ABSTRACT

Title of Dissertation:	**INCORPORATING PERFORMANCE REQUIREMENTS IN ASPHALT MIXTURE DESIGN**
	Anjuman Ara Akhter, *Doctor of Philosophy*, ***2023***
Dissertation directed by:	Associate Professor Dimitrios Goulias, Department of Civil and Environmental Engineering

In recent years transportation agencies have been focusing on performance-based asphalt mixture design to ensure durable pavements. Including performance in the design, phase allows the prediction of expected distresses, such as fatigue cracking, permanent deformation, and moisture damage. The main objective of this study was to identify a new approach to include performance testing in asphalt mixture design for the state of Maryland. The following specific objectives were identified to achieve this: (i) identifying the cracking and rutting criteria for asphalt mixtures in Maryland; (ii) assessing the repeatability of the selected performance tests; (iii) establishing model-based performance predictive approach for designed asphalt mixtures; (iv) adopting a non-destructive testing method (i.e., Ultrasonic Pulse Velocity – UPV) in Quality Assurance (QA) of asphalt mixtures.

Two well-accepted and suitable performance tests for Maryland conditions were selected to address the first objective. These tests included the IDEAL Cracking Test (IDEAL-CT) for fatigue

cracking and the High-Temperature Indirect Tensile Strength Test (HT-IDT) for permanent deformation. Such performance index tests were combined with volumetric requirements and benchmark analysis. Since mixture properties affect each of these typical distresses in asphalt mixtures and pavements differently, a Balanced Mix Design approach was adopted, BMD. The sources of variability in testing were quantified through round-robin testing between laboratories for the second objective. Based on the results and findings, an adjustment procedure was developed. For the third specific objective, a methodology was proposed for predicting field performance from laboratory testing and mixture volumetrics considering (i) well-accepted prediction models by the research community and (ii) fundamental asphalt material behavior parameters representing mix quality and well-related to performance. A sensitivity analysis of UPV regarding mixture volumetrics and testing conditions was carried out for the final objective. The resulting asphalt mixture stiffness from such an evaluation was then compared to the results from traditional destructive testing for pertinent conclusions. Based on these analyses and results, a framework was proposed for adopting UPV in the BMD mix design approach developed in this study. The research and methodology developed in this study can be used elsewhere, where similar materials are used.

INCORPORATING PERFORMANCE REQUIREMENTS IN ASPHALT MIXTURE DESIGN

by

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Dissertation submitted to the Faculty of the Graduate School of the
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Dedication

To my father, late Azizul Islam.

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List of Abbreviations

ASTM: American Society for Testing and Materials

BMD: Balanced Mix Design

COV: Coefficient of Variation

DM: Dynamic Modulus

FHWA: Federal Highway Administration

HT-IDT: High-Temperature Indirect Tensile Test

IDEAL-CT: IDEAL Cracking Test

IDT: Indirect Tensile Test

ILS: Interlaboratory Laboratory Study

JMF: Job Mix Formula

MDSHA: Maryland State Highway Association

NCHRP: National Cooperative Highway Research Program

NDT: Non-Destructive Test

QA: Quality Assurance

QC: Quality Control

TSR: Tensile Strength Ratio

UPV: Ultrasonic Pulse Velocity

VTM: Voids in Total Mix

VFA: Voids Filled with Asphalt

VMA: Voids in Mineral Aggregate

Chapter 1 : Introduction

1.1 Motivation and Background

More than ninety percent of the roadways are currently surfaced with asphalt mixtures. In the USA, this represents an industry of more than 3,600 hot mix asphalt plants nationwide, producing 400 million tons of asphalt mixtures yearly, worth more than 26 billion dollars (FHWA, 2021). In the current economic environment, long-lasting materials and pavements are a priority. In doing so, mixture design should consider performance testing complementing the volumetric analysis currently used by many agencies. Besides, state agencies are moving forward with sustainable pavements, which encourages reducing the use of virgin materials, hence increasing recycling. Furthermore, volumetric-based asphalt mixture design is insufficient to quantify the effect of novel technologies such as warm mix technology, polymer-based binders, fibers, and others.

Current volumetric-based practice lacks a performance optimization process for specific applications that consider factors other than traffic and climate. For instance, mixture location within a pavement structure, existing pavement conditions for overlays, and reflective cracking relief interlayers are a few. State Highway Agencies are currently focusing on developing Balanced Mix Design (BMD) for incorporating simple yet effective cracking and durability assessment of asphalt mixtures during design (Hajj et al., 2021). BMD considers performance criteria in asphalt mixture design regarding typical distresses, such as cracking and rutting (FHWA, 2016). Performance tests need to be identified and selected for such an approach considering loading and climatic effects in relation to mixture materials and properties. While there is no uniformity in the direction that each state considers, a provisional standard of practice was recently developed, AASHTO PP 105-20 "Standard Practice for Balanced Design of Asphalt Mixtures, (2020), in providing some guidance. Four different approaches were identified in this standard concerning the combination of volumetric and performance testing criteria considered.

Several Superpave mix design implementation studies have explored alternative testing methods for assessing mixture performance. For the development of BMD, the selection of the performance tests depends on the agency's experience with performance testing, field experience in implementation, equipment availability, and other parameters. Since contractors should also use these tests in quality control (QC) of asphalt mixtures during production, factors such as equipment cost, testing time, training, ease of operation, and results analysis are also critical factors for selecting such methods. Recent studies (Bennert et al., 2018; Yin et al., 2020) have found that Indirect Tensile Strength (IDT) based methods show promising results regarding the abovementioned factors. The indirect tensile asphalt cracking test (IDEAL-CT) developed recently is simple, practical, and efficient. The test is sensitive to crucial asphalt mix components and volumetric properties, including recycled materials content (such as RAP or asphalt shingles content), asphalt binder type, binder content, aging conditions, and air voids, as well as specimen thickness, loading rate, and testing temperature (Zhou et al., 2017). This study targeted IDEAL-CT and the Indirect Tensile Strength at high temperatures, HT-IDT, to develop the performance acceptance threshold requirements for a BMD approach for Maryland mixtures.

Incorporating new tests in the design and quality control procedures poses challenges. Although IDT-based test methods are widely accepted among state agencies for their accuracy and adaptability, variability in test results is still an issue. Since various sources of variability may skew the results, a unique approach to counter such effects should be identified.

Another critical aspect of implementing performance testing in the design and production phase of asphalt mixtures is that the results should accurately predict the mixtures' field performance. For that, historical field data are required for the mixtures to relate them to laboratory results. The lack of historical performance data for newly implemented testing methods may lead to developing predictive models for predicting field performance in lab testing. The IDEAL-CT and HT-IDT test methods have been studied for accuracy in terms of performance prediction in the design phase using short-term field aging data or lab permanent deformation testing results using testing such as the Asphalt pavement analyzer (Zhou et al., 2017; Bennert et al., 2018). However, assessing the long-term field performance is still needed. Thus, there is a need to develop an approach for predicting the long-term field performance (simulating climatic and traffic exposure) of the designed mixtures and, in the BMD scenario, to relate them to the IDEAL-CT and the HT-IDT test results.

Beyond these destructive testing methods, the potential use of Non-Destructive Tests (NDTs) is desired. Ultrasonic Pulse Velocity (UPV), Resonant Frequency Tests, and Infrared thermography are some examples considered in a laboratory setting for the characterization of asphalt materials. Speed and high repeatability of testing are two primary factors identified by researchers needed to characterize different building materials with NDTs (i.e., concrete, rocks, and asphalt).

1.2 Research Objectives

The study's main objective was to develop a rigorous framework to include performance tests in the design phase of asphalt mixtures. The specific objectives of this dissertation research are briefly described herein.

- The first objective of the study was the laboratory performance evaluation of asphalt mixtures to establish Maryland's balanced mix design criteria. This task was carried out by defining an appropriate BMD approach that will be easily adopted by the state agency and the asphalt industry. Asphalt mixtures were to be evaluated through selected performance tests that assess fatigue cracking and permanent deformation (i.e., rutting), Figure 1-1, and yet are easy to run by producers during mix design and production. This requires a balancing act between the two distresses, Figure 1-2, since the effect of binder content and volumetrics affect differently such performance parameters (fatigue cracking and permanent deformation). The selected testing methods included the IDEAL Cracking Test for fatigue cracking, IDEAL-CT, recently proposed by an SHRP2 IDEA project, and the High-Temperature Indirect Tensile Strength Test, HT-IDT for permanent deformation assessment.

- Along with mixture characterization, acceptance criteria were defined through alternative threshold selection approaches. Such analysis is a "benchmarking" approach from the characterization of 13 gap and dense graded mixtures used typically in the state of Maryland. Validation will require relating such lab results to long-term field performance.
- The next objective was establishing precision statements for the IDEAL-CT and HT-IDT test methods through variability analysis. The tasks included round-robin testing on selected mixtures with different laboratories to evaluate the variability components and sources and establish relevant conversion factors for comparing results between laboratories.
- The third objective of the study was to predict field performance, in terms of service life, for the asphalt mixtures with the BMD approach. A predictive modeling approach was first examined to assess whether fatigue cracking and permanent deformation (i.e., rutting) service life of mixtures can be estimated from the volumetric properties. The study then linked the relationship between laboratory performance and IDEAL-CT and HT-IDT to such predictive service life.
- The final objective was to explore the response of Ultrasonic Pulse Velocity (UPV) on asphalt mixtures for its potential adoption in the Quality Assurance/ Quality Control of BMD. This task was carried out by performing a sensitivity analysis of the method to the volumetric properties of asphalt mixtures and the testing conditions, determining the response of UPV by comparing it with common destructive methods, and developing a framework to adopt this test method in the BMD approach.

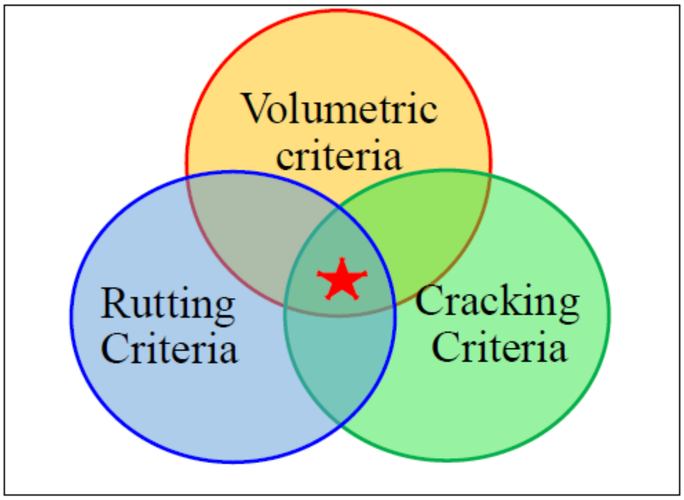


Figure 1-1 Integrating performance to volumetric requirements (FHWA 2022)

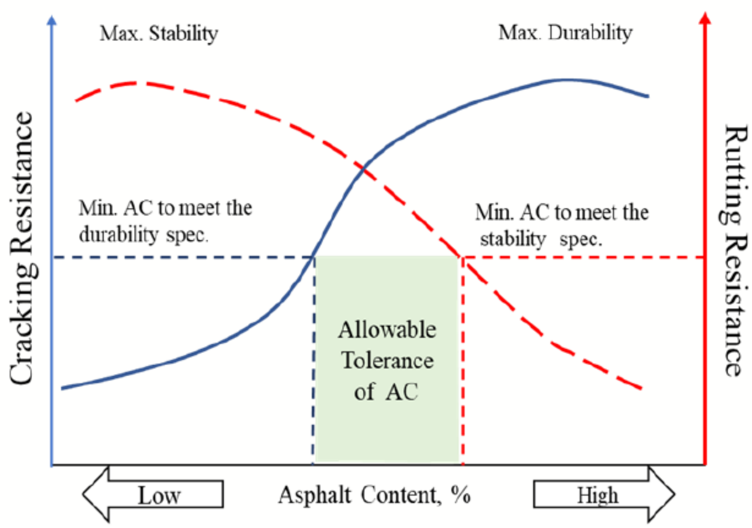


Figure 1-2 Balancing performance criteria (Al-Khayat et al., 2021)

1.3 Organization of The Dissertation

The research steps are illustrated in Figure 1-3, and the dissertation organization is described next.

This dissertation is organized in the following chapters. Chapter 1, Introduction, discusses the background and need for this study along with the objectives. Chapter 2 presents the initial selection of the performance criteria (through IDEAL-CT and HT-IDT) for Maryland's BMD asphalt mixture design. Chapter 3 presents the analysis of the sources of variability in IDEAL-CT and HT-IDT and the suggested approach for adjustments when testing data from alternative testing laboratories are used. Chapter 4 presents the modeling approach for predicting the asphalt mixture's field performance (fatigue cracking and permanent deformation) from the volumetric properties and considering the prediction of the dynamic modulus, rut depth, cracking potential, and pavement service life. The relationships between lab performance (through the IDEAL-CT and HT-IDT testing results) and predictive pavement service life are also explored. Chapter 5 presents the study results for assessing the potential adoption of ultrasonic pulse velocity (UPV) in the BMD mixture quality control process. Chapter 6 summarizes the study findings and the recommendations for future work.

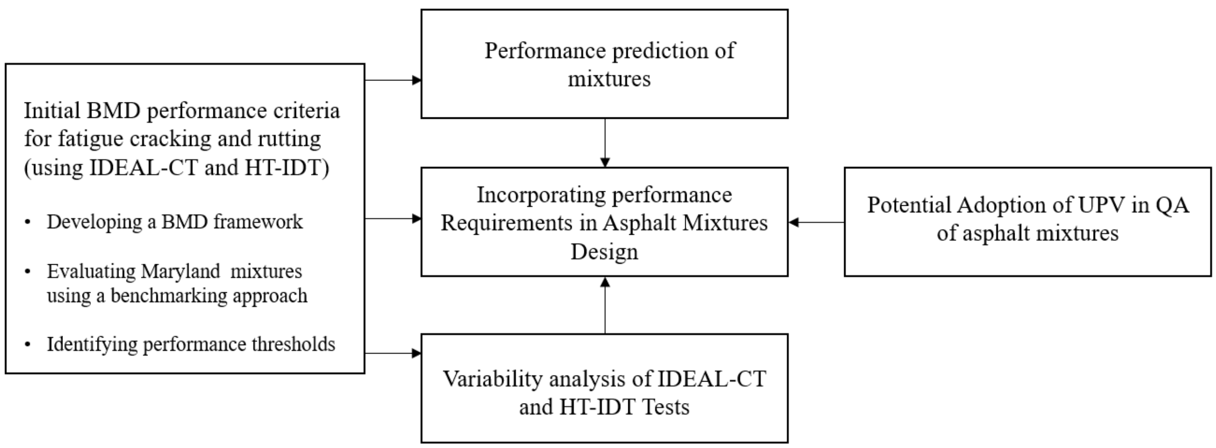


Figure 1-3 Schematic diagram of the study

Chapter 2 : Initial Quality Acceptance Threshold for Permanent Deformation and Fatigue Cracking for BMD

2.1 Introduction

For many years state highway agencies and the asphalt industry focused on including performance assessment of asphalt mixtures in their design for different modes of distress (i.e., fatigue cracking and permanent deformation) for producing durable pavements. Recently, several highway agencies developed a new approach, Balanced mix design, BMD (West et al., 2018; Hajj et al., 2021). Previously, most states used the volumetric-based mix design, such as the SUPERPAVE mix design, according to AASHTO M323 (AASHTO M323, 2017). While in most cases, this design method is mainly based on volumetric properties, and this approach is intended to include performance evaluation of the mixtures, especially for high traffic. However, due to complex testing requirements and challenges in implementation strategies, this second phase wasn't successful (McDaniel et al., 2022; Diefenderfer and Bowers, 2019; Tran et al., 2019; Yin and West, 2021). In several instances, volumetric-based design approaches did not always provide satisfactory performance. Furthermore, the use of asphalt additives, recycled materials (Recycled Asphalt Pavement, RAP; Recycled Asphalt Shingles, RAS, and so on), and fibers, to name some new technologies, further accentuated the need to incorporate performance testing in the mix design phase (Diefenderfer and Bowers, 2019; Zhou et al., 2021). Some of the many challenges reported in past studies that led to the need for BMD include:

- New Jersey identified that implementing the Superpave led to durability and cracking distresses, and adjusting the volumetric properties was not enough to counter that.
- Virginia experiences durability, and cracking distresses from Superpave implementation. Besides, the increased usage of recycled materials worsened the scenario regarding these distresses.
- Texas uses a high percentage of recycled materials in its mixtures for economic and environmental reasons (annual savings of approximately $80 million based on 15~20% RAP use) and moves towards BMD to address the premature failures from using recycled materials.

- The motivation of California towards BMD is to build long-life pavement that can serve for more than 30 years.
- Illinois also identifies the lack of performance of asphalt mixtures due to increased usage of recycled materials.

These significant reasons for moving towards a BMD approach were reviewed in a recent Federal Highway Agency, FHWA study (Hajj et al., 2021) with input from states such as Maine, New Jersey, Virginia, Louisiana, Texas, California, and Illinois. So, recently the BMD effort was investigated by the agencies. According to AASHTO PP 105-20, BMD is defined as the approach of asphalt mix design, which uses the performance tests on appropriately conditioned specimens to address several modes of distress considering a mixture of short-term aging, traffic levels, climate, and location-specific conditions (AASHTO PP 105, 2020). Under this Standard, four alternative approaches were identified that vary in mixtures' volumetric and performance assessment requirements. These include:

- Approach A "*volumetric design with performance verification*," where the design binder content of the mixture IS identified based on volumetrics, and the mixture is then tested for performance (i.e., rutting, cracking, moisture damage).
- Approach B "*volumetric design with performance optimization*." This approach is similar to approach A, but the performance evaluation is considered for identifying the design binder content.
- Approach C is "*performance-modified volumetric mix design,*" where the volumetrics identify initial materials, proportions, and binder content. The performance testing is then used to assess/modify binder content and/or mixture ingredients and properties.
- Approach D, *"performance design*," where mixture design is entirely based on performance testing criteria and little or no volumetric requirements.

Furthermore, various performance tests were proposed in the past decades for characterizing fatigue cracking, permanent deformation (i.e., rutting), and moisture damage. The objective of any BMD is to consider fast, reliable, and easy-to-use testing methods that both agencies and contractors can conduct during the design and production of asphalt mixtures.

2.2 The Maryland Experience & BMD Approach

As defined in NCHRP 20-406 study, nine steps are involved in determining a BMD to mix design (West et al., 2018). The crucial steps include identifying the primary mode of distress, selecting the BMD approach, and selecting the performance tests and acceptance criteria. Maryland State Highway Administration (MDSHA) has experienced an increase of RAP (as high as 45%) in the surface mixtures produced mixes that have a higher susceptibility to cracking (Goulias and Akhter, 2022). Thus, Maryland's primary mode of distress included fatigue cracking and permanent deformation (i.e., rutting). Implementation of the BMD in Maryland focused on Approach A-Volumetric design with performance verification. This reflected the belief that volumetric analysis provided good mixture properties for Maryland conditions but needed to be complemented with performance testing during the design process. In terms of performance testing, as indicated earlier objective of any BMD is to consider fast, reliable, and easy-to-use testing methods that both agency and contractors can conduct during the design and production of asphalt mixtures. Also, it was important to identify performance tests that use available equipment to the agency districts and producer's labs for quick implementation with minimal additional investment and personnel training. For such reasons, the IDEAL- Cracking Test (IDEAL-CT) (ASTM D8225, *Standard Test Method for Determination of Cracking Tolerance Index of Asphalt Mixture Using the Indirect Tensile Cracking Test at Intermediate Temperature*), and High-Temperature Indirect Tensile Test (HT-IDT) (ASTM D6931, *Standard Test Method for Indirect Tensile (IDT) Strength of Asphalt Mixtures*) were selected (ASTM D8825, 2019; ASTM D6931, 2017).

While the details and rationale of this approach are presented elsewhere (Akhter and Goulias, 2023; Goulias and Akhter, 2022), Figure 2-1 illustrates the overall BMD framework for Maryland. The delineated flowchart is applicable for surface mixtures commonly produced in Maryland. Besides the dense graded mixtures (DGs), gap-graded stone matrix asphalt mixtures (GGs) are included in the design approach, polymer-modified asphalt mixes specially designed for high-traffic corridors in Maryland. The volumetric design of these mixtures will be found elsewhere (MDSHA,2020). After the binder, aggregate blends, and recycled materials are selected, the asphalt mixtures will be designed according to current volumetric practice, which follows the design method of AASHTO M323 and R35. Besides controlling the densities, air void content, and aggregate blends, the moisture susceptibility of the mixtures was also tested according to Tensile Strength Ratio (TSR) test in this step. The volumetrically designed mixture was then used to compact test samples for IDEAL-CT and HT-IDT according to the standards' specifications. In the final step, all volumetric properties and performance test criteria should be met to pass the design as Job Mix Formula (JMF). The BMD approach and draft threshold analysis were showed cased in the recent FHWA BMD workshop (FHWA, 2022) and presented at the Maryland industry (Akisetty and Goulias, 2022).

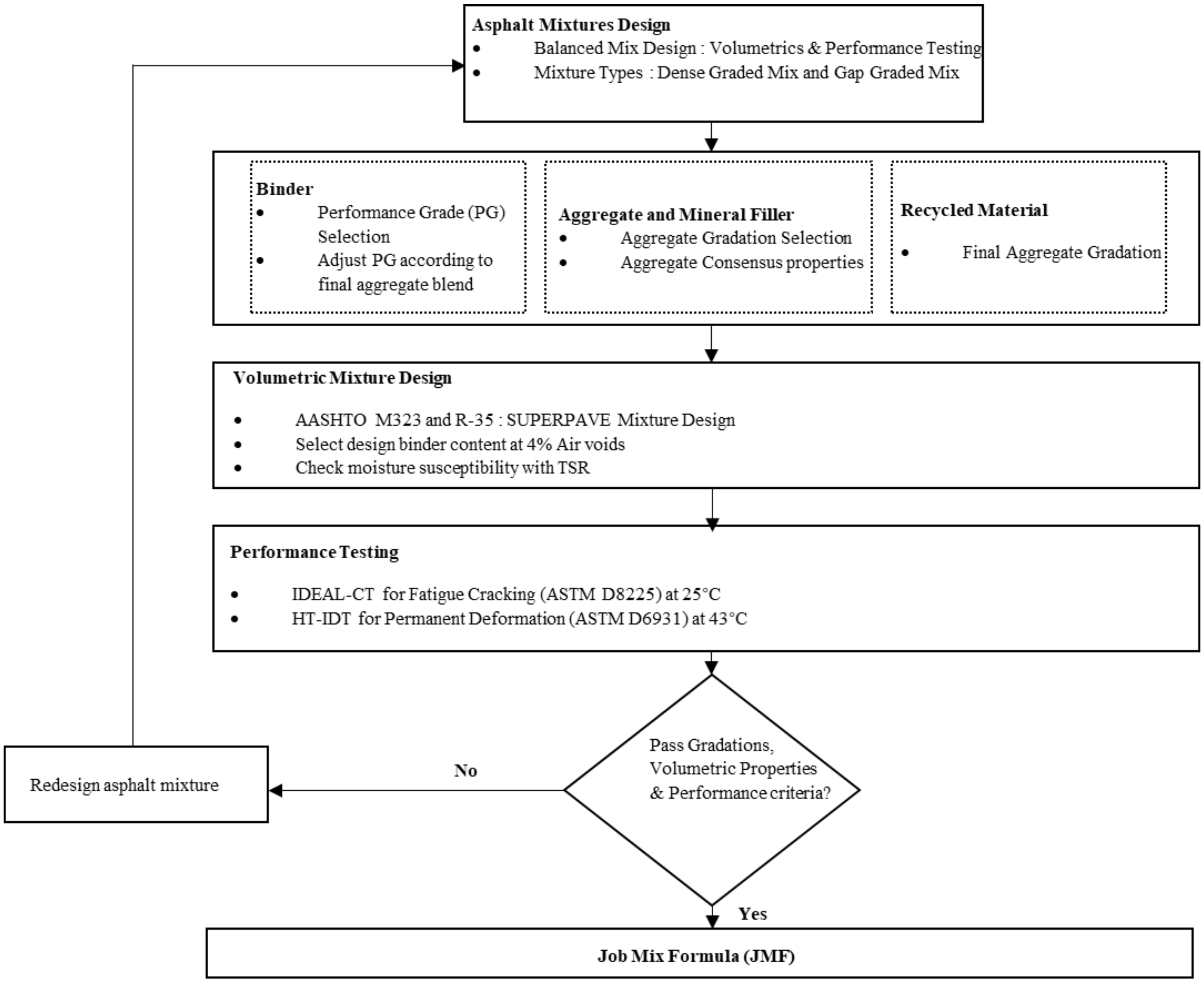


Figure 2-1 Balanced mix design framework for Maryland

2.3 Objectives and Scope

The experimental testing results pertinent to the performance assessment of Maryland mixtures are presented next. The objective of this effort was to:

- Conduct a performance evaluation of asphalt mixtures to establish Maryland's balanced mix design acceptance criteria. This task concerned the defined BMD approach in Figure 2-1. Asphalt mixtures were evaluated through the performance tests described next that assess fatigue cracking and permanent deformation (i.e., rutting). Producers' tests are relatively easy to run during mix design and production. The IDEAL Cracking Test, IDEAL-CT, was recently proposed by an SHRP2 IDEA project, while for rutting the High-Temperature Indirect Tensile Strength Test, HT-IDT was used.
- Along with mixture characterization, alternative acceptance criteria were explored for defining the initial acceptance thresholds. Such analysis is a "benchmarking" approach from the initial threshold acceptance values for these performance tests for the state's most used dense and gap-graded mixtures. While such analyses were based on a "benchmarking" approach, further validation will require fine-tuning such results with field performance.

2.4 Cracking Index Test (IDEAL-CT)

IDEAL-CT is a performance test that evaluates the cracking resistance potential of asphalt mixtures. In this test, a cylindrical asphalt sample is loaded in the diametrical plane at a loading rate of 50±2 mm/min till the load drops to 100 N. The Indirect Tensile test (IDT) frame is used with a Lottman's head, readily available in most producers' plants. The recommended sample for the test is a compacted gyratory sample, commonly compacted at 7±0.5% air void, with dimensions of 150±2 mm diameter and 62±1 mm thickness. The recorded load vs. displacement curve is used to evaluate the cracking characteristics of the mixture with the CT_{index} defined as follows (Zhou et al., 2017):

$$CT_{index} = \frac{G_f}{|m_{75}|} \times \left(\frac{l_{75}}{D}\right) \ldots\ldots\ldots\ldots\ldots (2\text{-}1)$$

Where l_{75} is the displacement for 75% of the peak load in the unloading portion of the curve, D is the diameter of the sample, in mm, m_{75} is the slope of the unloading curve = $\left|\frac{P_{85}-P_{65}}{l_{85}-l_{65}}\right|$, and G_f is the fracture energy in J/m^2. The above equation calculates the CT_{index} of a 62 mm height sample. For a sample size other than 62 mm in height, the following equation is used:

$$CT_{index} = \frac{G_f}{|m_{75}|} \times \left(\frac{l_{75}}{D}\right) \times \left(\frac{t}{62}\right) \ldots\ldots.(2\text{-}2)$$

Zhou et al. (2017) reported that the variability of the CT_{index} is less than 25%, and it is sensitive to binder content, RAP content, mixture aging, binder type, and aggregate gradation (Zhou et al., 2017). It was reported that CT_{index} is well correlated with dynamic loading cracking tests such as the Texas Overlay Tester (OT) (Zhou et al., 2021; Zhou et al., 2017; Newcomb and Zhou, 2018). Furthermore, it was reported that the CT_{index} correlates well with field performance regarding fatigue, reflective and thermal cracking (Zhou et al., 2017). Besides due to the simple enough procedure to be implemented in the contractor's lab during QC, IDEAL-CT has been considered or implemented in many agencies' BMD frameworks, such as Texas (Zhou et al., 2021), Virginia (Habbouche et al., 2022), Ohio (Abbas et al., 2021), is few to mention.

2.5 High-Temperature IDT (HT-IDT)

Indirect tensile test in high temperature is being assessed the permanent deformation of asphalt mixtures (Christensen and Bonaquist, 2004 and 2007). This method loads a compacted cylindrical sample along the diametrical plane until failure. The peak load is used to calculate the tensile strength with the following equation:

$$S_t = \frac{2P}{\pi D t} \ldots\ldots\ldots\ldots\ldots\ldots (2\text{-}3)$$

Where P is the peak load in N, D is the diameter in mm, and t is the sample's thickness in mm.

The high temperature is selected based on the location of the pavement. Christensen et al. (2007) used a temperature of 10 °C below the yearly 7-day average maximum temperature 20 mm below the pavement surface with 98% reliability, as determined by the Long-Term Pavement Performance Bind (LTPPBind)

(Christensen and Bonaquist, 2007). It was shown that there is a high correlation between permanent deformation and tensile strength at high temperatures. While a wide range of CT_{index} values has already been reported in the literature, fewer studies have explored HT-IDT values for rutting propensity. Christensen et al. (2007) reported that HT-IDT values minimize the rutting potential for specific mixtures examined in the past and in the function of traffic levels (i.e., Estimated Single Axle Load, ESAL) (Christensen and Bonaquist, 2007). Bennert et al. (2018) reported that HT-IDT is highly correlated with rutting evaluation tests, such as the Asphalt Pavement Analyzer (APA) (Bennert et al., 2018). The study indicated a good relationship between these two test parameters and reported a lower variability (in terms of Coefficient of Variation, COV) of HT-IDT (COV of 9%) in relation to APA (COV of 13%). Yan et al. (2020) reported that HT-IDT correlates reasonably well with the Hamburg Wheel Test (HWTT) for specific mixtures (Yan et al., 2020).

2.6 Materials and Sample Preparation

Dense-graded (DG) and gap-graded (GG) mixtures were collected from ongoing state construction projects. Eight dense-graded and five gap-graded mixes, all surface mixtures, were included in the study. The majority of the mixture used in the State is dense-graded mixtures. According to Maryland State Highway, pavements surfaced with these mixtures have not shown any rutting but are prone to cracking. The mixtures consist of different levels of recycled materials, Table 2-1. The RAP content ranged from 10% to 45% for dense-graded mixtures, while for gap graded was between 0% to 15%. Gap-graded mixtures are designed for high traffic levels with stiffer binder types (PG 76-22). Thus, the presence of RAP is limited to lower content since it promotes cracking susceptibility. For this study, three mixture collection points have been used for each mix: Plant, Modified, and Field (Figure 2-2). Figure 2-2 shows the current design and QA practice in Maryland. After selecting the ingredients, the asphalt mix is designed based on the Superpave design method (Maryland Department of Transportation and State Highway Administration, 2022). Plant mixes were collected from the production verification phase of the mixtures using JMF. These mixes were sampled from the dispensed truck right after production. Modified mixes (or Plant modified mixes) were collected from the contractor's plant after the initial modification of the mixes. According to Maryland specifications, the mixes must meet certain Percent Within Specification Limits (PWSL) to pass and go for further production. While details on the PWSL are found elsewhere in Standard Specifications for Construction and Materials (Maryland Department of Transportation and State Highway Administration, 2022), the key parameters to modify the mixes are aggregate gradations (No 200 through largest sieve sizes) and binder content. The plants' Modified mixes were also sampled right after production from the dispensed truck. After collecting these two plant mixes, field mixes were also sampled from the construction site and collected behind the paver.

