatures indicate this measurement highly correlates with the static plate load test (PLT) (Roland Anderegg & Kaufmann, 2004; Brandl & Adam, 1997; Kröber et al., 2001). Since this study focuses on the relative proportions of displacements for new and underlying layers, a static layer elastic stress and strain analysis was employed to evaluate the effects of underlying layers on CMV utilizing a classical software BISAR. Using the BISAR, the displacements during the compaction for both new layer (Layer 1) and underlying layers up to a depth of 50 cm (Layer 2, 3) were calculated for both projects. The differences in displacements between the first and the last pass of roller for each layer were also calculated. The drums of the breakdown rollers for both projects have a similar size: 2,000 mm in width and 1,400 mm in diameter for SAKAI SW880; and 1,980 mm in width and 1,370 mm in diameter for DD-110HF. When the drum is loaded on the asphalt layer, the width of the contact rectangular area will be proportional to the square root of the depth of the drum indentation (Å. Sandström, 1994). One single load of 100 KN was used to simulate the maximum load of roller, and a radius of 0.2256m was used to convert the actual rectangular loading area of drum. The modulus of layer beneath the depth of 50 cm was set as infinite in the first round calculation, then the same modulus of closest layer was adopted for the second round calculation. Both calculations have very similar results. The results of the first round of calculations for project 3 and 6 are shown in Table 7.1 and 7.2 respectively.

Table 7.1. Displacements of Different Layers for Project 3

Dynamic or resilient modulus (MPa)								
Location	1		2		3		4	
Layer 1	60	900	60	900	60	900	60	900
Layer 2	20000		20000		50000		50000	
Layer 3	200		100		200		100	
Displacement (µm)								
Layer 1	206.55	12.32	206.20	12.00	207.34	13.09	207.24	12.97
Layer 2+3	78.05	77.42	110.90	109.90	49.66	49.48	70.36	70.09
The difference of displacements between the first and last pass (µm)								
Layer 1	194.23		194.20		194.25		194.27	
Layer 2+3	0.63		1.00		0.18		0.27	

Table 7.2. Displacements of Different Layers for Project 6

	Dynamic or resilient modulus (MPa)							
Location	1		2		3		4	
Layer 1	10	700	10	700	10	700	10	700
Layer 2	200		200		100		100	
Layer 3	100		50		100		50	
	Displacement (μm)							
Layer 1	3020.6	36.7	2996	30.7	3021	40.0	2990	36.0
Layer 2+3	891.4	802.7	1147	996.3	1462	1281	1776	1517
	The difference of displacements between the first and last pass (μm)							
Layer 1	2983.9		2965.3		2981.0		2954.0	
Layer 2+3	88.7		150.7		181.0		259.0	

As shown in Table 7.1, for the resurfacing project, it is reasonable to assume that the CMV reflects the total displacement of the pavement, and the influence of underlying layers cannot be ignored. With the progress in the compaction, the displacement of the top layer decreases and the CMV starts to mainly reflect the stiffness of underlying layers. This influence can partly explain the poor correlation between the last pass CMV and asphalt layer density, since the CMV value of the last pass may mainly reflect the variation of underlying support. On the other hand, when observing the difference of displacements between the first and last pass in Table1, the displacements of underlying layers almost keep the same for the same location for each pass of the roller, even it may vary considerably for different locations.

In equation 2-1, a "cylinder deformation modulus" Ec can be expressed as the ratio of the force and the corresponding displacement. The displacement 's' is the sum of the displacement for the new asphalt layer and the displacement for the underlying layers, therefore, equation 1 can be transformed as follows:

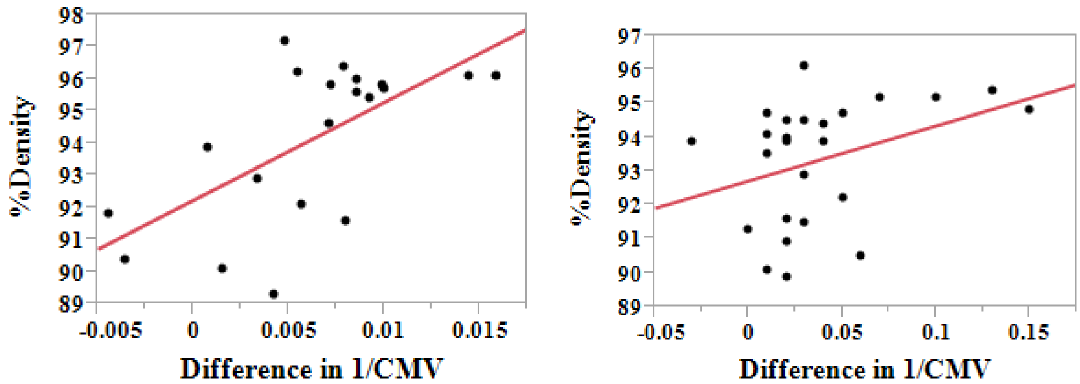
$$\frac{1}{CMV} = \frac{1}{E_C} = \frac{1}{c_1} \times \frac{S}{F} = \frac{1}{c_1} \times \frac{S_{Layer1} + S_{Layer2+3}}{F} \qquad (7\text{-}1)$$

For the same location, assuming the compaction force maintains constant, since the displacements of underlying layers do not change for different passes, the influence of underlying support on CMV can be eliminated through the difference in 1/CMV between different passes. Therefore, the difference in 1/CMV may mainly reflect the changes of displacement and modulus for the new asphalt layer.

As shown in Table 7.2, for the asphalt base layer compaction, the influence of underlying support on the CMV value is also significant; however, unlike project 3, the displacements of underlying layers may also vary significantly for the same location for different passes of the roller, depending on the strength of the underlying layers. It should be noted that the exact modulus for different locations and different passes may vary; however, the observations for project 3 and 6 still hold when the modulus changes within the scope.

Based on the above analyses, further correlation analyses were performed to explore any potential relationships between CMVs and asphalt densities. To find the difference in 1/CMV, at least two vibration passes are needed for the same location. For the project 3, there were a total of 20 cores accompanying with two or more vibration passes, and most of the continuous vibratory passes were finished in one minute for the same location according to the IC records. The amplitude of roller and the RMV values were checked to ensure that the roller was in stable behavior when the CMV value was obtained. For project 6, there were a total of 25 cores accompanying with two or more vibration passes. Simple linear regressions were performed between the difference in 1/CMV and core density, and the results are shown in Figure 7.9 and Table 7.3. For the project 3, the difference in 1/CMV has a significant relationship with the core density (Prob>|t|=0.0026), whereas the relationship is not significant for project 6 (Prob>|t|=0.077). The correlation can be further improved by considering the factor of temperature. According to Figure 7.8, both temperature and density can cause the change of asphalt mixture modulus if other parameters are kept the same. The starting temperatures of compaction were obtained from the IC records and a multivariate linear regression was performed to include both the starting temperature and the difference in

1/CMV. It shows that the starting temperature is a significant factor for both projects, and a much higher correlation can be observed when compared to the simple linear regressions (Project 1: R^2=0.74, Adjusted R^2=0.70; Project 2: R^2=0.41, Adjusted R^2=0.36).



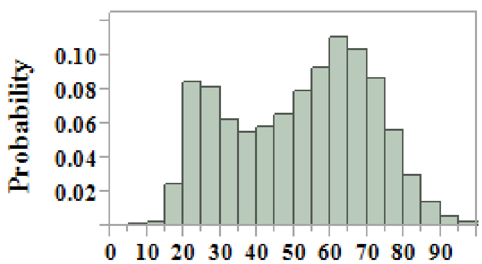
(a) Project 3 (b) Project 6

Figure 7.9. Correlations between difference in 1/CMV and density

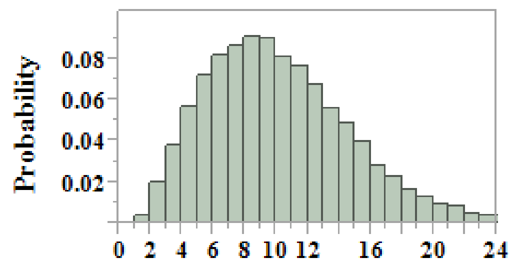
Table 7.3. Correlation Results between difference in 1/CMV and Core Density

Project	R^2	Adjusted R^2	Linear equation
3	0.40	0.37	Density = 92.24 + 305.82*Difference in 1/CMV
6	0.13	0.09	Density = 92.73 + 16.36*Difference in 1/CMV

The undesirable correlation results of project 6 can be partly explained by the effect of CMV conversion. As shown in Table 7.2, following the decrease of underlying support, the influence of underlying layers cannot be filtered effectively through this conversion. This result is also due to the characteristic of CMV measurement. Adam and Kopf (2004) investigated the relationship between ICMVs and soil modulus, which revealed the sensitivity of CMV to soil modulus within each operational mode. Their research revealed that CMV will increase linearly with the increase of soil modulus during partial uplift mode; however, it is insensitive to soil modulus during continuous contact when below 10. For project 6, the asphalt base layer is relatively thick and soft comparing to the asphalt surface layer, and the strength of underlying aggregate base layer is far smaller than the strength of underlying asphalt layers in project 3. Therefore, the vibratory roller was in a continuous contact behavior nearly half of the compaction time in project 6, comparing to a partial uplift mode for project 3. Figure 7.10 illustrates the distributions of CMV for one day for both projects. The correlation results of these two projects also have something similar to the findings of Mooney et al. (2010). In the research of IC soil compaction systems in 2010, it was found that the roller ICMV is more sensitive to thin lifts when the underlying subgrade is stiff (Similar to project 3), but it is insensitive to a stiff thin lift placed directly over a soft subsurface (Similar to project 6).



(a) Project 3



(b) Project 6

Figure 7.10. Distributions of CMV

Though similar IC rollers were applied on both asphalt pavement projects, the potentials of utilizing CMV to evaluate compaction level are different due to the properties of asphalt layer(s) being compacted and the stiffness of underlying layers. When it seems that the surface asphalt layer is more suitable for this approach, the problem is that at least two vibration passes for the same location are needed to find the difference in 1/CMV, and only one vibration pass is more common for the asphalt surface layer compaction. There were four other resurfacing projects also applying IC technology in Tennessee during the same time, but none of them could obtain enough CMV data.

Some other potential limitations should be considered when interpreting these results. A deformation modulus, measured from the drum, is an integral value which will be influenced not only by underlying layers, but also by the plastic deformation of the new asphalt layer. The plastic deformation is not considered in this study. The applicability of the Witczak predictive model for such temperature and air voids ranges needs to be verified further.

7.3 Summary

The relationships between the CMV of asphalt layer and other factors were thoroughly examined. Using the data of Project 3, the relationships between CMV and other roller operation parameters such as roller frequency, speed and amplitude were examined first, then by using Project 3 and 6, the effects of asphalt temperature and underlying support on CMV were analyzed further by utilizing the original Witczak model and the multilayer pavement analysis software BISAR. The conclusions are summarized as follows:

1. A significant linear relationship between CMV and amplitude could be observed among the operation parameters for Project 3, offering a criterion to adjust the CMV value.
2. For Project 3, the adjustment of CMV and the rising RMV value can help to identify the over compaction area and the optimum pass number.
3. For Project 3, linear relationships could be found between the CMVs of different passes for both raw data and adjusted data, and the adjusted data offer relatively stronger and more consistent correlation results.
4. A theoretical analysis showed that the difference in 1/CMV of the same point can filter the effect of underlying support on CMV and reflect the changes in compaction degree of the new asphalt layer.

5. There was a strong correlation between the difference in 1/CMV and core density for the resurfacing layer (Project 3), while no correlation could be found between the original CMV and core density.
6. No significant relationship could be found between the difference in 1/CMV and core density for the asphalt base layer (Project 6), mainly due to the properties of the asphalt layer and the stiffness of the underlying layers.
7. The fact that CMV is sensitive to road modulus indicates that it is not suitable for evaluating the compaction level of asphalt base layer.
8. A combination with asphalt temperature for a multivariate regression resulted in a higher R^2 value than any simple linear regression.

CHAPTER 8 RECOMMENDATIONS FOR ASPHALT RESURFACING QUALITY EVALUATION

In this study, the feasibility of utilizing IC parameters for evaluating the construction quality of asphalt resurfacing was evaluated so that they can be used as a supplement to traditional QC/QA procedures. Core samples from six projects were collected for this study. Lab tests, including density, permeability and indirect tensile strength (IDT), were performed. Additional IC data were also collected and analyzed, such as number of roller passes, surface temperature and ICMV. Linear models were utilized to identify the factors affecting the compaction quality of resurfacing layer.

8.1 Challenges of utilizing IC for asphalt QA purpose

There are some issues associated with the conventional methods for the density quality control (QC) and quality acceptance (QA) of asphalt compaction (AASHTO, 2001; FHWA, 2011). The conventional field density tests, either core-drilling or nuclear gauge test, are performed at selected spots where the test results are used for evaluating the quality for the entire asphalt layer. The density measurement on individual spots may result in the potential risk of misrepresenting the actual asphalt quality. Furthermore, even if the conventional methods can identify the weak areas, remedial actions are either impossible or expensive in that the compaction procedure is already finished. With 100 percent coverage, the IC technology can be a significant improvement over traditional QC/QA procedures involving tests at discrete point locations, and helps to evaluate the pavement performance under various situations (Žilioniene, De Luca, & Dell'Acqua, 2013).

While promising, current IC technology is still a relatively new technology for the asphalt compaction. Many factors such as non-linear behavior of roller and road interaction and multi-layer conditions limit the application of IC in the asphalt compaction. Strong linear relationships are rare to find between ICMVs and point tests of asphalt pavement, especially for the density measurements (Commuri et al., 2011; FHWA, 2011; HF Thurner, 2001). Thus, further researches are necessary to improve the application of IC for the asphalt compaction. In 2013, FHWA recommended IC specifications for asphalt materials based on the experiences of IC pooled fund and state projects, and encouraged the Department of Transportations (DOTs) to modify them to meet the specific state needs (DOT, 2014). The FHWA IC specifications recommended that test sections be evaluated to verify the mixture volumetric values and to determine a compaction curve of the asphalt mixtures with regard to the number of roller passes and to the stiffness of mixture. In addition, FHWA defined the target ICMV as the point when the increase in ICMV of the material between passes is less than 5 percent on the compaction curve. As an IC construction operation criterion, FHWA recommended that a minimum coverage of 90% of the individual construction area shall meet or exceed the optimal number of roller passes and 70% of the individual construction area shall meet or exceed the target ICMV values determined from the test sections. Since then, many DOTs have developed their own asphalt IC specifications based on the FHWA specifications. Some modifications are basically focused on the terms of payments and the compaction acceptance.

Although the compaction processes seem similar for the different asphalt layers, the situations are distinct due to the changes in the layer thickness and underlying supports. For the asphalt base layer, it is usually thick and has a weak underlying aggregate support; whereas the asphalt surface or resurfacing layer is relatively thin and has a strong underlying asphalt supporting layer. Various combinations of layers may result in different drum behaviors such as continuous contact or partial uplift, which eventually affect the potential of utilizing ICMV for asphalt compaction evaluation (M. A. Mooney, 2010; Å. Sandström, 1994). According to Chapter 7, it was found that unlike the asphalt surface layer, the vibratory roller may be in a continuous contact behavior during the asphalt base layer compaction, indicating that the ICMV is not suitable for evaluating the compaction level of the asphalt base layer (Hu, Huang, Shu, & Woods, 2016). Some DOTs have considered this difference and modified the specific IC specifications for different asphalt layers. California DOT regulates that ICMV be not collected for stiffness when the compacted asphalt layer is less than 4.5 cm (Caltrans, 2015). As for a thickness of 4.5 cm or greater, IC rollers provide additional real time QC for the asphalt density based on ICMV that is correlated to the specified asphalt target density. California DOT also specifies that the number of roller passes, asphalt temperature and ICMVs be for report only and not used for compaction QA.