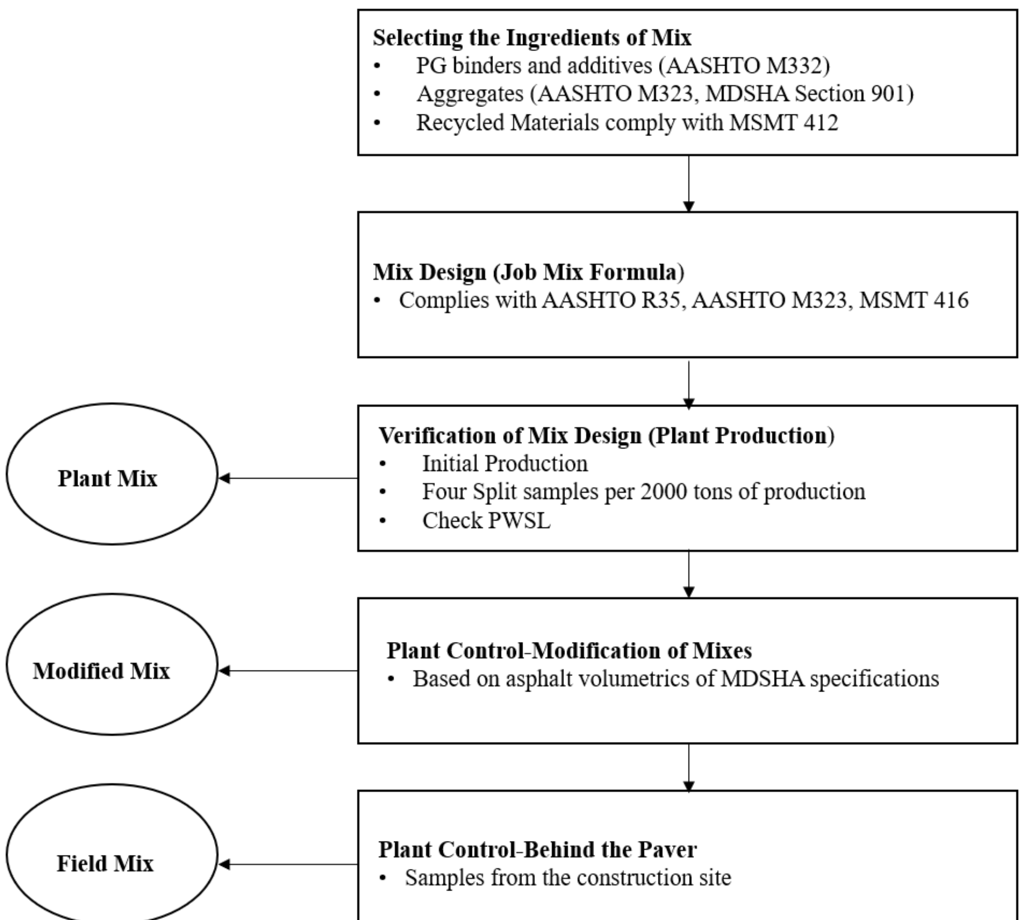


Figure 2-2 Current asphalt mix design practice in Maryland (Maryland Department of Transportation and State Highway Administration, 2022)

Table 2-1 Mixtures characteristics

Mixtures	Mix Method	Mix Band	RAP%	Traffic Level, ESAL (in millions)	Asphalt Type
GG1	Warm mix	12.5 mm	10%RAP	>30	64E-22(76-22)
DG1	Warm mix	19 mm	45%RAP	0.3 to <3	58S-28(58-28)
GG2	Hot mix	12.5 mm	0%RAP	>30	64E-22(76-22)
DG2	Hot mix	9.5 mm	45%RAP	0.3 to <3	58S-28(58-28)
DG3	Hot mix	9.5 mm	45%RAP	0.3 to <3	58S-28(58-28)
GG3	Warm mix	12.5 mm	15%RAP	>30	64E-22(76-22)
DG4	Warm mix	12.5mm	27% RAP	0.3 to <3	64S-22(64-22)
DG5	Warm mix	9.5 mm	29% RAP	0.3 to <3	64S-22(64-22)
DG6	Warm mix	9.5 mm	32%RAP	0.3 to <3	64S-22(64-22)
DG7	Warm mix	12.5 mm	10%RAP	0.3 to <3	64S-22(64-22)
DG8	Warm mix	12.5 mm	15%RAP	0.3 to <3	64S-22(64-22)
GG4	Hot mix	12.5 mm	15%RAP	>30	64E-22(76-22)
GG5	Hot mix	12.5 mm	15%RAP	>30	64E-22(76-22)

Volumetric mixture properties, such as maximum theoretical specific gravity, G_{mm}, bulk specific gravity, G_{mb}, voids in total mix, VTM, voids in mineral aggregate, VMA, voids filled with asphalt, VFA, effective binder content, P_{be}, aggregate gradations, and such, are presented in Table 2-2 for the three populations of each mixture. Aggregate gradation curves for each of these mixtures are shown in **Appendix A**. MD asphalt producers measure volumetric properties following MDSHA specifications for construction and materials and the SUPERPAVE mix design method (AASHTO M323, 2017; Maryland Department of Transportation and State Highway Administration, 2022). In this QC parameters determination, three samples were tested, and the average of the results was reported.

Table 2-2 Volumetric properties of asphalt mixes

Mixtures	Population	Max Sp Gr (G_{mm})	VTM (%)	VMA (%)	VFA (%)	Pbe (%)	G_{mb} at N_{des}	% passing #200	Cum % retained#4	Cum % retained 3/8"	Cum % retained 3/4"
DG1	Plant	2.624	3.0	13.2	71.0	4.4	2.545	5.4	57	32	8
	Modified	2.639	3.3	12.9	74.4	4.4	2.553	5.9	54	28	5
	Field	2.623	2.7	13.0	79.2	4.4	2.551	5.8	55	30	9
DG2	Plant	2.593	2.9	15.8	81.7	5.1	2.518	6.9	44	7	0
	Modified	2.587	2.7	15.8	83.1	5.2	2.518	6.9	42	8	0
	Field	2.605	3.5	15.9	78.1	5.0	2.514	5.8	46	8	0
DG3	Plant	2.602	3.0	15.4	80.3	5.1	2.523	7.6	46	8	0
	Modified	2.595	2.8	15.5	82.0	5.2	2.522	7.3	46	10	0
	Field	2.598	3.2	15.7	79.5	5.1	2.514	7.4	45	8	0
DG4	Plant	2.532	4.6	15.5	70.5	5.1	2.416	6.9	43	11	0
	Modified	2.533	3.9	14.6	73.3	4.7	2.435	7.2	44	13	0
	Field	2.537	4.8	15.5	68.8	4.9	2.415	6.1	43	12	0
DG5	Plant	2.506	4.1	15.1	73.0	5.2	2.404	4.8	35	2	0
	Modified	2.509	3.0	14.2	79.1	5.4	2.435	5.0	42	5	0
	Field	2.507	3.6	14.7	75.7	5.3	2.418	5.3	37	4	0
DG6	Plant	2.489	3.0	14.5	79.3	5.2	2.414	6.7	36	5	0
	Modified	2.478	4.3	16.2	73.5	5.4	2.371	5.5	35	5	0
	Field	2.500	3.4	14.3	76.2	5.0	2.416	6.1	35	5	0
DG7	Plant	2.618	5.4	14.5	62.9	4.3	2.477	6.1	47	13	0
	Modified	2.598	2.6	13.4	80.6	5.1	2.530	7.2	42	10	0
	Field	2.594	4.5	15.1	70.6	5.1	2.478	6.9	42	7	0
DG8	Plant	2.478	3.4	15.5	77.5	5.5	2.394	6.9	47	5	0
	Modified	2.478	3.4	15.1	77.5	5.4	2.394	6.7	41	8	0
	Field	2.485	4.1	15.5	73.5	5.5	2.384	7.0	36	5	0
GG1	Plant	2.619	3.9	17.3	77.5	6.3	2.518	6.9	58	18	0
	Modified	2.603	3.1	17.2	82.0	6.4	2.523	6.4	62	22	0
	Field	2.615	3.9	17.5	77.7	6.3	2.512	6.6	61	23	0
GG2	Plant	2.621	2.6	17.1	85.1	6.4	2.554	8.9	68	24	0
	Modified	2.647	4.5	17.8	74.9	6.2	2.530	8.1	69	26	0
	Field	2.614	3.6	18.3	80.1	6.4	2.519	8.5	69	26	0
GG3	Plant	2.576	3.6	18.6	80.7	6.4	2.484	8.2	69	15	0
	Modified	2.560	3.4	19.2	82.5	6.8	2.475	8.4	70	18	0
	Field	2.584	3.1	17.8	82.3	6.4	2.508	9.1	68	17	0
GG4	Modified	2.600	3.6	18.6	80.8	6.5	2.372	9.2	65	26	0
	Field	2.450	2.9	18.8	84.5	6.9	2.381	6.8	66	18	0
GG5	Plant	2.558	2.2	18.3	88.1	5.6	2.501	8.5	66	20	0
	Modified	2.585	4.6	19.5	76.4	6.8	2.467	6.8	70	21	0
	Field	2.568	4.6	19.7	76.9	6.4	2.451	8.0	67	20	0

The gyratory compactor compacted the asphalt mixtures at 7±0.5% air void content. The mixtures were reheated in the oven at mixing temperature for less than two and half hours to avoid the aging effect and then compacted by maintaining the compaction temperature. The sample size was 150±2 mm diameter and 62±1 mm height for IDEAL-CT and 150±2 mm diameter and 95±1 mm height for HT-IDT. The samples were tested according to ASTM D8225 and D6931 for IDEAL-CT and HT-IDT, respectively. For the IDEAL-CT, the testing temperature was 25°C. The samples were preconditioned in a water bath at 25°C for 2 hours. For the HT-IDT, the testing temperature was 43°C, representing a temperature of 10 °C lower than the yearly 7-day average maximum temperature in Maryland, 20 mm below the pavement surface, with 98% reliability. The asphalt samples for the HT-IDT were conditioned in a water bath at 43°C for 2 hours. Samples were tested immediately after removing from the water bath to maintain the temperature, especially when HT-IDT samples lose temperature quickly. After calibrating the time between removing from the bath to testing using an infrared camera, 2 minutes has been fixed as the proper time difference. Alternatively, an environmental chamber is suggested to maintain the high temperature while testing. A servo hydraulic-powered loading frame was used with a data acquisition system for testing. The loading rate for both tests was maintained at 50 mm/min. At least five samples were prepared for individual mixtures.

2.7 Results and Analysis

2.7.1 Cracking Index Test Results

Figure 2-3 shows the average CT_{index} for different dense-graded mixtures. The bars represent the one standard deviation from the average. Figure 2-3(a) shows the average CT_{index} for plant and modified mixes. As stated in the earlier section, plant mixes are modified in the plants by adjusting the volumetric properties, aggregate gradations, and binder content. The effect of the volumetric-based modification is identifiable by comparing the plant mix and modified plant mix results. DG1, DG2, DG6, and DG8 showed lower cracking resistance when mixes were modified based on only volumetrics. A Welch's t-test has been performed for all the mixes to estimate the statistical significance of the results from plant to modified plant mixes. Except for DG1, all mixes' average of the plant to modified plant mixes is statistically insignificant. For DG1, the mix has been modified by increasing the fines by 9% and coarse aggregate (sieve No 3/8) by 6%, which increases the VTM (%) from 3.0 to 3.3 but lowers the CT_{index} significantly. The average CT_{index} for plant mixes ranges from 54 to 230. The variability of these results was measured in terms of the coefficient of variation (COV%). Within mixtures, COV for CT_{index} ranges from 5.2% to 20.4%, which is acceptable (<25%).

Figure 2-3(b) presents the average CT_{index} for dense graded mixes for field mixes collected from behind the paver. Field mixes are used to establish quality acceptance and thus define the performance threshold for the specification. Overall, increased RAP content in the mixtures increases their cracking resistance. As discussed in the previous section, these mixtures are volumetrically adjusted to provide better performance and counter the high percentage of RAP content. For instance, DG2 and DG3 have similar mixture composition, Table 2-1, with the DG3 field mix having a higher effective binder content, $P_{be,}$ and D/B than DG2. This produced a more cracking-resistant mix. For the various mixes, the CT_{index} ranges from 59 to 174. The COV for these mixes' ranges from 9.3% to 17.6%.

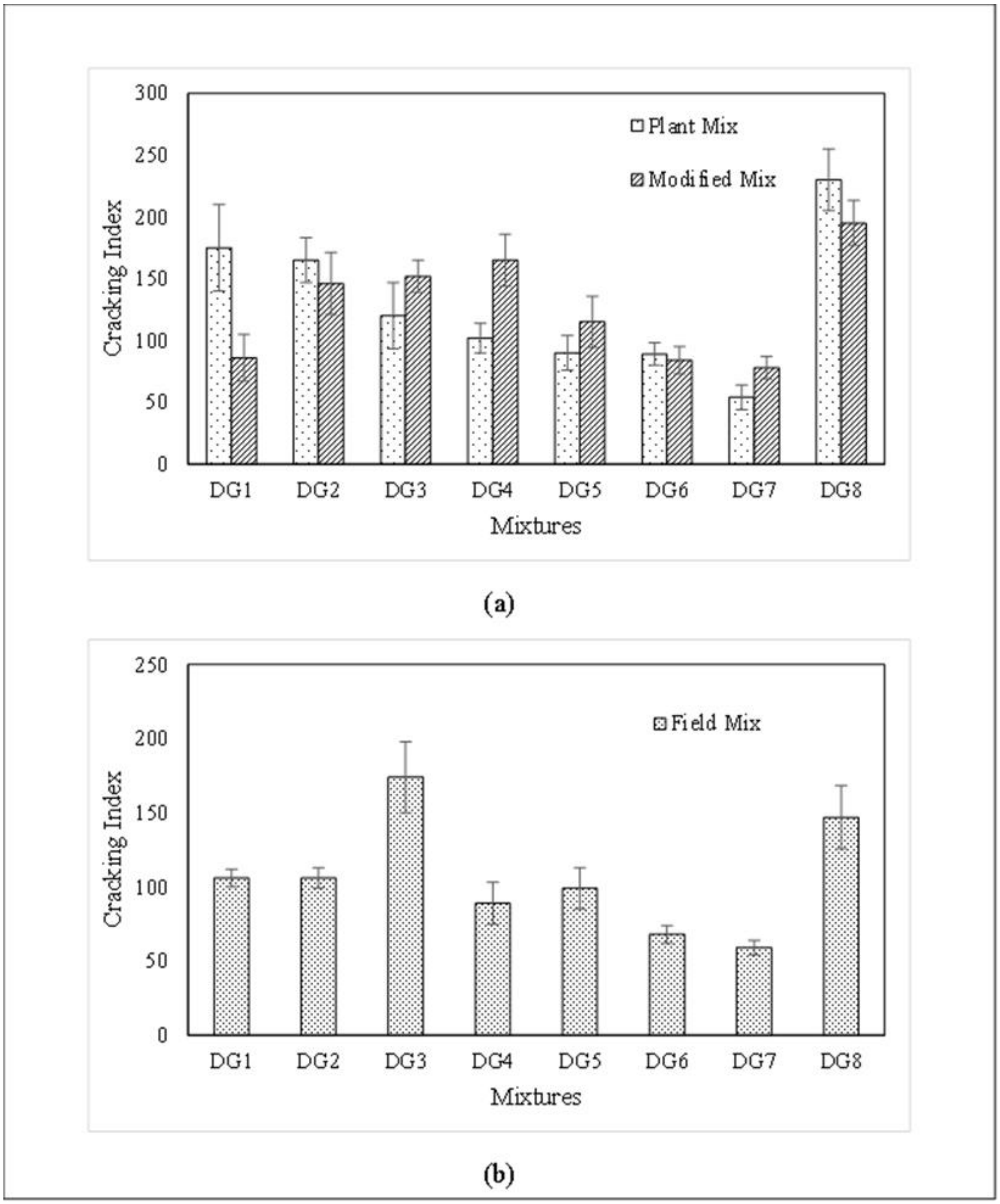


Figure 2-3 IDEAL-CT results for dense graded mixes: a) Plant and Modified Plant, b) Field

Figure 2-4 presents the average CT_{index} values for the gap-graded mixes. Figure 2-4(a) shows the results for the plant and modified plant mixes. The average CT_{index} ranges from 625 to 1671. For the GG4 mix, only field mixes were available. For mixes G2 and G5, the modified mixtures had drastic changes in volumetric properties producing a significant reduction in CT_{index}. Figure 2-4(b) presents the average CT_{index} for the field mixes, which ranged from 538 to 1436. The COV ranges from 8.3% to 22.4% for mixes. Like the dense-graded mixes, the field gap-graded mixtures also had a lower CT_{index} value than the plant mixes.

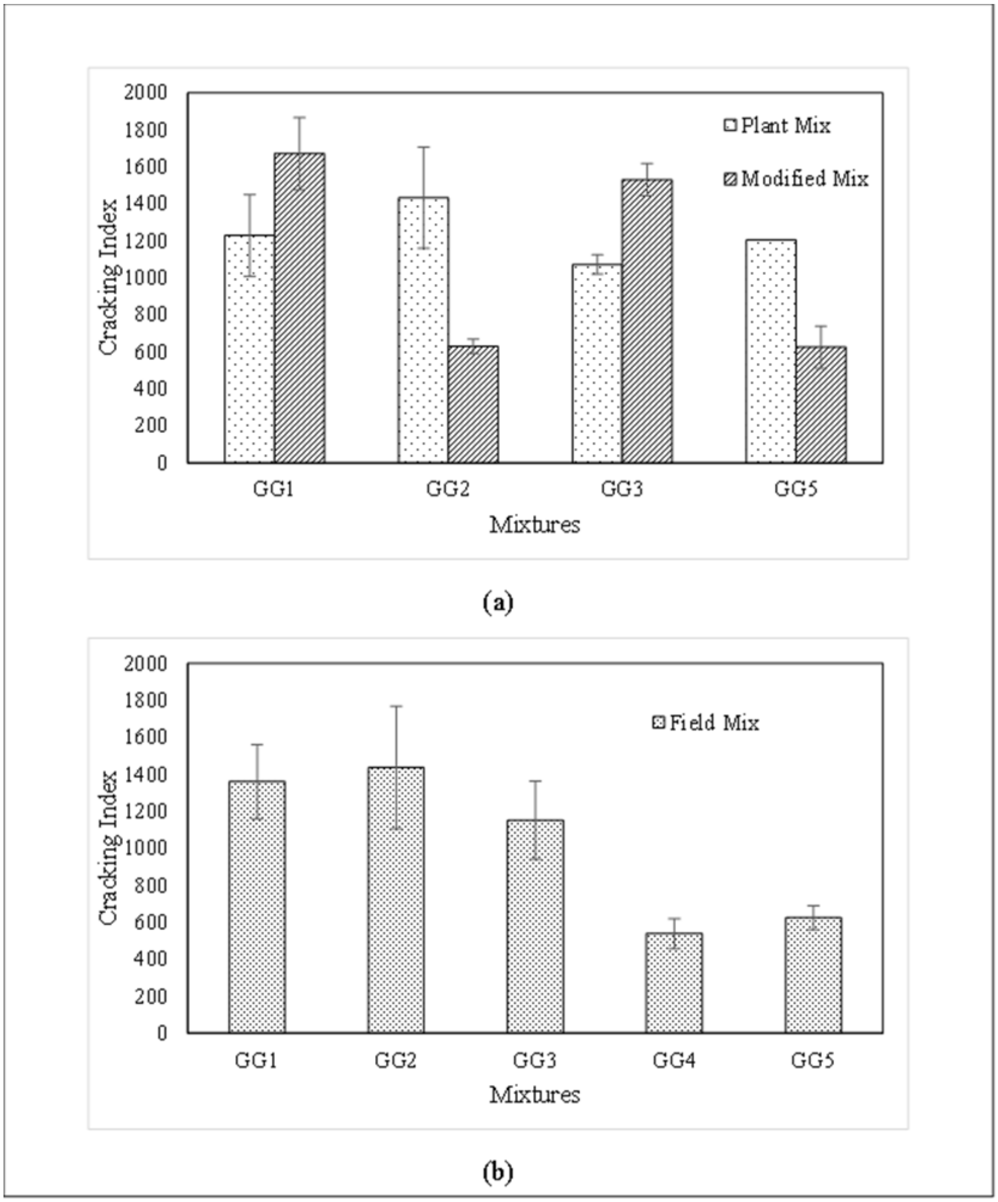


Figure 2-4 IDEAL-CT results for gap graded mixes: a) Plant and Modified Plant, b) Field.

Table 2-3 shows the correlation among the volumetric properties for all thirteen mixtures and their populations. Due to adjustments in the mixes, all populations have different JMF and are treated as such. Combining all the mixtures allows for exploring the effect of a wide range of volumetric properties. The correlation matrix provides the correlation among parameters, ranging from -1 to +1 range, with values close to ±1 representing a high correlation. The negative values indicate the linear inverse relation between parameters, while positive values indicate linear positive relation. High correlations (Pearson's correlation coefficient) have been observed between VTM and VFA; VMA and P_{be}; P_4 and VMA; P_{be} and P_4, and P_4 and $P_{3/8}$. Including these parameters in the multivariate linear regression analysis to relate with performance parameters (i.e., CT_{index} and HT-IDT) will introduce multicollinearity. One and linear regression analysis for each parameter has been performed and reported in Table 2-4, where the volumetric parameters are related to CT_{index} at a 95% confidence level. VMA, VFA, $P_{be,}$ and P_4 show the highest correlation with CT_{index}. Such correlation is essential when combining performance index criteria with volumetrics criteria in the design of the mixtures. The coefficient of determination should be further analyzed with more mixtures.

Table 2-3 Pearson coefficient (r) for mixture volumetric parameters

	G_{mm}	VTM	VMA	VFA	P_{be}	P_{200}	P_4	$P_{3/8}$	$P_{3/4}$	D/B
G_{mm}	1.00									
VTM	-0.05	1.00								
VMA	0.06	0.15	1.00							
VFA	0.05	-0.81	0.42	1.00						
P_{be}	-0.01	0.01	0.90	0.51	1.00					
P_{200}	0.29	-0.16	0.65	0.51	0.58	1.00				
P_4	0.44	-0.04	0.79	0.42	0.76	0.67	1.00			
$P_{3/8}$	0.57	0.01	0.46	0.15	0.45	0.46	0.84	1.00		
$P_{3/4}$	0.28	-0.11	-0.38	-0.20	-0.34	-0.25	0.06	0.44	1.00	
D/B	0.36	-0.19	-0.11	0.10	-0.30	0.60	0.06	0.11	0.05	1

Table 2-4 Correlation of mixture volumetric parameters to Cracking Index

Parameters	R Square	P-value
VMA	0.52	3.2E-07
VFA	0.46	1.2E-03
P_{be}	0.61	8.5E-09
P_4	0.66	5.9E-10

2.7.2 High-Temperature IDT Test Results

The high-temperature IDT testing results are summarized in Figure 2-5 with the average tensile strength of the dense graded mixes. Figure 2-5(a) presents the average tensile strength for plant and modified mixes. The tensile strength ranged from 139 kPa to 229 kPa. In almost all the cases, the volumetrically modified mixes had about equal or low tensile strength than plant mixtures. This reflects the combined effects of changes in mixture volumetrics and impact on stiffness. Thus, volumetrics alone are insufficient for assessing mixture propensity to permanent deformation.

Figure 2-5(b) presents the average tensile strength values for the field mixes, establishing the acceptance criteria for the rutting specification for Maryland mixes. The average tensile strength ranges from 128 kPa to 203 kPa. These values are comparable to, or slightly higher than, those from the plant mixes.

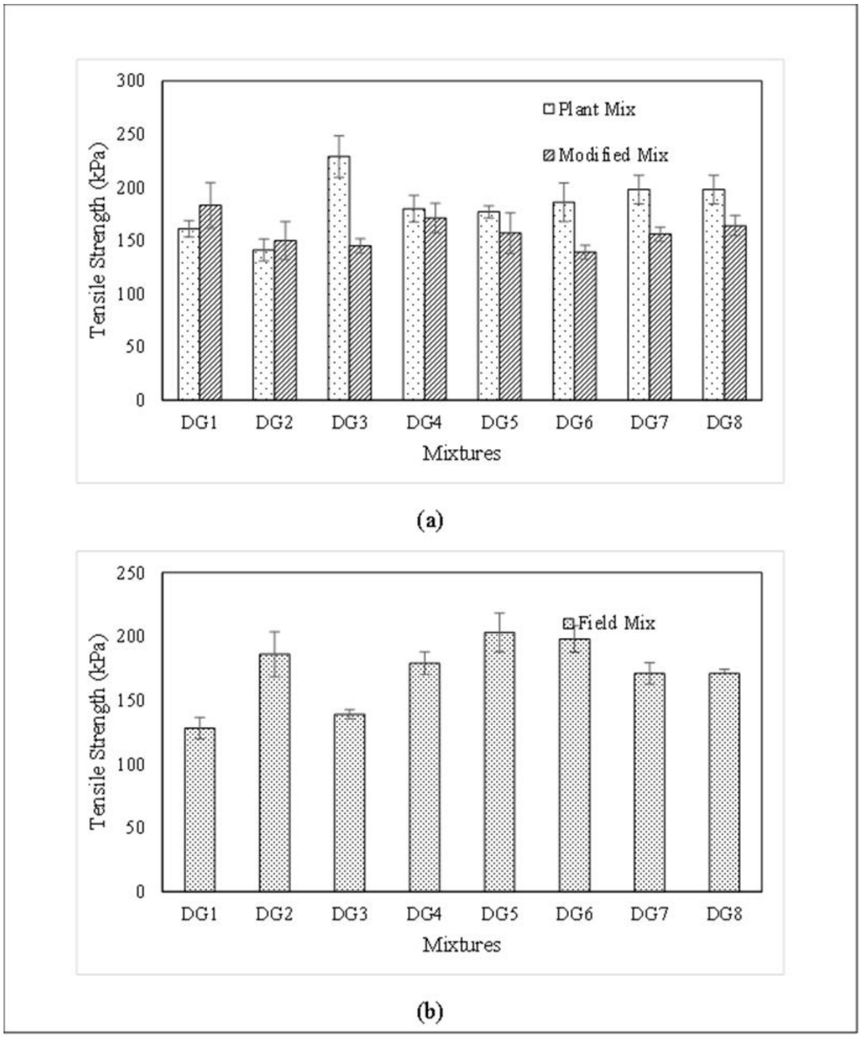
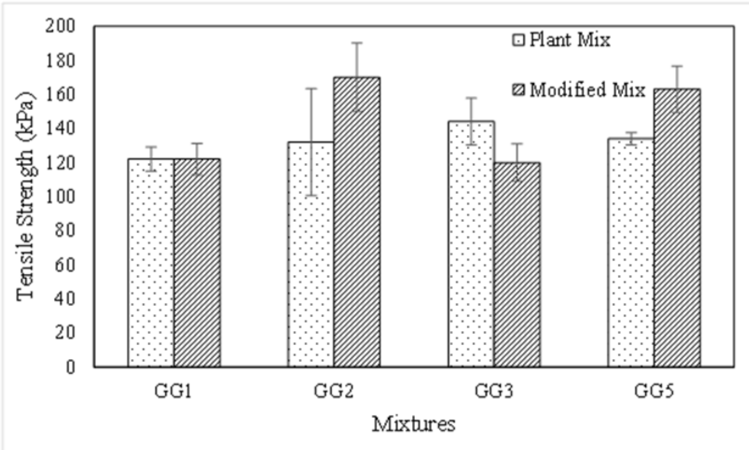
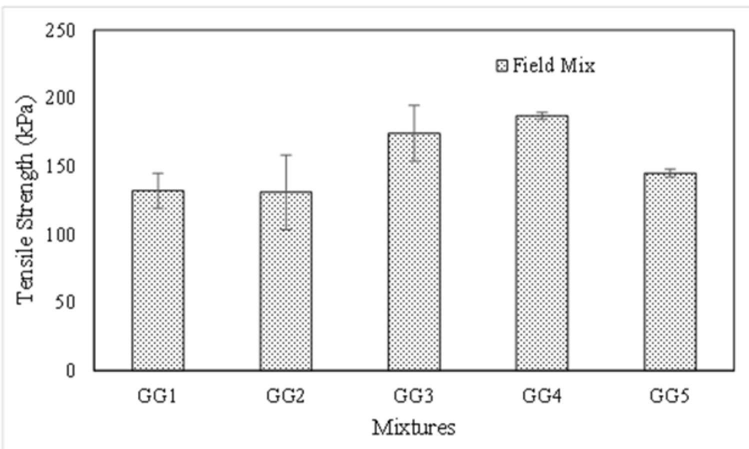


Figure 2-5 HT-IDT results for dense graded mixes: a) Plant and Plant Modified, b) Field.

Figure 2-6 shows the results for the HT-IDT of the gap-graded mixes. The average tensile strength for plant and modified plant mixes ranged from 120 kPa to 170 kPa. Similarly, to the dense-graded mixtures, volumetric changes in plant-modified mixtures do not always ensure improved rutting performance. For example, for Mix GG3, changes in mixture volumetrics produced a lower tensile strength from 120 kPa to 144 kPa. Figure 2-6(b) shows the average tensile strength for the field gap-graded mixes, ranging from 131 kPa to 187 kPa. These values are comparable to, or slightly lower than, those from the plant and plant-modified mixes.



(a)



(b)

Figure 2-6 HT-IDT results for gap graded mixes: a) Plant and Modified Plant, b) Field.

Similar operations as Table 2-4, One-on-one linear regression analysis for volumetric parameters with HT-IDT have been conducted and reported in Table 2-5, showing such relations at a 95% confidence level. Compared to CT_{index}, HT-IDT shows poor relation with volumetric parameters. But these coefficients should be further modified using more mixtures.

Table 2-5 Correlation of mixture volumetric parameters to HT-IDT

Parameters	**R Square**	**P-value**
VMA	0.14	0.02
VFA	0.17	0.01
P_{be}	0.16	0.01
P_4	0.16	0.01

2.7.3 Threshold Selection

Following the performance testing of these typical mixtures used in the state of Maryland, the study aimed to identify the initial draft design threshold criteria to be adopted within BMD. Several methods have been suggested over the years (Diefenderfer and Bowers, 2019; Zhou et al., 2020). Initially, three alternative approaches were examined:

- Method 1. Acceptance based on Minimum Threshold Values: Identify the minimum/maximum values of performance testing for the mixtures of interest. For example, the acceptance criteria in IDEAL-CT will be the minimum CT_{index} value observed from these pilot testing results. This less restrictive approach considers that such acceptance criteria might be loose until additional testing and more experience are gained from pilot projects where BMD is implemented.
- Method 2. Acceptance based on average Values: Identify the average values of performance tests for the mixtures of interest using all replicates. This approach represents the more restrictive case where a significant portion of the produced material may not meet these acceptance threshold values.
- Method 3. Acceptance based on Average ± x standard deviations: Identify the acceptance values based on the average and a given number of standard deviations from the mean of all tested mixtures of interest. The standard deviation is multiplied by 1.96 to attain the 95% confidence interval. In this case, it is expected that 95% of the tested values, or any other percentile of interest, will be above the acceptable threshold values.

In addition to these alternative methods, the trimmed average values were also considered an option. Trimmed average values are obtained by eliminating the maximum and minimum values among five or more replicates, and then the average values are calculated based on the remaining replicates. This approach reduces the effects of potentially large variability observed from sample to sample and testing.

2.7.4 Threshold Selection for Cracking Index

Table 2-6 provides the thresholds for CT_{index} for both dense and gap-graded mixes based on these alternative methods. Based on method 1, selecting the minimum values among all mixtures as thresholds will imply that all testing variability is accounted for in all mixtures and thus will initially include those poorly performing. Thus, such an approach may not promote improvement in mixture design and expected performance until the thresholds are revised with further experience in implementing the BMD. For approach 2, where the average values are considered, 50% of the mixes will be considered underperforming and thus will not meet the acceptance thresholds. For method 3, the average minus 1.96 times the standard deviation (i.e., 95% confidence level) considers some variability in performance testing results. Using method 3 with the trimmed average values (i.e., excluding the extreme observations) for the three populations will provide acceptance thresholds for CT_{index} ranging from 91 to 96. In contrast, the untrimmed averages range from 82 to 92.

Among the three populations, MDSHA believes field mix data are the most logical choice to select as performance threshold criteria for dense and gap-graded mixes, as they will best represent the field performance. Thus, the resulting CT_{index} thresholds for dense graded are 82 and 91 based on untrimmed and trimmed averages, respectively, and when the 95% percentile criteria are used.

Similarly, the threshold analysis for the gap-graded mixes is also shown in Table 2-6. The resulting threshold values for these mixes are 788 and 812 for the untrimmed and trimmed cases, respectively, and when the 95% percentile criteria are used.

Table 2-6 Threshold selection matrix for Craking Index

	Dense Graded Mixes			**Gap Graded Mixes**		
Criteria Value	**All populations**	**Plant mixes**	**Field mixes**	**All populations**	**Plant mixes**	**Field mixes**
Average	118	129	106	1081	1239	1022
Minimum	57	57	59	538	1091	538
Trimmed Average	116	124	105	1086	1253	1010
Trimmed Minimum	56	56	59	510	1106	510
Average - 1. 96Std.Dev	86	92	82	862	993	788
Trimmed Average - 1.96Std. Dev	94	96	91	915	1049	812

2.7.5 Threshold Selection for HT-IDT

The threshold selection for HT-IDT was carried out with the same approach. Table 2-7 reflects the analysis results. The tensile strength threshold values for dense graded mixes are 159 kPa and 165 kPa for untrimmed and trimmed values, respectively, based on the 95% criteria. Similarly, the threshold values for gap-graded mixes are 128 kPa and 135 kPa, respectively, for untrimmed and trimmed values. The rutting expectation in the asphalt cement layer in the flexible pavement for Maryland is 0.15 inches (Maryland Department of Transportation and State Highway Administration, 2022) for all HMA mixtures. So, implementing different HT-IDT thresholds for dense graded and gap-graded mixtures will require further assessment in coordination with MDSHA.