In the southeastern area of the United States, most asphalt resurfacing projects were constructed with similar mixtures and compaction procedures. In Tennessee alone, over 2,000 mile state and interstate roads are resurfaced each year, and an increasing number of resurfacing projects are beginning to apply the IC technology. However, how to utilize the IC parameters for the resurfacing projects in this area continues to be a major challenge and impedes the application of IC technology in asphalt compaction.

8.2 Identification of weak areas

The data from Project 1 to 6 were collected for this study. Around thirty cores for each project were collected after the compaction and lab tests including core density, permeability and IDT were performed. The selection of core locations was based on two concerns: One is the feasibility of using IC data to identify weak areas; another is to correlate IC data with asphalt quality for the QC/QA purpose. For the first four projects, the cores were divided into two groups for each project: One group contains cores associated with adverse IC records such as insufficient passes, low surface temperature or extremely high or low ICMV values, whereas the other group cores were randomly selected. For the subsequent two projects, all cores were randomly selected. Table 8.1 shows the basic information for six projects. The number of vibratory passes reveals how many vibratory passes of the breakdown roller were recorded for the same location. The asphalt job mix formulas are presented in Table 8.2. This type of asphalt resurfacing has been successfully applied by the TDOT for many years. As shown in Table 8.1 and 8.2, the asphalt mixing ratios for different projects were similar, but the projects were compacted under various rolling compaction combinations and construction time.

Table 8.1. Information of projects

Project	Number of cores	Breakdown roller	Intermediate roller	Number of vibratory passes	Compaction	Location (county)
1	30	HAMM HD120	HAMM HD120	1	Day	Crockett
2	30	HAMM HD120	Ingersoll Rand DD-110HF	0	Day	Lincoln
3	30	SAKAI 4000VPM	CAT CB54B	>2	Day	Hamilton
4	15	HAMM HD120	HAMM HD120	0~1	Night	Knox
5	30	HAMM HD120	HAMM HD120	1	Day	Dyer
6	30	Ingersoll Rand DD-110HF	BOMAG 278AD	0	Day	Davidson

Table 8.2. Job Mix Formula

Project	Sieve Size	Percent Passing, %					
		1	2	3	4	5	6
Aggregate Gradation	15 mm	100	100	100	100	100	100
	12.5 mm	97	98	99	98	99	98
	9.5 mm	88	88	92	64	93	87
	4.75 mm	68	65	57	58	66	66
	2.36 mm	52	44	36	42	48	51
	0.6 mm	29	25	24	23	29	29
	0.3 mm	17	13	12	12	16	13
	0.15 mm	9.4	7.7	4.9	7.0	7.9	7.8
	0.075 mm	6.2	5.4	3.6	4.7	5.8	5.3
Mix Properties	AC content, %	6.2	5.9	6.1	5.7	6.0	5.7
	AC Binder (PG)	70-22	64-22	70-22	70-22	70-22	64-22

One advantage of IC is the ability of identifying potential weak areas. As mentioned before, IC technology was initiated for the soil compaction, therefore the original IC specifications were applied on soil compaction in countries including Austria, Germany and Sweden. The German specifications for earthwork QC/QA using IC can be specified in two ways (M. A. Mooney, 2010). First, IC can be implemented through initial calibration of ICMVs to Plate Load Test (PLT) modulus or density and subsequent use of the correlation during QA. A second approach uses IC to identify weak areas for spot testing via PLT, or density methods. The Swedish specifications also permit the use of IC to identify weak spots for PLT.

Derived from the IC specifications for the soil compaction, the FHWA IC specifications for asphalt materials do not specify the use of IC to identify weak areas of asphalt layer; however, the specifications recommend pre-paving mapping with an IC roller of the existing support materials in order to identify the weak areas of underlying layers. And the IC construction operations criteria about the optimal number of roller passes and target ICMV value also imply the identification of weak areas of asphalt layer, since it is regulated that construction areas not meeting the IC criteria shall be investigated.

Several laboratory methods are available for density measurement based on the microstructure and void content of asphalt mixtures (Praticò & Moro, 2011). In this study, the CoreLok method (ASTM D 6752) was used to test the bulk specific gravity of core samples. Using 92% maximum specific density (G_{mm}) as the criterion for the weak points, core samples from six projects were evaluated with the associated IC data. For the first four projects, there are 22 cores out of total 105 cores which densities are under 92% G_{mm}, and most of them belong to the group associated with insufficient passes, extremely low CMV or low surface temperature. There are also two weak cores in project 3 associated with an abnormal number of vibratory passes (9 and 10 vibratory passes respectively), resulting in the issue of over-compaction. For the other two projects with all randomly selected cores, only two cores are under 92% G_{mm}, and unsurprisingly, these two cores were also associated with relatively low surface temperature: a starting surface temperature of breakdown roller under 200 °F (93 □) when the average temperature of cores is 237 °F (114 □). According to the statistic results of core densities, the weak compaction areas of the resurfacing layer could be accurately identified using the IC records. The weak cores and the corresponding IC data of project 2 are presented in Table 8.3. It should be noted that most of weak core samples were located on the edge of the pavement.

Table 8.3. Weak cores and the corresponding IC data of project 2

Core No.	%Density	Breakdown roller		Intermediate roller	
		Number of passes	Start Temp (°F)	Number of passes	Start Temp (°F)
1	86.3	1	180.7	1	136.2
2	83.4	1	171.1	0	NA
3	89.8	1	183.2	0	NA
4	86.4	1	185.7	0	NA
19	91.0	1	212.2	0	NA
Mean of all cores	93.9	3.0	245.3	4.0	224.3

8.3 Correlation analyses of core tests

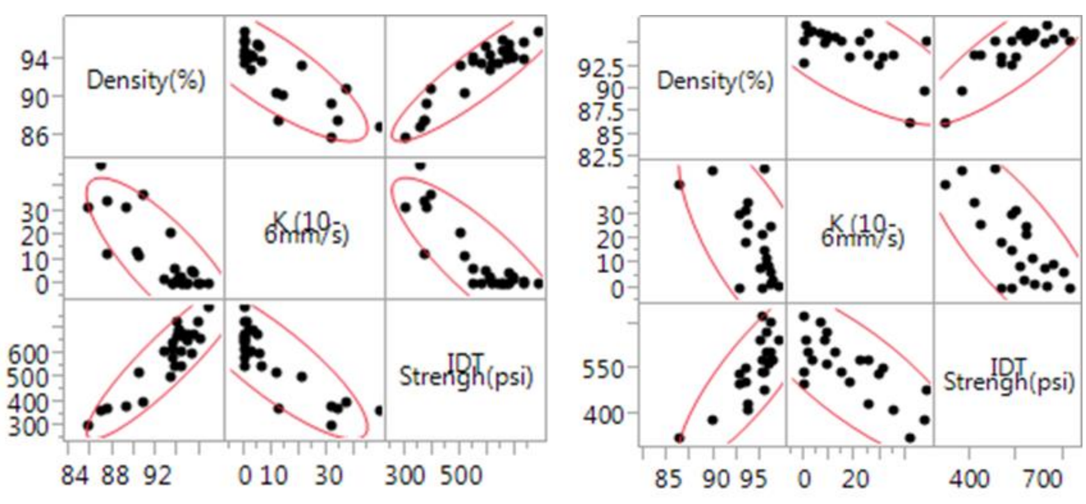
1.Correlations among core test results

Core density, IDT and permeability of cores were tested in this study to explore any potential relationship between IC data and the properties of asphalt mixture. Before the test results were correlated to the IC data, the results of core tests were firstly examined to determine the relationships among these properties of asphalt. Table 4 shows the correlation results among core density, permeability (K) and IDT strength. An absolute correlation value closer to 1.0 means a more linear dependent relationship of the two variables. As shown in Table 8.4, the density of cores for all projects correlated stably with both the permeability and IDT strength, while there is a fair good relationship between permeability and IDT strength for most projects. A scatterplot matrix is an ordered collection of bivariate graphs which demonstrate the relationships between multiple variables simultaneously. Figure 8.1 demonstrates the scatterplot matrix of project 1 and 2. As shown in Figure 8.1, there was linear relationship among these test results.

Table 8.4. Correlation pairs among various measurements of cores

Project	Density vs K	Density vs IDT	K vs IDT
1	-0.81	0.91	-0.84
2	-0.72	0.85	-0.79
3	-0.87	0.82	-0.69
4	-0.82	0.61	-0.47
5	na	0.73	na
6	-0.77	0.82	-0.79

Note: na = not applicable



(1) (2)

Figure 8.1. The scatterplot matrixes of project 1 and 2

2.Correlations between core test results and CMV

All rollers in the six resurfacing projects could obtain the CMV value when the roller is in a vibratory mode, which is designed to evaluate the stiffness or strength of pavement to a certain depth. The project 2 and 6 applied the static compaction process with no CMV obtained, and the project 4 only had four cores accompanied with CMV value. Therefore, the correlation analyses were performed between the test results of cores and the final CMV value for project 1, 3 and 5. As shown in Table 8.5, only the permeability tests of project 1 had a significant relationship with the last CMV value. Many factors may contribute to these poor correlations, including the non-linear behavior of an IC roller under specific conditions; the effect of paving temperature on ICMV while core tests are measured at room temperature. More importantly, CMV actually measures a combined stiffness of pavement structure up to a certain depth, usually greater than the asphalt layer thickness (M. A. Mooney, 2010). Therefore, the effects of the underlying layers on CMV cannot be ignored. In Chapter 7, the original Witczak model and the multi-layered pavement analysis software BISAR were utilized to analyze the deformation of pavement under compaction. Using the job mix formula and the roller's compaction frequencies, the dynamic modulus of the newly placed asphalt mixture was calculated for the possible range of air voids and temperature during compaction. The displacements were further calculated for the top and underlying layers during compaction. It was found that with the progress of the compaction, the displacement of the top layer decreases and the final CMV mainly reflects the stiffness of the underlying layers for the resurfacing projects. The results revealed that the final CMV cannot be adopted to evaluate the resurfacing asphalt quality directly.

To improve the correlation between CMV and asphalt properties, the influence of the underlying support on CMV should be eliminated. In the authors' another study, it was found that for the resurfacing project, there is no significant change on the displacements of the underlying layers under compaction for different passes comparing to the displacement of resurfacing layer, therefore, a theoretical analysis showed that the difference in 1/CMV between two passes may filter the influence of the underlying support on CMV, and mainly reflects the changes of the displacement and modulus for the new asphalt layer (Hu et al., 2016). To calculate the difference in 1/CMV between two passes, at least two vibration passes for the same location are necessary. According to Table 8.1, project 3 meets the requirements and the difference in 1/CMV was further correlated to the core test results. As shown in Table 8.5, it had a significant relationship with both the core density and IDT strength. It should be noted here that the intervals between two passes for project 3 were usually less than one minute, so the changes of temperature did not affect the modulus of asphalt dramatically.

The correlation between the difference in 1/CMV and the properties of asphalt offers the possibility of utilizing the CMV value to monitor the asphalt compaction quality. Nowadays, the nuclear density gauge is a usual tool to trace the change of asphalt density for the same location between passes, and to determine the optimal number of passes for the asphalt compaction. In this study, the correlation analysis shows that the difference in 1/CMV for the same location can also be used to monitor the change of the asphalt properties with less human and material costs.

Table 8.5. Correlation results between various test results of cores and CMVs

Project	Density		K (Permeability)		IDT	
	R^2	p-Value	R^2	p-Value	R^2	p-Value
1 (Last CMV)	0.04	0.3394	0.30	0.0098	0.10	0.1397
3 (Last CMV)	0.11	0.1522	0.15	0.1091	0.13	0.1166
5 (Last CMV)	0.00	0.8644	na	na	0.00	0.8231
3（1/CMV）	0.40	0.0026	0.16	0.0948	0.29	0.0170

3.Correlations between core test results and other IC data

Four other parameters can be directly obtained or derived from the IC records: the number of passes of the breakdown roller (Roller 1 passes); the starting surface temperature of the breakdown roller (Roller 1 temp); the number of passes of the intermediate roller (Roller 2 passes); the starting surface temperature of the intermediate roller (Roller 2 temp). One advantage of utilizing these parameters is that they can be obtained in all compaction modes, whereas CMV is only available in the vibratory mode. The linear correlation analyses were performed between lab test results and the selected IC parameters. As shown in Table 8.6, there was a stable and significant relationship between starting surface temperature of the breakdown roller and core densities for almost all projects, when the number of passes of the breakdown roller has a significant relationship with core densities for project 1and 2. As for the intermediate roller, the starting surface temperature and the number of passes have a similar but weaker relationship with core densities comparing to the breakdown roller. If the data of all the six projects were considered, the starting surface temperature of the breakdown roller is still the most significant factor for the asphalt density, followed by the starting surface temperature of the intermediate roller, the number of passes of the intermediate roller and the breakdown roller. The correlation analyses between permeability and IDT strength and IC data were also performed.

Table 8.6. Correlation results between various IC measurements and core densities

Project (Density)	Roller 1 passes		Roller 1 temperature		Roller 2 passes		Roller 2 temperature	
	R^2	p-Value	R^2	p-Value	R^2	p-Value	R^2	p-Value
1	0.42	<.0001	0.21	0.0092	0.21	0.0136	0.15	0.0383
2	0.23	0.0111	0.24	0.0085	0.21	0.0165	0.48	0.0006
3	0.06	0.17	0.30	0.0019	0.02	0.5352	na	na
4	0.10	0.2636	0.22	0.0916	0.02	0.6233	0.07	0.3987
5	0.01	0.6453	0.22	0.0199	0.00	0.8652	0.19	0.0294
6	0.06	0.1828	0.41	0.0001	0.00	0.9023	0.05	0.2556
All	0.04	0.0148	0.23	<.0001	0.10	0.0002	0.14	<.0001

In general, the significant relationships between permeability and IDT strength and the parameters of the breakdown roller existed in two projects, when almost no significant relationship could be identified for the parameters of the intermediate roller. Based on the analysis results, the asphalt density has a most significant correlation with the parameters of IC roller among various core tests. On the other hand, the starting surface temperature of the breakdown roller is the most important factor for the asphalt quality, regardless of which compaction pattern was chosen for the resurfacing projects.

8.4 Recommendations for asphalt resurfacing quality evaluation

Based on the results from the correlation analyses, the final CMV depends on the stiffness of underlying layers and its value may not be capable of directly evaluating the quality of resurfacing layer. However, one may find that the change of CMV between passes for the same location can filter out the effect of the underlying support and improve the correlation results to the asphalt properties. Other IC parameters, especially the starting surface temperature of the breakdown roller, are also helpful to identify the quality of asphalt resurfacing compaction. In fact, the Minnesota Department of Transportation (MNDOT) has attempted to evaluate the temperature records for quality management in the latest specification. Using paver mounted infrared temperature equipment, the segment temperature differential is determined by masking out paver stops and then cutting the lowest 1% and the highest 1.5% of the readings. The monetary price adjustments will be made based on the degree of temperature differential (MnDOT, 2014). It should be noted that this temperature records are from the paver not the roller, therefore the real compaction temperatures may vary depending on the compaction patterns. Also, this specification is more concerned about the temperature segregation, rather than the explicit compaction temperatures.

The quality of asphalt compaction is influenced by many factors including the environmental factors, the mix property factors and construction factors. Factors such as gradation, binder content and lift thickness will also affect the compaction directly; therefore, it is appropriate and necessary to focus on the same asphalt layer with similar mixture properties and lift thickness when recommending IC parameters for the asphalt compaction. As for the resurfacing projects in this study with similar mixture properties and same thickness, the most important factor for the compaction will be the asphalt temperature: as asphalt temperature decreases, the binder becomes more viscous, and finally no further reduction in air voids can be reached when it drops to the cessation temperature. Figure 8.2 shows the scatter plots between the asphalt density and the starting surface temperature of the breakdown roller. Among 23 cores which density is below 92% G_{mm}, there are 20 cores accompanying with a starting surface temperature under 110 °C（230 F°）. However, the temperature cannot be served as the only criterion since some cores with an accepted density are also in the range of low temperature as shown in Figure 8.2. Further investigation focusing on the cores with starting surface temperature under 110 °C revealed that most cores within the acceptable density range have a total number of passes larger than two (including breakdown and intermediate roller), whereas most weak cores (density < 92% G_{mm}) have a total number of passes equal to or less than two.

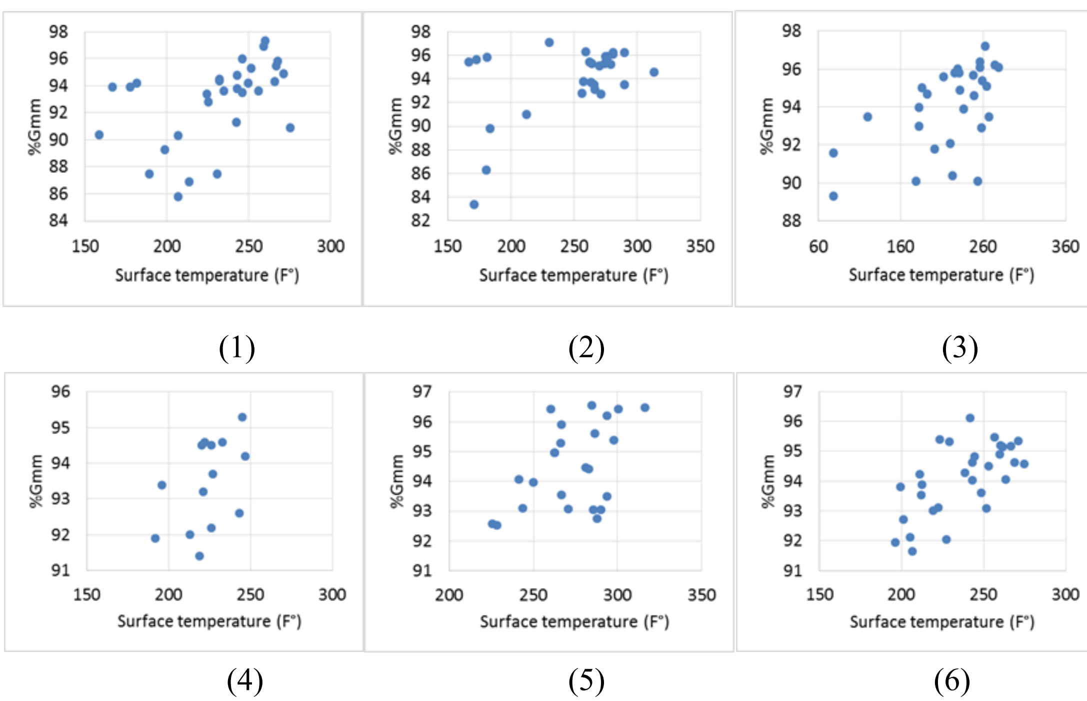


Figure 8.2. Core density and surface temperature

Based on the results in this study, unlike the FHWA IC specifications, the target ICMV value is not suggested for asphalt resurfacing compaction since the final ICMV value can be influenced by the underlying layer. As an alternative, the difference in 1/CMV, the starting surface temperature of compaction (>110 °C) and the total number of passes (>2 passes) are suggested for the resurfacing project, and the construction areas not meeting the criteria shall be investigated.

The use of the difference in 1/CMV is optional since the static compaction can also achieve a desirable asphalt density for the resurfacing project. For the project with a vibratory compaction, it is suggested that the correlation between the difference in 1/CMV between two passes and the asphalt properties should be established during the test section, and the daily CMV data should be checked based on the correlation results. It also should be noted that the surface temperature is not as same as the inner temperature of asphalt mixture, and the water used on the drums may drop the surface temperature significantly; however, as a parameter which can be directly obtained from the IC record, utilizing the surface temperature for the compaction of resurfacing project is simple and applicable.

8.5 Summary

In this study, IC parameters from six resurfacing projects in Tennessee were collected and evaluated for estimating the asphalt compaction quality. Field cores were taken from these projects and tested in the laboratory for density, permeability, and indirect tensile (IDT) strength. The lab results were correlated to different IC parameters. Based upon the results of correlation analyses, it is recommended that three IC parameters be used for evaluating the compaction quality of resurfacing project. The following conclusions can be drawn from this study:

1. As expected, the density of the cores from all projects showed a strong correlation

with the permeability and IDT strength; whereas, the permeability of core samples correlated well with IDT strength for most projects.

2. There was no significant relationship between the final CMV and core test results due to the effects of the underlying layers on CMV. A theoretical analysis showed that the difference in 1/CMV between two passes can filter the influence of the underlying support, and mainly reflects the changes of the new asphalt modulus. A correlation analysis indicated that the difference in 1/CMV between two passes had a significant relationship with asphalt properties.
3. As the most important factor for asphalt compaction among IC parameters, the starting surface temperature of the breakdown roller had a stable and significant relationship with core densities for all projects.
4. Based on the test results, it is recommended that three IC parameters be selected for evaluating the quality of the resurfacing project: the difference in 1/CMV, the starting surface temperature of compaction (>110°C), and the total number of passes (>2 passes).

CHAPTER 9 GEOSTATISTICAL ANALYSIS FOR ASPHALT COMPACTION

In this study, the geostatistical analysis procedures for the asphalt compaction was examined. Utilizing the semivariogram model, the spatial variability of asphalt layers from Project 3 and 6 are analyzed using both the ICMV data and the conventional point-wise measurements. The results of spatial statistics and univariate statistics are compared to identify the model's capacity in characterizing the spatial uniformity, and challenges involved in performing the geostatistical analysis for the asphalt compaction using the IC technology are also identified.

9.1 Challenges and benefits of utilizing IC data for geostatistical analysis

Achieving a uniform and desirable density has been adopted as the criterion for the asphalt pavement compaction over the years. Univariate statistics are typically used to describe the uniformity of the compacted asphalt nowadays. However, univariate statistics are incapable of addressing the issue of spatial uniformity (Gringarten & Deutsch, 2001; Yarus & Chambers, 2006). Two datasets with identical means and variances can have distinct spatial characteristics, therefore, it is necessary to combine the method of geostatistical analysis to better quantify the spatial uniformity, to improve the process control and to identify the poorly compacted locations during the asphalt compaction.

A fundamental assumption of geostatistics is the existence of spatial autocorrelation (Olea, 2006), which can be simply described as the phenomenon that in the vicinity of large values there are other large values, while small values may close to other small values. Although constant material inputs are generally used in pavement design, the engineering properties of asphalt mixtures can vary significantly in the spatial direction. Geostatistical analysis tools including semivariogram model are useful for evaluating the spatial variation and the actual performance of asphalt layer (Gringarten & Deutsch, 2001). An increasing number of researches have been conducted to understand the effect of spatial variability on the actual performance of pavement structures (Thompson & White, 2007; Vennapusa et al., 2009). However, detailed information with accurate location is essential for the geostatistical analysis, and the conventional point-wise measurements for asphalt compaction are difficult to meet the requirements (AASHTO, 2001).

The spatially referenced ICMV data offer the opportunity to perform the geostatistical analysis on the asphalt layer. Some researchers have performed the geostatistical analysis successfully on the soil compaction using IC technology (Gallivan, Chang, & Horan, 2011; M. A. Mooney, 2010; Vennapusa et al., 2009; White & Thompson, 2008; White et al., 2008). However, how to utilize the ICMV for the asphalt layer compaction remains a challenge due to many factors including the change of asphalt temperature and the measuring depth of IC roller. With suitable geostatistical models, the benefit of ICMV should be utilized and new insights into the spatial uniformity for asphalt layer should be developed.

CMV obtained from project 3 and 6 were analyzed using geostatistical models in the ArcGIS software, and other IC recordings such as RMV and vibration amplitude were also checked to clarify their influences on the CMV value. The trend of the data was checked using the trend analysis tool in the ArcGIS, and no obvious trend could be found. Two projects represent two typical scenarios for the asphalt compaction respectively. Project 6 is a new asphalt pavement including aggregate base, bituminous base layer, bituminous binder layer and surface layer, whereas Project 3 is a resurfacing project including only one thin resurfacing layer.

9.2 Geostatistical analysis for Project 6

A new asphalt pavement using the IC technology was performed in Davidson County, Tennessee， in August 2014. Three asphalt layers, including base layer, binder layer, and surface layer with a thickness of 10.2 cm, 4.4 cm and 3.2 cm respectively, were constructed on an aggregate base layer in accordance with the TDOT specification.

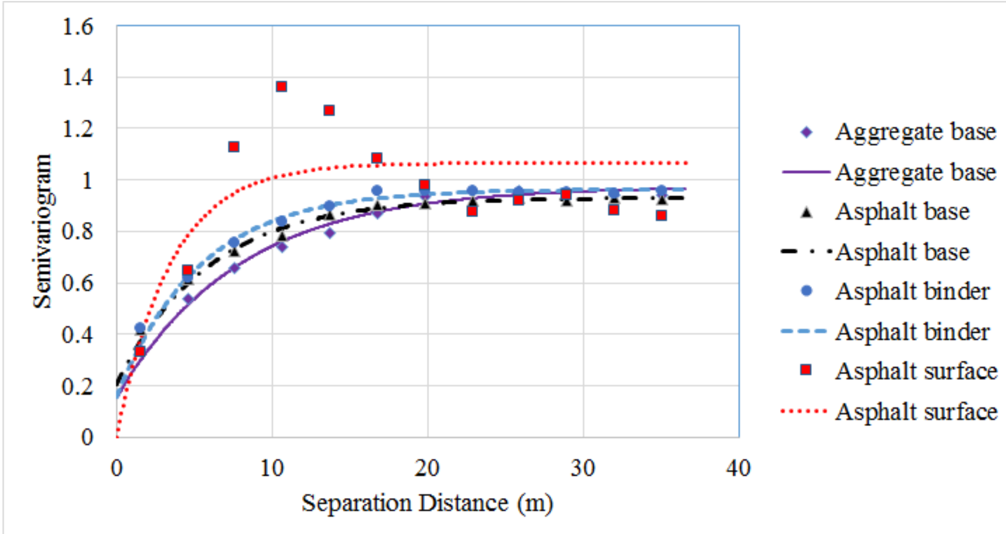
For all the three asphalt layers, a high frequency double drum vibratory roller equipped with IC system, Ingersoll Rand DD-110HF, was served as the breakdown roller. The roller was operated at a frequency of 55 Hz and an amplitude of approximately 0.45 mm when in the vibration mode. Another vibratory roller, BOMAG 278AD, was used for intermediate rolling in a static mode. The IC rollers were operated by the same operators with the same roller parameters such as speed and frequency for all the asphalt layers, which offers a basis for comparing the geostatistical models among different layers. The aggregate base layer was compacted by a single drum vibratory roller, CAT CS54, equipped with the same IC system. To explore any correlation between the aggregate base layer compaction and the asphalt layer compaction, the geostatistical analysis was performed both on the CMV of the aggregate base layer and asphalt layers. A total of 30 cores for each asphalt layers were randomly collected after the compaction within a length of 300 m. All the core locations were recorded and the lab densities (G_{mm}%) were measured. The possibility of utilizing the conventional point-wise measurements for the geostatistical analysis was also checked using the cores data.

For the same section of the pavement in which the cores were collected, the geostatistical analyst tool in the ArcGIS was used to check the data, obtain the semivariogram and create the contour maps using Kriging method. A summary of spatial and univariate statistics of CMV for different layers are presented in Table 9.1. Both the original semivariogram and semivariogram after the normal score transformation were included for comparison. After the transformation, the data will have a normal distribution with a mean of zero and a variance of one. As shown in Table 9.1, all the Sills for different layers are close to one after the transformation.

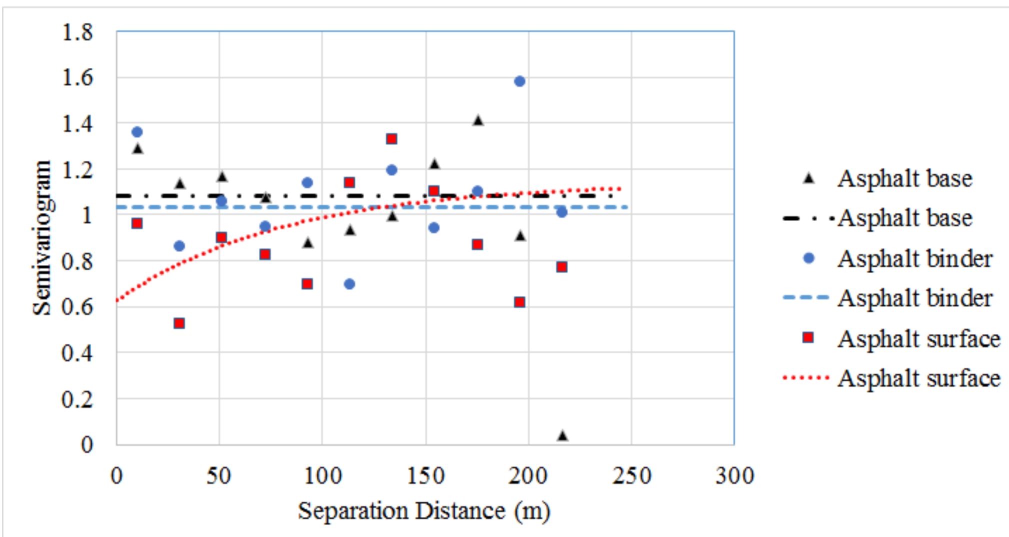
Table 9.1 Spatial and univariate statistics of CMV for different layers

Layer	Univariate statistics		Semivariogram			Semivariogram with normal score			Sample size
	μ	σ	Nugget	Sill	Range (m)	Nugget	Sill	Range (m)	
Aggregate base	43.9	31.8	159.1	947.3	19.0	0.2	1.0	23.7	76696
Asphalt base	10.2	4.6	3.7	19.4	12.9	0.2	0.9	17.6	57555
Asphalt binder	17.6	9.1	11.5	80.2	13.0	0.2	1.0	16.1	68749
Asphalt surface	30.7	16.1	0	285.5	11.4	0	1.1	10.3	5353

The CMV of the aggregate base layer has the largest mean and standard deviation, and the sill and range value of this layer are also significantly different from the asphalt layers. Though using the same IC system, the single drum vibratory roller for the aggregate compaction had significant differences from the double drum vibratory roller for the asphalt compaction on many aspects. Besides the disparity in weights, the single drum vibratory roller has a frequency of 30 Hz and an amplitude of 2.5 mm, comparing to a frequency of 55 Hz and an amplitude of 0.45 mm for the double drum roller. Moreover, it is obvious that the machine-ground interactions are different for the aggregate and asphalt layer. Since the Nugget and Sill value of the original semivariogram were affected by the CMV value, it was suggested that the semivariogram after the normal score transformation be used to track the changes of spatial variation between layers, and in this project, the larger range value for the aggregate base indicates a better spatial continuity than the asphalt layers when considering the CMV value only.