Table 2-7 Threshold selection matrix for HT-IDT

	Dense Graded Mixes			**Gap Graded Mixes**		
Criteria Value	**All populations**	**Plant mixes**	**Field mixes**	**All populations**	**Plant mixes**	**Field mixes**
Average	183	184	172	146	140	149
Minimum	128	141	127	120	122	131
Trimmed Average	181	180	172	146	143	148
Trimmed Minimum	127	139	127	121	123	128
Average - 1. 96Std.Dev	163	165	159	128	121	128
Trimmed Average - 1.96Std. Dev	169	171	165	134	131	135

2.7.6 Acceptance Performance Diagram for Balanced Mix design

Based on the findings of the threshold analysis, the following performance diagrams were defined, identifying four potential regions, Figure 2-7 and Figure 2-8. These performance acceptance criteria will be used with the volumetric requirements during mix design as outlined in the BMD approach for Maryland in Figure 2-1. For the dense-graded mixes, the thresholds for CT_{index} and HT-IDT are 90 and 165 kPa, respectively, while for the gap-graded mixes, the thresholds for CT_{index} and HT-IDT are 800 and 135 kPa.

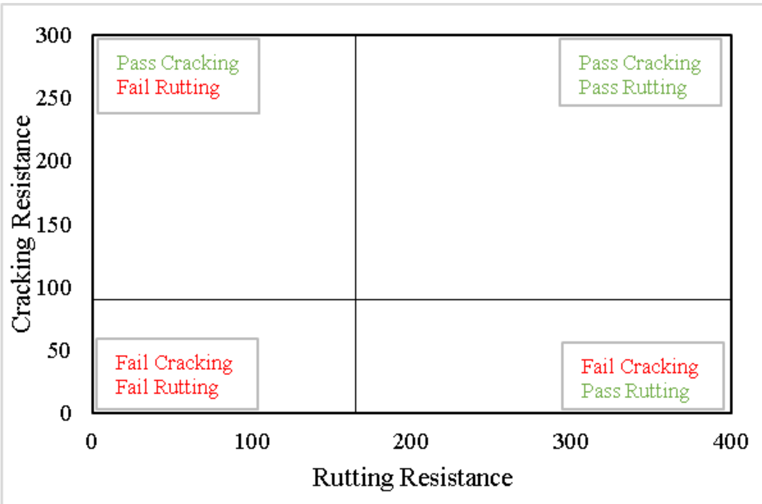


Figure 2-7 Performance diagram for dense graded mixes

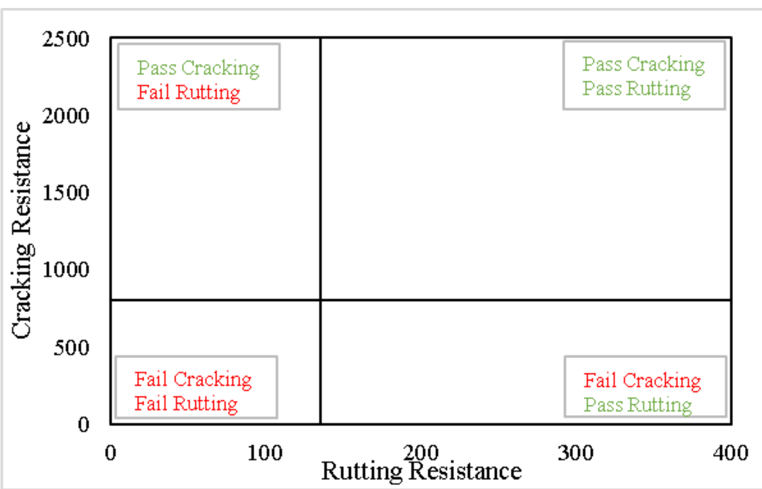


Figure 2-8 Performance diagram for gap graded mixes

2.8 Conclusion

The performance testing results regarding fatigue cranking and permanent deformation (i.e., rutting) for the most common mixtures used in Maryland were presented herein. Such values were considered for defining acceptance threshold values to be incorporated in the Maryland BMD. Three populations were considered in this study from mixtures used in various projects across the state: plant mixtures, plant modified, and field mixtures. While such mixtures represent different populations, field mixtures (i.e., behind the paver) results define acceptance thresholds for the BMD. In contrast, plant and plant-modified data can be used for verification during mix design and production. Based on the experimental results of these mixtures and correlation analysis, specific volumetric properties, such as VMA, VFA, $P_{be,}$ and $P_{4,}$ correlate well with CT_{index}, thus confirming that specific mixture volumetrics influence cranking resistance potential. Such correlations were insignificant for rutting potential (i.e., HT-IDT).

Alternative approaches were considered for defining acceptance thresholds for both dense and gap-graded mixes in Maryland for fatigue cracking and permanent deformation (i.e., rutting). These included the more restrictive option, where the average values of the benchmarking testing are used, to the least conservative approach, where the minimum values are considered. Fine-tuning the acceptance thresholds is expected to continue as BMD is implemented with pilot construction projects in the state. Furthermore, field performance studies should be considered for linking lab performance results to the actual field performance of the asphalt mixtures in the state. The intermediate step in this process is to relate CT_{index} and HT-IDT lab testing results to predicted performance using available and well-established models. This step is currently underway. Ultimately the BMD approach could lead to the development of performance-based specifications where predicted performance could be linked to a pay factor schedule relating quality to rewards. The proposed BMD and study findings may be transferable elsewhere, where similar materials and asphalt mixtures are used.

Chapter 3 : Variability Analysis of IDEAL-CT and HT-IDT Tests

3.1 Introduction

Mitigating the risks of controlling the performance of the asphalt mixtures during the design, production, and construction is a long-experienced problem. Recent efforts to include performance test results, such as cracking and rutting tests, under Balanced Mix Design (BMD) approach will have associated risks if the sources of variability in the performance results are not appropriately addressed. In this study, IDEAL-CT has been evaluated for the cracking assessment, and HT-IDT is for the rutting assessment. These two tests have been adapted to the BMD approach established for Maryland (Goulias and Akhter, 2021; Akhter and Goulias, 2023). IDEAL-CT test method met all the essential Quality Control (QC) criteria, such as equipment costs, the tests' accuracy, and the test results' repeatability, to name a few. Hence, IDEAL-CT has been studied by many researchers and agencies for potential adoption and implementation in the design and Quality Assurance (QA) process of asphalt mixtures. Studies have also been focused on increasing the accuracy of performance tests by identifying the precision of the test results. Although precision and bias statement is yet to be established for the IDEAL-CT test, research efforts are ongoing on assessing the sources of variability and amount of the repeatability (i.e., the coefficient of variation, COV%) of the test results. Some studies reported common sources of variability in IDEAL-CT results, which are included but not limited to:

- Specimen Preparation - proper sampling, segregation, consistency (Tylor et al., 2019a, 2019b)
- Materials (Boz et al., 2022)
- User training in testing and interpretation/analysis of experimental results (Tylor et al., 2019a)
- Testing system and instrumentation – "closed loop servo-hydraulic system" vs. "screw-type" actuators for displacement control, loading head setup and friction, calibration compliance, and loading rate (Habbouche et al., 2021; Tylor et al., 2019a).

To assess the variability of IDEAL-CT at intermediate temperature, NCAT runs the round-robin test with fourteen participant laboratories from single laboratory prepared specimen at 7.0±0.5% air voids to find the single-laboratory COV at 18.8% and multi-laboratory COV at 20.2% (Taylor at al., 2019; Taylor 2019). Rutgers ran similar evaluations using five mixtures and nine participant laboratories, where single laboratories compacted samples at 5.5±0.5% air voids (Bennert et al., 2020). The reported single-laboratory COV is 15.2%, and multi-laboratory COV is 23.0%. In VTRC, two mixtures were tested among forty-one participant laboratories to report the single laboratory COV at 18.3% and multi-laboratory COV at 21.3% (Boz et al., 2021) Habbouche et al., 2021).

Although limited information has been found on the variability of HT-IDT, there are variability reports on the IDT at intermediate temperatures. Boz et al. (2021) performed an IDT test in three laboratories and reported COV as high as 10.7% for different equipment setups.

In both test methods, establishing a single acceptance threshold in the design and QA becomes challenging due to this high range of variabilities. So, the main objective of this study is to identify all the possible sources of variabilities and quantify them. Besides, the study also suggests the correction procedures due to these variabilities. The study has been conducted to review the current interlaboratory standard procedures and specifications for variability in test methods. And also run interlaboratory tests by identifying suitable labs and mixtures.

*3.2 **Experimental Plan***

3.2.1Methods and Indices

- IDEAL-CT

The IDEAL-Cracking test is an index-based cracking evaluation of asphalt mixtures at an intermediate temperature under static loading conditions. The test is performed according to ASTM D8225. A cylindrical specimen with a shape of 150mmx62mm is compacted at 7±0.5% air void and loaded at a diametrical plane at 50±2 mm/min loading rate till the load reaches 100N. Resulted in the load-displacement curve is used to estimate a unique index parameter called CT_{index}. CT_{index} is defined as the following formula (3-1).

$$CT_{index} = \frac{G_f}{|m_{75}|} \times \left(\frac{l_{75}}{D}\right) \times \left(\frac{t}{62}\right) \qquad (3\text{-}1)$$

Where l_{75} is the displacement for 75% of the peak load in the unloading portion of the curve, D is the diameter of the sample, in mm, m_{75} is the slope of the unloading curve = $\left|\frac{P_{85}-P_{65}}{l_{85}-l_{65}}\right|$, and G_f is the fracture energy in J/m^2. Before each test, the specimen must be conditioned in a water bath for 2 hours at 25 °C.

- HT-IDT

The indirect tensile test is a well-adopted performance test for stiffness measurements among agencies and industries. When performed in high temperatures, IDT can characterize the rutting performance of asphalt mixtures (Christensen and Bonaquest,2004, 2007; Bennert et al., 2018). The selection of the test temperature depends on the climate condition of the location. Researchers (Christensen and Bonaquest, 2007) used a temperature of 10 °C below the yearly 7-day average maximum temperature 20 mm below the pavement surface with 98% reliability, as determined by the Long-Term Pavement Performance Bind (LTPPBind). For Maryland, this temperature is 43°C. For this test, a cylindrical specimen with 150mmx95mm is compacted at 7±0.5% air voids and compacted and loaded at a diametrical plane at a deformation rate of 50±2mm/min until the failure. The peak load, P, is then recorded and converted to tensile strength using the following formula.

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

Where D is the diameter of the sample; t is the thickness of the sample. The specimen is required to be conditioned for 2 hours in a water bath or environment chamber at 43°C and tested maintaining the temperature. Maintaining this high temperature is somewhat challenging, so the results' variability may rise from laboratory to laboratory.

- ILS Standards

The most common standard for carrying out the variability analysis for construction materials is *ASTM E691-21 Standard Practice for Conducting an Interlaboratory Study to Determine the Precision of a Test Method.* The standard describes the procedures for estimating the repeatability and reproducibility of any test methods. Repeatability is measured for within-laboratory results, while reproducibility is measured for inter-laboratory results. The variation can be measured in terms of range, standard deviation, coefficient of variation, and variance, to name a few parameters. ASTM D4867-14 describes the test method for assessing moisture on asphalt concrete paving mixtures. The multi-laboratory standard deviation has been reported as 8% in this standard. The maximum allowable difference in the tensile-strength ratio between the results of tests performed on samples of the same mixture by two different laboratories' different two-sigma limits (d2s limit) was set to 23%. In ASTM *D6931-17 Standard test method for Indirect Tensile (IDT) strength of asphalt mixtures,* the within-laboratory repeatability standard deviation has been determined to be 80 kPa, and multi-laboratory precision is yet still to be published.

- Correction Factor (CF) Procedures

Establishing correction factors in QA of the asphalt mixtures design is another practice besides identifying the precision and bias. A CF methodology must be specified when the resulting standard deviation from the ILS studies shows a big difference, which will cause a substantial error in the reported QC results from laboratory to laboratory. For instance, the CF procedure is undertaken for the *AASHTO T308, the Standard Method of Test for Determining the Asphalt Binder Content of Hot Mix Asphalt (HMA) by the Ignition Method*. This method tests asphalt mixtures in an ignition furnace with a forced draft exhaust system by exposing them to 1000°F. Due to materials, temperature, and equipment conditions, aggregates are constantly lost. This results in the incorrect measurement aggregate to binder ratio when compared to the initial mass of the total mixture. A unique CF is determined for individual mixtures/furnaces to counter the error. Although establishing CF for the ignition method is more challenging, given the sophistication of the procedures of asphalt content determination, it significantly helps mitigate risk associated with the asphalt mixture design.

3.2.2 Mixtures

A total of six surface mixtures were collected from the ongoing construction projects in Maryland. The mixtures varied in composition and mixing method. There were two warm mixes, and four hot mixes were collected. Details on the basic characteristics of these mixtures are shown in Table 3-1. Among the six mixes, five are dense graded mixes (designed for traffic levels of 0.3 to 3 million), with varied Reclaimed Asphalt Contents (RAP) ranging from 10% to 45%. Mix 2, Mix 5, and Mix 6 also consist of aggregate with high dynamic friction value (DFV). All dense graded mixes are designed with soft binder type of PG 64-22 or 58-28. On the other hand, mix 1 is a Gap Graded Stone Matrix Asphalt (GAP) mix designed with a polymer-modified stiff binder, PG 76-22.

Table 3-1 Basic characteristics of asphalt mixtures

Mixtures	**Mix Method**	**Mix Band**	**Mix Type**	**RAP%**	**Traffic Level**	**Asphalt Type**
Mix 1	Hot mix	12.5 mm	RAP/GAP	15%RAP	>30	64E-22(76-22)
Mix 2	Warm mix	9.5 mm	RAP/High DFV	15%RAP	0.3 to <3	64S-22(64-22)
Mix 3	Hot mix	12.5 mm	RAP	45%RAP	0.3 to <3	58S-28(58-28)
Mix 4	Hot mix	12.5 mm	RAP	10%RAP	0.3 to <3	64S-22(64-22)
Mix 5	Warm mix	9.5 mm	RAP/High DFV	32%RAP	0.3 to <3	64S-22(64-22)
Mix 6	Hot mix	12.5 mm	RAP/High DFV	19%RAP	0.3 to <3	64S-22(64-22)

A further distinction of the mixtures is shown in Table 3-2, showing the asphalt mixtures' volumetric properties, such as specific gravities, G_{mm} and $G_{mb,}$; air voids, VTM, VMA, VFA; effective binder content, P_{be} and aggregate gradations.

Table 3-2 Volumetric properties of the asphalt mixtures

Mixtures	**Design Binder Content (%)**	**G_{mm}**	**VTM (%)**	**VMA (%)**	**VFA (%)**	**P_{be} (%)**	**G_{mb} at N_{des}**	**%passing #200**	**Cumulative %retained#4**	**Cumulative %retained 3/8"**
Mix 1	6.5	2.468	3.5	18	80.7	6.3	2.382	8.5	68	28
Mix 2	5.2	2.516	4	15.9	74.8	5.1	2.414	7.2	33	4
Mix 3	4.8	2.538	4	14.6	72.4	4.5	2.436	6.7	50	15
Mix 4	5.8	2.471	4	15.9	74.9	5.2	2.372	6.4	42	12
Mix 5	5.2	2.497	4	15.1	73.5	4.8	2.397	6.4	34	5
Mix 6	4.6	2.695	4	14.5	72.6	4.2	2.588	4.7	45	13

3.2.3 Sample preparation and conditioning

After collecting the mixtures, two sets of samples were compacted, one set for IDEAL-CT tests with a dimension of 150mm x 62mm (diameter to height) and the other set for HT-IDT with a dimension of 150mm x 95mm (diameter to height). Both sets of samples were compacted at 7±0.5% air void content. NCAT conducted ILS for IDEAL-CT in two phases; in the first phase, participating labs got the loose samples, and in the second phase, labs got the compacted discs to conduct the test. The variability reduces from 33% to 11.1%. So, the study concluded that two-thirds of the variability of the test is related to differences in sample fabrication (Hajj et al., 2021). So, to reduce the sample preparation-related variability, all samples were compacted in one laboratory and distributed among other laboratories. But the samples' conditioning (2 hours in a water bath) was done in the respective laboratories. The samples must be tested within a short period after removing from the water bath to maintain the test temperature. For conditioning, the participating laboratories used different environmental chambers and conventional ovens.

3.2.4 Participated Laboratories

For this study, laboratories are selected so that the state highway agency, the producers, and the main research laboratory (responsible for identifying the performance criteria values and BMD framework) are all involved.

3.3 Data Analysis and Results

As mentioned in the previous section, the sources of variability in performance test results can arise from sample preparation to equipment issues. In this section, various sources of variability have been evaluated. Table 3-3 shows an experimental plan for assessing variability that complements the results. For example, five mixtures (Mix 2 to Mix 6) were tested using the split sample method, where samples were compacted in one lab and distributed to the other laboratories. Mix 1, however, was samples prepared and tested in separate laboratories. IDEAL-CT and HT-IDT tests required 2 hours of asphalt samples conditioning in the water bath at intermediate and high temperatures. Although the temperature needed to be maintained strictly for the entire period, using different conditioning equipment (e.g., Conventional Oven versus Environment Chamber) introduced variability in results. In this study, all three laboratories used different conditioning equipment. Other types of testing equipment are available for loading the samples in the diametrical plane. Loading systems can be screw-driven systems or servo-hydraulic systems. Differences in the instruments may cause a difference in loading rate, peak load, and recording of the data points, which all lead to significant variability in the results. Mix 1 through Mix 5 were tested in three laboratories to assess the instrumentation-related variability. In each case, different operators tested mixtures samples in each laboratory.

Table 3-3 Sources of variability

Mixture	**Sample Preparation**	**Conditioning Equipment**	**Testing Equipment**	**Operator**
Mix 1	x	x	x	x
Mix 2	√	x	x	x
Mix 3	√	x	x	x
Mix 4	√	x	x	x
Mix 5	√	x	x	x
Mix 6	√	x	x	x

Note. √ Means same; x means different.

Testing variability can also be associated with sample preparation. Potential sources include variations in the heating of mixtures, segregation while handling the mixtures, compaction equipment, and the number of air voids in the compacted samples, to name a few. Mix 1 was collected from the plant and sent directly to two different laboratories in this study. The samples were compacted for IDEAL-CT and HT-IDT tests according to the standards and procedures described in the previous section. The result in interlaboratory COV for the Cracking Index between lab 1 and lab 2 is 9.0%, and HT-IDT is 46.3%.

The effect of the air void content during the compaction influences the performance of the mixtures. According to Zhou et al. (2017), the IDEAL-CT test's cracking index varies with the air void contents' variation. But the study shows variation in results for high variation in air void content, such as 5% to 8%.

Figure 3-1 was plotted to see the effect of the air void content on the cracking index and tensile strength variations. Here the air void content variation is 6.5% to 7.5%. According to these results, air voids variations within the specified ±0.5% interval have a small impact on testing results. Thus, for both IDEAL-CT and HT-IDT tests, the range of air void content of the compacted samples can be kept at 7±0.5%.

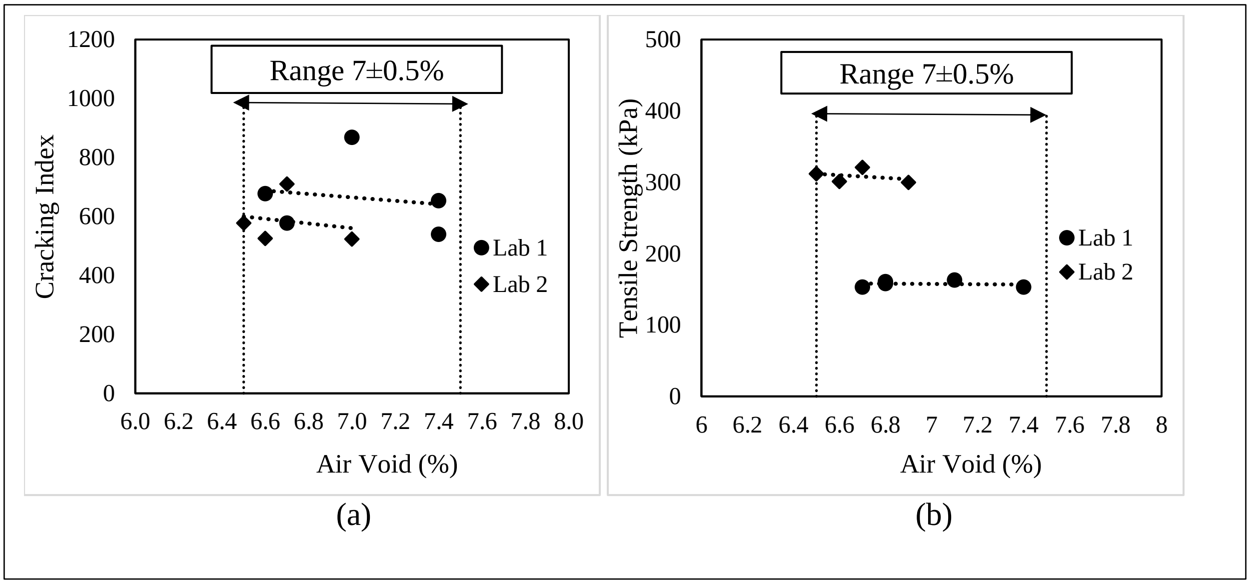


Figure 3-1 Effect of air void content on cracking index and the tensile strength

While Mix 2 and Mix 3 were tested in Lab 1 and lab 2, Mix 4 to Mix 6 was tested in three laboratories with different testing equipment. The results from Mix 2 and Mx 3 needed to discard due to equipment malfunction. The performance results of Mix 4 to Mix 6 are shown in Table 3-4 and

Table *3-5*. The average CT_{index} for Mix 4 is 104 to 165, Mix 5 is 45 to 56, and Mix 6 is 83 to 122. In a previous study, the initial performance criteria for dense graded mixes in Maryland were set to 90 (Akhter et al., 2022). While Mix 4 and Mix 5 are consistently overperforming or underperforming, respectively; however, in Mix 6, only one laboratory result showed an underperforming mixture. This acknowledges the importance of identifying the variability between labs, establishing a precision statement, and eventually suggesting the necessary correction factors. The variability in test results for IDEAL-CT ranges from 10.1% to 18.4% for all three mixes among the three laboratories. This is comparably lower than other studies incorporating more laboratories or mixtures in the variability testing of IDEAL CT. For instance, Habbouche et al., 2021, reported the variability among six mixtures ranges from 9.1% to 40.7%. This brings questions are the COV can be correlated to the mixture characteristics, as some mixtures show such high variability compared to others.

Figure 3-2 has been created to show the effects of the volumetric properties of mixes on the variability of test results. Overall, mixtures with lower RAP are softer and present higher variability in test results. This is important as softer mixes may be prone to show more skew in the threshold selection of the mixtures.

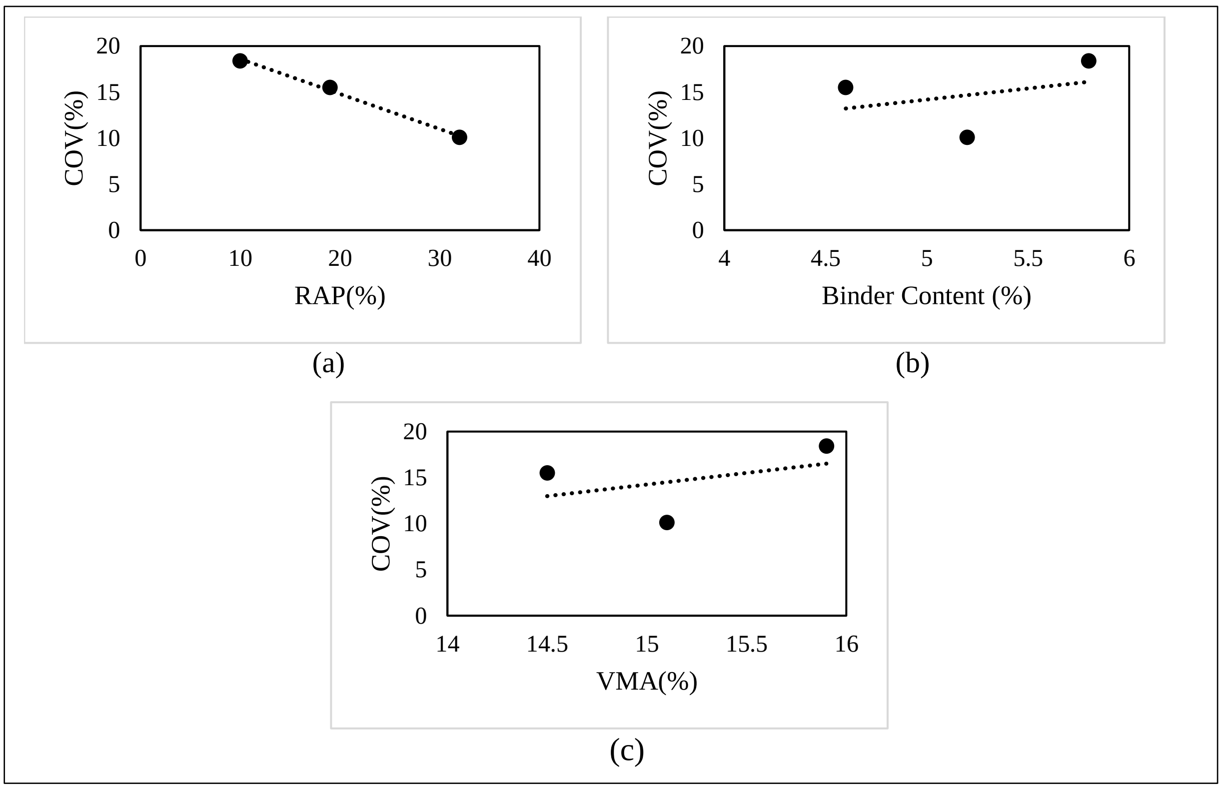


Figure 3-2 Effect of volumetric properties of asphalt mixtures in the variability of IDEAL-CT results (n= 4)

The COVs here are estimated for interlaboratory variabilities. The average HT-IDT for Mix 4 is 212 to 325 kPa, Mix 5 is 189 to 311 kPa, and Mix 6 is 190 to 247 kPa. All these mixtures meet the Maryland mixtures' initial performance criteria, which is set as 165 kPa (Akhter et al., 2022). The variability for HT-IDT is 17% to 22.4%. Among the three labs, Lab 1 has constantly shown statistically significantly lower values than the other two labs. The performance measurements were based on four replicates for individual laboratories.

Table 3-4 Interlaboratory variability in IDEAL-CT results

	Mix 4			Mix 5			Mix 6		
	Lab 1	**Lab 2**	**Lab 3**	**Lab 1**	**Lab 2**	**Lab 3**	**Lab 1**	**Lab 2**	**Lab 3**
Average Peak Load (KN)	12.6	15.2	15.9	18.3	17.8	17.5	18.4	17.6	13.5
COV (%), Peak Load	3.1	3.5	3.3	1.9	1.0	2.5	6.1	1.5	1.0
Average CT_{index}	165	139	104	45	56	46	83	105	122
COV (%), CT_{index}	17.7	30.2	15.3	11.6	5.2	8.6	19.2	13.3	21.2
Interlaboratory COV (%), CT_{index}	18.4			10.1			15.5		

Table 3-5 Interlaboratory variability in HT-IDT results

	Mix 3			Mix 4			Mix 5		
	Lab 1	**Lab 2**	**Lab 3**	**Lab 1**	**Lab 2**	**Lab 3**	**Lab 1**	**Lab 2**	**Lab 3**
Average Peak Load (KN)	4.5	6.3	7.3	4.2	6.9	7.3	4.3	6.8	5.5
COV (%), Peak Load	1.1	7.1	2.4	2.1	8.1	0.6	3.4	1.3	5.1
Average Tensile strength (kPa)	212	283	325	189	311	328	190	306	247
COV (%), Tensile Strength	1.1	7.1	2.4	2.4	8.1	0.5	3.3	1.3	5.2
Interlaboratory COV (%), Tensile Strength	17.0			22.4			19.1		

The first investigation of the source of the machine-related variability is the loading rate during the tests. Table 3-6 and Table 3-7 show the average deformation rate from all these three labs, COV of the rates in each data point, and tolerance limits. Lab 1 shows the lowest deformation rate (around 48 mm/min) among the three instruments with the highest COV (approximately 46%). This is a potential cause for the significantly lower HT-IDT values, shown in

Table *3-5*. Here's to be noted, Lab 2 and Lab 3 use the screw-driven loading system, while Lab 1 uses the servo-hydraulic loading system.

Table 3-6 Deformation rate analysis for IDEAL-CT tests

	Mix 3			**Mix 4**			**Mix 5**		
	Lab 1	**Lab 2**	**Lab 3**	**Lab 1**	**Lab 2**	**Lab 3**	**Lab 1**	**Lab 2**	**Lab 3**
Average Deformation Rate	48.8	51.5	50.3	48.0	51.3	50.3	48.5	51.5	50.3
COV (%), Deformation Rate	45.9	14.3	17.0	46.5	16.0	17.3	46.2	14.0	13.1
%Load Readings within Tolerance	4.1	26.8	41.8	21.6	20.9	5.2	4.7	26.5	25.2

Table 3-7 Deformation rate analysis for HT-IDT tests

	Mix 3			**Mix 4**			**Mix 5**		
	Lab 1	**Lab 2**	**Lab 3**	**Lab 1**	**Lab 2**	**Lab 3**	**Lab 1**	**Lab 2**	**Lab 3**
Average Deformation Rate	48.0	51.5	50.5	49.2	51.7	50.5	49.0	51.6	50.5
COV (%), Deformation Rate	45.9	13.4	13.8	45.8	12.8	16.1	45.7	13.0	17.3
%Load Readings within Tolerance	3.1	30.0	24.4	2.6	31.7	20.6	3.2	32.1	21.0

While there is less variability between lab 2 and lab 3, a deeper investigation within these two laboratory results was performed and shown in Figure 3-3. Figure 3-3 shows the load, displacement, and deformation rate of two samples tested in Lab 2 and Lab 3. Both samples are compacted from the same Mix 5, in the same lab, by the same personnel at an air void content of 6.8%. But the samples were conditioned in different lab water environment chambers and tested by different personnel. The resulting CT_{index} are 52 and 43, respectively, with 13.4% variability.

So, it is to be noted that even with identifying the sources and percentage of variability, some correction factors for instruments should be established, along with the acceptance criteria and COV.