(a) Semivariograms of CMV for different layers



(b) Semivariograms of core densities for different layers

Figure 9.1. Semivariograms for different layers

Figure 9.1 shows the semivariograms after the transformation for different layers using both CMV and core densities, which demonstrates the benefit of IC technology. Although 30 cores for each layer were collected within a length of 300 m, which had a much higher sampling density than routine quality inspection, no spatial trend could be observed for all the layers using core data as shown in Figure 9.1 (b). In other words, the conventional point test results at limited locations cannot capture the spatial variation of the compaction. Even if the core tests can identify the weak locations, determining the area for remedial action remains a challenge since no spatial autocorrelation information could be obtained from the core tests. As shown in Figure 9.1 (a), the experimental dots of CMV fit well with the exponential curves for all the layers except the asphalt surface layer. An examination on CMV data indicates that the roller was usually operated in a static mode for the surface layer, resulting in an inadequate sample size for spatial statistics (only one tenth of sample size of other asphalt layers as shown in Table 9.1). With the help of semivariogram and IC technology, it is possible to identify the weak areas accurately. For example, the Ranges after the transformation for both asphalt base and binder layer are close to 17 m, which means that the data of compaction are only spatially correlated within this range. If core tests identify the flawed location, further investigation should be performed according to the Range value to determine the repair area.

For different asphalt layers, the univariate statistics results also show a notable difference. The values of mean and standard deviation increase gradually from the asphalt base layer to the surface layer, despite using the same roller with the same settings. This phenomenon should be clarified since the value of CMV will affect the value of Sill. During compaction, a vibration roller puts a dynamic parabolic load on a rectangular area of the asphalt pavement surface, and a CMV value can be regarded as an integral cylinder deformation modulus to a certain depth, and this depth is usually larger than the thickness of the compacting layer (HF Thurner, 2001). This modulus (CMV) can be expressed as the ratio of the force and the corresponding displacement. Since the force can be assumed as a constant for different asphalt layers with the same roller settings in this project, the corresponding displacement during compaction becomes a key factor to account for the variations of CMV values between different layers.

To calculate the displacement under compaction, the Witczak dynamic modulus predictive model and the finite element analysis software Abaqus FEA were utilized in this study. The Witczak model was developed to estimate the dynamic modulus of asphalt mixture from material properties and volumetrics (Andrei et al., 1999). The dynamic moduli of different asphalt layers were calculated using the job mix formula, roller setting frequency, the temperature from the IC data, and air voids from the nuclear gauge (NG) test. It should be noted that the modulus of asphalt at high temperature under compaction is far less than that at ambient temperature when serving as the underlying layer. The resilient modulus of aggregate base was set as 200 MPa. The literature shows inconclusive results about the measurement depth of a vibration roller (Roland Anderegg & Kaufmann, 2004; Vennapusa et al., 2009). To simplify the problem, the concept that an IC roller can be assessed at a depth of 1,000 times the amplitude by Anderegg and Kaufmann was adopted here to calculate the deformations under the roller (Roland Anderegg & Kaufmann, 2004). The calculated displacements for three asphalt layers under compaction comparing with the univariate statistics results are shown in Figure 9.2. One can refer to the authors' another study for the detailed information about the displacement assumption and calculation (Hu et al., 2016).

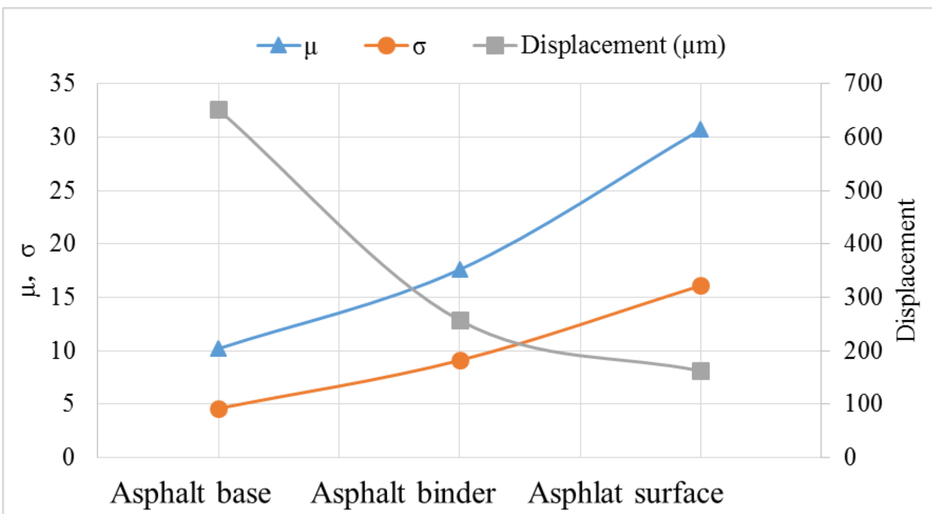


Figure 9.2. Displacements vs. univariate statistics of CMV

As discussed above, the cylinder deformation modulus to a certain depth can be expressed as the ratio of the force and the corresponding displacement. When the compaction force remains stable, the CMV value will mainly reflect the changes of displacement. This relationship is well illustrated in Figure 4, which shows a decrease in the displacement from the base to the surface layer, corresponding to an increase in the mean value of CMV. Based on the calculation results, the variation of CMVs between different layers is not because of the compaction quality, but rather due to the measuring depth of IC roller. This characteristic of IC measurements impedes the comparison of compaction quality between different asphalt layers even with the same roller and settings. It was suggested that the range of semivariogram after the normal score transformation be used to analyze the compaction qualities between different layers. As shown in Table 9.1 and Figure 9.1, the ranges of the asphalt base and asphalt binder layer have similar values, indicating similar spatial uniformities.

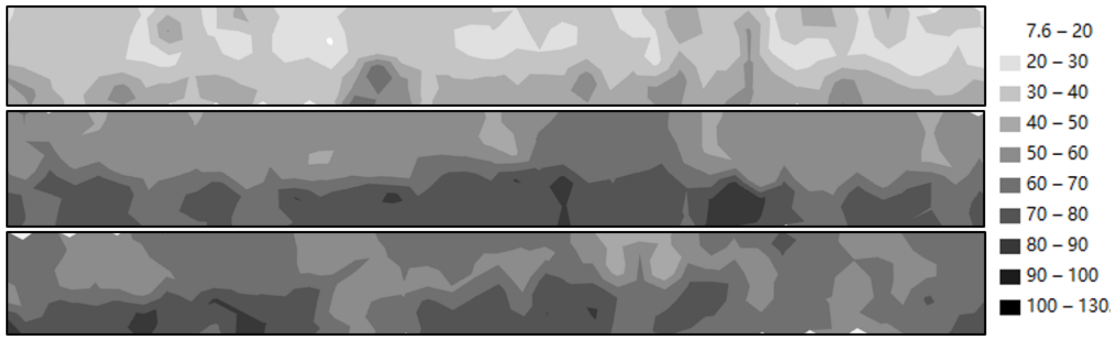
9.3 Geostatistical analysis for Project 3

Project 3 was constructed in Hamilton County, Tennessee in October 2013. A 3.2 cm thick overlay was applied on an existing asphalt pavement in accordance with the TDOT specifications. A high frequency double drum vibratory roller equipped with IC system, SAKAI SW880, was served as the breakdown roller. The roller was operated at a frequency of 69 Hz and an amplitude of approximately 0.5 mm. Another vibratory roller, CAT CB54B, also equipped with IC but in a static mode, was used for intermediate compaction. The breakdown roller was in the vibration mode for most of the compaction time, and the CMV data were obtained for different roller passes. It offers an opportunity to monitor the change of spatial uniformity during the compaction process. The semivariograms of different compaction passes for the same asphalt layer were calculated and compared in this case study.

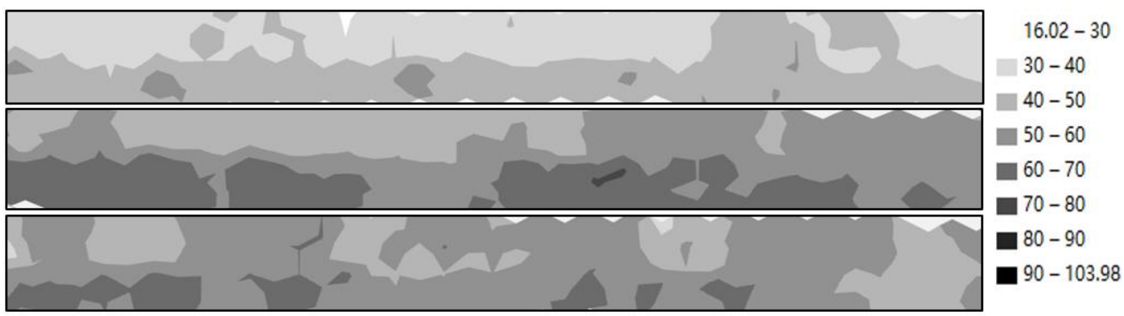
Table 9.2 Spatial and univariate statistics of CMV for different passes

Passes	Univariate statistics		Semivariogram			Semivariogram with normal score			Sample size
	μ	σ	Nugget	Sill	Range (m)	Nugget	Sill	Range (m)	
1	29.8	10.1	24.8	93.0	20.4	0.2	0.9	18.3	44331
2	58.8	12.0	10.2	112.5	28.4	0.1	0.8	28.2	40664
3	61.7	14.1	46.7	162.1	23.7	0.2	0.8	27.7	35983

Table 9.2 shows the spatial and univariate statistics of CMV for different passes for a same day compaction. Since the CMV values are comparable for the same asphalt layer, and the information of Nugget and Sill is difficult to interpret after the transformation, the original semivariogram is suggested and utilized here to analyze the compaction of the same asphalt layer. For the univariate statistics, the CMV of the first pass has the smallest mean value followed by a significant increase at the second pass, indicating an increase in the stiffness of surface layer under compaction. The mean of CMV still increases at the third pass. As for the spatial statistics, an increase of range from the first pass to the second pass indicates that the spatial uniformity increases at the beginning of compaction, but a significant increase of nugget and sill and a decrease of range occurred simultaneously at the third pass, suggesting a decline in the spatial uniformity during the compaction. Figure 9.3 (a) shows the changes of the kriged contour maps of CMV for different passes at the same location with a length of 1600 ft (488 m). As shown in Figure 9.3 (a), the contour map of the second pass shows greater spatial uniformity than both the first pass and the third pass. Based on the univariate statistics of CMV, the third pass compaction will help to increase the CMV value; however, the results of spatial statistics suggest that two passes of the breakdown roller are enough for the resurfacing compaction.



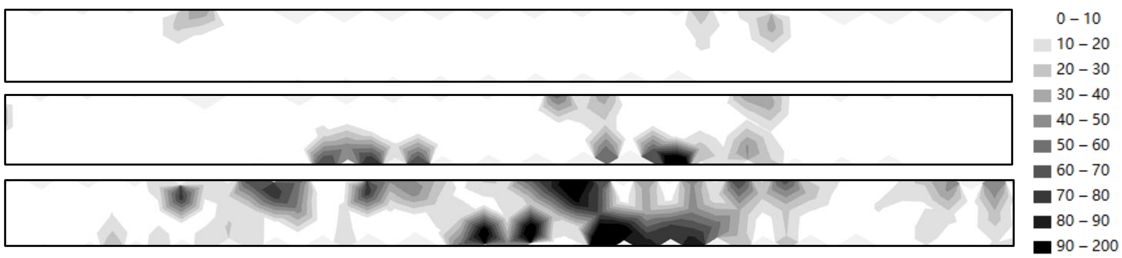
(a) Original contour maps of CMV (Top to bottom are pass1, 2 and 3)



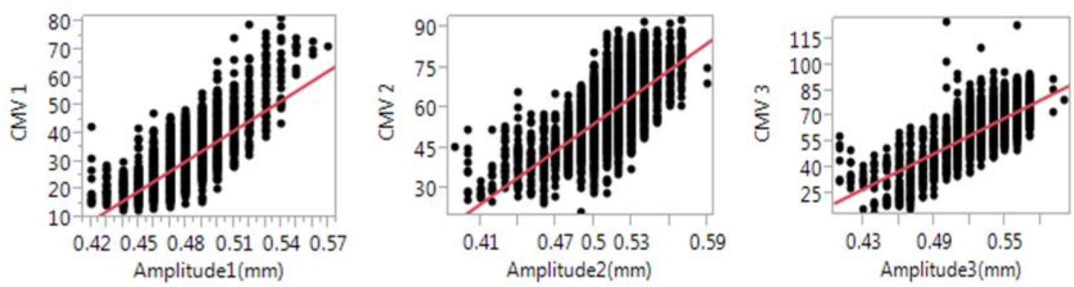
(b) Contour maps of CMV after the conversion (Top to bottom are pass1, 2 and 3)
Figure 9.3. CMV contour maps

As mentioned above, CMV is a dimensionless compaction parameter that depends on roller dimensions (i.e., drum diameter and weight) and roller operation parameters (e.g., frequency, amplitude, and speed). On the other hand, CMV must be interpreted in conjunction with the RMV since the drum behavior affects the CMV measurements. Therefore, it is necessary to acquire a better understanding of CMV by examining the relationships among these parameters. After an examination among various parameters, it was found that a significant and consistent correlation exists between CMV and roller amplitude as shown in Figure 9.4. A high amplitude will force the roller into a double jump mode and the correlation between CMV and stiffness will show a discontinuity (Å. Sandström, 1994). It is important that the roller remains in the same mode with a constant amplitude when utilizing CMV for the compaction verification. Figure 6 also illustrates the change of RMV at the same location during the compaction. The contour maps of

RMV show that the area and the value of RMV which is not close to zero grew steadily from the first pass to the third pass, suggesting a double jump or rocking mode of drum occurring due to the increasing stiffness of asphalt layer. At these areas, the asphalt cannot be compacted further and the CMV may not reflect the actual stiffness of asphalt layer. To eliminate the influence of amplitude on the CMV value, the CMV value was adjusted based on the linear relationship between CMV and amplitude for all roller passes. Furthermore, in areas with RMV >20, the CMV value were replaced with a fixed number of 60 to represent the possible highest compaction level (Vennapusa et al., 2009).



RMV (Top to bottom are pass1, 2 and 3)



Pass 1 Pass 2 Pass 3

Figure 9.4. Distribution of RMV and relationship between amplitude and CMV

Table 9.3 and Figure 9.3(b) show the statistics results and contour maps after the conversion. The mean of CMV is increased and the standard deviation is decreased for the first pass comparing to the values of raw CMV, and similar trend is also found in semivariogram model with a larger range value and a smaller sill value. It reveals that the CMV value of the first pass is more affected by the roller amplitude, and a conversion to a standardized amplitude can help to eliminate this influence. The spatial and univariate statistics of the second pass have similar results with the original data, indicating that the influences of RMV and amplitude are relatively small. As for the third pass, the RMV value has a significant effect on the statistics analysis of CMV. After the conversion, the mean of CMV for the third pass is less than that for the second pass, and the range of semivariogram also drops significantly during the third pass in Table 9.3, which is consistent with the observation in Figure 9.3(b). Through the conversion, the trend of spatial uniformity during the compaction becomes clearer, and the weak areas can be identified more accurately. When a vibratory roller passes over the asphalt materials that have already been adequately compacted, it can perceptively or imperceptibly bounce on the pavement surface and cause a reduction in asphalt density. This phenomenon is known as over compaction. The spatial statistics of CMV after the conversion suggest that the third pass of compaction for the resurfacing project was unnecessary and resulted in a problem of over compaction, whereas it is difficult to identify using the univariate results only.

Table 9.3 Spatial and univariate statistics of CMV after the conversion

Passes	Univariate statistics		Semivariogram		
	μ	σ	Nugget	Sill	Range (m)
1	37.6	6.5	7.4	43.2	24.5
2	55.1	8.5	12.3	73.3	29.0
3	53.2	9.0	23.4	76.1	21.8

9.4 Summary

In this study, geostatistical analysis was employed to evaluate the construction quality of asphalt pavement through the IC technology. Compared with the conventional quality control method, the geostatistical analysis is capable of evaluating the spatial uniformity and identifying the weak locations during compaction. On the other hand, the measurement depth of the ICMV and its relationship with the roller operation parameters should be considered during the analysis. In general, the geostatistical method can serve as a supportive tool for evaluating the compaction quality with the IC technology. To fully establish the quality control framework of IC technology, two asphalt pavement projects, including one new pavement construction and one resurfacing project, were investigated and evaluated by using geostatistical method in this study.

The new pavement project with different asphalt layers well demonstrates the benefit of IC technology. The experimental dots of CMV fit well with the exponential curves for all the layers with sufficient IC data, whereas no spatial trend could be observed using conventional core data. Besides, the range value of semivariogram is helpful to determine the repair area. An increase in CMV from asphalt base layer to surface layer was not due to the change in compaction quality, but rather due to the measuring depth of the IC roller. It is suggested that the range of semivariogram after the normal score transformation be adopted to compare the compaction quality of different layers. For this project, the ranges of semivariogram were very close for the asphalt base layer and binder layer, indicating that both layers had similar spatial uniformities.

For the resurfacing project, using the semivariogram model from the original CMV data could identify the trend of compaction quality during the compaction. However, various factors such as amplitude and RMV may affect the value of CMV, and their influences should be filtered out if necessary. The spatial statistics of CMV after the conversion could identify the change of spatial uniformity more accurately. For this project, the spatial statistics of CMV after the conversion indicated that the third pass of compaction was unnecessary and possibly resulted in the issue of over-compaction, when using the univariate results only was unable to identify this problem.

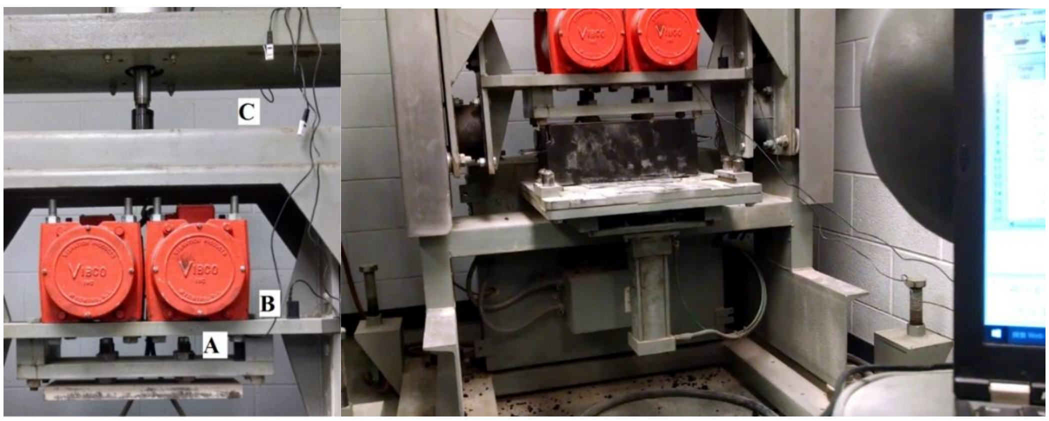
CHAPTER 10 A LABORATORY SIMULATION OF IC

Currently, at least six manufacturers (Ammann, Bomag, Case/Ammann, Caterpillar, Dynapac, and Sakai) offer ICMV on their machines. All the manufacturers employ proprietary data filtering, recording, and display methods using proprietary software. To better understand the potential of using IC on the asphalt QA procedure, it is necessary to design a laboratory method to simulate the vibratory compaction process. In this study, the Asphalt Vibratory Compactor (AVC) was used to simulate the influence of breakdown rollers on the compaction of asphalt mixture in the field.

10.1 Asphalt Vibratory Compactor and materials

AVC was employed to simulate the IC compaction process in the labroatory. AVC mainly consists of a compactor assembly and a vibratory compactor unit. The compactor assembly is a steel loading frame equipped with compactors, noise absorbing supports and isolators. The vibratory compactor unit consists of two vibratory motors with an output frequency of 3560 round per minute (RPM) and a force of 1886 lbs. Therefore, the fundamental frequency of vibrator is round 60 Hz.

Two accelerometers are used to monitor the vibratory characteristic of AVC during compaction. The accelerometer measures acceleration through an integrated circuit in which there is flexible "fingers" carved in silicon. These fingers flex and lead to the changes of capacitance when subjected to acceleration and deceleration. By monitoring and recording the change of voltage due to the capacitance changes, the accelerometer uses this "analogy signal" to indicate the change of acceleration. In this study, two single axis accelerometers with different measuring ranges, ± 50 m/s^2 with an accuracy of ±0.5 m/s^2 and ± 245 m/s^2 with an accuracy of ± 2.45 m/s^2 respectively, was utilized to record the acceleration. Both accelerometers have a same frequency response from 0 to200 Hz, and the accelerometer readings were sampled at 1000 Hz. Three locations on the vibrating compaction assembly were tested first as shown in Figure 1, and the results showed that the signal from location C was more damped than other locations since it is close to the pneumatic actuator, which can offer an adjustable static compaction force similar to the static load of roller. Therefore, two accelerometers were installed on location A and B correspondingly. During the test, it was found that the amplitude of the fundamental frequency was usually larger than the scope of low-*g* accelerometer. Therefore, the data analysis was primary based on the record of high-g accelerometer, and was checked with the data form the low-g accelerometer. Discrete-Time Fourier Transform was utilized to perform the response analysis.



(a) (b)

Figure 1. AVC and accelerometers installation and testing

In this study, both dense graded and open graded asphalt mixture were included. Two dense graded asphalt mixtures were selected. They are common used asphalt mixture for wearing course and binder course in Tennessee, called “D-mix” and “B-mix”, respectively. The aggregate for D-mix consisted of gravel, #10 limestone and natural sand, whereas that for B-mix consisted of gravel, #7 limestone and natural sand. As for the open graded friction course (OGFC), to ensure the bonding between aggregate particles, it employed PG 76-22 bitumen as the binder, and the aggregate for OGFC consists of gravel and #7 limestone. The gradation and volumetric properties of each mixture were listed in Table 10.1.

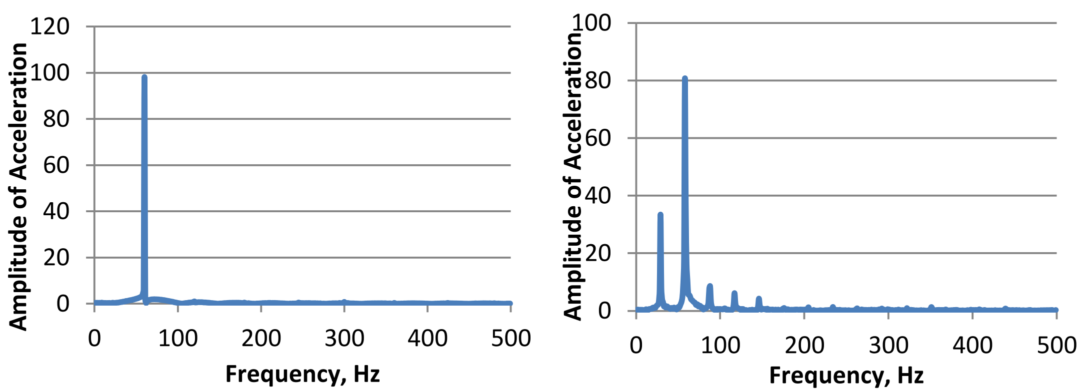
Table 10.1. Gradation and volumetric properties for each asphalt mixture

Mixture properties	Sieve size (mm)	OGFC	D-mix	B-mix
Gradation Passing Percentage,%	26.5	100	100	100
	19	100	100	93
	16	100	100	N/A
	12.5	97	98	N/A
	9.5	69	87	71
	4.75	16	66	44
	2.36	5	51	37
	0.6	N/A	29	23
	0.3	N/A	13	11
	0.15	N/A	7.8	7.3
	0.075	2.1	5.3	5.1
Asphalt content,%		7.50	5.70	4.20
PG grade		PG76-22	PG64-22	PG64-22
G_{mm}		2.300	2.410	2.500
G_{mb}		1.879	2.320	2.406
Air voids,%		18.3	3.80	3.80
VMA,%		N/A	16.60	13.60
VFA,%		N/A	77.40	72.20

The loose asphalt mixture was placed in a rectangular mold with 38.1 cm in length, 16.51 cm in width and 8.89 cm in depth. It should be noted that the mold shape has an edge effect on asphalt mixture compaction. The asphalt mixture compacted in tetragonal mold may have less even air voids distribution in lateral direction than that in cylinder mold (Chen, Huang, & Shu, 2012). For each sample, the loose thickness was 8.89 cm. All samples were compacted for a fixed time of 25 seconds. The compaction process was firstly characterized by the time-frequency spectrum of acceleration of compact unit. Then the evolution of compaction for asphalt mixture was investigated by analyzing both stiffness properties and volumetric properties. Finally, the influence factors on compactability of asphalt mixture were discussed.

10.2 Different compaction stages of vibratory compaction

Figure 10.2 illustrated the amplitude of acceleration at frequency domain under two loading scenarios: empty loaded and loaded. Figure 9.2(a) indicated the oscillatory properties of the vibrator at empty loaded condition. The single-peak of amplitude at 60Hz was generated by the vibrators in the compact unit. The fundamental component of vibration was determined by the output force and frequency of compact unit. As a comparison, when loaded, the peak value of amplitude at fundamental component of vibration decreased as illustrated in Figure 9.2(b). Meanwhile, there were several peaks at harmonic and/or subharmonic frequency components. The presence of amplitude peaks at fundamental component, harmonic and subharmonic component are believed to be associated with the increase of stiffness for compacted material. Therefore, it is possible to evaluate the stiffness of compacted materials by analyzing the change of amplitude of these vibratory components.

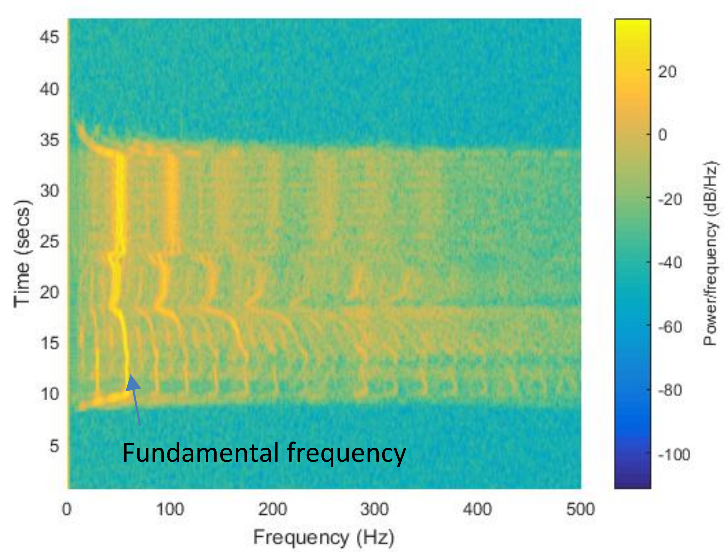


(a) Empty loaded (b) Loaded

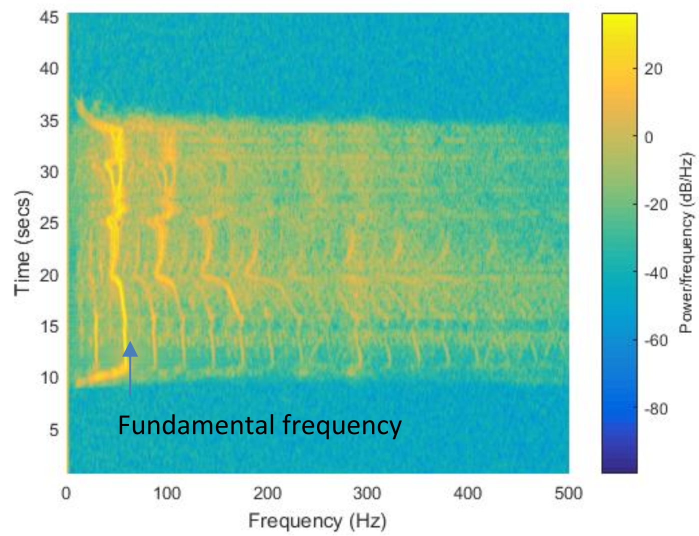
Figure 10.2. Relationship between frequency and Amplitude of acceleration under two loading scenarios

Figure 10.3 illustrated the time-frequency spectrum of acceleration for different asphalt mixtures. The fundamental frequency of compactor-material contact system was pointed out in the figures. During the compaction process, there was an increase of fundamental frequency from 0 Hz to oscillation frequency of compact unit (58 Hz), followed by a stably decrease. Then, there was a fluctuation of fundamental frequency for each mixture. Finally, there tended to be a stable stage after that fluctuation. The color scale in the spectrum indicated the strength of acceleration signal. There were stronger signals at fundamental frequency and each harmonic and subharmonic frequencies. The stronger signals correspond to the peaks in the frequency domain of acceleration as shown in Figure 10.2.

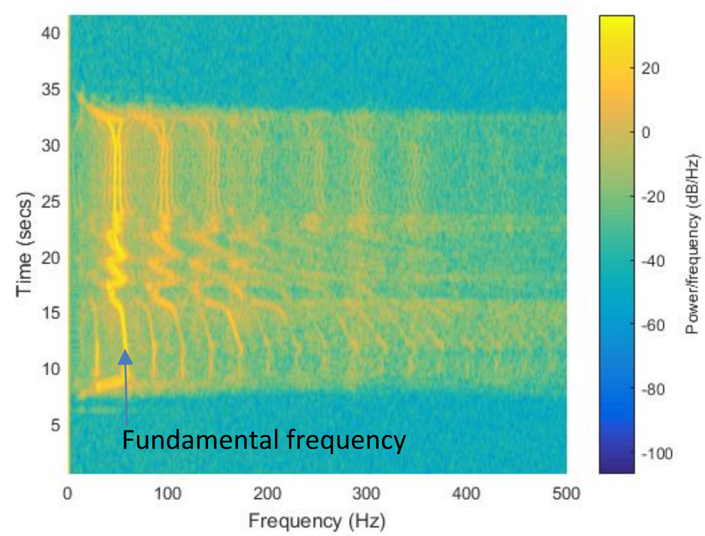
One can infer from Figure 10.3 that the process of lab vibratory compaction may consist of different stages. During the initial stage of compaction, the fundamental frequency of compact unit was close to that of "empty loaded". As the loose mixture becomes denser, the stiffness of mixture increases. Accordingly, the fundamental frequency of compactor-material system decreased. There was a fluctuation in time-frequency spectrum during the entire compaction process. Following this fluctuation, there seemed a "stable" stage in which there were no significant changes of signal strength in time-frequency spectrum of acceleration.