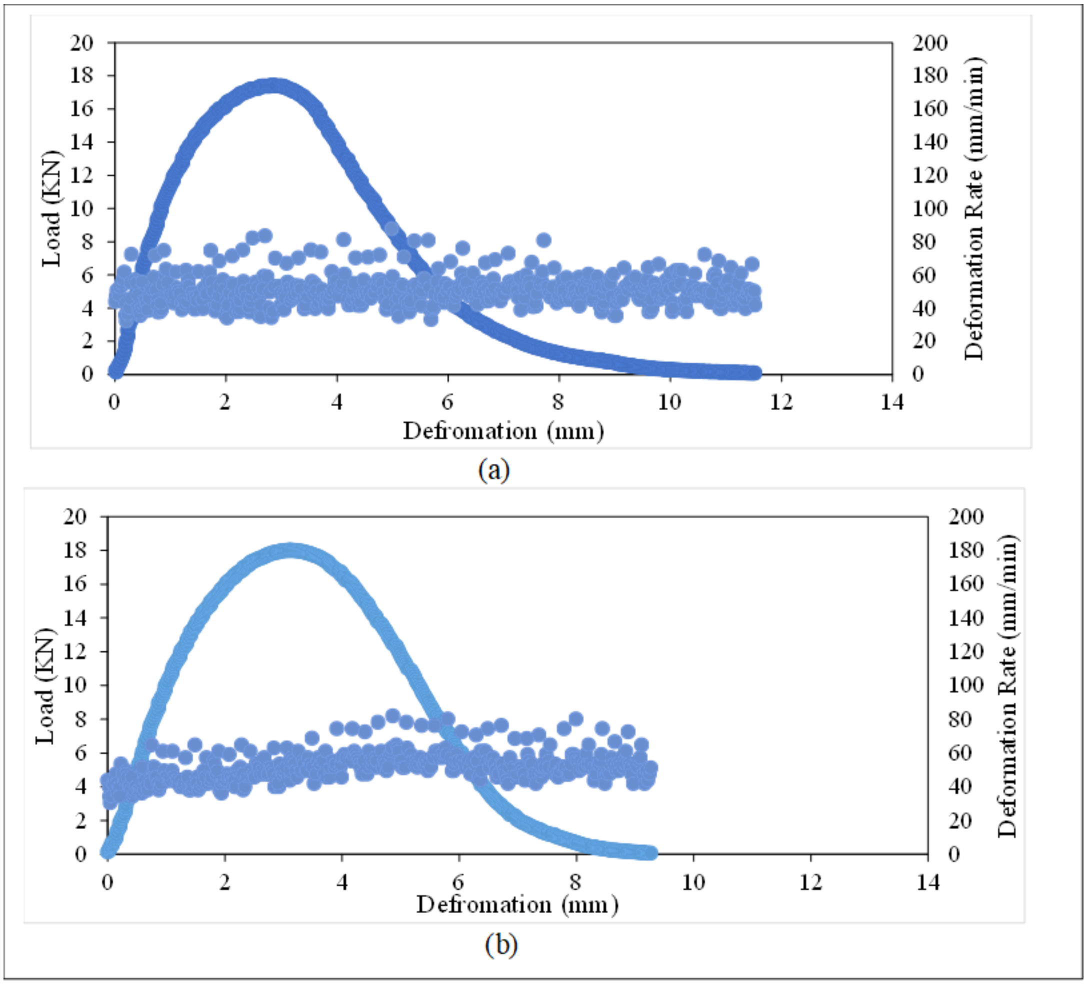


Figure 3-3 Deformation rate for different devices

3.4 Adjustment Factor Procedures

3.4.1 Establishing Correction Factors for Cracking Index

As evident, the IDEAL-CT test produces results that are impacted by the variability issues due to instrumentation among laboratories. These variabilities may be countered by identifying correction factors with the acceptance criteria. CT_{index} is defined in the previous section in equation (3-1), where G_f is the fracture energy, ratio of the area under the load-displacement curve (W_f) divided by the area of the fracture. Based on the following substitutions, the K adjustment factor can be calculated:

$$CT_{Index} = \frac{W_f}{|m_{75}|\ D * t} \times \left(\frac{l_{75}}{D}\right) \times \left(\frac{t}{62}\right)$$

$$CT_{Index} = \frac{W_f * l_{75}}{|m_{75}|} \times \left(\frac{1}{D^2\ * t}\right) \times \left(\frac{t}{62}\right)$$

$$CT_{Index} = K \times \left(\frac{1}{D^2\ * t}\right) \times \left(\frac{t}{62}\right)$$

Thus, the ratio of the CT_{index} between labs is equivalent to the K ratios.

$CT_{index,\ BaseLab}$ / $CT_{index,\ Lab2}$ = $K_{BaseLab}$ / K_{Lab2}

Corrected $CT_{index,Lab2}$ = ($K_{BaseLab}$ / K_{Lab2}) * $CT_{index,Lab2}$

If assumed Lab 1 is a base laboratory, which can be the state agency lab, then the correction of the CT_{index} for other labs, such as Lab 2, will be as shown in Table 3-8. The adjustment of the CT_{index} can be sample specific, where the correction factors will be applied to specimens with the same density (i.e., air void content) or mixture specific. It is to be noted there were two specimens prepared for the particular air voids. In both cases, the interlaboratory COVs have reduced significantly. Table 3-9 shows the paired t-test of the results. As attaining exact air void content is somewhat challenging while compacting the asphalt mixture samples, a mix-specific QA approach is recommended. More examples of applying the correction to other mixtures are shown in **Appendix B**.

Table 3-8 Example of adjusted CT_{index} values between labs.

Mixture	**Air Voids**	**K (Lab 1)**	**K (Lab 2)**	**Ratio of K**	**Lab1 CTindex**	**Lab2 CTindex**	**Ratio of CTindex**	**Corrected Lab2 CTindex (specific air voids)**	**Corrected Lab2 CTindex (Mix specific)**
Mix 5	6.6	58.571	82.7	0.7085	42	59	0.7119	42	48
	6.7	73.017	78.48	0.9304	52	56	0.9286	52	46
Interlaboratory COV						10.1		0.1	0.1

Table 3-9 Paired t-test results for CT_{index}

Mix	**Corrected Lab 2 Strength (specific air voids)**	**Corrected Lab 2 tensile Strength (Mix specific)**
Mix 5	H0	H0

3.4.2 Establishing Correction Factors for Tensile strength.

As evident from

Table *3-5*, the inter-laboratory COV is significantly higher than the interlaboratory COV; a correction factor with the acceptance threshold should be provided, given the sources of variability arising due to laboratory environment and equipment. The equation relating the peak load to the indirect tensile strength is provided by equation (3-2) which is as follows:

$$S_t = \frac{2P}{\pi D t}$$

Thus, considering the difference in peak loads between labs observed for the data reported in Table 3-10, the ratio of the peak loads between the labs will translate into a ratio between corresponding tensile strengths.

$S_{t,Lab1} = (2 * P_{Lab1})/(\pi D t)$

$S_{t,Lab2} = (2 * P_{Lab2})/(\pi D t)$

$S_{t,Lab1} / S_{t,Lab2} = P_{Lab1} / P_{Lab2}$

Thus, the adjusted Lab2 Tensile Strength = $(P_{Lab1} / P_{Lab2}) * S_{t,Lab2}$

A similar approach to the cracking index has been taken here to correct the Lab2 tensile strength, eliminating interlaboratory variability. No statistically significant variation has been found for Mix 4 results (Table 3-11). More examples of applying a correction to other mixtures are shown in **Appendix B**.

Table 3-10 Example of adjusted tensile strength between labs.

Mixture	**Air Voids**	**Lab1 Load (KN)**	**Lab2 Load (KN)**	**The ratio of Peak Load**	**Lab1 Strength**	**Lab2 Strength**	**The ratio of Tensile Strength**	**Corrected Lab2 strength (specific air voids)**	**Corrected Lab2 tensile Strength (Mix specific)**
					(kPa)	(kPa)		(kPa)	(kPa)
Mix 4	7	4.4	6.9	0.6377	196	308	0.6364	196	195
	7.1	4.4	6.7	0.6507	195	299	0.6522	195	189
	7.2	4.2	6.9	0.6087	188	309	0.6084	188	195
Interlaboratory COV						22.5		0.0	0.0

Table 3-11 Paired t-test results for tensile strength.

Mix	Corrected Lab 2 Strength (specific air voids)	Corrected Lab 2 tensile Strength (Mix specific)
Mix 4	H0	H0

3.5 Conclusion

Quality assurance of the asphalt mixes is essential in ensuring long-lasting asphalt pavement systems. Performance tests (cracking, rutting, moisture damage, and so on) are being adopted by the agencies and asphalt industries, which lead to QA performance evaluation for these test criteria besides the current specifications of densities, air content, and aggregate gradations. This study evaluated variability issues for IDEAL-CT and HT-IDT and adopted tests for Maryland and other states (i.e., Virginia, Texas, and New Jersey) for their BMD. The primary sources of variability have been identified, which include, but are not limited to, the sample preparation, sample condition equipment, loading equipment, and operator. Variabilities of these test results may have been sensitive to the volumetric properties of the mixtures. The study suggests that there is a possibility of variability in the performance results might depend on the volumetric properties of the mixtures. Considering the IDT test setup without confinement, softer mixtures may result in bigger variability, hence prone to skew in the acceptance threshold. The study has established correction factors for both these tests, significantly reducing the variability in the test parameters. Thus, these correction factors will be along with the acceptance criteria of the cracking and rutting for the use of the producer's QC during production.

Chapter 4 : Performance Prediction of Asphalt Mixtures

4.1 Introduction

Following the laboratory performance testing of asphalt mixtures, the next step was to predict the field mixtures' performance. With BMD under development and a lack of historical field performance data, an alternative approach for performance prediction is needed.

Some efforts from the Virginia Department of Transportation (VDOT) have undergone to collect samples immediately after construction from behind the paver to predict the field performance of mixtures (Diefenderfer et al., 2019). However, the major drawback of this approach is that field performance cannot be quantified unless the pavement has undergone traffic loads and environmental impact over a long period of time. Some studies (Elias et al., 2021; Zhang et al., 2021) explored long-term aging protocols to predict this aging effect from laboratory specimens. This addresses oxidation effects but not the in-service performance of asphalt mixtures in the field.

Pavement design and analysis are moving towards mechanical-empirical design methods such as MEPDG. This approach uses mechanistic-empirical models to evaluate the pavement responses to traffic loading and environmental factors regarding stress, strain, or deformation. Then it uses distress prediction models to predict pavement performance, Figure 4-1 (NCHRP 2004b). Material inputs are required in pavement response and distress models to predict pavement performance.

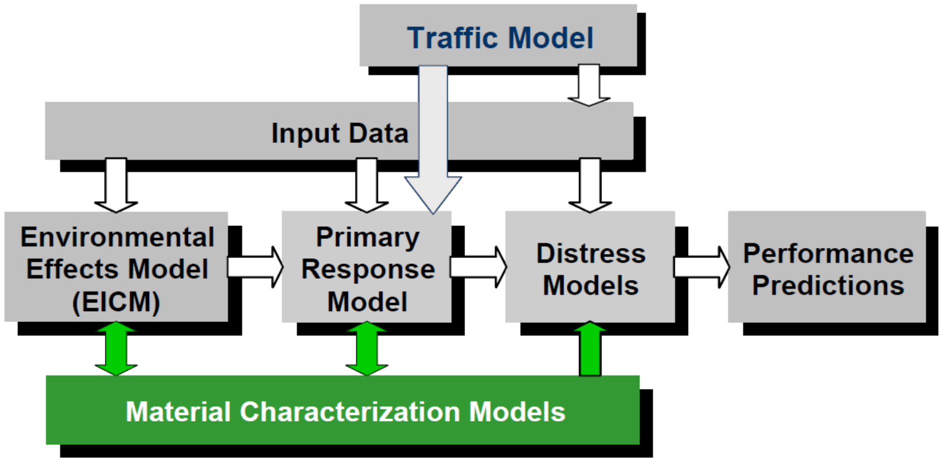


Figure 4-1 MEPDG framework for Pavement Design (NCHRP 2004b).

This study's objective was to develop an approach to predict the field performance of asphalt mixtures regarding rutting and cracking. In this regard, the modeling proposed in past studies was reviewed and analyzed with experimental data. The predictive models were assessed and fine-tuned using the mixtures' material properties, volumetrics, and mechanical response parameters. Eventually, such analysis could produce to define target criteria for cracking and rutting in the field identified as failure thresholds (i.e., rutting depression depth, number of load applications to fatigue failure, other).

4.2 Literature Review on Predictive Models

This section presents the proposed predictive models in the literature for rutting and fatigue cracking of asphalt pavement systems. First, the predictive models for the key performance parameter of HMAs, dynamic modulus, were reviewed. Following E* prediction, mixture rutting and fatigue prediction models were examined using E*. Then the service life pavement prediction models were examined and used in the analysis. After an in-depth review of the proposed models in the literature, those recommended in a national study (NCHRP 704) were selected since they were developed based on an exhaustive data set from various asphalt mixtures around the US. These are presented in the following sections, along with the analysis.

4.2.1Prediction of E

Dynamic modulus is a key parameter in predicting field performance in the pavement design process. Dynamic modulus, E*, defines the stiffness characteristics of HMA as a function of the frequency of loading and temperature. The Mechanical Empirical Prediction Design Guide (MEPDG) guide provides three hierarchical levels of inputs (Level 1, 2, and 3) for E*. In level 1, E* is measured directly from the laboratory tests. In levels 2 and 3, E* is predicted from the volumetric properties and binder properties of the compacted samples from prediction models. Due to the availability of the equipment and the time consumption in running tests and analyses, Level 1 is not always a feasible option. So, the prediction model for E* is highly desirable for further pavement performance analysis.

Some of the most common AC dynamic modulus prediction models are the Shell oil equation (Bonnaure et al., 1996), the Witczak model (NCHRP 1-37A), the Hirsch model (Christensen et al., 2003), the Al-Khateeb model (2013), are to name a few. Among these Witczak model has gained the most acceptability over the years to predict the AC dynamic modulus. The Witczak E* prediction Equation produces results comparable to the E* values estimated from the lab tests (Loilizi et al., 2006; Lundy et al., 2005; Azari et al., 2007, El-Boussany et al., 2011). In a more recent study (Batioja-Alvarez et al., 2019), several Indiana mixtures were produced and tested for dynamic modulus to evaluate the efficiency of the Witczak, Hirsch, and Al-Khateeb model. The Witczak 1-37A model is the most efficient predictive model regarding accuracy, repeatability, and reproducibility. Witczak 1-37A model produced in 1969, Shook and Kallas first developed the first set of dynamic modulus prediction equations, which was letter developed by Witczak and colleagues (Fonsceca and Witczak, 1996) utilizing 1,430 points from 149 conventional asphalt mixes. The current Witczak model (NCHRP 1-37A) is based on 2,750 points from an additional 56 mixes, which predict dynamic modulus as a function of aggregate gradation, binder content and viscosity, air void, and loading frequency.

$$log_{10}|E^*| = -1.249937 + 0.02923p_{200} - 0.001767\,(p_{200})^2 - 0.00284p_4 - 0.05809V_a - 0.082208\frac{V_{beff}}{V_{beff}+V_a} + \frac{3.871977-0.0021p_4+0.003958p_{3/8}-0.000017(p_{3/8})^2+0.00547p_{3/4}}{1+\exp(-0.603313-0.31335logf-0.393532log\eta)} \tag{4-1}$$

Where,

p_{200} = percentage of aggregate passing sieve #200
p_4 = percentage of aggregate retained in sieve #4
$p_{3/8}$ = percentage of aggregate retained in sieve 3/8 inch
$p_{3/4}$ = percentage of aggregate retained in sieve 3/4 inch
V_a = percentage of air voids
V_{beff} = percentage of effective asphalt content
f = loading frequency, Hz
η = binder viscosity at a temperature of interest, Poise

4.2.2 Prediction Model for Permanent Deformation (Rutting)

Rutting occurs in the asphalt pavement due to the accumulation of traffic loads. Although rutting occurs in the different layers of the multilayer pavement system, this study focuses on predicting the rutting in the AC layer. Rutting is directly related to the stiffness of the pavement and evaluated from the dynamic modulus of the asphalt mixes. In MEPDG rutting, the AC layer is predicted from accumulated plastic strain in different depths (NCHRP 2004a). Equation (4-2) shows the prediction model for the plastic strain to vertical strain relationship of the AC layer under traffic load at a specific temperature.

$$\frac{\varepsilon_p}{\varepsilon_r} = aT^bN^c \tag{4-2}$$

Where,

ε_p = total plastic strain at N repetitions of traffic load
ε_r = resilient strain of the asphalt mixtures
N = number of load repetitions
T = pavement temperature
a , b, c = Non-linear regression coefficients

The relationship between plastic strain and vertical resistant strain at any point of the AC layer provides the opportunity to estimate the plastic strain, predict the rut depth according to the following model (4-3), and then sum up all the rutting for the entire layer (4-4).

$$\Delta R_{d_i} = \varepsilon_{p_i} . \Delta h_i \quad (4\text{-}3)$$

$$R_d = \sum_{i=1}^{n} \Delta R_{d_i} \quad (4\text{-}4)$$

As dynamic modulus, E* is the key parameter representing the mixture properties; NCHRP 704 study describes the rut depth as a function of E*, according to the following formula.

$$RUT = aE^{*b} \quad (4\text{-}5)$$

Where,

RUT = rutting depth, inch

E* = dynamic Modulus

MEPDG Version 1.0 (preliminary version of AASHTOware Pavement Design) was used to run simulations to establish an empirical relationship (R^2 = 0.985) between the E* and rutting of the asphalt pavement layer, where a = 0.8159 and b = 0.631. About 4356 simulation runs were performed to establish this relationship. These runs were based on input parameters reflecting the asphalt pavement scenarios nationwide. The input parameters used in these simulation runs are listed below:

- Pavement Structure: El-Basnyouan (2011) proves from a series of sensitivity analyses that subgrade soil characteristics have minimal effect on the rutting characteristics of the AC layer. So, for this analysis, only full-depth pavement systems were used with asphalt layers on top of subgrade soil.
- Environmental Sites: 12 climatic locations
- Design Life: 20 years
- Design Traffic: 4 traffic levels, 10^5,10^6, 10^7, 10^8 ESALs
- Vehicle Speed: Combination of vehicle speeds (0.5,15,45, and 60 mph)
- AC layer thickness: Various (1,2,3,4,6,8,12, and 20 in.)
- Asphalt Mix Characteristics: Various binder types range from soft to stiff.

4.2.3Prediction model for fatigue cracking

Fatigue cracking is a load-induced distress caused by bending the AC layers. These cracks initiate at the bottom of the AC layer and propagate to the surface under repeated load repetitions. The fatigue model used in MEPDG for cracking evaluation in the asphalt concrete layer is shown in the following equation 4-6.

$$N_f = 0.00432 * C * \beta_{f1} * k_1 * \left(\frac{1}{\varepsilon_t}\right)^{k_2 * \beta_{f2}} \left(\frac{1}{E}\right)^{k_3 * \beta_{f3}} \quad (4\text{-}6)$$

Where,
N_f = number of load repetitions for fatigue cracking
ε_t =tensile strain at a critical location, in/in
E = stiffness of asphalt material, psi
β_{f1} , β_{f2} , β_{f3} = local calibration coefficients
C = 10^M
M = $4.84\left(\frac{V_b}{V_a + V_b} - 0.69\right)$
V_a = Air voids, %
V_b = Effective binder content, %
k_1 , k_2 , k_3 = national coefficients = 0.007566, 3.9492, and 1.281, respectively.

After estimating the number of load repetitions till cracking, the following transfer function (Equation 4-7) is used to assess the percent of alligator cracking in the AC layer, a function of the estimated percentage of damage.

$$F.C._{(A)} = \left(\frac{6000}{1+e^{\left(C_1 * C_1' + C_2 * C_2' * log10(Damages)\right)}}\right) * \left(\frac{1}{60}\right) \tag{4-7}$$

Where,
$F.C._{(A)}$ = Fatigue alligator cracking, %
$C_2' = -2.40874 - 39.748 * (1 + h_{ac})^{-2.856}$
C_1' =-2C_2'

El-Badway et al. (2009) have proposed a methodology to predict fatigue cracking in the HMA from the AC dynamic modulus. In this study, authors developed an accurate closed-form solution for the fatigue cracking prediction based on the dynamic modulus of HMA from simulations run of MEPDG version 1.0, using a combination of inputs significant for the load-associated bottom-up fatigue cracking damage. It's important to note, firstly, the study has developed a fatigue cracking model for a two-layer pavement system and then developed a predicted methodology to transform a multilayer system into a two-layer system.

- Pavement Structure: one HMA layer over a composite foundation layer
- Environmental Sites: Grand Forks, North Dakota (Cold); Oklahoma City, Oklahoma (Moderate) and Key West, Florida (Hot).
- Design Life: 20 years
- Design Traffic: $2x10^6$ ESALs
- Vehicle Speed: Combination of vehicle speed
- AC layer thickness: Various
- Asphalt Mix Characteristics: Various binder types range from soft to stiff.
- Composite foundation modulus: Six E_{cf} values for simulating soft to rigid foundations.

Based on 4536 simulation runs of MEPDG, a general comprehensive model to predict fatigue cracking is proposed as a function of dynamic modulus of AC, AC thickness, VFB, and E_{cf}. The prediction model is shown as follows,

$$\log N_f = 8.3014 - \{[(b_1 \log(h_{ac})^2 + b_2 \log(h_{ac}) + b_3) \log(E^*) + b_4 \log(h_{ac})^2 + b_5 \log(h_{ac}) + b_6] \log(E_{cf})^2 + [b_7 \log(E^*)^2 + b_8 \log(E^*) + b_9] \log(E_{cf}) + b_{10} \log(h_{ac})^2 + b_{11} \log(h_{ac}) + b_{12}] \log(VFB)^2 + [b_{13} \log(h_{ac})^2 + b_{14} \log(h_{ac}) + b_{15}] \log(VFB) + b_{16} \log(E^*)^2 + [b_{17} (h_{ac})^2 + b_{18}(h_{ac}) + b_{19}] \log(E^*) + b_{20} \tag{4-8}$$

Where,
N_f = number of repetitions for fatigue failure
E^* = dynamic modulus of AC layer, ksi
h_{ac} = AC layer thickness, inch
E_{cf} = dynamic modulus of the composite foundations (may include base, subbase, and subgrade), ksi
b_1 to b_{20} = regression coefficient
VFB = voids filled with asphalt, %

In function of the AC layer thickness, the developed model's regression coefficients were reported in Table 4-1.

Researchers have developed two sets of coefficients based on the thickness of the AC layer. The thickness primarily affects the cracking propagation from the bottom of the layer to the surface. The thin model is developed for AC later thickness smaller than 3 inches (76.2 mm), and the thick model is for greater than 3 inches (76.2 mm).

Table 4-1 Regression coefficients for fatigue damage prediction model (Adopted from NCHRP 704)

Regression Coefficients	Thin Model	Thick Model
b1	-0.0095	0.0645
b2	-0.0756	-0.0144
b3	-0.0438	0.0416
b4	-0.5414	-0.6003
b5	1.4319	0.7046
b6	-1.0252	-1.0276
b7	-0.0208	-0.0218
b8	0.7040	0.6280
b9	-4.1171	-3.2499
b10	-4.1659	28.9186
b11	-3.0733	-51.9588
b12	-6.4418	12.7671
b13	-1.5883	15.8844
b14	-2.8014	-28.6128
b15	-9.2885	0.9160
b16	-0.1177	-0.1792
b17	0.0681	0.0024
b18	-0.3789	-0.1009
b19	0.8989	1.2623
b20	2.9330	1.4613

4.2.4 Prediction of Service Life

NCHRP 704 study has proposed a performance-related specification (PRS) methodology for quality assurance (QA) of hot-mixes asphalt (HMA) construction. The study creates a program that employs a database of pre-solved solutions of the MEPDG (currently AASHTOware Pavement Design Software). In the study, HMA rutting and fatigue cracking performance prediction models are assessed and validated for a wide range of parameters through MEPDG simulations. The most noteworthy achievement is to employ the dynamic modulus as the key parameter to predict the rutting and fatigue cracking and the service life accordingly.

After evaluating the rutting for each mixture, the rut life can be predicted using the following Equation.

$$Y = \frac{\log\left(\left(\frac{RUT}{RUT_C} * \frac{E^*}{E_C^*}\right)^{\frac{1}{0.479244}}\left((1+r)^{Y_C}-1\right)+1\right)}{\log(1+r)} \tag{4-9}$$

Where,
Y = predicted service life, years
Y_c = design life, years
RUT = rut depth, inch
RUT_c = rut depth criterion value, inch
E* = dynamic modulus, ksi
E^*_c = dynamic modulus criterion value, ksi
r = growth rate (rate of traffic increase per year), %

Similarly, the fatigue life can be estimated based on the following model stated in the NCHRP 704 study, where fatigue distress, N_f, has been translated to the fatigue service life, Y.

Figure 4-2 shows a general example of the fatigue life of the as-designed (JMF) mixes, where the criteria value for fatigue damage is 30% after the 20 years of service life. Based on this, the field performance of the mixes after production (MCS_1 and MCS_2) can be compared. Mixture MCS_1 reached 30% fatigue damage in less than 20 years of time.

$$Y = \frac{\log\left(\frac{N_f}{N_{fc}}((1+r)^{Y_c}-1\right)+1)}{\log(1+r)} \tag{4-10}$$

Where,
Y = predicted service life, years
Y_c = design life, years
N_f = allowable failure traffic repetitions
N_{fc} = criterion value for failure traffic repetitions
E* = dynamic modulus, ksi
E^*_c = dynamic modulus criterion value, ksi
r = growth rate (rate of traffic increase per year), %

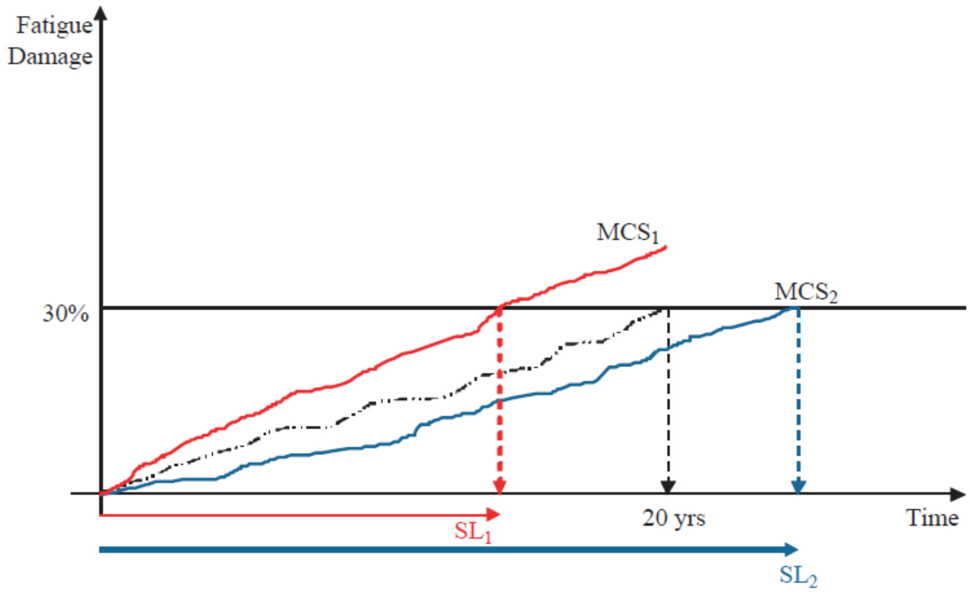


Figure 4-2 Fatigue service life prediction (Adopted from NCHRP 704)

Note: MCS = Mixture Designation   SL = Service Life

This formula is further simplified in NCHRP 704 study by substituting Equation for N_f and N_{fc}, where the critical N_f is taken as the initial ESAL value for which the mixtures are designed. Constants are obtained for the damage of 30% in the study from Monte Carlo simulations. These constants should be modified for local materials.

$$Y_i = \frac{\left(\left\{10^\wedge\left(\beta_1\left(\log\frac{E_i^{*2}}{E_c^{*2}}\right)+\beta_2\left(\log\frac{E_i^*}{E_c^*}\right)+\beta_3\right)\right\}\{(1+r)^{Y_c}+1\}+1\right)}{\log(1+r)} \tag{4-11}$$

where

$$\beta_1 = a_{17} + a_8 \log E_{cf}$$

$$\beta_2 = \alpha_1(\log E_{cf})^2 + a_9 \log E_{cf} + \alpha_4$$

$$\beta_3 = \alpha_2\left(\log\frac{VFB_i^2}{VFB_c^2}\right) + \alpha_3\left(\log\frac{VFB_i}{VFB_c}\right)$$

where

$\alpha_1 = a_2 \log h^2_{ac} + a_3 \log h_{ac} + a_4$
$\alpha_2 = a_{11} \log h^2_{ac} + a_{12} \log h_{ac} + a_{13}$
$\alpha_3 = a_{14} \log h^2_{ac} + a_{15} \log h_{ac} + a_{16}$
$\alpha_4 = a_{18}h^2_{ac} + a_{19}h_{ac} + a_{20}$
$a_2 = -0.0095$ for thin model or 0.0645 for thick model
$a_3 = -0.0756$ for thin model or -0.0144 for thick model
$a_4 = 0.0438$ for thin model or 0.0416 for thick model
$a_{11} = 4.1659$ for thin model or 28.9186 for thick model
$a_{12} = -3.0733$ for thin model or -51.9588 for thick model
$a_{13} = -6.4418$ for thin model or 12.7671 for thick model
$a_{14} = -1.5883$ for thin model or 15.8844 for thick model
$a_{15} = -2.8014$ for thin model or -28.6128 for thick model
$a_{16} = 9.2885$ for thin model or 0.9160 for thick model
$a_{17} = -0.1177$ for thin model or -0.1792 for thick model
$a_{18} = 0.0681$ for thin model or 0.0024 for thick model
$a_{19} = -0.3789$ for thin model or -0.1099 for thick model
$a_{20} = 0.8989$ for thin model or 1.2623 for thick model

4.3. Methodology

This section describes the step-by-step approach selected for the field performance prediction of the asphalt mixtures in terms of pavement service life.

- The first step is to select the most suitable prediction model for the estimation of the dynamic modulus of the mixtures. The Witczak 1-37A model has been selected for this study. As mentioned in the previous section, this model produced the most relatable results to laboratory tests.
- As the dynamic modulus is a function of the volumetric properties of the selected asphalt mixtures and test conditions (Equation 4-1), the next step will be to collect these data.
- Rutting service life and fatigue service life of the asphalt pavement will be evaluated for the selected asphalt mixtures using models described in previous sections (Subsection 4.2.2 to 4. 2.4).

4.3.1 Asphalt Mixtures

Volumetrically designed asphalt mixtures were collected from plants and fields from different Maryland pavement construction projects. These dense-graded surface mixtures were also used to establish Maryland's BMD approach. These mixtures vary in binder type and content, aggregate gradation and sources, recycled materials content, etc. Basic characteristics have been shown in

Table 4-2. It is significant to point out that this study includes mixtures with a wide range of Recycled Asphalt Pavement (RAP) contents as state agencies are highly concerned about the unsatisfactory performance of asphalt mixtures, including higher content of recycled materials. Furthermore, warm mix mixtures were also included in the study, besides typical hot mix asphalts. The inadequacy of the warm mix technology was a concern for the asphalt industries for some time.

Table 4-2 Basic characteristics of the asphalt mixtures

Mixtures	**Mix Method**	**Mix Band**	**Mix Type**	**RAP%**	**Traffic Level**	**Asphalt Type**
DG1	Warm mix	19 mm	RAP	45%RAP	0.3 to <3	58S-28(58-28)
DG2	Hot mix	9.5 mm	RAP	45%RAP	0.3 to <3	58S-28(58-28)
DG3	Hot mix	9.5 mm	RAP	45%RAP	0.3 to <3	58S-28(58-28)
DG4	Warm mix	12.5mm	RAP	27% RAP	0.3 to <3	64S-22(64-22)
DG5	Warm mix	9.5 mm	RAP	29% RAP	0.3 to <3	64S-22(64-22)
DG6	Warm mix	9.5 mm	RAP/High DFV	32%RAP	0.3 to <3	64S-22(64-22)
DG7	Warm mix	12.5 mm	RAP/High DFV	10%RAP	0.3 to <3	64S-22(64-22)
DG8	Warm mix	12.5 mm	RAP/High DFV	15%RAP	0.3 to <3	64S-22(64-22)
DG9	Warm mix	9.5 mm	RAP/High DFV	15%RAP	0.3 to <3	64S-22(64-22)
DG10	Hot mix	12.5 mm	RAP	45%RAP	0.3 to <3	58S-28(58-28)
DG11	Hot mix	12.5 mm	RAP	10%RAP	0.3 to <3	64S-22(64-22)
DG12	Hot mix	12.5 mm	RAP/High DFV	19%RAP	0.3 to <3	64S-22(64-22)
DG13	Hot mix	12.5 mm	RAP/High DFV	35%RAP	0.3 to <3	58S-28(58-28)
DG14	Hot mix	12.5 mm	RAP	19%RAP	0.3 to <3	64S-22(64-22)
DG15	Hot mix	12.5 mm	RAP/High DFV	40%RAP	0.3 to <3	64S-22(64-22)
DG16	Hot mix	12.5 mm	Virgin	0	0.3 to <3	64S-22(64-22)
DG17	Hot mix	9.5 mm	RAP/High DFV	35%RAP	0.3 to <3	64S-22(64-22)
DG18	Hot mix	9.5 mm	RAP	20%RAP	0.3 to <3	64S-22(64-22)
DG19	Hot mix	12.5 mm	RAP	13%RAP	0.3 to <3	64S-22(64-22)
DG20	Hot mix	12.5 mm	RAP	10%RAP	0.3 to <3	64S-22(64-22)
DG21	Hot mix	9.5 mm	RAP	30%RAP	0.3 to <3	64S-22(64-22)
DG22	Hot mix	9.5 mm	RAP/High DFV	35%RAP	0.3 to <3	58S-28(58-28)

Table 4-3 shows the value of such parameters for all the collected asphalt mixtures. These volumetric properties are part of the QC during production and laying in the field of the asphalt mixtures.

Table 4-3 Dense graded mixtures volumetric properties

Mixtures	**G_{mm}**	**VTM (%)**	**VMA (%)**	**VFA (%)**	**Pbe (%)**	**G_{mb} at N_{des}**	**%Passing in sieve No 200**	**Cum %retained in #4**	**Cum %retained in #3/8"**	**Cum %retained in #3/4"**
DG 1 (P)	2.624	3.0	13.2	71.0	4.4	2.545	5.4	57	32	8
DG 1 (M)	2.639	3.3	12.9	74.4	4.4	2.553	5.9	54	28	5
DG 1 (F)	2.623	2.7	13.0	79.2	4.4	2.551	5.8	55	30	9
DG2 (P)	2.593	2.9	15.8	81.7	5.1	2.518	6.9	44	7	0
DG2 (M)	2.587	2.7	15.8	83.1	5.2	2.518	6.9	42	8	0
DG2 (F)	2.605	3.5	15.9	78.1	5.0	2.514	5.8	46	8	0
DG3 (P)	2.602	3.0	15.4	80.3	5.1	2.523	7.6	46	8	0
DG3 (M)	2.595	2.8	15.5	82.0	5.2	2.522	7.3	46	10	0
DG3 (F)	2.598	3.2	15.7	79.5	5.1	2.514	7.4	45	8	0
DG4 (P)	2.532	4.6	15.5	70.5	5.1	2.416	6.9	43	11	0
DG4 (M)	2.533	3.9	14.6	73.3	4.7	2.435	7.2	44	13	0
DG4 (F)	2.537	4.8	15.5	68.8	4.9	2.415	6.1	43	12	0
DG5 (P)	2.506	4.1	15.1	73.0	5.2	2.404	4.8	35	2	0
DG5 (M)	2.509	3.0	14.2	79.1	5.4	2.435	5.0	42	5	0
DG5 (F)	2.507	3.6	14.7	75.7	5.3	2.418	5.3	37	4	0
DG6 (P)	2.489	3.0	14.5	79.3	5.2	2.414	6.7	36	5	0
DG6 (M)	2.478	4.3	16.2	73.5	5.4	2.371	5.5	35	5	0
DG6 (F)	2.500	3.4	14.3	76.2	5.0	2.416	6.1	35	5	0
DG7 (P)	2.618	5.4	14.5	62.9	4.3	2.477	6.1	47	13	0
DG7 (M)	2.598	2.6	13.4	80.6	5.1	2.530	7.2	42	10	0
DG7 (F)	2.594	4.5	15.1	70.6	5.1	2.478	6.9	42	7	0
DG8 (P)	2.478	3.4	15.5	77.5	5.5	2.394	6.9	47	5	0
DG8 (M)	2.478	3.4	15.1	77.5	5.4	2.394	6.7	41	8	0
DG8 (F)	2.485	4.1	15.5	73.5	5.5	2.384	7.0	36	5	0
DG9 (P)	2.516	4.0	15.9	74.8	5.1	2.414	7.2	33	4	0
DG10 (P)	2.538	4.0	14.6	72.4	4.5	2.436	6.7	50	15	0
DG11 (P)	2.471	4.0	15.9	74.9	5.2	2.372	6.4	42	12	0
DG12 (P)	2.695	4.0	14.5	72.6	4.2	2.588	4.7	45	13	0
DG13 (P)	2.563	4.0	15.4	73.9	4.7	2.460	6.2	53	17	0
DG14 (P)	2.585	4.0	14.4	72.1	4.3	2.481	7.0	46	14	0
DG15 (P)	2.582	4.0	14.7	72.8	4.4	2.479	6.3	46	16	0
DG16 (P)	2.546	4.0	15.1	73.5	4.7	2.444	6.8	41	14	0
DG17 (P)	2.492	4.0	15.1	73.4	4.8	2.385	6.5	33	4	0
DG18 (P)	2.513	4.0	15.4	73.9	4.9	2.412	5.7	37	4	0
DG19 (P)	2.530	4.0	14.3	72.0	4.4	2.429	6.0	54	17	0
DG20 (P)	2.471	4.0	15.9	74.9	5.2	2.372	6.4	42	12	0
DG21 (P)	2.562	4.0	15.2	73.6	4.7	2.459	6.7	43	5	0
DG22 (P)	2.563	4.0	15.4	73.9	4.7	2.460	5.8	41	3	0

4.4. Data Analysis

4.4.1 Dynamic Modulus

The first step estimates dynamic modulus E* from the volumetric properties and the loading frequency. The binder viscosity for different binder types and testing temperatures (IDEAL-CT is 25°C and HT-IDT is 43°C) can be evaluated using the following empirical relations (Mirza and Witczak 1995). The A and VTS values are available for different binder types from the NCHRP 1-37A report, Table 4-4.

$$loglog(\eta) = \begin{cases} A + VTS\ (T_R) & T_R > T_{critical} \\ 1.095 & T_R \leq T_{critical} \end{cases} \quad (4\text{-}12)$$

Where,

η = viscosity (cP)
A = Intercept of temperature susceptibility relationship
VTS = Slope of the temperature susceptibility relationship
T_R = Temperature in Rankine
$T_{critical}$ = Temperature in Rankine at which the viscosity is equal to 2.7×10^{12} cP

Table 4-4 Viscosity parameters for different asphalt binder grades (Adopted from NCHRP 1-37A)

Asphalt Binder Grade	**A**	**VTS**	**Asphalt Binder Grade**	**A**	**VTS**
PG 46-34	11.5040	-3.9010	PG 70-28	9.7150	-3.2170
PG 46-40	10.1010	-3.3930	PG 70-34	8.9650	-2.9480
PG 46-46	8.7550	-2.9050	PG 70-40	8.1290	-2.6480
PG 52-10	13.3860	-4.5700	PG 76-10	10.0590	-3.3310
PG 52-16	13.3050	-4.5410	PG 76-16	10.0150	-3.3150
PG 52-22	12.7550	-4.3420	PG 76-22	9.7150	-3.2080
PG 52-28	11.8400	-4.0120	PG 76-28	9.2000	-3.0240
PG 52-34	10.7070	-3.6020	PG 76-34	8.5320	-2.7850
PG 52-40	9.4960	-3.1640	PG 82-10	9.5140	-3.1280
PG 52-46	8.3100	-2.7360	PG 82-16	9.4750	-3.1140
PG 58-10	12.3160	-4.1720	PG 82-22	9.2090	-3.0190
PG 58-16	12.2480	-4.1470	PG 82-28	8.7500	-2.8560
PG 58-22	11.7870	-3.9810	PG 82-34	8.1510	-2.6420
PG 58-28	11.0100	-3.7010	AC-2.5	11.5167	-3.8900
PG 58-34	10.0350	-3.3500	AC-5	11.2614	-3.7914
PG 58-40	8.9760	-2.9680	AC-10	11.0134	-3.6954
PG 64-10	11.4320	-3.8420	AC-20	10.7709	-3.6017
PG 64-16	11.3750	-3.8220	AC-3	10.6316	-3.5480
PG 64-22	10.9800	-3.6800	AC-40	10.5338	-3.5104
PG 64-28	10.3120	-3.4400	PEN 40-50	10.5254	-3.5047
PG 64-34	9.4610	-3.1340	PEN 60-70	10.6508	-3.5537
PG 64-40	8.5240	-2.7980	PEN 85-100	11.8232	-3.6210
PG 70-10	10.6900	-3.5660	PEN 120-150	11.0897	-3.7252
PG 70-16	10.6410	-3.5480	PEN 200-300	11.8107	-4.0068
PG 70-22	10.2990	-3.4260			

Figure 4-3 shows the E* values for different mixtures at intermediate and high temperatures. These values were estimated using Equation (4-1) for loading frequency at 0.1Hz. The viscosity was calculated for the MD mixtures at the intermediate temperature of 25 °C and High temperature of 43 °C, which are reference temperatures for performance evaluation of fatigue cracking and rutting, respectively. At intermediate temperature, dynamic modulus ranges from 457 ksi (3151 MPa) to 917 ksi (6322 MPa) for different MD mixtures. The range is 245 ksi (1689 MPa) to 358 ksi (2468 MPa) at high temperatures.

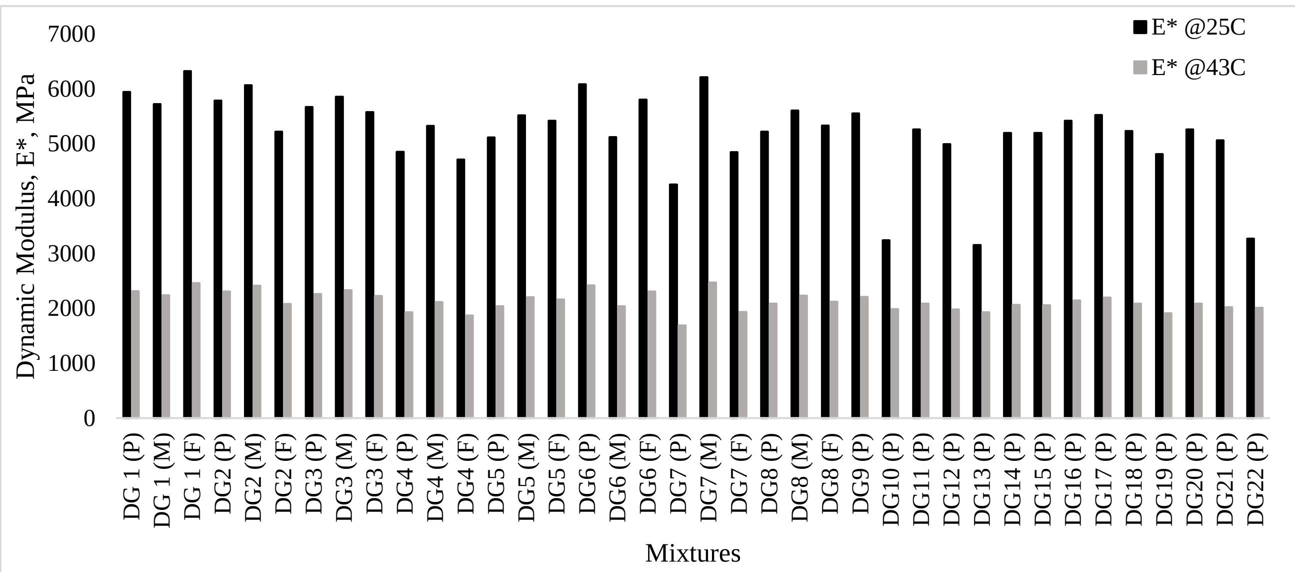


Figure 4-3 Dynamic modulus for individual mixes in different temperatures

4.4.2 Rutting Service Life

Following Figure 4-4 has been reconstructed from the model established in the NCHRP 704 in accordance with Equation (4-5) and modified for the effective temperature (43 °C) and frequency (0.1 Hz) used for the Maryland asphalt mixtures.

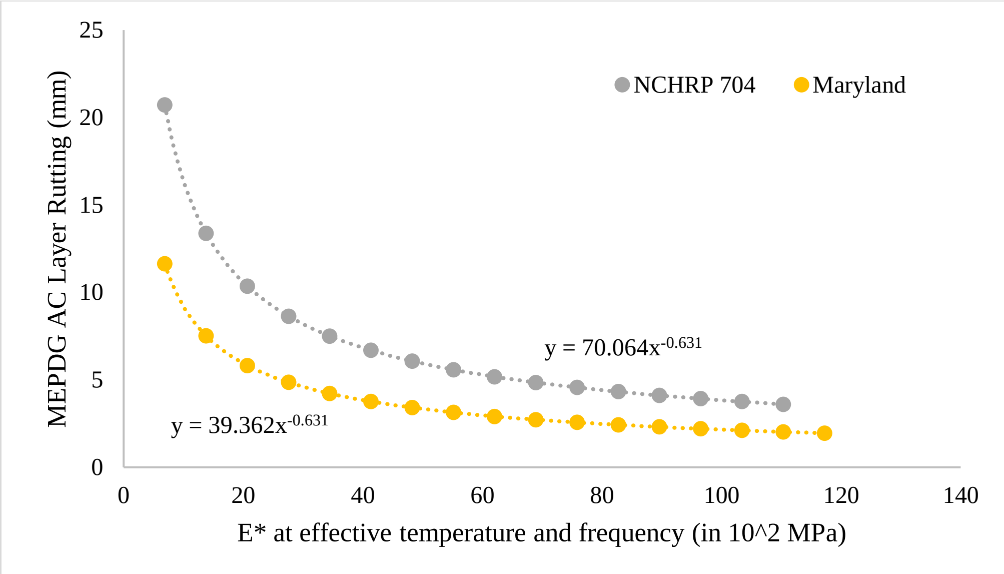


Figure 4-4 E* and Rutting relationships at effective temperature and frequency

E* values translated to the rut depth using the previously stated model, shown in Table 4-5. Rut depth for volumetrically designed mixtures ranges from 5.1 mm to 6.6 mm (0.20 to 0.26 inches). Some important parameters, such as design life, criteria values for E* and RUT, and annual traffic growth rate, r, should be determined to evaluate the service life for rutting.

- Selection of the Design Life

  For Maryland pavements, the structural design life is 25 years during the design of the new pavement. But the flexible pavement, especially the HMA surfaces, doesn't last for 25 years to be structurally sufficient. The goal is to prevent structural distress rather than functional distresses to provide a sustainable pavement system. So, the functional design life for all new HMA pavement is 15 years. (MDDOTSHA 2022)

- Annual Traffic Growth Rate

  The assumed annual traffic growth rate is 4%, which remained constant for the entire pavement service life.

- Criteria Value for Rut Depth and E*

The criteria values for E* and Rut depth have been selected based on benchmarking approach, where threshold values were selected at average and average plus or minus standard deviation from the results of selected mixtures. For this study, average and standard deviation minus 1.96*Standard deviation was selected for the criteria for the rut depth and corresponding E* value.

Based on the above criteria, Table 4-5 shows the predicted rutting service life for different mixtures, where criteria values are selected at average minus 1.96 * standard deviation (at 95% confidence interval). The criteria E* for all the dense graded mixtures is 1786MPa (259 ksi), whose corresponding rutting is 6.4 mm (0.25 inch). The service life for this criterion varies from 14.7 to 18.2 years, Figure 4-5.

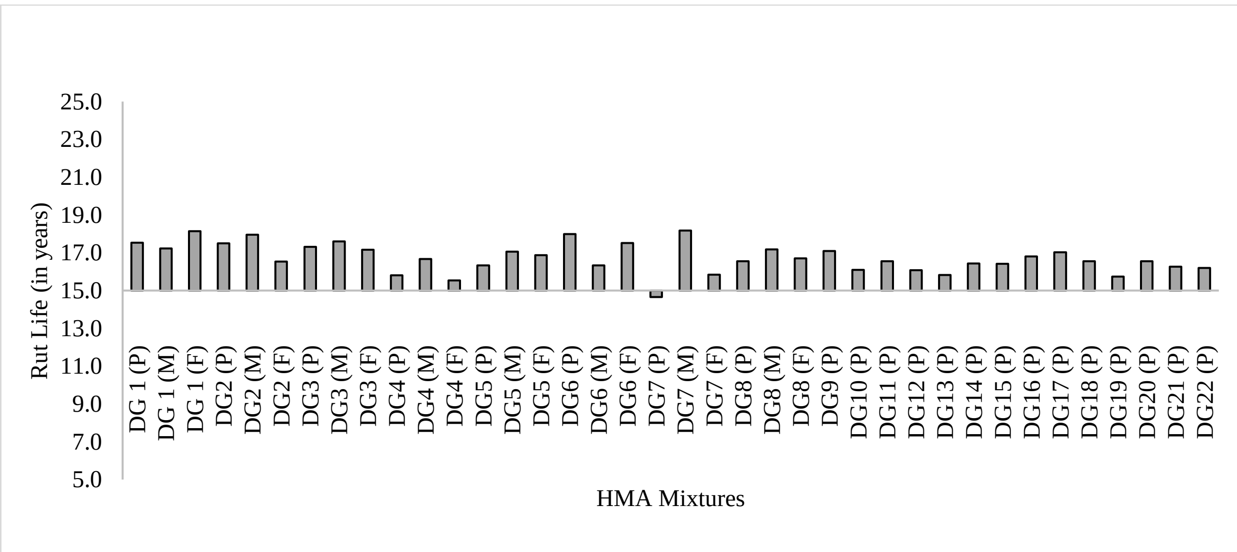


Figure 4-5 Rutting service life for different asphalt mixtures.

Table 4-5 Modulus, rutting depth, and predicted rut service life.

Mixture	E (in MPa)	Rut depth (mm)	Rutting Service Life (in Years)
DG 1 (P)	2310	5.3	17.5
DG 1 (M)	2241	5.6	17.2
DG 1 (F)	2461	5.3	18.1
DG2 (P)	2303	5.3	17.5
DG2 (M)	2413	5.3	18.0
DG2 (F)	2082	5.8	16.5
DG3 (P)	2261	5.6	17.3
DG3 (M)	2330	5.3	17.6
DG3 (F)	2227	5.6	17.2
DG4 (P)	1924	6.1	15.8
DG4 (M)	2110	5.8	16.7
DG4 (F)	1868	6.1	15.5
DG5 (P)	2041	5.8	16.3
DG5 (M)	2199	5.6	17.1
DG5 (F)	2158	5.6	16.9
DG6 (P)	2420	5.3	18.0
DG6 (M)	2034	5.8	16.3
DG6 (F)	2310	5.3	17.5
DG7 (P)	1689	6.6	14.7
DG7 (M)	2468	5.1	18.2
DG7 (F)	1931	6.1	15.8
DG8 (P)	2089	5.8	16.6
DG8 (M)	2227	5.6	17.2
DG8 (F)	2124	5.8	16.7
DG9 (P)	2206	5.6	17.1
DG10 (P)	1986	6.1	16.1
DG11 (P)	2089	5.8	16.5
DG12 (P)	1979	6.1	16.1
DG13 (P)	1931	6.1	15.8
DG14 (P)	2062	5.8	16.4
DG15 (P)	2055	5.8	16.4
DG16 (P)	2144	5.6	16.8
DG17 (P)	2193	5.6	17.0
DG18 (P)	2082	5.8	16.5
DG19 (P)	1910	6.1	15.7
DG20 (P)	2089	5.8	16.5
DG21 (P)	2020	5.8	16.3
DG22 (P)	2006	5.8	16.2

Note: E* for all the dense graded mixtures is 1786 MPa, whose corresponding rutting is 6.4 mm (0.25 inch).

4.4.3 Fatigue Service life

Firstly, allowable traffic load repetitions, $N_{f,}$ have been estimated for individual mixtures. But as the primary goal is to simplify the analysis, Equation (4-11) has been used to assess fatigue service life for individual mixtures. The major assumptions for the estimation of the fatigue life were as follows:

- Selection of design service life and annual traffic growth

  Design service life, Yc, and traffic growth, r, are the same as the rutting life estimation, 15 years and 4%, respectively.

- Asphalt layer thickness, Voids filled with bitumen, and Foundation modulus.

  For the estimation of the fatigue service life, either a thin (h_{ac} < 3inch) or thick (h_{ac} >3inch) model can be used. Only thin models with AC layer thickness of 3 inches or 76.2 mm have been used for now. The criteria value for voids filled with bitumen, VFB, has been taken as 71.5%, the average of the required allowable range in Superpave mix design (AASHTO M323) for traffic level with ESAL at 0.3 to 3 million. The foundation modulus has been assumed at 21 MPa (3000 psi).

- Criteria value for Dynamic modulus

  The dynamic modulus thresholds have been taken at the average of all mixture results. These results were also used for the benchmarking approach of selecting the BMD criteria for Maryland. The average E* at room temperature is 5226 MPa (758 ksi). The study also concentrates on literature to find the E* values for similar mixtures from direct laboratory tests, such as the triaxial test. Based on the literature (Bennert et al., 2009), the criteria value for E* is assumed at 4137 MPa (600 ksi) at room temperature, which is close to the average minus 1.96* standard deviation (at 95% confidence interval).

Table 4-6 and Figure 4-6 show the fatigue life of the asphalt mixtures, while the criteria value for dynamic modulus has been taken at 4137 MPa (600 ksi). Table 4-7 shows the fatigue service life while the criteria value for dynamic modulus has been taken at 5226 MPa (758 ksi).

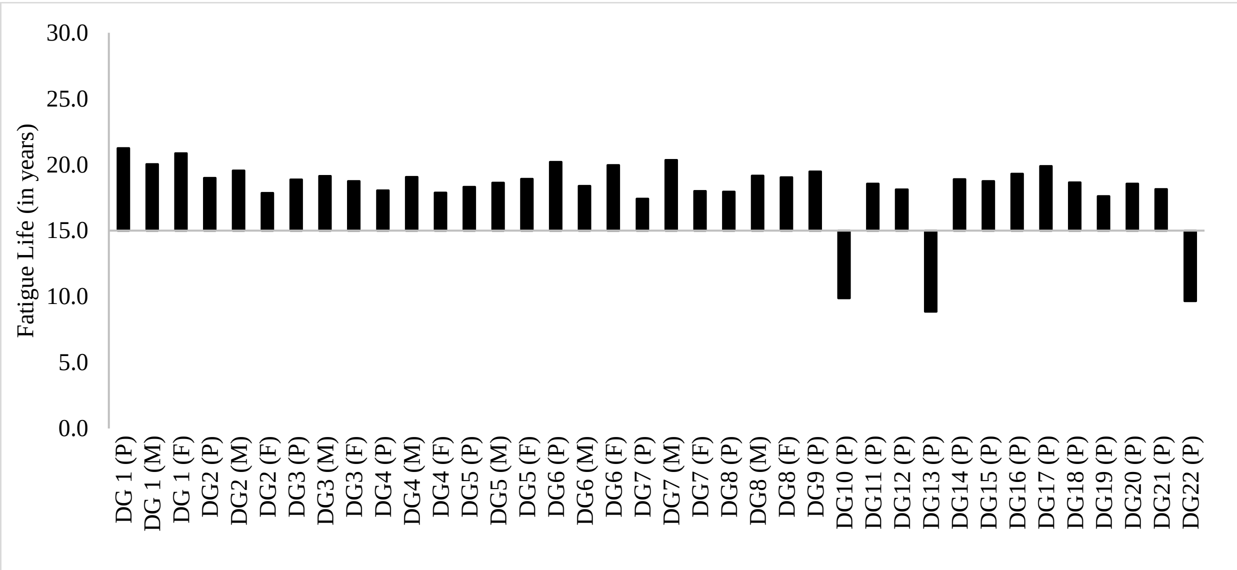


Figure 4-6 Fatigue service life for different asphalt mixtures

Table 4-6 Modulus, and predicted service life (E_c at 4137 MPa)

Mixture	E (in MPa)	Fatigue Service life (Years)
DG 1 (P)	5943	21.2
DG 1 (M)	5723	20.1
DG 1 (F)	6322	20.9
DG2 (P)	5785	19.0
DG2 (M)	6067	19.6
DG2 (F)	5219	17.9
DG3 (P)	5667	18.9
DG3 (M)	5854	19.1
DG3 (F)	5578	18.7
DG4 (P)	4854	18.0
DG4 (M)	5323	19.1
DG4 (F)	4709	17.9
DG5 (P)	5116	18.3
DG5 (M)	5516	18.6
DG5 (F)	5419	18.9
DG6 (P)	6081	20.2
DG6 (M)	5123	18.4
DG6 (F)	5805	20.0
DG7 (P)	4254	17.4
DG7 (M)	6212	20.3
DG7 (F)	4847	18.0
DG8 (P)	5219	18.0
DG8 (M)	5605	19.2
DG8 (F)	5330	19.0
DG9 (P)	5550	19.5
DG10 (P)	3241	9.9
DG11 (P)	5261	18.6
DG12 (P)	4992	18.1
DG13 (P)	3151	8.8
DG14 (P)	5199	18.9
DG15 (P)	5199	18.8
DG16 (P)	5419	19.3
DG17 (P)	5523	19.9
DG18 (P)	5233	18.7
DG19 (P)	4813	17.6
DG20 (P)	5261	18.6
DG21 (P)	5061	18.2
DG22 (P)	3268	9.7

Note: E* is assumed at 4137 MPa at room temperature, relative to the average minus 1.96* standard deviation (at 95% confidence interval).

Table 4-7 Modulus and predicted service life (E_c = 5226 MPa, average)

Mixture	E (in MPa)	Fatigue Service life (Years)
DG 1 (P)	5943	17.4
DG 1 (M)	5723	16.1
DG 1 (F)	6322	16.9
DG2 (P)	5785	14.8
DG2 (M)	6067	15.4
DG2 (F)	5219	13.5
DG3 (P)	5667	14.7
DG3 (M)	5854	15.0
DG3 (F)	5578	14.5
DG4 (P)	4854	13.8
DG4 (M)	5323	14.9
DG4 (F)	4709	13.7
DG5 (P)	5116	14.1
DG5 (M)	5516	14.4
DG5 (F)	5419	14.8
DG6 (P)	6081	16.2
DG6 (M)	5123	14.2
DG6 (F)	5805	15.9
DG7 (P)	4254	13.2
DG7 (M)	6212	16.3
DG7 (F)	4847	13.8
DG8 (P)	5219	13.7
DG8 (M)	5605	15.0
DG8 (F)	5330	14.9
DG9 (P)	5550	15.4
DG10 (P)	3241	4.3
DG11 (P)	5261	14.4
DG12 (P)	4992	13.9
DG13 (P)	3151	3.1
DG14 (P)	5199	14.8
DG15 (P)	5199	14.6
DG16 (P)	5419	15.2
DG17 (P)	5523	15.9
DG18 (P)	5233	14.5
DG19 (P)	4813	13.3
DG20 (P)	5261	14.4
DG21 (P)	5061	13.9
DG22 (P)	3268	4.0

4.5. Mixture Performance Test Results and Service Life of Pavements

As described in earlier chapters, Balanced Mix design, BMD is a new path different state agencies adopt to ensure more durable pavement systems nationwide. The BMD approach balances fatigue cracking and rutting of the asphalt mixture to choose the optimal design binder content. AASHTO PP105-04 describes several test methods for evaluating rutting and cracking characteristics in the laboratory to assess the BMD criteria. IDEAL-CT and HT-IDT were the two performance-related tests adopted for the Maryland BMD. Dynamic Modulus, E, predicted rut life from volumetric properties, and the HT-IDT results are shown in

Table *4-8* for Maryland Field mixtures for comparison. Figure 4-7 is also plotted to identify the relation between the predicted service life from the volumetric properties (through E*) and the ratio of HT-IDT concerning the target desired HT-IDT value. In the y-axis, the ratio of rutting service life (Service life vs. the criteria Service life) is used, while in the x-axis is the ratio of the tensile strength (tensile strength to criteria tensile strength). The criteria for service life are the design life of an asphalt pavement, which is 15 years, and HT-IDT$_c$ is 165 kPa, established for Maryland Mixtures in Chapter 2. A polynomial relation (R^2 =0.73) was obtained between these two parameters. This relationship needs to be further validated with additional field mixes.

Table 4-8 Comparison of predicted permanent deformation (rutting) service life to the tensile strength of asphalt mixes.

Mixture	**E (in MPa)**	**Rut depth (mm)**	**Rutting Service Life (in Years)**	**HT-IDT (kPa)**
DG 1 (F)	2460	5.3	18.1	128
DG2 (F)	2082	5.8	16.5	186
DG3 (F)	2224	5.6	17.2	139
DG4 (F)	1868	6.1	15.5	179
DG5 (F)	2160	5.6	16.9	203
DG6 (F)	2309	5.3	17.5	198
DG7 (F)	1932	6.1	15.8	171
DG8 (F)	2121	5.8	16.7	171

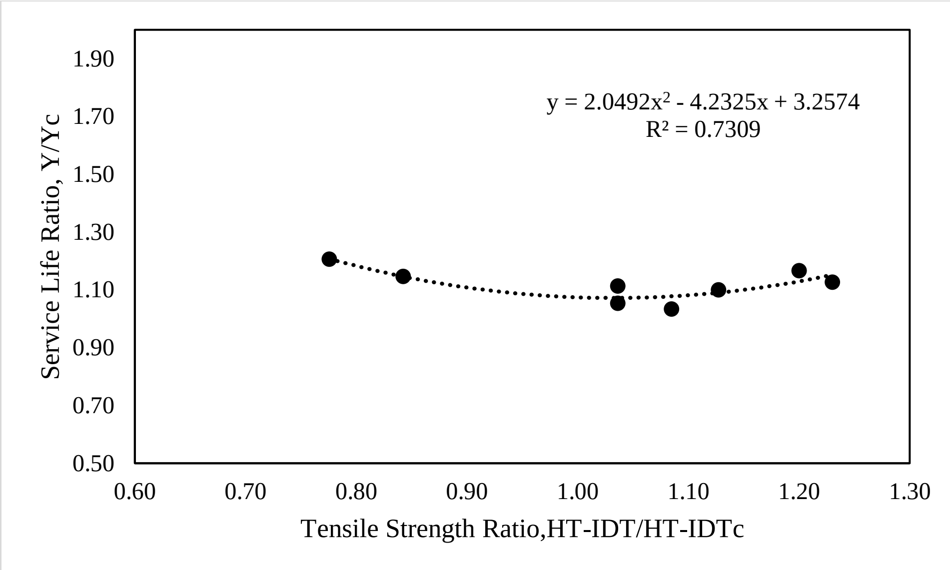


Figure 4-7 Relationship between the tensile strength of mixtures to rutting service life.

A similar effort was undertaken for the fatigue life and IDEAL-CT results. For comparison, the dynamic Modulus, E from volumetric properties, Fatigue service life from E, and the IDEAL-CT results are shown in Table 4-9 for Maryland field mixtures. Figure 4-8 was plotted to assess the relation between the predicted service life from the volumetric properties and E* and the predicted-performance of the asphalt mixtures in terms of CT_{index}. The CT_{index} criteria value is 90 (Chapter 2) for Maryland mixtures. The Figure shows a poor relationship between these two parameters (R^2 = 0.45). Thus, the fatigue model requires further calibration for the local materials.

Table 4-9 Comparison of predicted fatigue service life to cracking index of asphalt mixes.

Mixture	E (in MPa)	Fatigue Service life (Years)	CTindex
DG 1 (F)	6322	16.9	106
DG2 (F)	5219	13.5	106
DG3 (F)	5578	14.5	174
DG4 (F)	4709	13.7	89
DG5 (F)	5419	14.8	99
DG6 (F)	5805	15.9	68
DG7 (F)	4847	13.8	59
DG8 (F)	5330	14.9	147

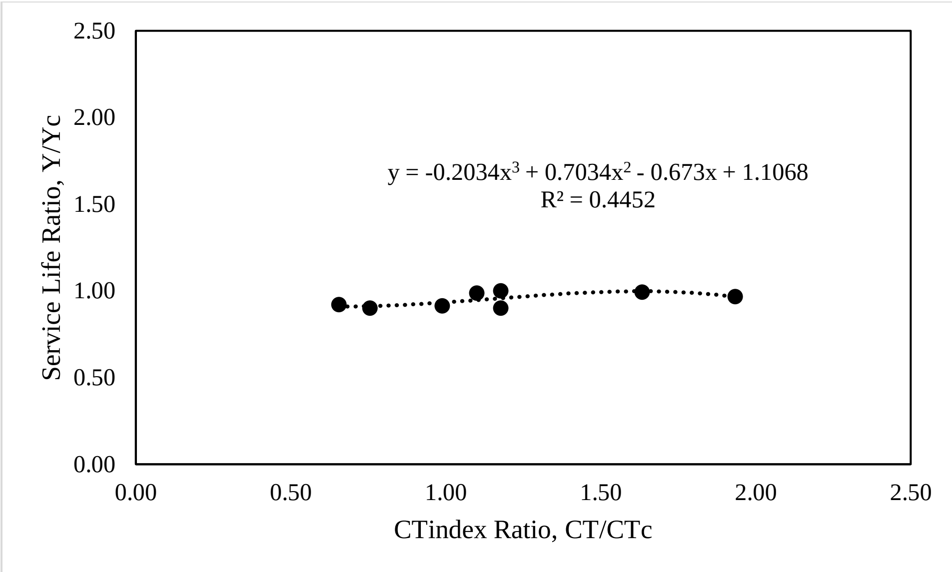


Figure 4-8 Relationship between cracking index of mixtures to fatigue service life.