(a) B-mix



(b) D-mix



(c) OGFC

Figure 10.3. Time-frequency spectrums of acceleration

Measurement values are defined based on the amplitude peaks at specified frequencies. These frequency components include fundamental component, harmonic/subharmonic components. The spectral properties can be quantitatively evaluated by calculating the ratios of amplitude values. The changes of measurement values during vibratory compaction for different mixtures were illustrated from Figure

10.4 to Figure 10.6. The measurement value was calculated at the time interval of one second with the sample rate of 1000 Hz. The error bars represented the standard deviation from replications.

Figure 10.4 illustrated the change of CMV during vibratory compaction for different mixtures. CMV is the ratio of acceleration amplitude at the first harmonic component to the fundamental component. As shown in Figure 10.4, CMV gradually increased within the first ten seconds, followed by a sudden increase with an increased standard deviation. Finally, the CMV value remained stable for each mixture. It was found that the CMV for B-mix was slightly higher than 50, while those for D-mix and OGFC were slightly lower than 50.

Generally, three stages can be identified in accordance with the change of CMV during compaction. At the first stage, CMV gradually increases with a relatively small standard deviation. Following the first stage, there was a distinct increase in CMV at the second stage. It seems that CMV also exhibited large variability within this stage, which may be attributed to the chaos state due to the presence of noise wave. After the second stage, the CMV decreased and became stable at the third stage, indicating no significant change of CMV for this period.

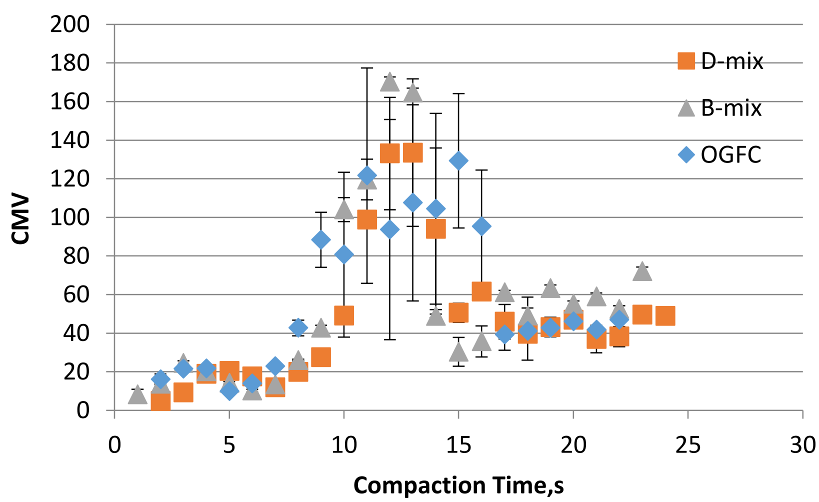


Figure 10.4. Relationship of compaction time versus CMV

Figure 10.5 illustrated the change of RMV during the vibratory compaction for different mixtures. Similar to CMV, RMV is the ratio of acceleration amplitude of subharmonic component, which is half of the fundamental component, to fundamental component. This measurement value was developed to characterize the behavior of drum jump (Adam & Kopf, 2004a). With RMV=0, the drum is considered in a continuous contact or partial uplift mode. RMV > 0 indicates that the drum enters rocking and chaotic modes from double jump mode. In this study, RMV was employed to indicate the properties of acceleration spectrum during compaction.

There was a sharply increase in RMV at the beginning of the compaction, followed by a sudden decrease. Then there is no significant change of RMV for each mixture. It was found that the compaction times to a stable RMV are: 7 s for B-mix, 8 s for D-mix and 6 s for OGFC. The change of RMV can be classified into "unstable" stage and "stable" stage. Similar to CMV, there was a large variability in RMV at "unstable" stage. At the stable stage, both the RMV values and standard deviations decreased.

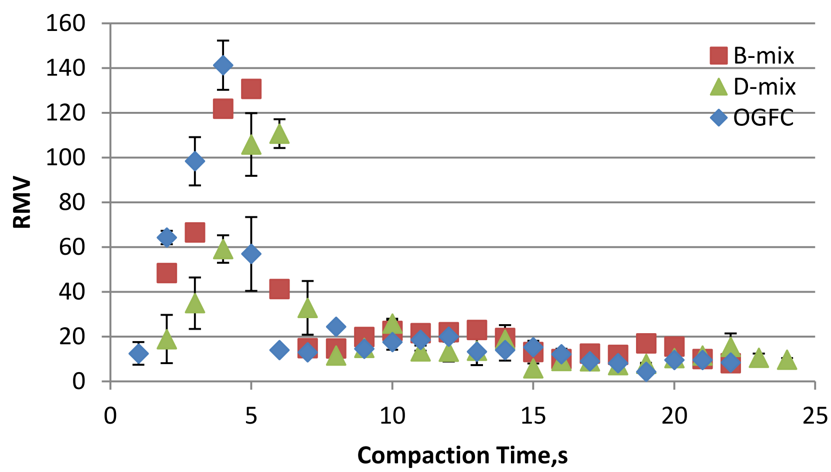


Figure 10.5. Relationship of compaction time versus RMV

Figure 10.6 illustrated the change of CCV during the vibratory compaction for different mixtures. The calculation of CCV includes fundamental component as well as several harmonic/subharmonic components. Comparing with CMV and RMV, CCV contains more frequency components and signal information. Therefore, this value may represent the general trend of changes in the spectrum of acceleration. It was found that the CCV increased at early compaction time. Then, the CCV slightly decreased and remained stable. The CCV also exhibited a smaller standard deviation within this period. After this period, there was significant fluctuation in CCV and an increase in the standard deviation. Finally, similar to the CMV trend, the CCV value tended to be stable, which indicated there was no significant change in the spectrum of acceleration at the end of the compaction process.

One may find that the increase of CCV at initial compaction time was in consistent with that of RMV. Considering the definition of CCV and RMV, it can be concluded that the increase of CCV at the early compaction time was attributed to an increase at half fundamental component. Although there was difference in CCV for each mixture, change of CCV may also be roughly classified into three stages: 1) At the first stage, there was an increase in CCV; 2) At second stage, there was fluctuation and large standard deviation of CCV; and 3) CCV remains stable at the third stage. Furthermore, it seemed that the boundary of the first and second stage for CCV was the same as RMV, whereas the boundary of the second and third stage was the same as CMV.

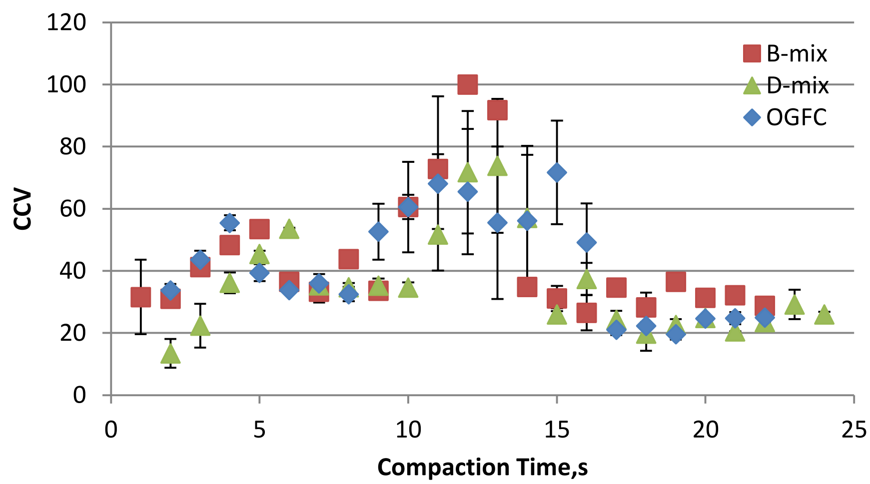


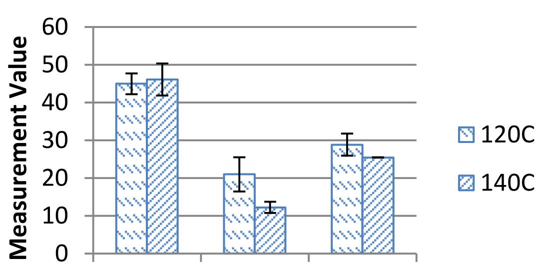
Figure 10.6. Relationship of compaction time versus CCV

10.3 Factors affecting the compactability of asphalt mixture

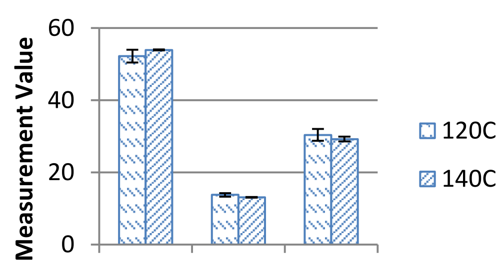
1 Influence of compaction temperature on compactability of asphalt mixture

Compaction temperature is believed to be an important factor influencing the compactability of asphalt mixture. There is a recommended range for compaction temperature, within which the optimum densification state for asphalt mixture can be obtained. Based on the engineering experience, two compaction temperatures, 120 Celsius and 140 Celsius, were used for evaluating the influence of temperature on compaction of asphalt mixture.

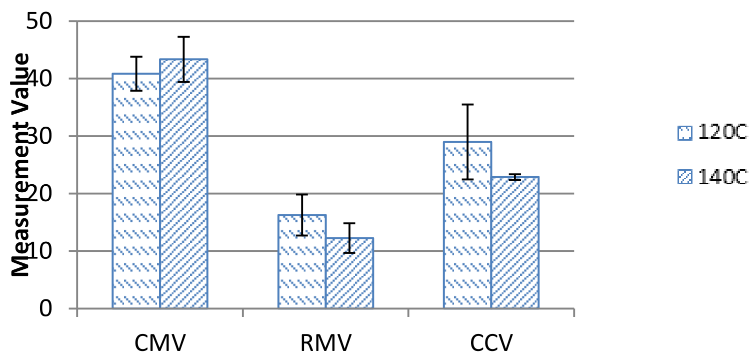
Figure 10.7 illustrated the comparison of measurement values for different asphalt mixture at two different compaction temperatures. The measurement values in the figure are the averaged value collected from the third compaction stages (“stable stage”). The error bar represented for the standard deviation of replications. In terms of CMV and CCV, the value for B-mix was the highest, followed by D-mix and OGFC. In terms of RMV, D-mix was ranked as the highest. RMV for OGFC tended to be slightly higher than B-mix. As for the influence of temperature, the RMV and CCV decreased with the increase of compaction temperature, whereas no significant difference of CMV between two temperatures.



(a) D-mix



(b) B-mix



(c) OGFC

Figure 10.7. Influence of compaction temperature on MV

Figure 10.8 illustrated the comparison of air voids for different mixtures. The "Standard" series represent the target air voids determined by Marshall Design method. It was found the air voids at two different temperatures were generally higher than the target air voids. For example, the air voids of B-mix at 140 □ and 120 □ were 5% and 6% respectively, comparing to the target air voids of 4%. As mentioned before, the edge effect of rectangular mold may result in relative high air voids as compared to the standard one. Nevertheless, the purpose of evaluating the air void in this study is not for mix design but for inspecting the influence of temperature on air void.

It was also found that the difference between the designed air void and final air voids for D-mix was larger than B-mix. This means D-mix was probably at under-compacted stage in terms of air voids, which may be attributed to the effective compaction thickness for asphalt mixture. Asphalt mixtures with thicker lift thickness and smaller nominal maximum aggregate size (NMAS) are harder to compact than those with thinner lift thickness and larger NMAS. The recommended maximum lift thickness for asphalt mixture with NMAS of 12.5 mm is generally from 5 cm to 6.35 cm, which is lower than the lift thickness in this study. Since the lift thickness for B-mix was within the optimum lift ranges, the difference between the final air void and designed air void is much lower than the others.

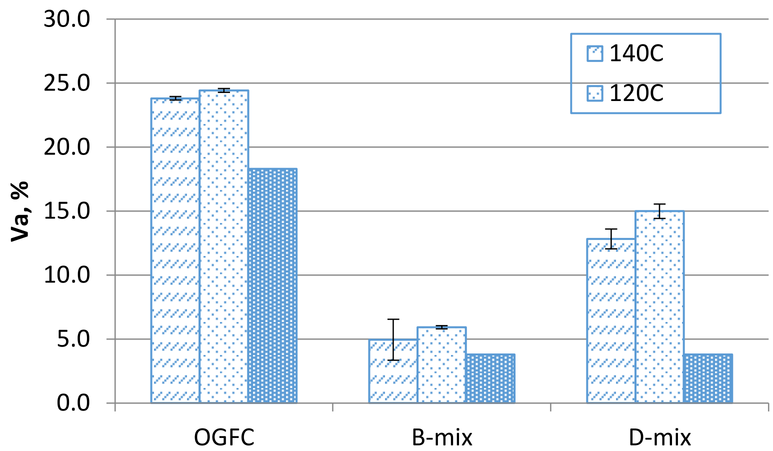


Figure 10.8. Comparison of air voids of different mixtures

Since the measurement values are served as an indicator of compaction stiffness in the IC technology, it is reasonable to compare the measurement values with the dynamic modulus of mixture. The Witczak model was developed to estimate the dynamic modulus of asphalt mixture from simpler material properties and volumetrics (Andrei, Witczak, & Mirza, 2005). In this model, the dynamic modulus at a given loading frequency and temperature depends on a number of factors such as the viscosity of the asphalt, the effective asphalt content and the percentage of air voids. Using the job mix formula and the actual air voids, the dynamic modulus for B-mix, D-mix and OGFC at 140 □ were calculated as 186, 76 and 17 MPa respectively, comparing to 241, 83 and 24 MPa respectively at 120 □. The rank of the calculated modulus of asphalt mixtures in agreement with that of CMV and CCV. It can also be observed that the dynamic modulus are close at two temperatures for the same mixture, which can be explained by the combined effects of a decrease of temperature and an increase of air voids. Based on these observations and analysis, CMV seems be more capable of differentiating materials. According to the definition, the RMV only contains one subharmonic frequencies components. The evolution of RMV indicated that RMV tended to remain stable from the beginning of second compaction stages. Therefore, it is insensitive to the changes of stiffness during the compaction, and not suitable for characterizing the stiffness properties of asphalt mixture during the compaction.