The framework presented in

Figure *4-9* is suggested for predicting the field performance of asphalt mixtures under the BMD approach. After estimating the dynamic modulus from the volumetric properties of asphalt mixes, rutting, and fatigue service lives may be predicted based on the proposed empirical models. These models require calibration with local materials and conditions such as temperature. Then, the service lives can be compared with the BMD performance criteria, such as the cracking index for fatigue and HT-IDT for rutting (as selected for Maryland). To be noticed that further calibration of the service prediction models will be required with additional materials and mixture parameters (such as, for example, binder type), asphalt layer thickness, and other key parameters.

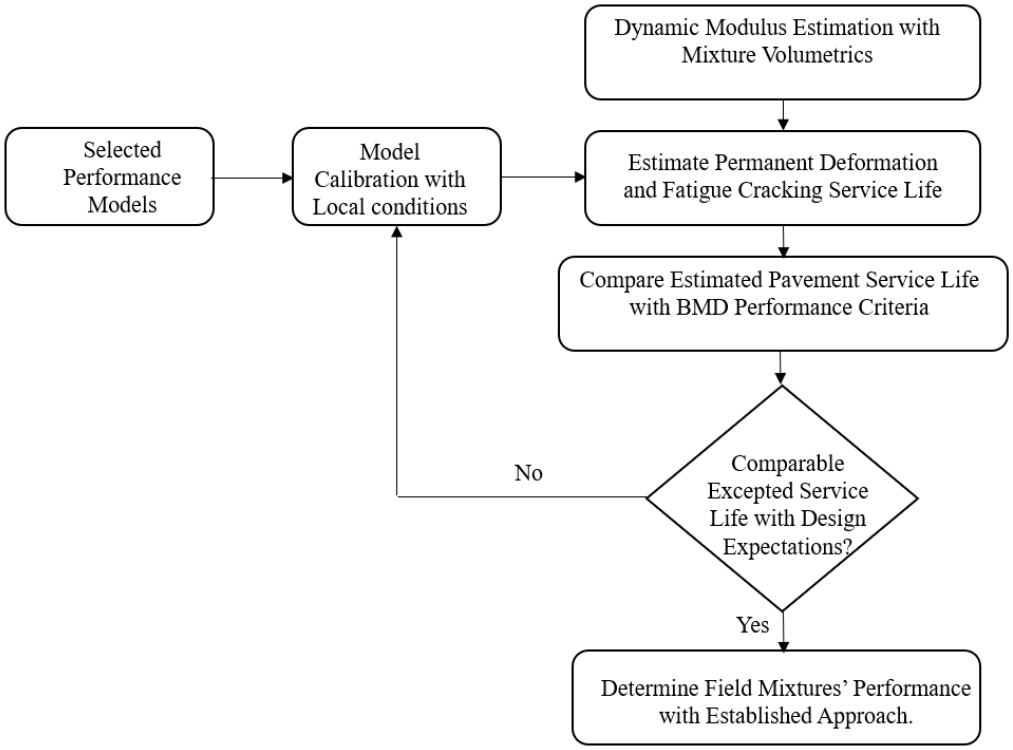


Figure 4-9 Approach to predict field performance of asphalt mixtures within BMD.

4.6. Conclusion

Predicting pavement performance is a vital part of designing a durable pavement structure. Agencies have been concerned with the performance of pavements, especially flexible pavements. The key concern is related to the current practice of designing and producing asphalt mixtures based primarily on volumetric properties. The asphalt industry and state agencies are moving towards implementing performance-based asphalt mix design methods (such as BMD). While such efforts are gaining traction across the USA, a significant limitation is predicting field performance. Thus, this study proposed a framework for predicting field performance for MD mixtures. The proposed framework can be easily adaptable for asphalt mixture performance prediction elsewhere. The permanent deformation and fatigue cracking prediction models estimate the service life of the MD mixtures around the design life target of 15 years. The permanent deformation service life for the mixes ranged from 14.7 years to 18.2 years, and for fatigue life, 9.9 years to 21.2 years. A significant component of such assessment is related to identifying the acceptable value of

E* for MD mixtures. This is based on historical data E* from laboratory tests and pertinent assessments of mixture performance testing. The criteria value for intermediate temperature (25 °C) was identified in this study at 4137 MPa, and for high temperature (43 °C), a value of 1786 MPa.

The relationship between predicted service life (for both permanent deformation and fatigue cracking) and the BMD testing results can be established. Such relationships can assess how BMD design mixtures are expected to perform in the field by identifying the expected pavement service life since production.

Chapter 5 : Incorporating Ultrasonic Pulse Velocity in Quality Assurance of Asphalt Mixtures

5.1. Introduction

In recent years performance evaluation has been given precedence for the design of asphalt mixtures by highway agencies (Bennert et al., 2018; Diefenderfer and Bowers, 2019; Akhter and Goulias, 2021). As mentioned earlier in the dissertation, in asphalt mix design, a new approach, Balanced Mix Design (BMD), is currently being explored (Ashchenbrener, 2016; hall, 2016; Buchanan, 2017). BMD focuses on the performance assessment of mixtures regarding predicted distresses such as fatigue cracking and permanent deformation. The approach is explored by agencies to be integrated into the design of the mixtures, as well as the Quality Assurance (QA), especially in the Quality Control phase (Hajj et al., 2021a) and/or the independent assurance testing by agencies phase (Hajj et al. 2021b) of the mixtures. For this purpose, alternative tests are explored by highway agencies and include the Asphalt Pavement Analyzer (APA), the Hamburg Wheel Test Tracker (HWTT), the Flow Number (FN), the Indirect Tensile Test (IDT) for rutting assessment, the Flexural bending beam fatigue test, the Illinois flexibility index test (I-FIT), the semi-circular bending (SCB) test, the Texas Overlay test (OT) and the IDEAL-CT for fatigue cracking. Besides permanent deformation (i.e., rutting) and fatigue cracking, moisture damage evaluation is performed with the tensile strength ratio (TSR) or the HWTT for durability assessment. All previously mentioned tests are destructive in nature, where test specimens must go through crushing or bending. Besides, these test methods produce large variability (sample to sample, mixture to mixture, laboratory to laboratory) in test results. Thus, these testing procedures require the production of a large number of samples preparation, which is time-consuming and labor-intensive. On the other hand, non-destructive tests (NDT) can be used on specimens again and again without demolishing them. Research (Goulias, 2019) showed that NDTs are highly repeatable, fast, accurate, and reliable (Saremi et al., 20202; Goulias, 2019). So, incorporating a non-destructive test (NDT) in the quality assessment and, thus, the performance criteria of BMD will provide added value.

Various non-destructive tests are now being explored to assess the mechanical properties of highway materials (Saremi et al., 2023; Goulias, 2019). Some examples of NDT methods for assessing highway materials and pavements include Falling Weight Deflectometer (FWD); Ground Penetration Radar (GPR); Resonant frequency methods (RTG); Ultrasonic Pulse Velocity (UPV); Spectral Analysis of Surface Waves (SASW); Infrared thermography or image analysis (Celaya et al., 2006; Celaya and Nazarian, 2007; Yehia et al., 2007; Tarefder et al., 2013; Lin et al., 2015&2016; Joshaghani, 2019; Goulias et al., 2020; Gagarin et al., 2020). While several of these NDTs are used in in-situ QA assessment of pavements, some NDTs, such as UPV, RTG, and image analysis, are being explored in the characterization of materials in laboratory settings. Researchers have explored the use of UPV in laboratory settings to characterize asphalt mixtures besides other materials, concrete or rock. For example, Tavassoti-Kheiry et al. (Tavassoti-Kheiry et al. 2017) have coupled the UPV test with the Dynamic Modulus (DM) testing of asphalt mixtures to investigate the validity of the modulus assumptions in AASHTO TP 79-09 and assess the prediction of master curves. Jimoh et al. (Jimoh et al. 2015) computed the resilient modulus of asphalt mixes using UPV and compared the results with IDT. Pan et al. (Pan et al., 2019) examined this NDT method to assess freeze-thaw effects on asphalt mixtures in lab-produced specimens.

The main purpose of this study was to assess UPV for potential adoption in the BMD and QA process and, thus, complement destructive tests, such as IDT. The study was carried out with the scope of assessing the sensitivity of the UPV to the mixture properties (such as densities) and testing conditions. Also, the ability of this NDT to be related to and evaluate the performance characteristics of asphalt mixtures, such as stiffness properties and moisture susceptibility, was of interest as well. A framework has been suggested for incorporating UPV in the BMD method in the performance evaluation of asphalt mixtures.

5.2. Literature Review

5.2.1 Ultrasonic Pulse Velocity (UPV)

The Ultrasonic Pulse Velocity method uses high-frequency (greater than 20 kHz) acoustic waves to excite the particles in the direction of propagation. In an isotropic elastic medium, the wave's velocity depends on the materials' mechanical properties. Although asphalt is a viscoelastic material, UPV responded well to the dynamic elastic properties of asphalt (Birgission et al., 2003). This assumption is valid since, at low strain levels, the deformation and the corresponding strain are very small (Arbani et al., 2009). The compressive wave velocity (the fastest wave among the ultrasonic pulses) can be expressed as a function of the mechanical properties of asphalt mixtures (Norambuena-Contreras et al., 2010; Nazarian et al., 2002).

$$V_p = \sqrt{\frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)}} \qquad (5\text{-}1)$$

Where, V_p It is the compressive wave velocity, E is the dynamic elastic modulus, ν is the Poisson's ratio, and ρ is the density of the mix. A set of transducers are used to transmit and receive the ultrasonic pulses propagated through the medium. The transducers contain piezoelectric crystals, which generate stress waves during the test and travel through the medium. The receiver transducer captures the stress wave. Meanwhile, the oscilloscope captures the mechanical energy due to this wave propagation transformed into electrical energy as transmit time in microseconds. The velocity of the pulses estimated from this transmit time, t, is equal to:

$$V_p = h/t \qquad (5\text{-}2)$$

Where h is the thickness of the specimen when the transducers are arranged in the opposite face of the specimen. From Equations (5-1) and (5-2), the dynamic elastic modulus of asphalt mixes can be derived (Norambuena-Contreras et al., 2010),

$$E = 1.27 \ x \ 10^9 \frac{Mh(1+\nu)(1-2\nu)}{d^2t^2 \ (1-\nu)} \qquad (5\text{-}3)$$

Where E is the dynamic Modulus in MPa, M is the weight of the asphalt sample in kg, h is the height of the asphalt sample in mm, d is the diameter of the sample in mm, t is the propagation time of the pulse in µs, and ν is the Poisson's ratio which depends on the testing temperature and humidity of the sample and usually ranges from the 0.3 to 0.35 for asphalt mixtures.

5.2.2 Indirect Tensile Test (IDT)

IDT has been used for evaluating asphalt mixture stiffness, rutting, fatigue cracking potential, and moisture damage. The load is applied in the diametrical plane of the asphalt sample, and the maximum load is recorded until sample failure. The tensile strength is calculated based on the maximum load, P, sample diameter, D, and thickness, t:

$$S_t = \frac{2P}{\pi D t} \tag{5-4}$$

Moisture susceptibility of asphalt mixtures is also determined using IDT by producing two sets of samples: one set of unconditioned samples and a second one conditioned at 60 °C for 24 hours in a water bath to simulate moisture damage. The Tensile Strength Ratio (TSR) is then calculated from both test results:

$$TSR = \frac{S_{t,wet}}{S_{t,dry}}\ x\ 100 \tag{5-5}$$

where, $S_{t,wet}$, is the indirect tensile strength of the wet (conditioned) samples and $S_{t,dry}$, is the indirect tensile strength of the dry (unconditioned) samples. According to AASHTO T283, the acceptance of an asphalt mixture in regard to moisture susceptibility is considered when TSR is higher than or equal to 80%.

5.2.3. Past Studies on UPV

Many researchers have explored UPV in the characterization of asphalt mixture properties and performance. This section presents an exhaustive literature review to understand the behavior of UPV in asphalt mixes. UPV has been explored in determining the dynamic elastic properties of asphalt mixtures by relating ultrasonic pulses to the mechanical properties of the mixes. Medina et al. (2018) have used UPV to evaluate the AC modulus for possible incorporation in the AASHTOWare pavement ME design. An attempt was made to combine the UPV modulus prediction with the Witczak and Hirsch models to improve modulus estimates at low frequencies. The scope of that study was limited to the assessment of the dynamic modulus, DM, as input to AASHTOWare pavement ME design and did not provide a comparison of the complex modulus with other testing methods. Tavassoti et al. (2017) studied the application of UPV to define the DM master curve accurately by combing UPV results with the traditional destructive DM tests. The study concludes that the Hirsch model can underpredict the maximum limiting modulus. Birgisson et al. (2003) evaluated the UPV method to monitor the change in the integrity of asphalt mixtures due to the moisture condition. In that study, both P-wave and S-wave were collected to estimate the modulus and assess the effects of moisture conditioning. The study provided a good agreement of UPV to mixture density and compaction level. It concluded that UPV is sensitive to the pore water effect after moisture conditioning of the specimen. The UPV response is more sensitive for the high absorptive aggregates where a high amount of water is absorbed by aggregates (Dave et al., 2018). Tigdemir et al. (2004) estimated the fatigue life of asphalt mixtures using UPV. The fatigue life was initially modeled with the mixture and testing parameters from the repeated IDT test. Then additional parameters, such as seismic modulus, pulse velocity, and shear strain, were added to the model. Several studies concluded that pulse velocity is sensitive to key asphalt mixtures properties, such as binder, air voids, and aggregate gradation. Some of the findings from past research are summarized in Table 5-1.

Table 5-1 Ultrasonic Pulse Velocity sensitivity to different mixture and testing parameters

Parameters	**UPV Response**	**Reference**
Air Void Content	Pulse velocity decrease with an increase in air voids in compacted samples.	(Pan et al., 2019)
Filler Content	Increase in pulse velocity due to increase in filler content.	(Arbani et al., 2009)
Binder Content	With the increase of binder content, pulse velocity increases till optimum binder content and then decreases.	(Arbani et al., 2009)
Temperature	Decrease in UPV with the increase in temperature	(Biligiri et al., 2009; larcher et al., 2015)

5.3. Experimental Plan

Experimentation has been carried out in two phases for this study. In the first phase, UPV was tested for sensitivity concerning the volumetric properties of asphalt mixtures and testing conditions. Based on past studies, UPV has been tested on specimens compacted from plant-produced mixtures designed solely based on the volumetric approach. In the second phase, the experimentation was designed to focus on the potential incorporation of UPV in BMD and the QA process by assessing the methods regarding stiffness and moisture susceptibility. For this purpose, mixtures with different binder content were designed in the laboratory and tested with UPV and traditional IDT testing. Details on the experimentation were provided in an earlier section under "Mixtures and Samples Characteristics, Data Collection and Results."

5.4. Response of UPV To Mixture Properties and Testing Conditions

The sensitivity of UPV to mixture properties and testing conditions was examined with a select number of asphalt mixtures. The results are presented next.

5.4.1 Asphalt Mixtures and Sample

Dense-graded mixtures were collected from five different construction projects in Maryland. Both plant and field mixtures (behind the paver) were used. The characteristics of the mixes are presented in Table 5-2. All five mixes have different aggregate gradation (Mix Band), binder content, and recycled asphalt pavement content (RAP). Increasing RAP content stiffens mixture properties, making them crack-prone and susceptible. Among the five mixes, Mix 1 has the highest amount of RAP content of 45%; thus, a stiffer binder was used. For the remaining four warm mix mixtures, a PG64-22 was used.

Table 5-2 Mixture characteristics of the asphalt mixtures

Mixtures	**Mix Method**	**Mix Band**	**RAP%**	**Traffic Level**	**Asphalt Type**	**Design Binder Content (%)**
Mix 1	Hot mix	9.5 mm	45%RAP	0.3 to <3	58S-28(58-28)	5
Mix 2	Warm mix	12.5mm	27% RAP	0.3 to <3	64S-22(64-22)	4.8
Mix 3	Warm mix	9.5 mm	29% RAP	0.3 to <3	64S-22(64-22)	5
Mix 4	Warm mix	9.5 mm	32%RAP	0.3 to <3	64S-22(64-22)	5.2
Mix 5	Warm mix	12.5 mm	10%RAP	0.3 to <3	64S-22(64-22)	4.7

The volumetric properties of the mixtures are shown in Table 5-3. For each construction project, the volumetrics of plant and field mixes and the Job Mix Formula (JMF) are presented. The mixtures were prepared according to the AASHTO M323 specification. Densities and binder content are presented in the Table along with maximum specific gravities, G_{mm}, total void content, VTM, voids in the mineral aggregate, VMA, and voids filled with asphalt, VFA. For example, the design VTM is 4% for dense graded mixtures. VMA has a minimum of 15% for the 9.5 mm aggregate band and a minimum of 14% VMA for the 12.5mm aggregate band. The VFA should range between 65 to 78%. According to Maryland specifications, a tolerance of ±1.2% applies to VMA, and 4±1.2% for VTM is used, while VFA should meet the AASHTO M323 specifications. Among the mixtures, the plant mixes of Mix 1, 3, and 5 have VFA outside the range of 65 to 78%, and thus adjusted field mixtures were developed.

Table 5-3 Volumetrics of the mixtures

Mixtures	**Population**	**G_{mm}**	**VTM (%)**	**VMA (%)**	**VFA (%)**	**P_{be} (%)**
Mix 1	JMF	2.609	4.0	15.6	74.4	4.80
	Plant	2.595	2.8	15.5	82.0	5.19
	Field	2.598	3.2	15.7	79.5	5.14
Mix 2	JMF	2.529	4.0	14.9	73.1	4.60
	Plant	2.533	3.9	14.6	73.3	4.72
	Field	2.537	4.8	15.5	68.8	4.94
Mix 3	JMF	2.498	4.0	15.1	73.7	4.80
	Plant	2.509	3.0	14.2	79.1	5.41
	Field	2.507	3.6	14.7	75.7	5.34
Mix 4	JMF	2.497	4.0	15.1	73.5	4.80
	Plant	2.478	4.3	16.2	73.5	5.43
	Field	2.500	3.4	14.3	76.2	5.04
Mix 5	JMF	2.600	4.0	14.2	72.0	4.20
	Plant	2.598	2.6	13.4	80.6	5.11
	Field	2.594	4.5	15.1	70.6	5.08

The aggregate gradations for the 9.5mm and 12.5 mm nominal aggregate size, NMAS, mixtures are presented in Table 5-4. These represent the most common blends used in Maryland surface mixes. The noticeable difference between plant and field mixtures is observed in the fines (sieve size 0.075 mm).

Table 5-4 Aggregate gradation of the mixes

Sieve, mm	**Mix 1**		**Mix 2**		**Mix 3**		**Mix 4**		**Mix 5**	
	Plant	**Field**	**Plant**	**Field**	**Plant**	**Field**	**Plant**	**Field**	**Plant**	**Field**
25	100	100	100	100	100	100	100	100	100	100
19	100	100	100	100	100	100	100	100	100	100
12.5	100	100	96	98	100	100	100	100	97.1	97.2
9.5	90	92	87	88	95.4	96.1	95	95	90	93
4.75	54	55	56	57	57.7	63	65	65	57.8	57.9
2.36	33	34	35	35	39.2	43	40	40	35.8	35
1.18	24	23	24	23	30	32.5	28	29	25	24.2
0.6	18	17	16	15	23.9	25.5	20	21	19.5	18.8
0.3	14	13	12	11	13.9	14.4	12	13	12.8	12.7
0.15	10	10	9	8	7.4	7.6	8	9	9	8.9
0.075	7.3	7.4	7.2	6.1	5	5.3	5.5	6.1	7.2	6.9

Two samples were compacted for each mixture using the Superpave gyratory compactor. The first set (bigger samples) is for high-temperature IDT testing with 150 mm x 95mm dimensions. The second set (smaller samples) for intermediate temperature IDT testing was with dimensions of 150mm x 62 mm. This second set had smaller dimensions since companion samples were also used for Cracking Index (According to ASTM D8225) testing as part of a BMD study that is beyond the scope of the analysis herein. At least three samples were compacted for each set for the five mixtures' plant and field mixes. The compacted air void content was kept at 7±0.2%, reflecting design requirements according to the BMD study.

5.4.2 Data Collection

The Ultrasonic Pulse Velocity (UPV) test was performed according to the *ASTM C597 "Standard Test Method for Pulse Velocity Through Concrete*." The test equipment consists of two transducers (one transmitter and one receiver) and a high-resolution data acquisition unit. The piezoelectric transducers used in this study had a 50 mm diameter and 54 kHz center frequency, Figure 5-1. Petroleum jelly was used to establish good contact between the transducers to the sample surface and to avoid signal loss due to surface voids. The test was performed by placing the transducers on two opposite surfaces of each sample. While placing the transducers, the pulse transmission time was collected at the center of both surfaces. The testing configuration is presented in following Figure 5-1.

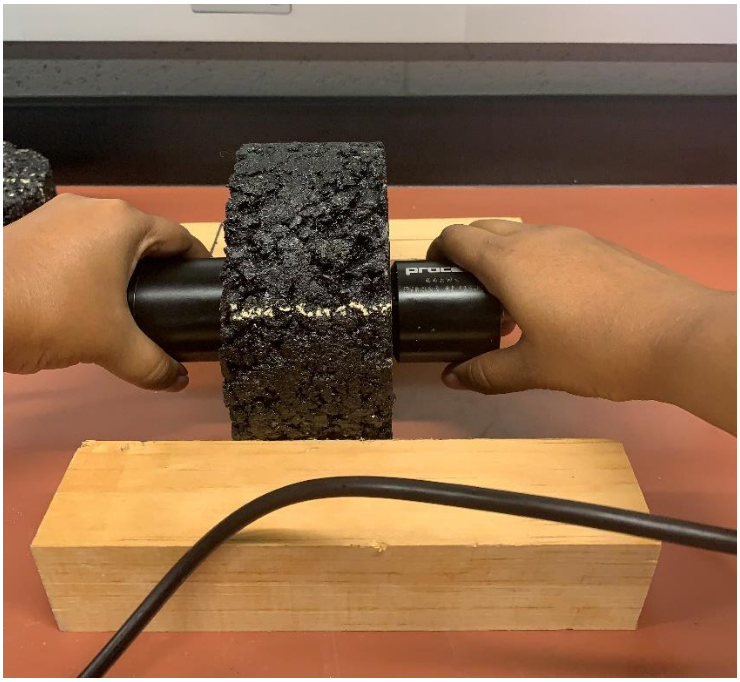


Figure 5-1 UPV testing setup

UPV readings were taken at the midpoint of the samples to measure the sensitivity of UPV to moisture and temperature. The larger samples were cured in a water bath for 2 hours at 43°C, while the smaller samples were conditioned for 2 hours at 25°C. Transmission times were recorded before and after the curing condition for the IDT testing.

Three replicate measurements on each sample were collected to assess UPV repeatability. Suppose the measurements varied by more than 10% from the average. Three additional readings were collected. The repeatability of transition time for measurements on the same sample was between 0.2 to 1.2% (coefficient of variation, COV), and between samples from the same mixture was 1% to 4%. This level of repeatability agrees with the findings of previous studies in asphalt mixtures (Celaya and Nazarian, 2007; Lin et al., 2015,2016) and are within the commonly acceptable range of ± 3% reported when using NDTs in quality assurance of highway materials (Akhter and Goulias, 2021; Celaya and Nazarian, 2007).

5.4.3 Data analysis and Results

5.4.3.1 Sensitivity to Testing Conditions

Figure 5-2 provides the average transition time results (average of three samples, n=3) for the dry (unconditioned) and wet-conditioned samples. Lower propagation times are observed for wet-conditioned samples since ultrasonic pulses travel faster in water-filled voids than in air voids. The short-term moisture conditioning on mixture degradation was minimal since it did not significantly affect UPV. This effect is in agreement with past studies that examined the moisture effect in long-term conditioning, such as 24 hours (Birgisson et al., 2003). The error bars represent the one standard deviation testing variability.

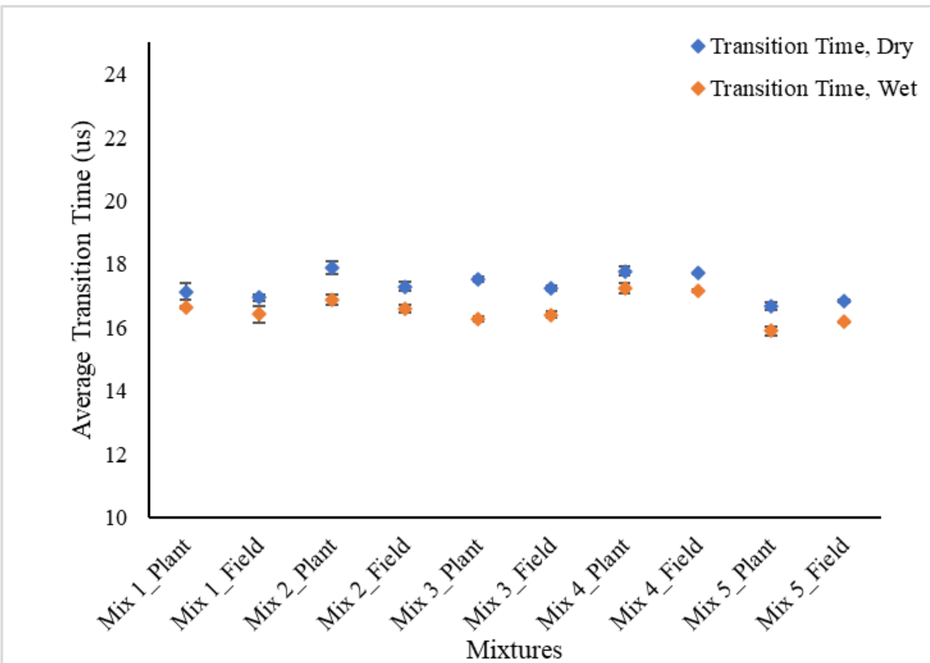


Figure 5-2 Effect of moisture conditioning on transition time of ultrasonic pulses (n=3)

Previous studies (Tarefder et al.,2013; Lin et al., 2015) showed that temperature has a clear effect on UPV. Figure 5-3 presents the average transition time of pulses at 25°C and 43°C. The trend shows approximately 3-microsecond differences in transmission time in these two temperatures. At high temperatures, ultrasonic pulses traveled slower in asphalt mixtures than at room temperature. When asphalt mixtures are exposed to moisture at high temperatures, mixture degradation occurs with a loss of bonding between the binder and the aggregate. Thus, the mixture loses strength and stiffness and has a lower density affecting UPV transition time.

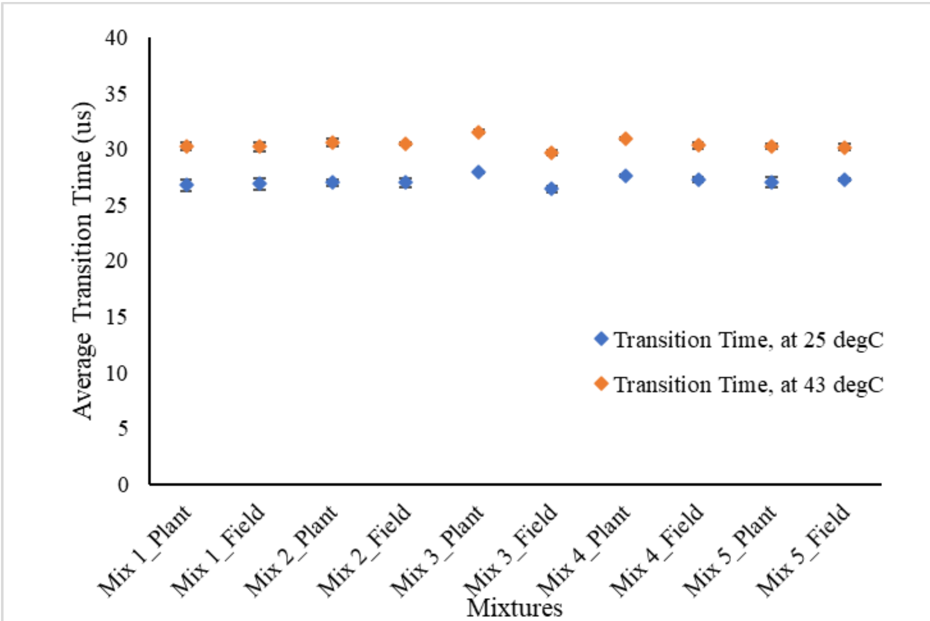


Figure 5-3 Effect of temperature on the transition time of ultrasonic pulses (n=3)

5.4.3.2 Sensitivity to Mixture Density

Current volumetric asphalt mixture design and quality control practices are based on the measurement of mixture densities in terms of air void content, specific gravities, and effective binder content. To adopt a test for quality assurance of asphalt mixtures, it is of interest to be sensitive to different density values. So, this section aims to verify the sensitivity of UPV to the densities of mixtures. Past studies showed evidence that ultrasonic pulse velocities are responsive to the density of the medium, such as concrete, asphalt, rock, etc.

Figure 5-4 shows the average ultrasonic pulse velocity in relation to mixture properties.

Figure 5-4(a) and Figure 5-4(b) provide the response of UPV to bulk specific gravity (G_{mb}) and theoretical specific gravity (G_{mm}), respectively. The higher the specific gravity, the lower the air void content, which represents a denser mixture that pulses travel through. This reflects the response of UPV from plant mix to field mixtures. For example, for Mix 1, G_{mb} changes from 2.514 to 2.522 from plant to field. UPV response is 3727 m/sec to 3778 m/sec, respectively, since the field mix is denser than the plant mix.

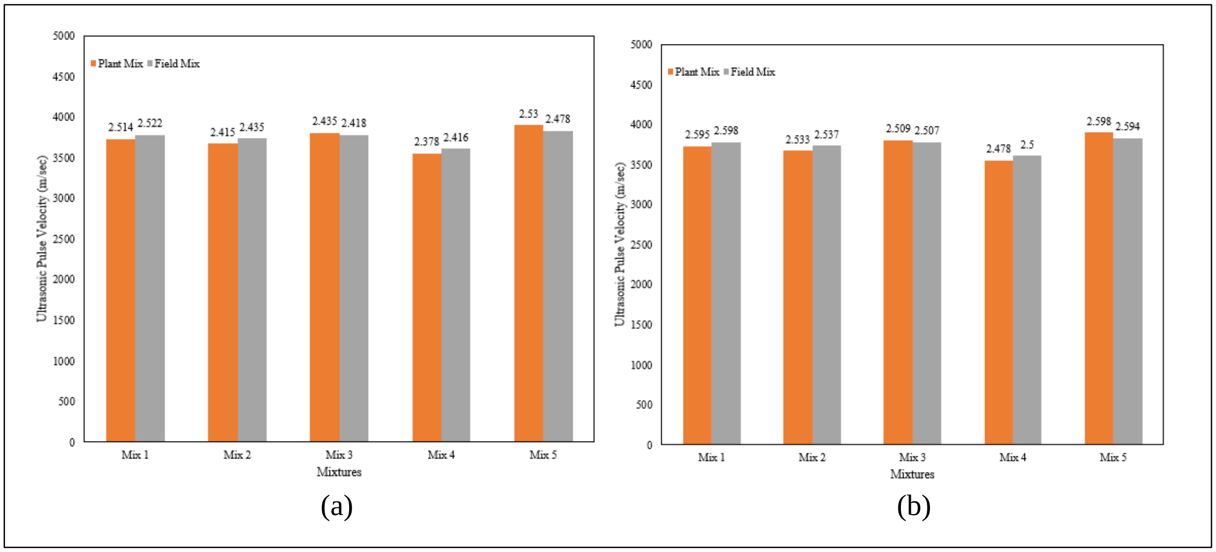


Figure 5-4 UPV response to mixture properties: a) Bulk Specific Gravity, G_{mb}; b) Theoretical Specific Gravity, G_{mm} (n=3)

5.5. Asphalt Mixture Performance Evaluation

5.5.1 Asphalt Mixtures and Samples (Mix 6)

The initial investigation included one of the most frequently used mixtures in the state of Maryland, let's say, Mix 6. This includes a binder with a performance grade of 64-22 and a dense graded aggregate gradation with a Nominal Maximum Aggregate Size (NMAS) of 12.5 mm. Figure 5-5 shows the gradation of the aggregate, which includes 5% fines.