2 Influence of underlying layer on compactability of asphalt mixture

The block specimens of asphalt mixture with a thickness of 4 cm were employed as underlying layer to identify the influence of underlying layer on the lab compaction. It was placed into the modulus at ambient temperature prior to the loose asphalt mixture. Figure 10.9 illustrated the time-frequency spectrums for D-mix with an underlying layer. The fundamental frequency initially decreases and stably increases. Comparing with Figure 5, there was no evident fluctuation in fundamental frequency for asphalt mixture with underlying layer during the compaction process. This means the compaction process for D-mix with underlying layer tends to be more stable than D-mix without underlying layer. It should also be noted that the lift thickness of D-mix for Figure 10.9 was within the optimum lift thickness determined by NMAS.

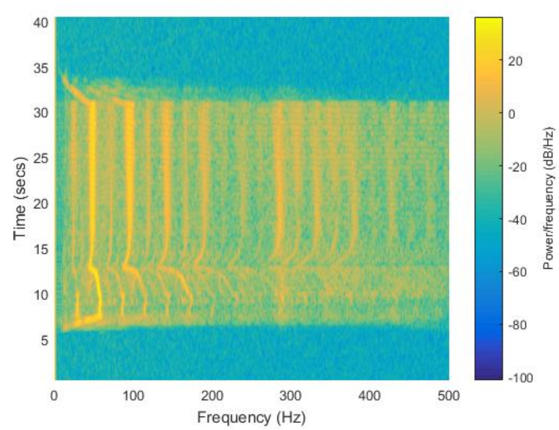
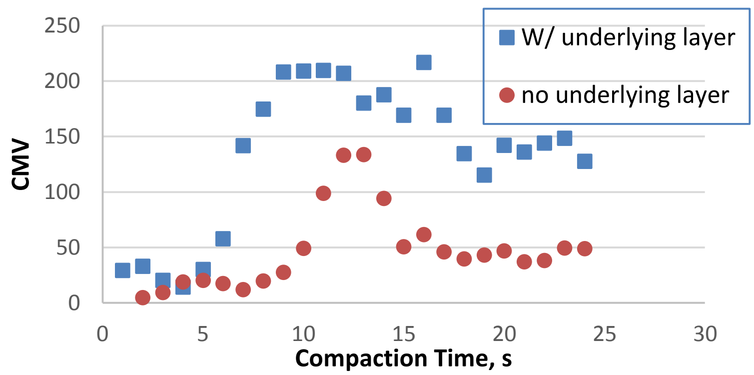
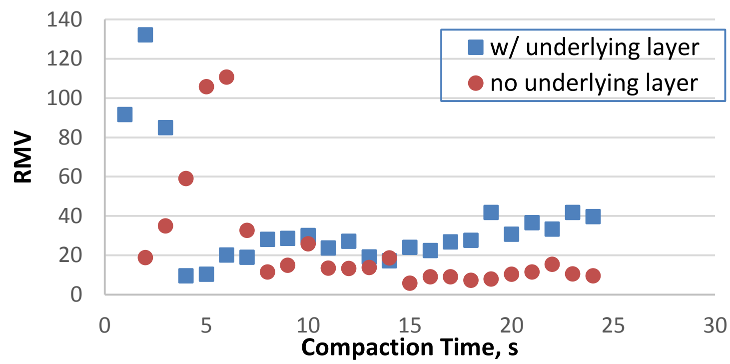


Figure 10.9. Time-frequency spectrum for asphalt mixture with underlying layer

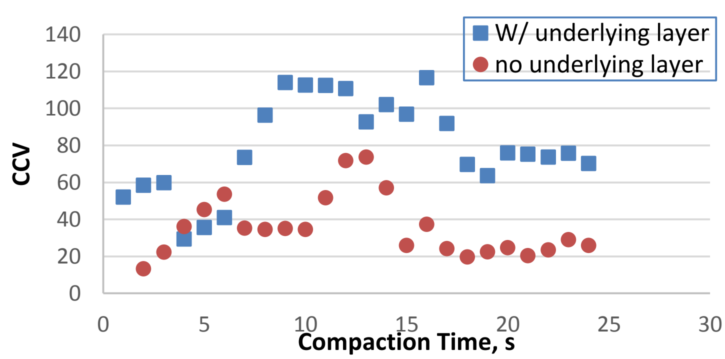
Figure 10.10 illustrated the change of measurement values with compaction time. D-mix without underlying layer from pervious study was also included for comparison. Comparing with D-mix without underlying layer, three compaction stages can be also identified. Generally, it is obvious that all the measurement values (CMV, RMV and CCV) for D-mix with underlying layer were greater than that without. This is because the stiffness for underlying layer at ambient temperature is considerably higher than the topping loose asphalt mixture, which increases these stiffness-related measurement values for the composite specimens.



(a) CMV



(b) RMV



(c) CCV

Figure 10.10. Change of measurement values with compaction time

10.4 Volumetric properties of asphalt mixture during vibratory compaction

In order to investigate the relationship between volumetric properties and vibratory properties, asphalt mixture specimens were compacted at different compaction time. The duration of compaction was 26 seconds with the interval of 2 seconds. At each interval, two specimens were compacted individually to determine the air voids. Both the evolution of air voids of asphalt mixtures and measurement values were monitored. Efforts were made to correlate the change of volumetric properties with measurement values.

Table 10.2 listed the results of linear regression models between air voids and measurement values (CMV and CCV). Except for CMV for OGFC, the coefficient of determination for each model was generally low, less than 0.5. In fact, it is difficult to find a model that fit the data well. Hence, one may conclude that there was no significant linear relationship between air voids and measurement values.

Table 10.2. Results between air voids and measurement values

Type of Mix	CMV	CCV
D-mix	-1.64Va+59.09 (R^2=0.046)	-0.02Va+47.47 (R^2=4E-06)
B-mix	-2.31Va + 43.00 (R^2 = 0.475)	-0.36Va + 45.16 (R^2 = 0.0076)
OGFC	-3.01Va + 103.67 (R^2 = 0.787)	-2.09Va + 97.23 (R^2 = 0.1406)

The evolution of measurement value indicates that there is a fluctuation in MVs during the compaction process. Therefore, uncertainties are introduced by this fluctuation. By excluding the unstable stage for each measurement values, the models were re-constructed through linear regression. Table 10.3 listed the results for linear model without unstable stage. It was found that the linear model fits well with the data.

Most coefficients of determination were greater than 0.5, which may be considered as linearly correlated.

Table 10.3. Results between air voids and measurement values (without unstable compaction stage)

Type of Mix	CMV	CCV
D-mix	-2.95Va+78.86 ($R^2 = 0.737$)	-1.70Va + 68.35 ($R^2 = 0.678$)
B-mix	-2.36Va+45.10 ($R^2 = 0.530$)	-0.81Va+ 41.39 ($R^2 = 0.153$)
OGFC	-3.13Va +106.04 ($R^2 = 0.834$)	-1.96Va+ 85.48 ($R^2 = 0.719$)

Comparing Table 10.2 with Table 10.3, the general trend of measurement values changing with air voids can also be identified. As the air void decreases, both CMV and CCV increase. This means the increase of compaction efforts results in an increase of CMV and CCV. As more compaction efforts are made, asphalt mixture becomes denser, resulting in an increase in stiffness for asphalt mixture. From the above results, it seems that both CMV and CCV are potential indicators of compaction properties for asphalt mixture. These values correlate well with air voids. The changes of these values during compaction are in agreement with that of stiffness.

Figure 10.11 illustrated the changing rate of air voids at each compaction stage. The changing rate is expressed as the ratio of change of air void to compaction time. The three compaction stages were determined in terms of CCV. It was found that stage 1 exhibited the highest changing rate of air voids, whereas stage 3 exhibited the lowest. It was also found that the changing rate between mixtures at stage 3 were almost identical. This means, at stage 3, additional compaction efforts may not necessarily result in a decrease in air voids or an increase in density. The compact properties for different asphalt mixture at stage 3 tended to be identical.

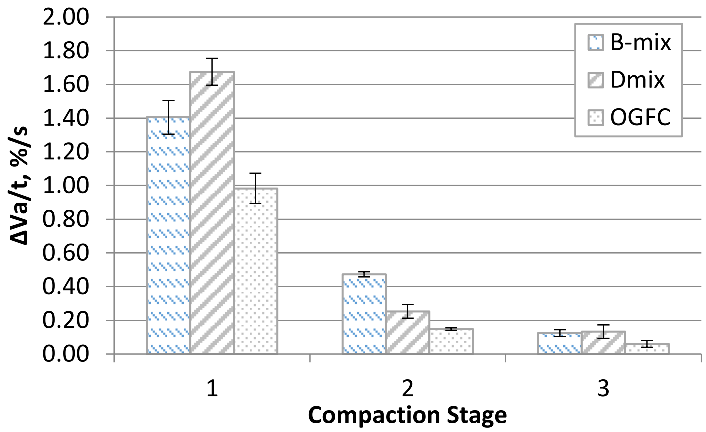


Figure 10.11. Changing rates of air voids at each compaction stage

Figure 10.11 also indicated the densification of asphalt mixture was mainly contributed by the compaction efforts at stage 1. Although there is a fluctuation in stiffness, which is represented by MV, during the compaction stage 2, the changes in density of asphalt mixture is relative low comparing with that at stage 1. At stage 3, there is a stable change of density for each mixture. However, the compaction efficiency is the lowest one comparing with the former two stages. It should be noted that the air voids for D-mix specimens were still above 12% at stage 3, while the changing rates of air void were only 0.13% per second. This means although D-mix specimen is under-compacted, additional compaction efforts provide limited contribution to the increase of density. As mentioned above, the lift thickness for D-mix is out of the optimum compaction thickness. In order to increase the density of asphalt mixture, one may decrease the lift thickness of asphalt mixture. However, it should also be pointed out that the shape of sample mold is also responsible for the results. It is hard to compact and obtain sufficient compaction degree at the edge and corner of a rectangle specimen. It is recommended that further tests be conducted on cylindrical specimens to validate the findings from this study.

10.5 Summary

In this study, the AVC was used to simulate the IC compaction of asphalt mixture in the field. The one-dimension accelerometer was employed to monitor the oscillation properties of the vibrator based on which the response-derived stiffness properties of asphalt mixture were characterized. Fourier Transform was employed to construct time-frequency spectrum of acceleration. The stiffness properties of asphalt mixture during compaction were evaluated by response-derived measurement values (CMV, RMV, and CCV) developed from IC technology. The main conclusions were summarized as below:

1. The change in time-frequency spectrum indicated that the vibratory compaction process can be classified into three different stages. At the first stage, the CMV or CCV gradually increases. At the second stage, there is a significant increase in values with higher standard deviations. At the third stage, the measurement value of asphalt mixture tended to be stable.
2. CMV seems more suitable to characterize the changes in stiffness properties of asphalt mixture during compaction and was used for determining the compaction stages in this study.
3. The final CMV for B-mix was the highest, followed by D-mix and OGFC both at 140 ℃ and 120 ℃, which was in agreement with the rank of the calculated modulus of asphalt mixtures.
4. There was a significant increase in measurement values for asphalt mixture with underlying layer during compaction comparing with the original values without underlying layer, around 150 vs. 50 for CMV and 80 vs. 20 for CCV.
5. There was a linear relationship between air voids and measurement values with the data from the second stage excluded. Most coefficients of determination were greater than 0.5 for three mixtures using CMV and CCV.
6. The densification of asphalt mixture was mainly contributed by the compaction

efforts at stage 1, whereas additional compaction efforts at stage 3 may result in a limited decrease in air voids.

CHAPTER 11 ECONOMIC ANALYSES

In this study, economic analyses were included to show the financial benefits of IC technology.

11.1 Introduction and Methodology

As a feature of IC rollers, reduced construction and maintenance costs have been reported by the FHWA (Savan, Ng, & Ksaibati, 2015). Previous researches and the practices in this study revealed that IC technology can have the following benefits in the overall quality of pavements (Beainy, Commuri, & Zaman, 2011):

- Providing a complete coverage of the compaction area,
- Ensuring uniform compaction,
- Reducing the overall cost of construction,
- Reducing the life-cycle cost by increasing the service life of pavements,
- Providing the quality results in real time, thereby aiding the QC/QA process,
- Providing mapped and stored information for later use in the long-term pavement management.

However, limited cost-benefit data are available according to the result of a literature review by Savan et. al (Savan, Ng, & Ksaibati, 2016). They also performed a survey to investigate the current knowledge of IC among professionals. According to their survey, several DOTs were in the process of implementing or considering IC; however, the amount of data on costs is very limited. Ten respondents of the survey reported an increase in construction related costs when using IC; however, this increase can be associated with the training for roller operators and being unable to take advantage of economies of scale. Long-term benefit and cost data were less available for this survey.

In this study, a benefit-cost analysis will be performed to systematically evaluate the economic benefits of IC based on two separate periods: Construction Cycle Cost and

Roadway Life Cycle Cost. The differences between the conventional and intelligent compaction types was analyzed and compared for each time period.

Construction Cycle Cost

The construction cycle begins with the preparations for conducting pavement compaction and ends at the completion of QC/QA testing. This includes the costs for rollers, labor to operate the rollers, and conducting QC/QA testing. The cost for a new IC roller is about 3 to 5 percent more than the cost of a conventional roller, and the cost of retrofitting an IC system on a conventional roller ranges from $50,000 to $75,000, depending on the manufacturer and desired features (Nieves, 2013).

Conventional compaction includes any method of roadway compaction and subsequent QC/QA methods that does not use a roller equipped with on-board stiffness measuring devices. QC/QA data for conventional compaction is obtained by in-situ field tests, primarily using nuclear gauge and core sampling for pavements. As a comparison, IC compaction means the compaction of a roadway section by use of a IC roller, including the use of an accelerometer, GPS unit, and on-board computer. QC/QA data is obtained from the roller and is analyzed by a QC/QA technician or engineer.

For conventional compaction, construction costs regarding roller equipment and labor are obtained using pricing data from Savan et. al as a reference (Savan et al., 2016). The costs are set as an hourly rate as shown in Table 11.1 so that they can be used for different projects in this study. The amount of time for the conventional compaction to complete the projects in this study needs to be estimated since that all projects in this study actually used IC rollers. In order to calculate the number of hours for conventional compaction, a 30 percent increase in the number of hours it would take a IC roller is applied. The increase rate was based on the number of roller passes from IC rollers compared to conventional rollers to perform similar compaction work as observed by Briaud and Seo (Briaud & Seo, 2003). The actual compaction hours using IC rollers were gathered using the IC recording. The QC/QA costs are another part of the total construction cost. The information for the cost of testing can be obtained by surveying contractors on their costs related to QC/QA in either an hourly or unit area rate. In this study, the QC/QA costs were decided according to the research of Savan et. al in a unit area rate as shown in Table 11.1. For the conventional compaction, the in-situ field tests as QC/QA data will continue until the finish of the project, so the total QC/QA costs equal the unit area rate multiplying the area of the project. The construction costs for conventional compaction will be calculated using the equation as follows:

$$Construction\ cost = Compaction\ time * (Roller + Operator\ cost\ per\ hour) + QC/QAunit\ cost * Total\ area \tag{11.1}$$

For IC compaction, construction costs were calculated based on the actual construction time and the cost of an IC roller. In this study, each project had an Intelligent Compaction Bid Price which gave the specific cost of retrofitting IC systems on conventional rollers. However, this retrofitting cost should be considered as a long-term investment which will benefit the other projects in the future. Assuming that the IC roller has an average service life of 10 years and 500 work hours in a year, the additional cost of IC roller for a specific project should be based on the ratio between the work hours of this project and the total possible work hours of roller (5000 hours). One benefit of IC compaction is a reduction of QC/QA costs comparing to the conventional compaction. By providing a method to gather compaction data for 100 percent of the pavement, QC/QA costs for IC can be reduced to the area of the test section required to calibrate conventional testing methods with the ICMVs, and the compaction quality of the rest of construction area can be monitored using the target ICMV and no further in-situ point tests are needed. Therefore, the construction costs for IC compaction can be calculated using the equation as follows:

$$Construction\ cost = Compaction\ time * (Roller + Operator\ cost\ per\ hour) + IC\ retrofitting\ cost * Compaction\ time / Service\ life\ of\ roller + QC/QAunit\ cost * Test\ section\ area \quad (11.2)$$

The input data for benefit-cost analysis are shown in Table 11.1.