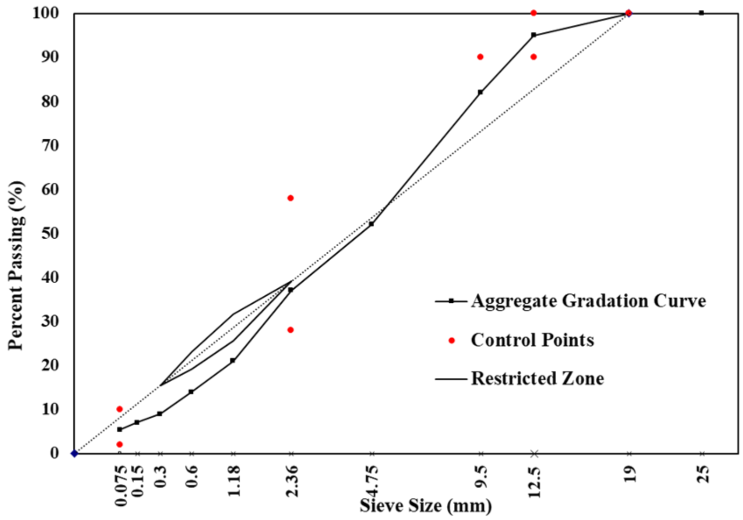


Figure 5-5 Aggregate gradation of the Mix 6

To determine the initial design binder content (DBC) with volumetrics, mixtures with four binder contents, 4.2%, 4.8%, 5.6%, and 6.2%, were prepared. The replicate samples (n=3) were compacted at each binder content using a gyratory compactor. As identified by Superpave and state requirements, the volumetric properties at 4% air void content are shown in Table 5-5 and were evaluated following AASHTO M323 "Standard specification for Superpave Volumetric Mix Design." The corresponding binder content was equal to 5.9%.

Table 5-5 Volumetric properties of designed Mix 6

Binder Content (%)	5.9
Theoretical Specific Gravity, G_{mm}	2.655
Bulk Specific Gravity, G_{mb}	2.553
VMA (%)	18.3
VFA (%)	73.2

In the next step, three mixtures were prepared at DBC±0.5% (at binder content 5.9%, 5.4%, and 6.4%) with 3 replicates for each case. The specimens were 150 mm x 95 mm. This size was selected primarily to have enough thickness for the UPV testing. All three mixtures were compacted using the gyratory compactor with a target air void of 7%, as identified in the BMD criteria. Two sets of samples were prepared to run moisture susceptibility tests: (i) unconditioned - dry samples; (ii) conditioned -wet samples (conditioned in a water bath at 60°C for 24 hours as per AASHTO T283).

5.5.2 Data Collection

UPV Data were collected, described in section 5.4.2, on both unconditioned and conditioned samples. Testing repeatability was assessed from three readings per sample. Table 5-6 for both conditioned and unconditioned samples.

Table 5-6 UPV repeatability

BC%	COV (%)
Unconditioned	
5.9	0.21
5.4	1.43
6.4	1.40
Conditioned	
5.9	0.91
5.4	0.50
6.4	0.32

5.5.3 Data analysis and results

5.5.3.1 Stiffness Assessment

As mentioned earlier, the stiffness of asphalt mixtures has been linked to permanent deformation and cracking potential (Bennert et al., 2018; Diefenderfer and Bowers, 2019; Aschenbrener, 2016). Figure 5-6 shows the summary results based on the average ultrasonic pulse velocity (n=3) values and the corresponding dynamic elastic modulus calculated from equation 3 with an assumed Poisson's ratio of 0.3. Error bars represent the one standard deviation percentile. For these mixtures, the average pulse velocity in unconditioned samples ranges from 2263 m/s to 2395 m/s. As expected, the effect of moisture exposure (i.e., moisture-conditioned samples) produced lower pulse velocity in relation to the unconditioned samples at the same binder content. The observed values for these ranged from 2225 m/s to 2297 m/s. The observed

UPV values agree with those reported in past studies with pulse velocities in relation to mixture type (Lin et al., 2015, 2016).

The tensile strength of asphalt mixtures with varying binder content was compared with the Dynamic Modulus from the pulse velocities of UPV, Figure 5-7. For these mixtures, the IDT values for the unconditioned samples ranged from 742 to 820 kPa. As expected, the effect of moisture exposure (i.e., moisture-conditioned samples) produced lower IDT values in relation to the unconditioned samples at the same binder content.

The sensitivity of UPV to binder content is shown in Figure 5-6. A change in binder content of ±0.5% produces small variations in UPV (i.e., around 50 to 100 m/s, equivalent to a change of 2 to 4% in relation to the value observed at the design binder content of 5.9%). This is also reflected in the calculated values of the dynamic modulus from equation 3 (i.e., of the order of 3 to 8%). Comparable ranges in values are observed from the IDT results in relation to the design binder content (i.e., of the order to 3 to 10%).

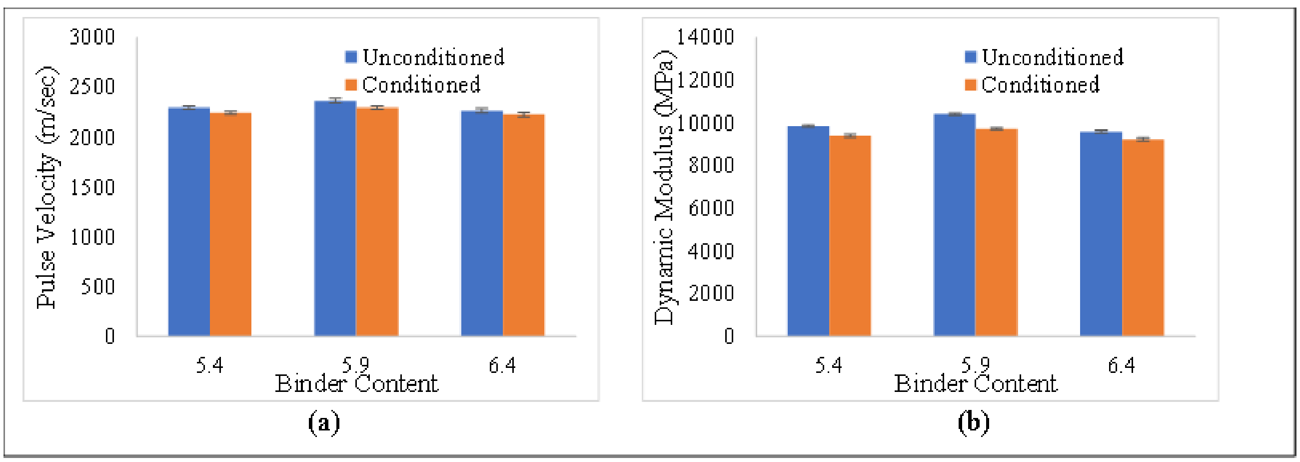


Figure 5-6 Pulse velocity and dynamic modulus with binder content. (n=3)

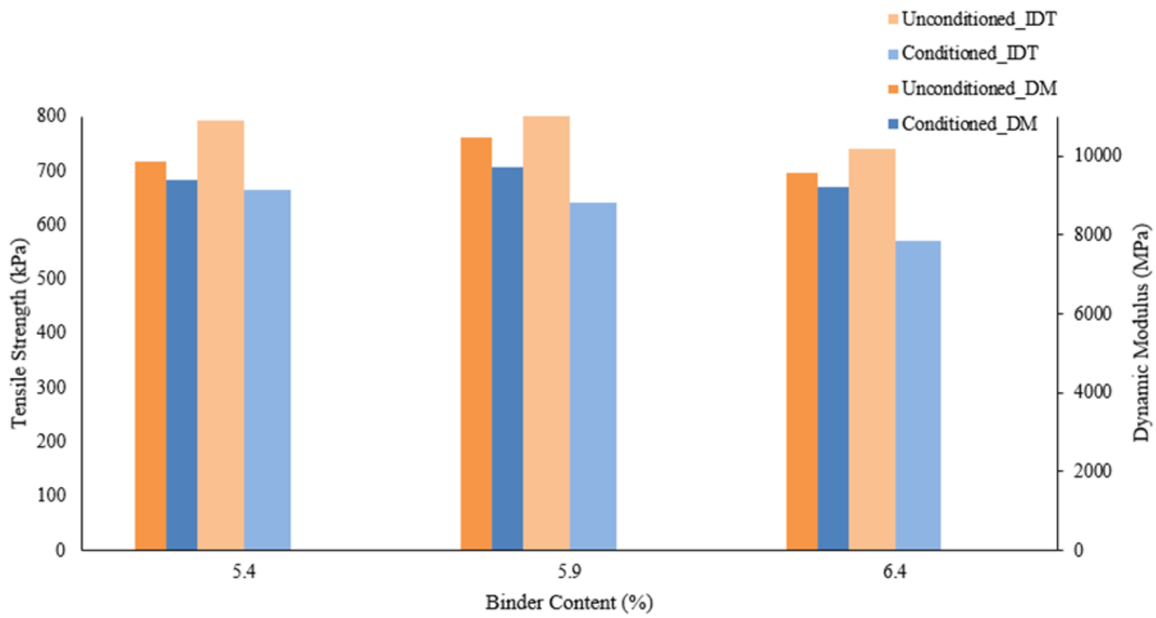


Figure 5-7 Indirect tensile strength and dynamic elastic modulus (n=3)

5.5.3.2 Moisture Damage

Moisture in asphalt mixtures causes loss of adhesion in the asphalt-aggregate interface, which leads to loss of strength and durability. As described in the previous section (subsection 5.2.2), the tensile strength ratio (TSR) is used as an indicator of moisture susceptibility. The TSR should have a minimum value of 80% for a mixture to be accepted according to AASHTO T283 and state specifications. Since the dynamic modulus calculated from UPV is related to IDT, UPV has the potential to be adopted in the moisture susceptibility assessment. Using UPV, the dynamic modulus (in MPa) was computed using equation 3. Like the definition of TSR, the reduction in dynamic modulus ratio (DM_R) due to moisture exposure can be calculated with the following equation:

$$DM_R = \frac{DM_{Wet}}{DM_{Dry}}\, x100 \qquad (5\text{-}6)$$

Where DM_{wet} and DM_{dry} represent the dynamic modulus on conditioned and unconditioned samples, respectively. Figure 5-8 presents the comparison of TSR (%) and Dynamic Modulus Ratio (%). Among the three mixtures, only the first mixture with BC of 5.4% meet the TSR criteria. The other two mixes are susceptible to moisture damage. The dynamic modulus ratio shows a higher value than TSR, but all the DM_R values agree with the TSR values. Thus, a threshold value for acceptance based on DM_R should be established. If a single DM_R the acceptance threshold value for durability is to be defined for all mixtures; as in the case of TSR, it will be necessary to examine such parameters for materials and mixtures used in a region. The established DM_R threshold can replace TSR, reducing the production of the samples, as UPV can be run on the same set of samples before and after conditioning.

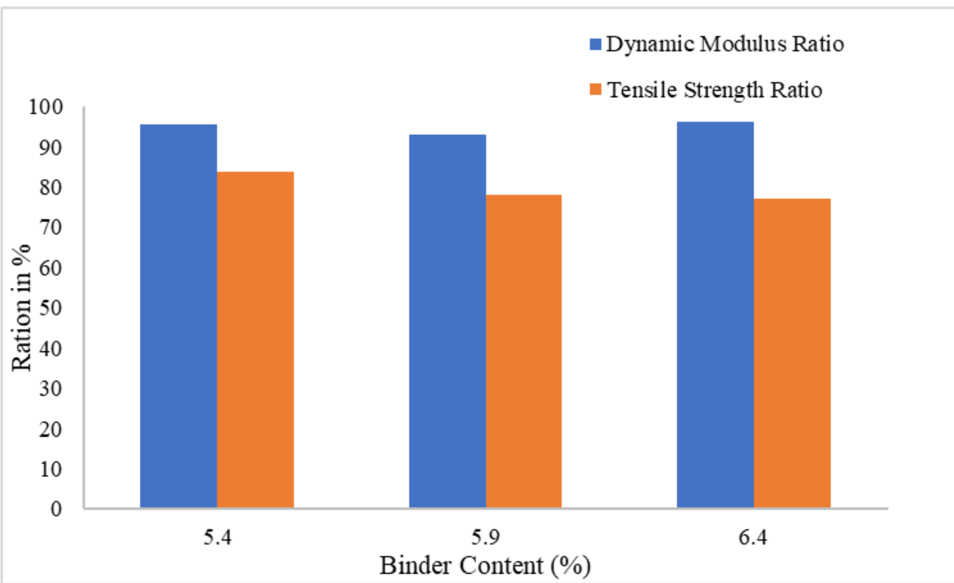


Figure 5-8 Dynamic modulus ratio and tensile strength ratio

5.6. Suggested Framework for Incorporating UPV In QA

Based on the study findings, a framework for incorporating this NDT method in the QA process of the BMD approach is suggested, Figure 5-9. After selecting the asphalt mixture components such as a binder, aggregate blends, recycled materials, and so on, the mixture will be designed according to current practice following AASHTO M323 guidelines (i.e., mixtures evaluated based on densities, effective binder content, and other volumetric properties). In the next step, mixtures will be assessed through performance testing for cracking, permanent deformation, moisture susceptibility, and stiffness. IDEAL-CT, HT-IDT and TSR have been selected for such an assessment in Maryland. In this design phase, UPV can be used for moisture susceptibility and stiffness evaluation, which will complement the performance characterization of the mix without producing extra samples. UPV can be run on the samples produced for TSR and IDT (described in subsection 5.2.2) right before destructive testing. The suggested number of samples is presented in Table 5-7. If all the volumetrics and performance criteria pass, the mixture will be accepted as the job mix formula (JMF). Otherwise, the mix will be redesigned from the volumetric design phase. The next step upon approval of the JMF is the quality assessment phase during production. In this step, the producers and highway agencies will desire faster performance evaluation. In this phase, TSR can be eventually replaced with DM_R from UPV testing. Since there are three performance testing requirements (cracking index, high-temperature tensile strength, and stiffness from dynamic modulus), the number of samples per test can be reduced to 3. Thus, the total required number of samples could be reduced to 9 instead of the 16 used in the design phase (Table 5-7).

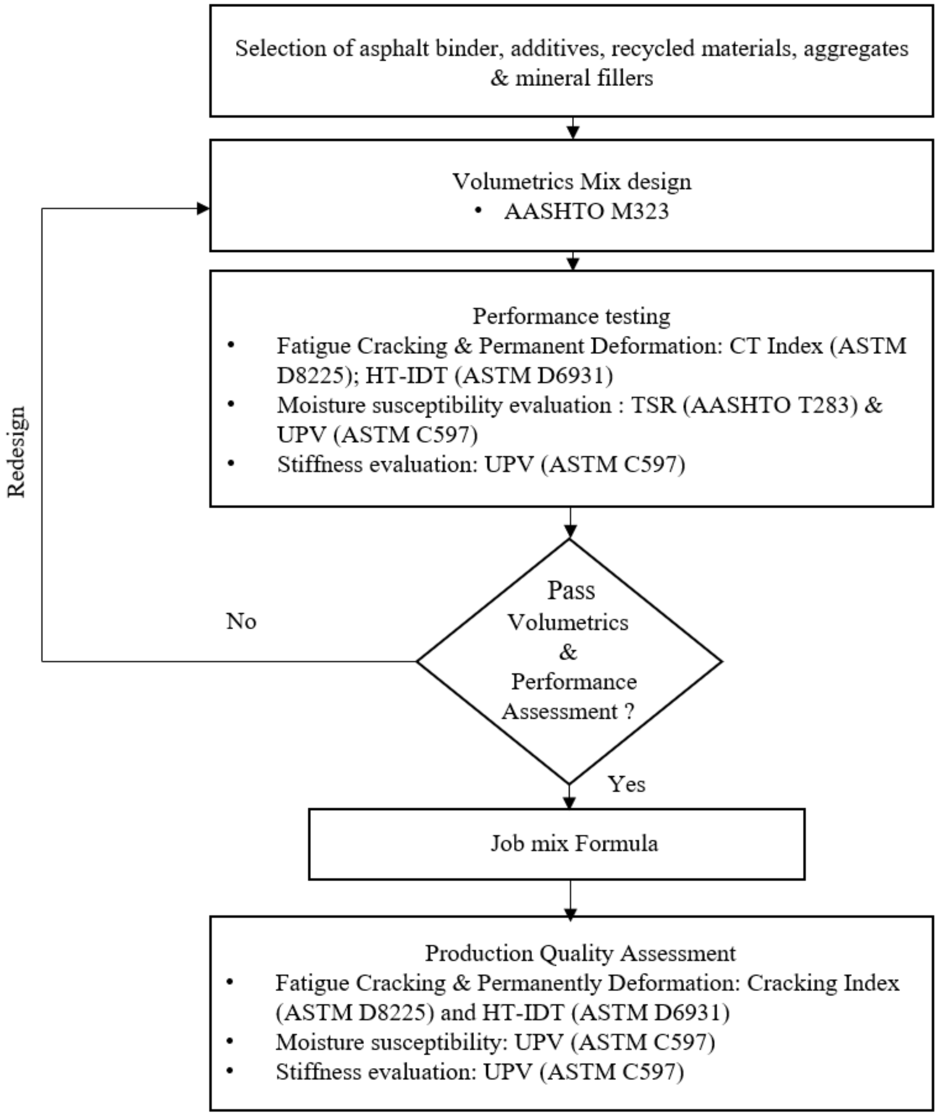


Figure 5-9 Framework to incorporate UPV in the BMD approach.

Table 5-7 Sample requirements for performance testing

	Fatigue Cracking	**Permanent deformation (Rutting)**	**TSR**	**UPV (DM$_R$ & Stiffness)**	**Total**
Design	5	5	6	6 (from TSR)	16
QA	3	3	0	3	9

5.7. Conclusion

During the design and production of asphalt mixtures assessing the effects of mixture ingredients on properties and performance is an important step in ensuring pavement longevity. Several highway agencies are now focusing on the performance characterization of asphalt mixes during design and production. Historically agencies are evaluating several tests, most of them destructive. Due to the significant benefits of NDTs, agencies are now focusing on their potential adoption in QA, mix design, and performance assessment. This study provided an initial assessment and framework for adopting UPV in a mix design and QA of asphalt mixtures. The findings of this study include:

- UPV is responsive to moisture presence. Short-term moisture conditioning (i.e., 2 hours) has minimal effect on mixture degradation; thus, no significant effect on UPV was observed.
- UPV is sensitive to testing temperature. At high temperatures, ultrasonic pulses travel slower in asphalt mixtures. This reflects the higher degree of mixture degradation at higher curing temperatures, thus reflecting longer transmission times.
- UPV relates well with mixture density. Faster propagation is observed in denser mixtures. Further assessment is needed based on representative mixtures for the specific region to account for the effects of the remaining mixture properties on UPV and the impact on the dynamic modulus.
- UPV testing has high testing repeatability within samples (0.2% to 1.2%) and between samples (1% to 4%).
- The study results indicated that stiffness from UPV compares well with IDT results.

Since this study was exploratory in nature in (i) assessing the potential use of UPV in a mix design and QA and (ii) proposing an initial framework for its potential adoption, a limited number of mixtures were used. As indicated in the various steps of the analysis, the relationships between UPV and IDT are mixture specific and, thus, should be examined and developed for the variety of asphalt mixtures used in a region. Further testing is thus needed to assess performance criteria for mixture properties and durability for various asphalt mixes in Maryland. The methodology presented herein is transferable to regions with similar materials and mixtures.

Chapter 6 : Conclusions and Recommendations

6.1 Summary of Findings

The research developed in this study provided the following findings and contributions to the state of knowledge in incorporating performance requirements in asphalt mixture design.

6.1.1 Initial Quality Acceptance Threshold for Permanent Deformation and Fatigue Cracking for BMD

- The primary source of distress experienced in Maryland asphalt pavements are permanent deformation (i.e., rutting) and fatigue cracking. Although several alternative approaches have been explored for addressing such issues, performance-based mix design needs to be addressed and implemented.
- The proposed Balanced Mix Design (BMD) approach in this study considers the design of asphalt mixtures based on volumetric analysis combined with performance testing for the first time in Maryland. This overcomes the current limitation of linking volumetrics to predicted field performance during the design phase.
- IDEAL-CT and HT-IDT, both indirect tensile strength-based test methods, provide an initial assessment of lab performance for Maryland dense-graded and gap-graded asphalt mixtures for cracking and rutting, respectively.
- A threshold analysis approach was proposed for defining acceptance values for inclusion into Maryland's revised BMD-based asphalt mixture specifications.

6.1.2 Variability Analysis of IDEAL-CT and HT-IDT Tests

- Testing and analysis on the most representative mixture used in Maryland provided comparable variability levels (i.e., in terms of COV) for the IDEAL-CT and HT-IDT. These ranged from 10.1% to 18.4% and 17% to 20.4%, respectively, for inter-laboratory conditions.

- The main sources of variability in IDEAL-CT and HT-IDT were related to (i) sample preparation and conditioning (particularly for HT-IDT due to the high temperature involved) and (ii) testing instrumentation within and between labs (i.e., deformation rate).
- Testing repeatability is affected by the volumetric properties of asphalt mixtures. And particularly RAP percentage and binder content. This indicated that softer mixtures may be associated with larger testing variability. Nevertheless, such effects should be further assessed with additional testing.
- A correction approach for IDEAL-CT and HT-IDT testing results was proposed in order to account for the effects of various laboratories and materials testing systems used in the design of asphalt mixtures. This is particularly critical since both agency and contractors will use their own laboratories for designing and assessing such mixtures.

6.1.3 Performance Prediction of Asphalt Mixtures

- Beyond the laboratory performance assessment of Maryland mixtures using the BMD approach and lab testing procedures identified in this dissertation, this study aimed to predict field performance. As indicated earlier, the proposed methodology based on well-accepted fatigue cracking and permanent deformation prediction models were used with the Maryland mixtures in order to predict service life. In this regard, mixture properties were linked to the dynamic modulus of asphalt mixtures. Criteria values for dynamic modulus for asphalt mixtures in Maryland, both at intermediate (25°C) and high temperature (43°C), were identified based on the excepted pavement life expectancy. This provided E* values of 4137 MPa for intermediate temperature (25°C) and 1786 MPa for high temperature (43°C).
- Based on the dynamic modulus-based performance prediction approach suggested in this study, it was concluded that current Maryland asphalt mixtures are associated with service life expectancy ranging from about 8.8 to 21.2 years in regard to fatigue cracking and/or permanent deformation.

6.1.4 Incorporating Ultrasonic Pulse Velocity in Quality Assurance of Asphalt Mixtures

This study explored whether UPV is responsive to mixture properties and moisture exposure in order to be used in the QA of the BMD approach. The following conclusions were obtained, and the suggested approach for using such NDT in QA was proposed.

- UPV is responsive to mixture properties and moisture exposure. Short-term moisture conditioning (i.e., 2 hours) has minimal effect on mixture degradation; thus, no significant effect on UPV was observed.
- UPV is sensitive to testing temperature. At high temperatures, ultrasonic pulses travel slower in asphalt mixtures than at room temperature. This reflects the higher degree of mixture degradation at higher temperatures, thus reflecting longer transmission times.
- UPV relates well with mixture density. Faster propagation is observed in denser mixtures.
- UPV testing has high testing repeatability within samples (0.2% to 1.2%) and between samples (1% to 4%).
- The study results indicated that stiffness from UPV compares well with IDT results.

6.2 Recommendations and Future Work

The analysis and findings of this study need to be further extended to address the following aspects:

- *Aging Effects.* In this study, both cracking and rutting acceptance criteria have been identified for short-term aging (i.e., 2 hours of conditioning). The effects of long-term exposure reflecting field conditions could be considered in such assessment in order to account for longer-term environmental exposure.
- *Expand the study on additional Gap Graded Mixtures.* While a finite number of gap-graded mixtures are used in Maryland, additional mixes should be considered to further validate the acceptance threshold for these. These mixtures contain a high amount of binder and better aggregate skeleton affecting to a different degree performance than dense-graded mixtures.

- *Validate Field Performance Prediction Models.* The proposed methodology for predicting field performance from laboratory results was based on well-accepted models in the research community. Such models were developed from various mixtures in the US and used in this study with Maryland materials and mixtures. Nevertheless, the model predictions should be verified with the actual field performance of asphalt mixtures and pavements designed and built with the BMD approach. Similar considerations should be used when the proposed approach herein is used with mixtures and pavements in other states.
- *Field Core Testing.* Further validation of the performance models and their predictions could be achieved by testing cores from pavement sites to capture field performance in the short and long term. This assessment will directly link the properties of designed mixtures and actual performance accounting for traffic and climate exposure.
- *Implementation of UPV in QA.* This study examined the possibility of using UPV during design and QA activities under the BMD approach. Since the findings were based on a limited number of mixtures, further assessment should incorporate additional mixtures.

Appendix A

This section provides the aggregate gradations of the asphalt mixtures used in the study of Chapter 2. Aggregate is a vital part of the asphalt mixtures, constituting 96 % to 86%. The amount of fines, coarse aggregate, and fine aggregate in the mixtures affects the performance of the mixtures. For instance, mixtures with high fines are more prone to permanent deformation than mixtures with fewer fines. Figures A1 to A13 show aggregate gradations plots for three populations (Plant, Modified Plant, and Field) along with JMF. All the gradation charts have been prepared according to AASHTO M323.