Table 11.1 Input data for benefit-cost analysis. From (Savan et al., 2016)

Item	Unit cost (US$)/quantity
QC/QA per square meter	$ 0.04784
Conventional roller cost per hour	$ 36
Roller operator per hour	$ 30

Roadway Lifecycle Cost

As one of the largest benefits, IC can provide a more uniform compaction, resulting in an extended pavement life. To evaluate the effects of compaction uniformity on pavement performance, Xu et al. used three-dimensional finite element model and the mechanistic empirical pavement design guide (MEPDG) model to estimate pavement performances in terms of rutting and fatigue life based on the Bomag IC roller (Xu, Chang, Gallivan, & Horan, 2012). It was found that the mean value of fatigue lives for the heterogeneous pavement model is 38.2% of those for the uniform pavement model. In other words, if the pavement life is determined by its fatigue life, a uniform pavement achieved using IC roller will have more than twice the fatigue life of a pavement compacted by a conventional roller. In order to calculate the benefit from using IC, the total costs of projects in this study were acquired from TDOT. The regular service life for asphalt overlay using conventional compaction in Tennessee is between 10 to 12 years; therefore, the cost of resurfacing per year for the conventional compaction should be the total costs of projects divided by the remaining service life (10 years in this study). The service life of asphalt resurfacing using IC compaction should be decided based on the field observation and testing. However, all the projects using IC compaction in Tennessee were finished less than four years and no service life data about the IC compaction is available now. According to the research of Xu et al., it is reasonable to claim that the service life of the uniform resurfacing using IC compaction will be similarly increased by a factor of 2.6. It should be noted that the suggested service life for IC compaction was obtained from only one study, and the oxidation process is inevitable and more serious for the surface layer. The service life of the resurfacing project is hard to estimate accurately. Therefore, a sensitivity analysis will be presented later to illustrate the effect of different service lives on associated lifecycle cost savings.

11.2 Case Studies

The cost comparison between the two compaction methods is comprised of a summation of the costs from both the construction cycle and roadway lifecycle. To illustrate the economic benefit of IC technology, four resurfacing projects in this study were utilized to perform the benefit-cost analysis. Table 11.2 contains the data obtained from TDOT and the IC record, which will be used for the analysis.

Table 11.2 Project data for benefit-cost analysis.

Project No	Contract Number	County	Lane Miles Paved	IC hours	Total Contract Amount	IC Bid Price
1	CNM194	Crockett	33.56	81	$4,229,507.61	$55,000.00
2	CNM278	Lincoln	25.5	63	$1,689,998.68	$112,620.00
3	CNM201	Hamilton	12.95	51	$1,673,709.32	$27,085.95
4	CNM165	Knox	21.72	45	$1,527,093.85	$31,580.11

Construction Cycle Cost

As discussed before, the construction cycle begins with the preparations for conducting pavement compaction and ends at the completion of QC/QA testing. This includes the costs for rollers, labor to operate the rollers, and conducting QC/QA testing. For conventional compaction, the unit costs for the roller, operator, and QC/QA for are listed in Table 11.1, and the total areas for QC/QA testing are calculated using the lane length in Table 11.2 and a lane width of 3.7 m. To calculate the number of hours for conventional compaction, a 30 percent increase in the number of hours it would take a IC roller as shown in Table 11.2 is applied. By using Equation 11.1, the construction cycle costs for four projects were calculated as shown in Table 11.3. The roller cost includes both the breakdown roller and the intermediate roller since usually these two rollers were retrofitted with IC equipment in this study.

For IC compaction, construction costs were calculated based on the actual construction time and the cost of an IC roller using Equation 11.2. The IC compaction time is shown in Table 11.2. The IC retrofitting cost is the IC bid price in Table 11.2, which will be prorated to the cost of project according to the compaction time and the service life of the roller. The QC/QA costs for IC can be reduced to the area of the test section required to calibrate conventional testing methods with the ICMVs. For example, project 3 used a test strip around 1 km in length to calibrate the NG tests with the ICMVs. The test section area for four projects were calculated by multiplying a length of 1 km and a lane width of 3.7 m. By using Equation 11.2, the construction cycle costs for four projects using IC technology were calculated as shown in Table 11.3.

Table 11.3. Construction cost for conventional and Intelligent compaction.

No	Conventional compaction (US$)			Intelligent compaction (US$)				Cost ratio
	Roller cost	QC/QA cost	Total cost	Roller cost	Retrofitting cost	QC/QA cost	Total cost	
1	13899.6	9558.1	23457.7	10692.0	891.0	177.0	11760	0.50

2	10810.8	7262.5	18073.3	8316.0	1419.0	177.0	9912.0	0.55
3	8751.6	3688.2	12439.8	6732.0	276.3	177.0	7185.3	0.58
4	7722.0	6186.0	13908.0	5940.0	284.2	177.0	6401.2	0.46

As shown in Table 11.3, when comparing the construction cost between conventional compaction and Intelligent compaction, the roller cost for Intelligent compaction is less than that of conventional compaction for all projects due to an improvement of compaction efficiency. There is an additional retrofitting cost for Intelligent compaction; however, a total roller cost including the retrofitting cost for Intelligent compaction is still less than the roller cost for conventional compaction. The main cost difference between conventional compaction and Intelligent compaction lies in the QC/QA cost since that the QC/QA tests should cover the whole compaction area for conventional compaction, whereas they were only performed on the test section for Intelligent compaction. As shown in Table 11.3, the total construction cost for Intelligent compaction is about half of that for conventional compaction. It should be noted that the assumption of IC efficiency was based on one study.

Roadway Lifecycle Cost

One of the largest benefits of IC is that it can provide a more uniform compaction, resulting in an extended pavement life. As for Intelligent compaction, the total costs of four projects were calculated by adding the total contract amount and the IC bid price in Table 11.2. Then the total cost was divided by the service life increase from the improvement to obtain the cost per year. As mentioned before, the service life using conventional compaction was ten years; however, the service life using intelligent compaction can be increased to 26 years according to the research of Xu et al. (Xu, Chang, Gallivan, et al., 2012). Therefore, the total cost for intelligent compaction was divided by 26 years to yield the annual cost for IC. For conventional compaction, the total cost can be considered equivalent to the total contract amount in Table 11.2. There is a difference for QC/QA costs between conventional compaction and intelligent compaction as shown in Table 11.3; however, the QC/QA costs is negligible in the total cost, and no further adjustment for the QC/QA cost was performed. The total cost for conventional compaction was divided by ten years to yield the annual cost as shown in Table 11.4.

Table 11.4. Lifecycle cost for conventional and Intelligent compaction.

Project	Conventional compaction cost per year (US$)	Intelligent compaction cost per year (US$)
1	422950.8	164788.8
2	168999.9	69331.49
3	167370.9	65415.2
4	152709.4	59949

As shown in Table 11.4, the cost savings for IC compared to conventional compaction is significant for all four projects when assuming that IC results in 2.6 times the service life of a conventionally compacted pavement. The cost savings using IC resulted from increased compaction uniformity. However, this suggested increase in service life was obtained from one study, and the oxidation process is more serious for the surface layer. Due to the limited amount of research on increased service life from using IC, a sensitivity analysis was performed to illustrate the effect of different service lives on associated lifecycle cost savings. Figure 11.1 shows the lifecycle cost savings based on a variety of service life multipliers from using IC.

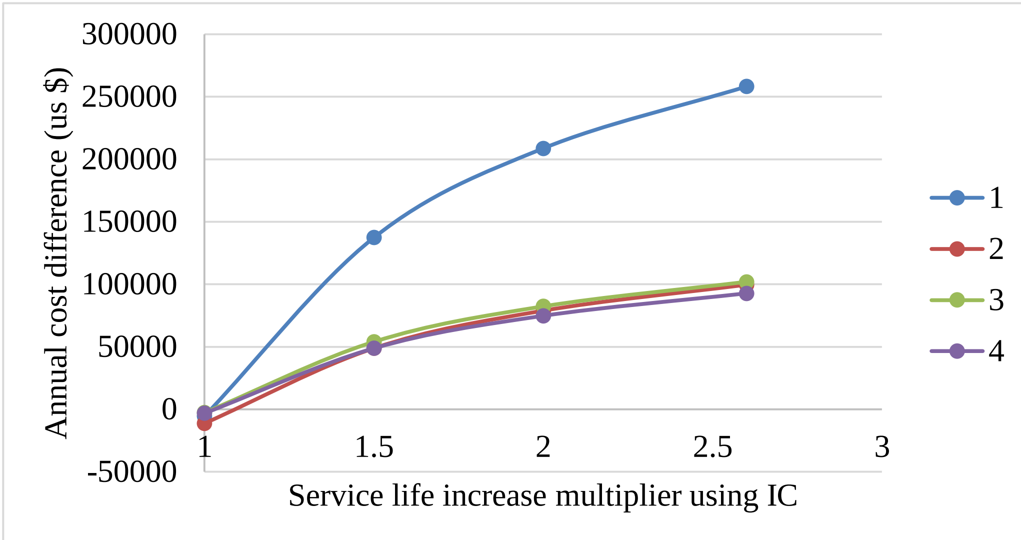


Figure 11.1. Lifecycle cost saving based on service life improvement from IC

As shown in Figure 11.1, following an increase of service life by using IC, the annual cost savings for all four resurfacing projects increase. The multiplier of 2.6 times the service life of a conventionally compacted pavement for project 1 results in an annual cost savings of US$ 258162. A 1.5 times improvement for the same project results in an annual savings of US$ 137316, and annual cost savings decreases rapidly as the multiplier approaches one. When the service life of pavement using IC is the same as the one using conventional compaction, the annual cost saving for project 1 has a negative value of US$ 5500 due to the extra IC retrofitting cost. The same trend exists for the other projects.

11.3 Summary

The performance of a pavement is related to the quality of the compaction. Conventional compaction and testing methods are insufficient for evaluating the compaction quality, whereas IC can provide both a more uniform compaction and a method to increase QC/QA testing coverage to 100 percent.

In this study, benefit-cost analysis was performed to systematically evaluate the economic benefits of IC. Two specific cycle costs were developed based on costs for construction of a pavement and savings from improved compaction uniformity over the pavement lifecycle. The benefit-cost analysis in this study demonstrated a nearly 50% decrease in construction costs for all four projects when using IC. The increased service life from using IC was determined based on increased compaction uniformity, which resulted in a significant annual cost savings for all projects. However, it should be noted that the assumption of IC efficiency and the suggested service life for IC compaction was based on one study, and the applicability of the analysis for the projects in Tennessee should be investigated in the future.

CHAPTER 12 CONCLUSIONS AND RECOMMENDATIONS

In the research project, a total of six asphalt mixture projects and one soil project were constructed to evaluate the IC technologies for various construction materials. All asphalt mixture projects were performed in Tennessee, and the soil project was finished in China to provide additional data for soil compaction.

Three different in-situ testing methods including Portable Soil Moisture Meters, Core cutter method, and FWD were employed to evaluate the correlation between in-situ soil tests and ICMVs, whereas four in-situ and laboratory testing methods, including NG and core density, permeability, and IDT test, were employed to evaluate the relationship between physical and mechanical properties of asphalt mixture and ICMVs. Simple linear and multiple linear regression analyses was performed to develop correlations between ICMVs and various in-situ test results. Geostatistical semivariogram analysis was performed on spatially referenced ICMVs to evaluate the spatial uniformity of IC measurements and to quantify the compaction quality of soil and asphalt mixtures. The original Witczak model and the multilayer pavement analysis software BISAR were employed to calculate the displacements of the newly constructed asphalt layer and underlying layers under a vibratory roller. Based on the results of the laboratory and on-site tests, the following conclusions can be summarized:

1. For soil compaction, water content of soil had a significant effect on CMV value. A strong and stable linear relationship could be identified between CMV and deflection of soil layer when water content of soil was consistent.
2. For asphalt compaction, a significant linear relationship could be established between CMV and roller vibration amplitude among the operation parameters for Project 3, which provides a basis for adjusting CMV value. The adjustment of CMV and the

rising RMV value could help identify the over-compacted area and the optimum pass number.

3. A theoretical analysis showed that the difference in 1/CMV of the same point could filter the effect of underlying support on CMV and reflect the changes in compaction degree of the newly placed asphalt resurfacing layer.
4. There was a strong correlation between the difference in 1/CMV and core density for Project 3, while no correlation could be found between the original CMV and core density. However, no similar correlation could be identified for asphalt base layer (Project 6), mainly due to the properties of the asphalt layer and the stiffness of the underlying layers. The fact that CMV is sensitive to road modulus indicates that it is not suitable for evaluating the compaction level of asphalt base layer.
5. For asphalt core tests, the density of the cores from all projects showed a strong correlation with the permeability and IDT strength; whereas, the permeability of core samples correlated well with IDT strength for most projects. As the most important factor among IC parameters for asphalt compaction, the starting surface temperature of the breakdown roller had a stable and significant relationship with core densities for all projects.
6. Based on the core test results, it is recommended that three IC parameters be selected for evaluating the compaction quality of the resurfacing project: the difference in 1/CMV, the starting surface temperature of compaction (>110℃), and the total number of passes (>2 passes).
7. Compared with conventional quality control methods, the geostatistical analysis is capable of evaluating the spatial uniformity and identifying the weak locations during compaction. On the other hand, the measurement depth of ICMV and its relationship with the roller operation parameters should be taken into account during the analysis. In general, the geostatistical method can serve as a supportive tool for evaluating the compaction quality with IC technology.

8. The IC compaction of asphalt mixture was simulated by utilizing AVC in the laboratory. A one-dimension accelerometer was employed to monitor the oscillation properties of the vibrator, based on which the response-derived stiffness properties of asphalt mixture were characterized. The stiffness properties of asphalt mixture during compaction were evaluated with response-derived measurement values (CMV, RMV, and CCV), and linear relationships could be found between air voids and measurement values.
9. Benefit-cost analysis was performed to systematically evaluate the economic benefits of IC. Two specific cycle costs were developed based on costs for construction of a pavement and savings from improved compaction uniformity over the pavement lifecycle. The benefit-cost analysis indicated a nearly 50% reduction in construction costs for all four projects using IC. The increased service life resulting from using IC was determined based on increased compaction uniformity, which led to a significant annual cost saving for all projects.

The following were recommended for future studies:

10. To improve the understanding of IC for asphalt compaction application, more future studies are needed. For example, the plastic deformation of compaction layer should be considered when performing a theoretical analysis on CMV value. Also, further research should be performed to define a more appropriate index of IC to reveal the compaction level of asphalt layer in the future.
11. It is also recommended that IC data be incorporated into a pavement management system (PMS) so that long term benefits of IC technology may be identified in the future.

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