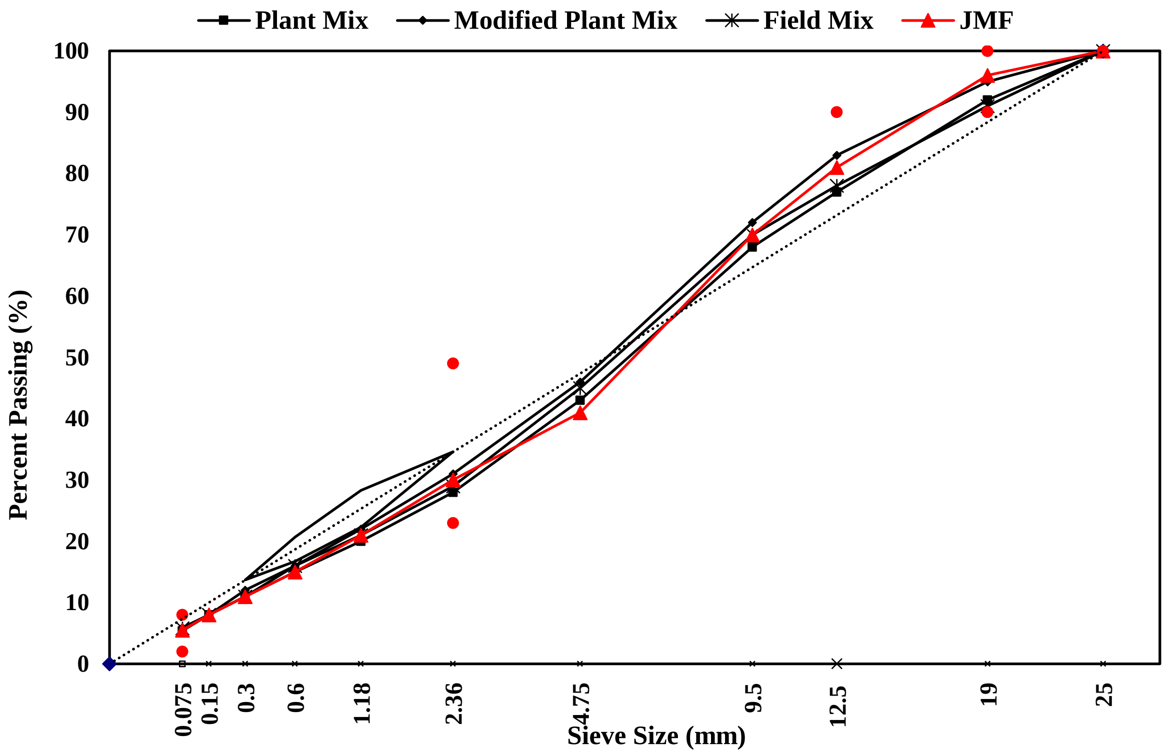


Figure A-1 Aggregate gradation curves for Mix DG1

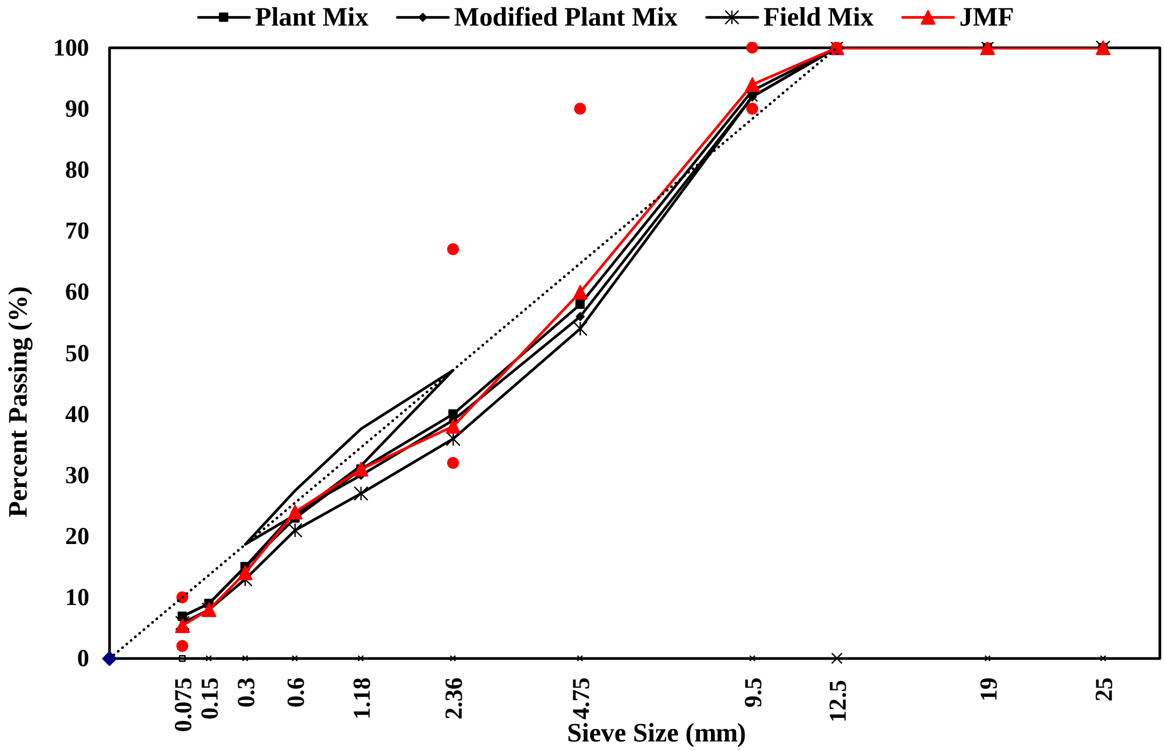


Figure A-2 Aggregate gradation curves for Mix DG2

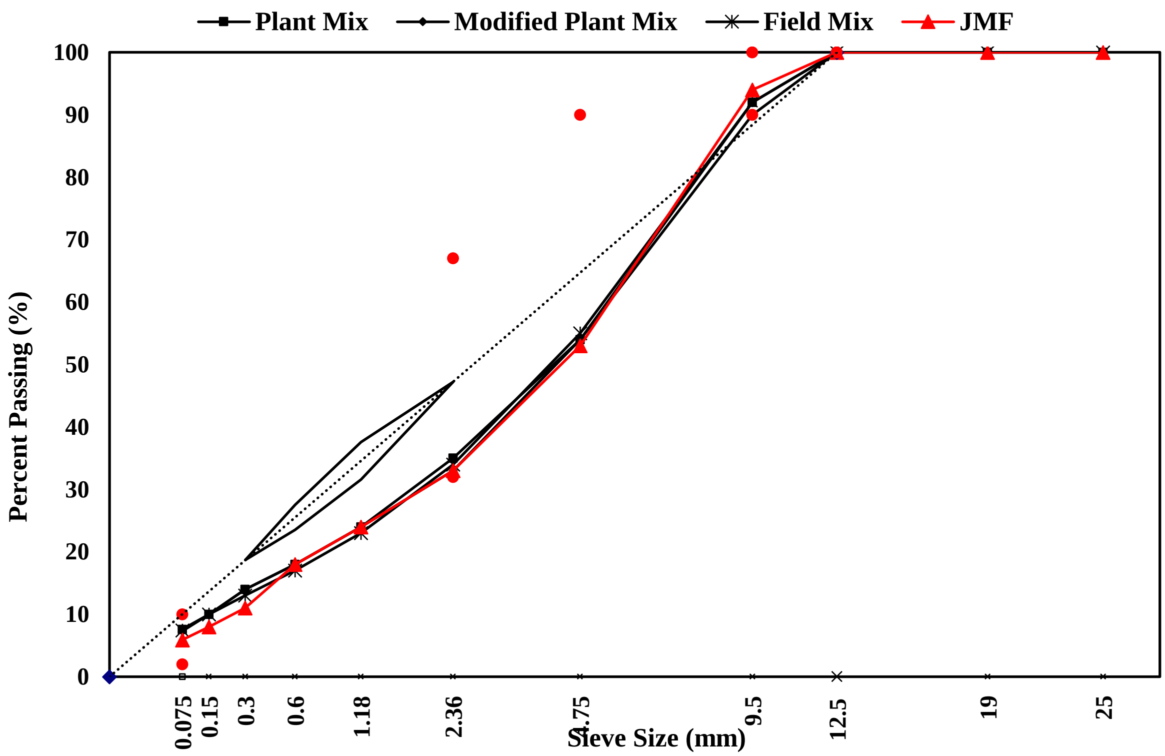


Figure A-3 Aggregate gradation curves for Mix DG3

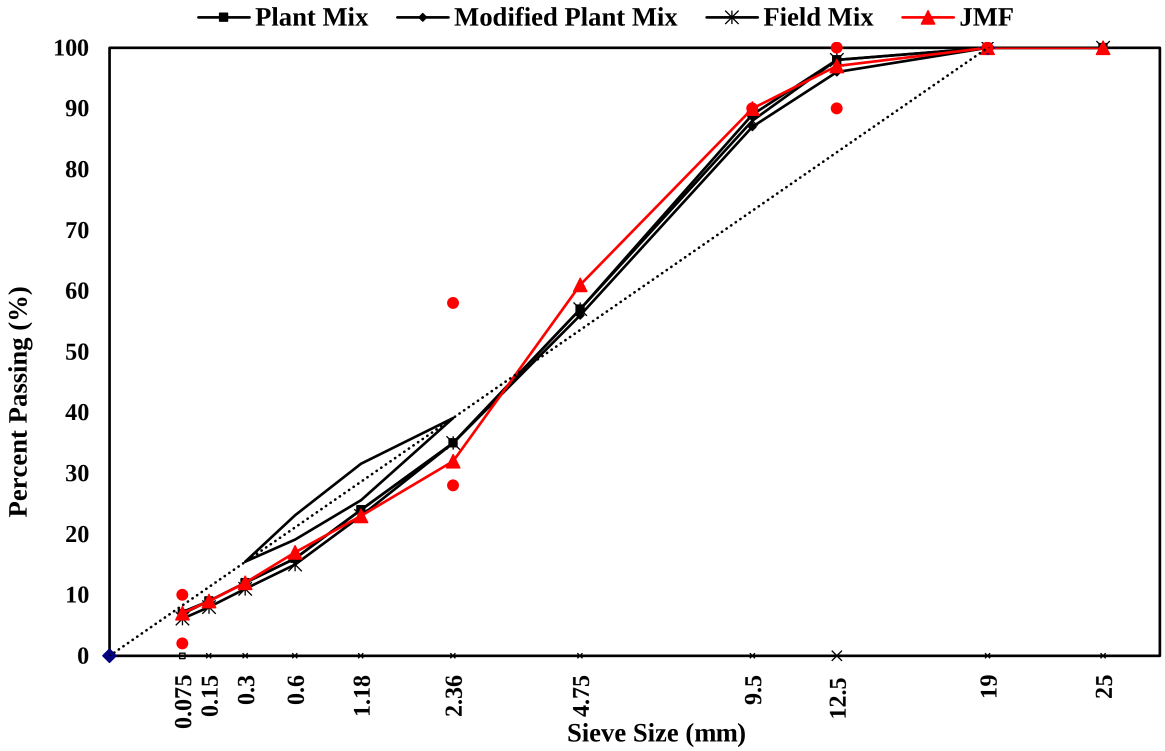


Figure A-4 Aggregate gradation curves for Mix DG4

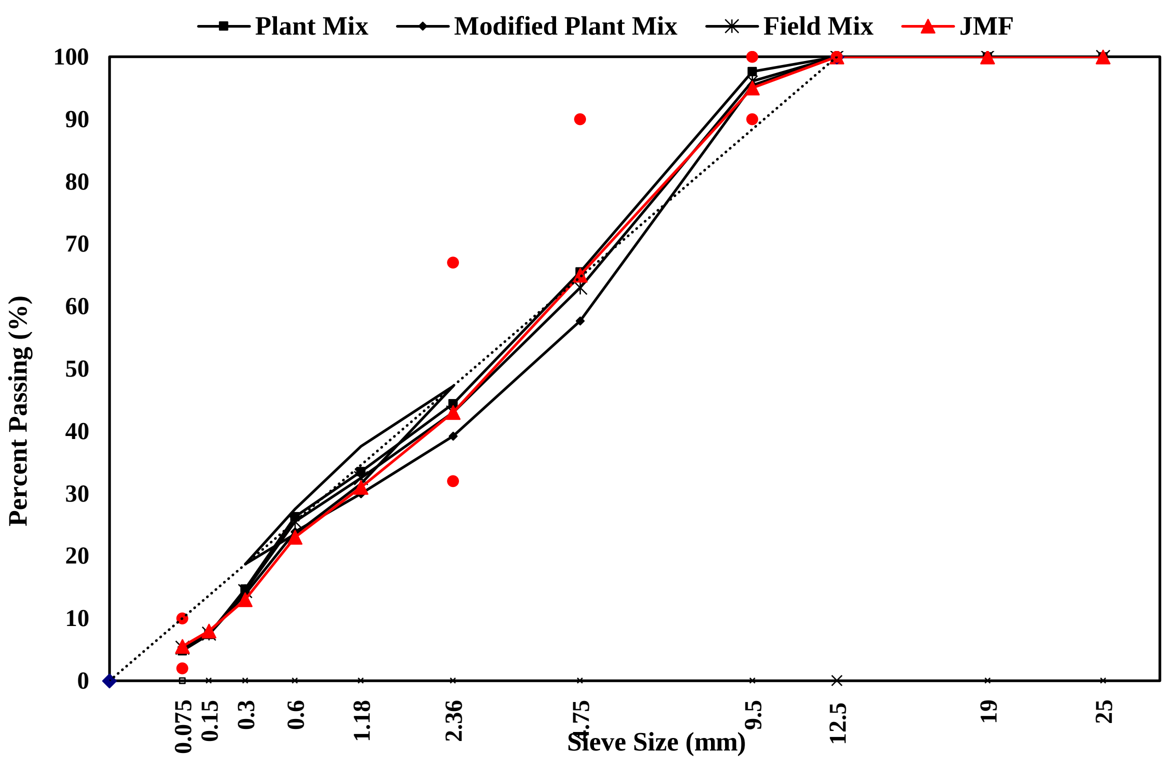


Figure A-5 Aggregate gradation curves for Mix DG5

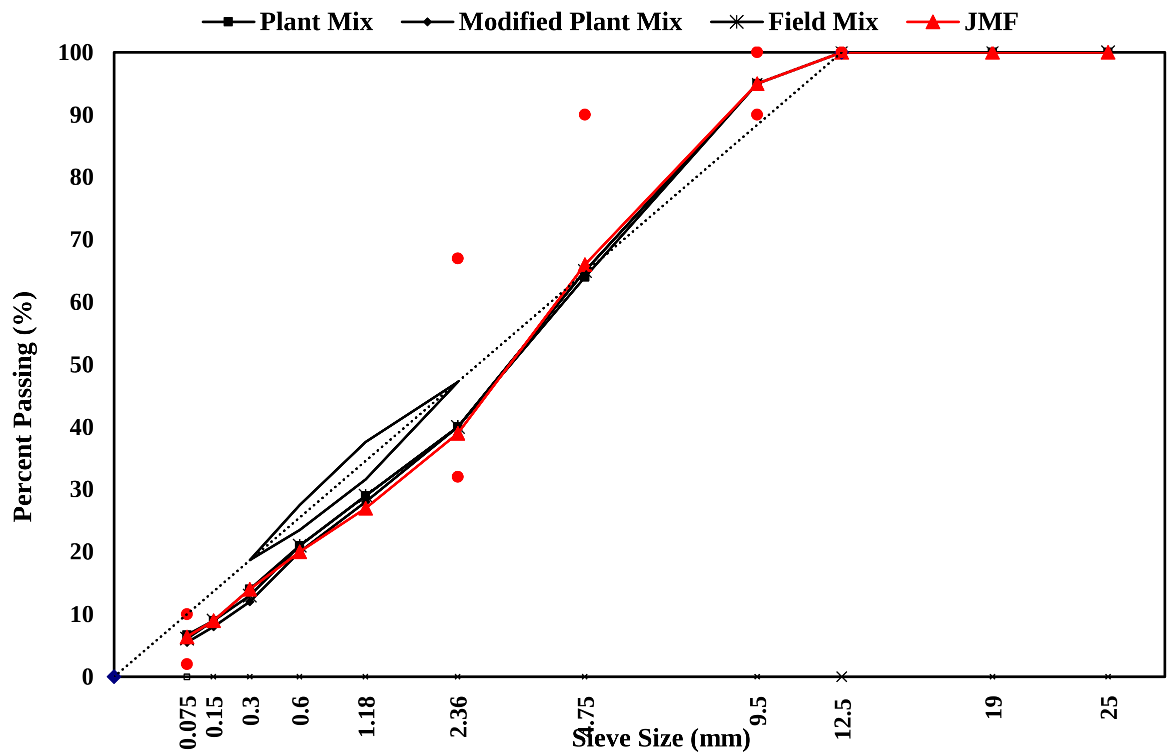


Figure A-6 Aggregate gradation curves for Mix DG6

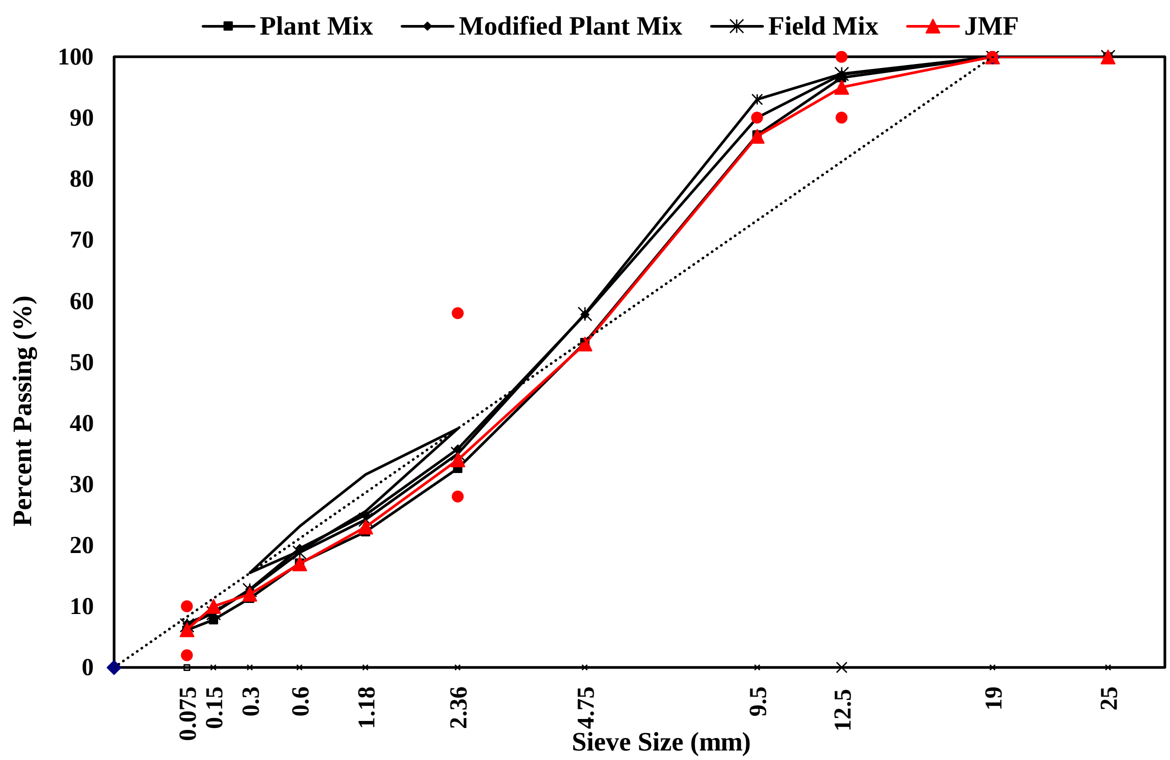


Figure A-7 Aggregate gradation curves for Mix DG7

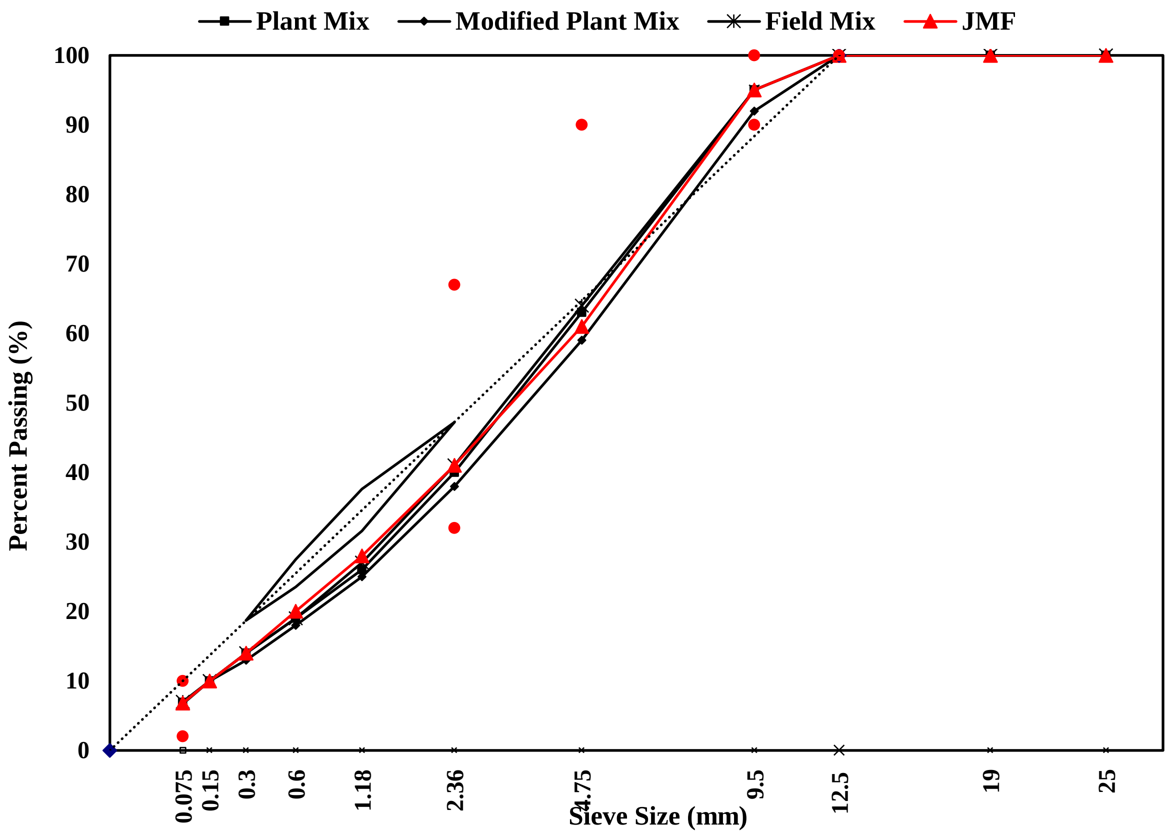


Figure A-8 Aggregate gradation curves for Mix DG8

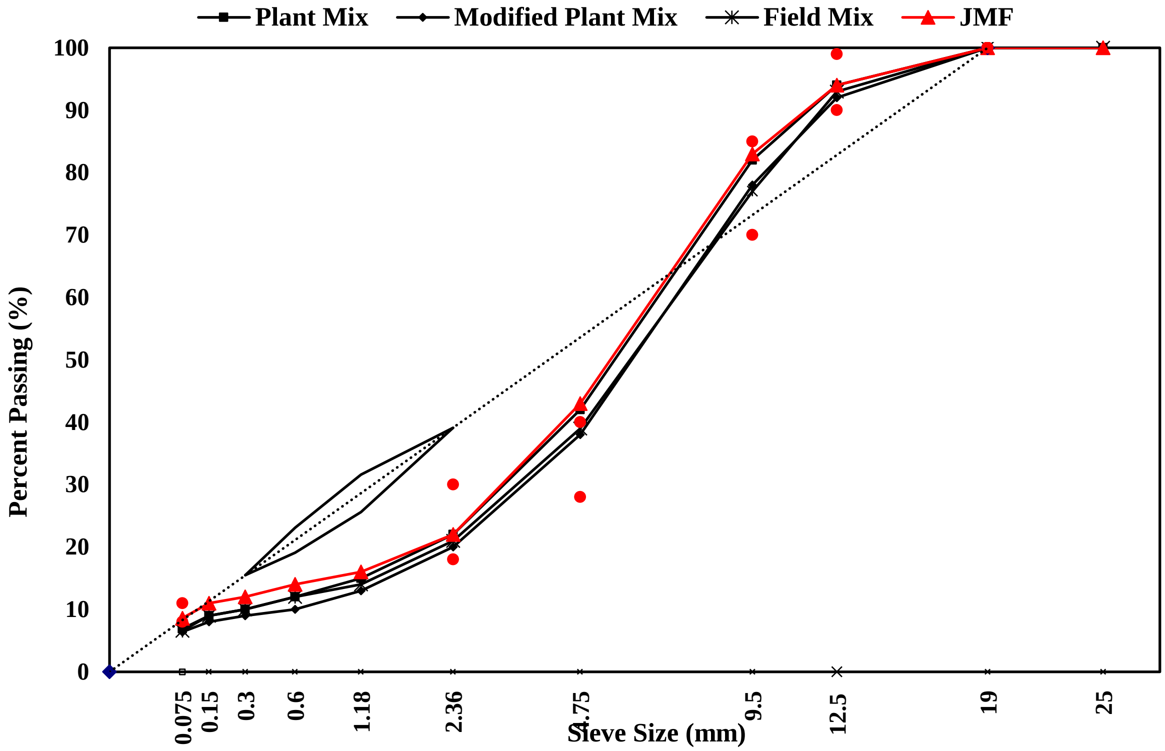


Figure A-9 Aggregate gradation curves for Mix GG1

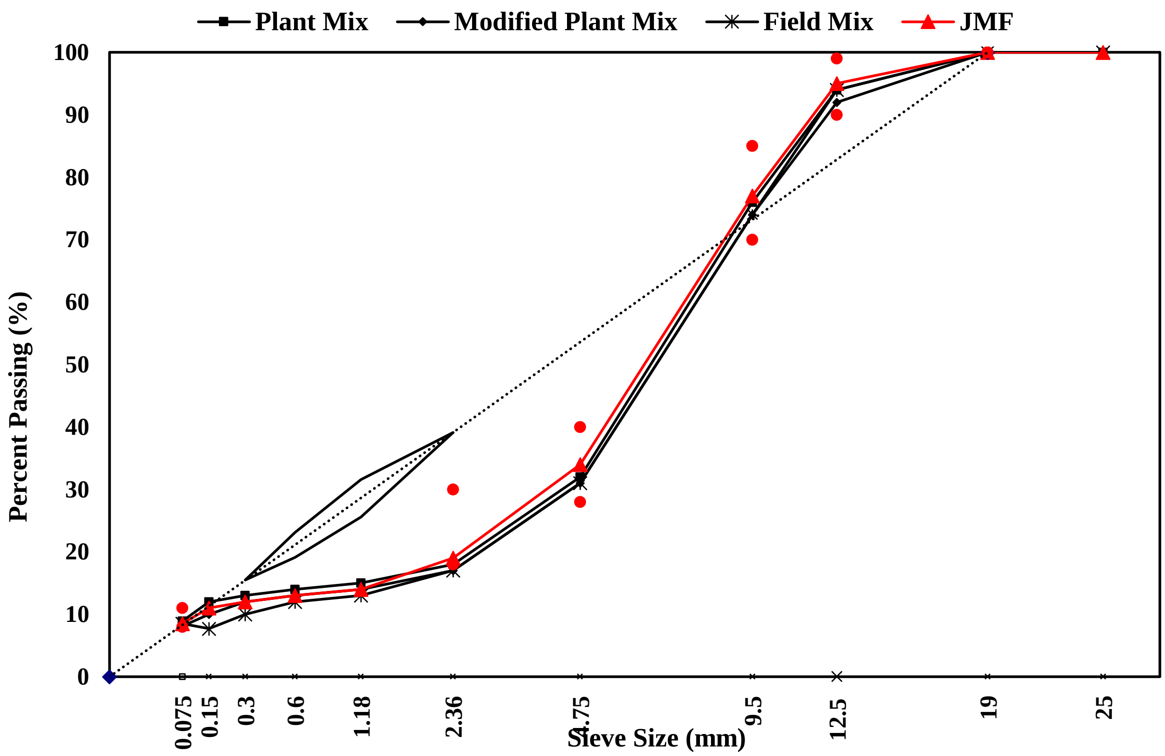


Figure A-10 Aggregate gradation curves for Mix GG2

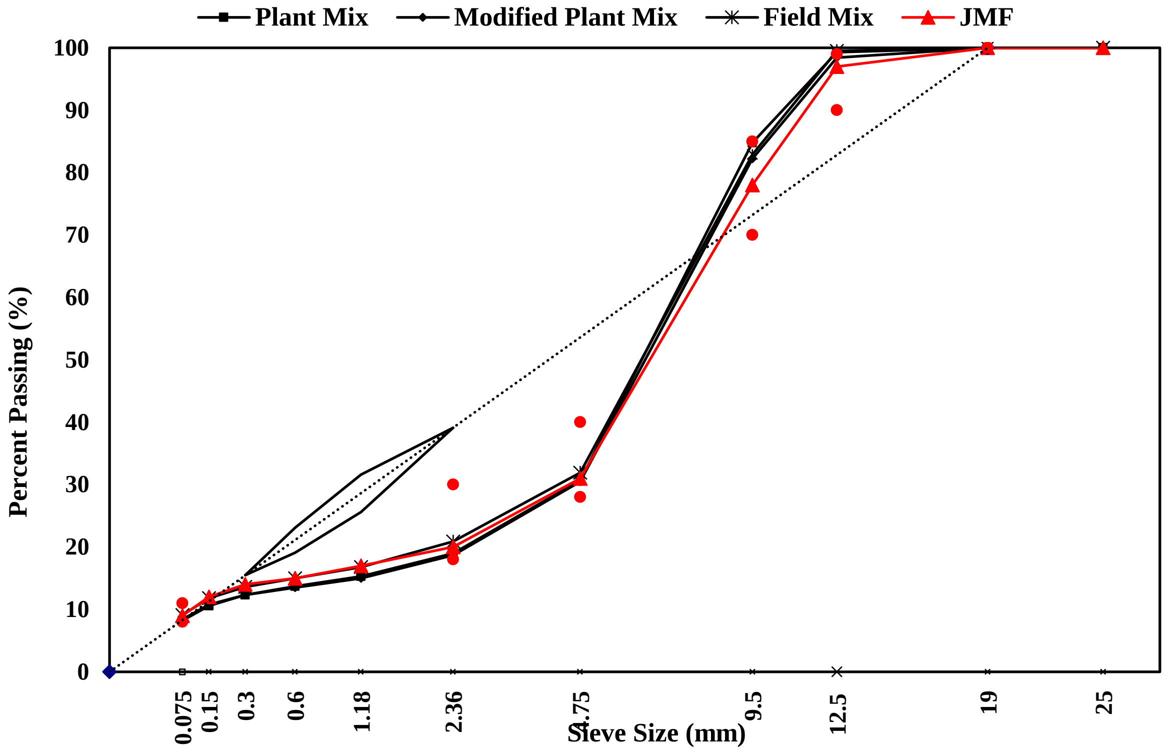


Figure A-11 Aggregate gradation curves for Mix GG3

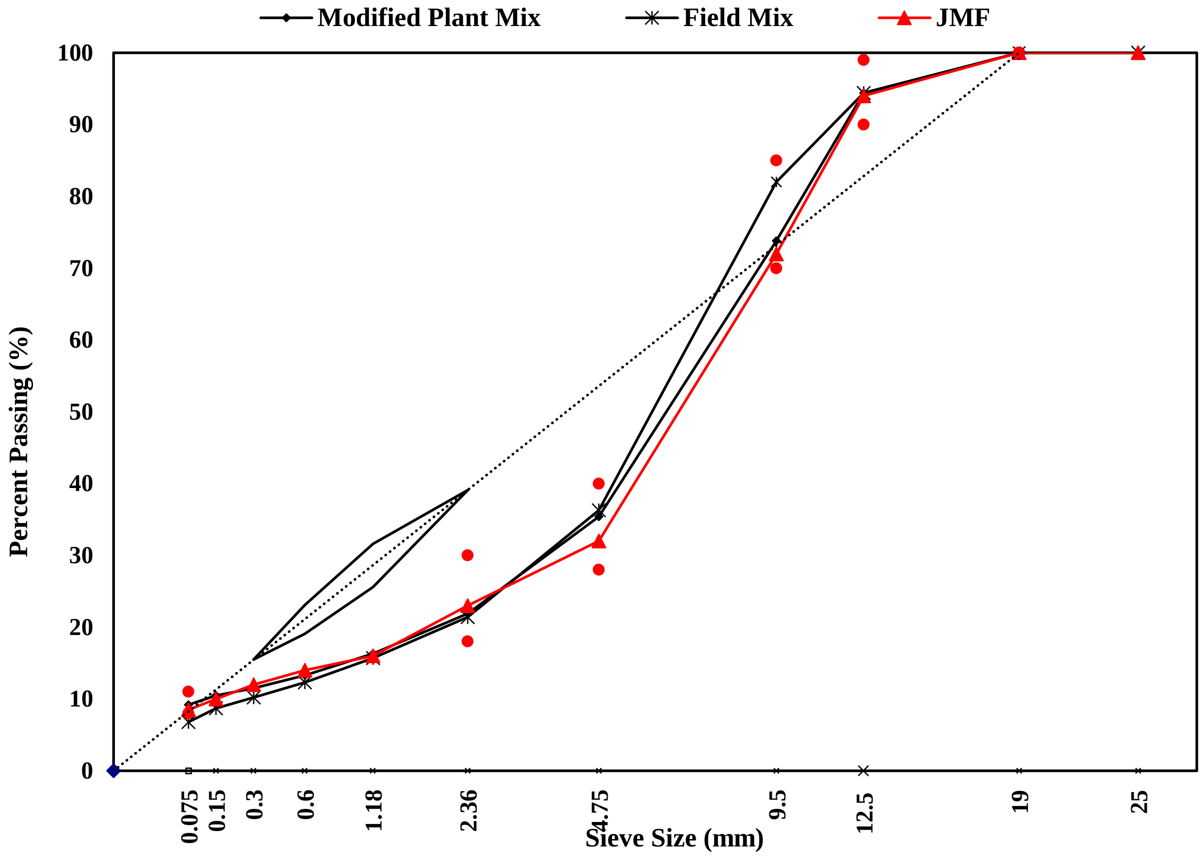


Figure A-12 Aggregate gradation curves for Mix GG4

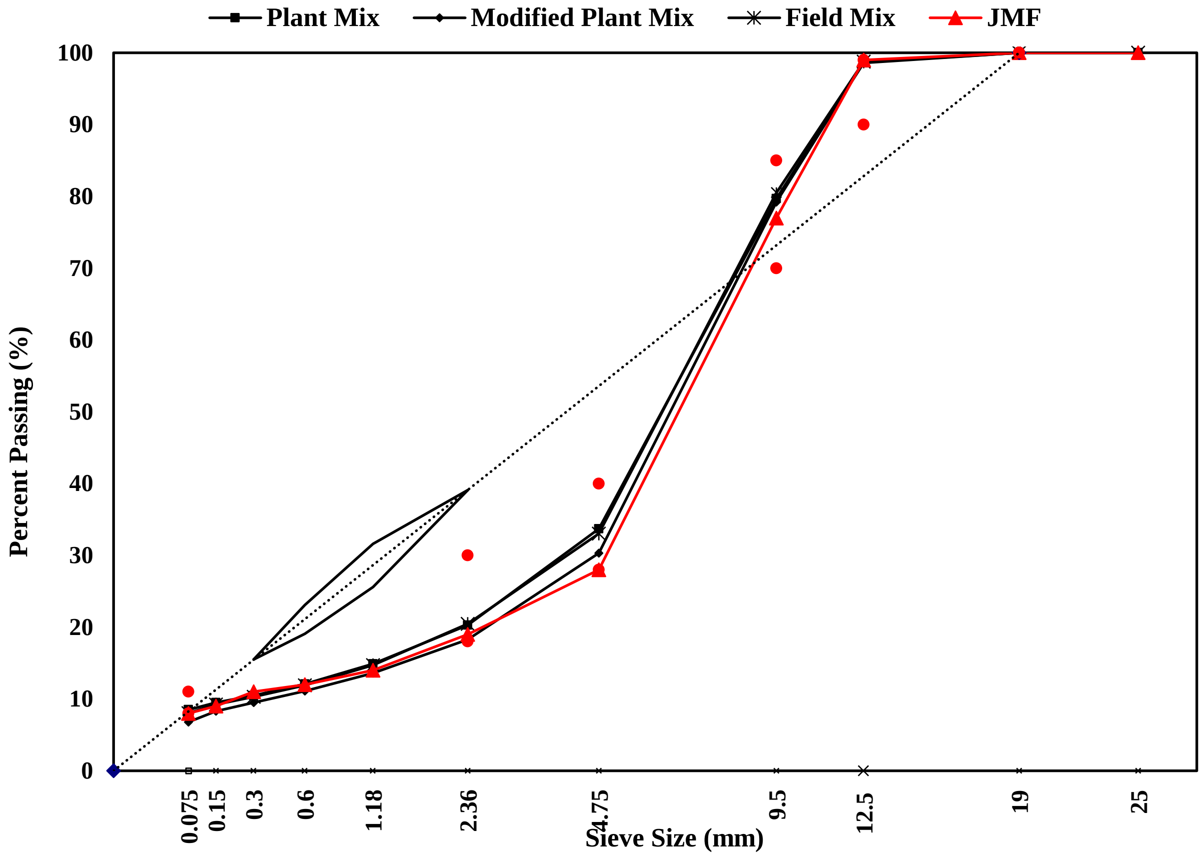


Figure A-13 Aggregate gradation curves for Mix GG5

Appendix B

As a follow-up of the correction procedures developed and presented in Chapter 3 for the IDEAL-CT and HT-IDT, examples are provided herein. Table B-1 provides the corrected COV for Mix 4, Mix 5, and Mix 6 for CT_{index}. Table B-2 includes the t-test results for the corrected CT_{index} between Lab 1 and Lab 2. No statistically significant difference is observed between the average CT_{index} of Lab 1 and Lab 2 after the use of the correction factors approach. Figure B-1 represents the significant reduction of the interlaboratory COV of the corrected CT_{index} values. Similarly, Tables B-3, B-4, and Figure B-2 present the effect of the correction factors on the interlaboratory COV of the HT-IDT results.

Table B-1 CT_{index} corrections

Mixture	Air Voids (%)	K (Lab1)	K (Lab2)	Ratio of K	Lab 1 CT_{index}	Lab 2 CT_{index}	Ratio of CT_{index}	Corrected Lab 2 CT_{index} (specific air voids)	Corrected Lab 2 CT_{index} (Mix specific)
Mix 4	7	145.88	160.3	0.91	104	116	0.8966	106	93
	7.2	109.2	118.7	0.9197	78	85	0.9176	78	68
	7.4	91.44	156.1	0.5858	66	113	0.5841	66	91
Interlaboratory COV						11.7		0.4	0.1
Mix 5	6.6	58.571	82.7	0.7085	42	59	0.7119	42	48
	6.7	73.017	78.48	0.9304	52	56	0.9286	52	46
Interlaboratory COV						10.1		0.1	0.1
Mix 6	7.1	143.73	118.35	1.2144	104	85	1.2235	103	103
Interlaboratory COV						10.1		0.4	0.4

Table B-2 Results of paired t-test

Mix	Initial CT_{index}	Corrected Lab 2 CT_{index} (specific air voids)	Corrected Lab 2 CT_{index} (Mix specific)
Mix 4	H1	H0	H0
Mix 5	H1	H0	H0
Mix 6	H1	H0	H0

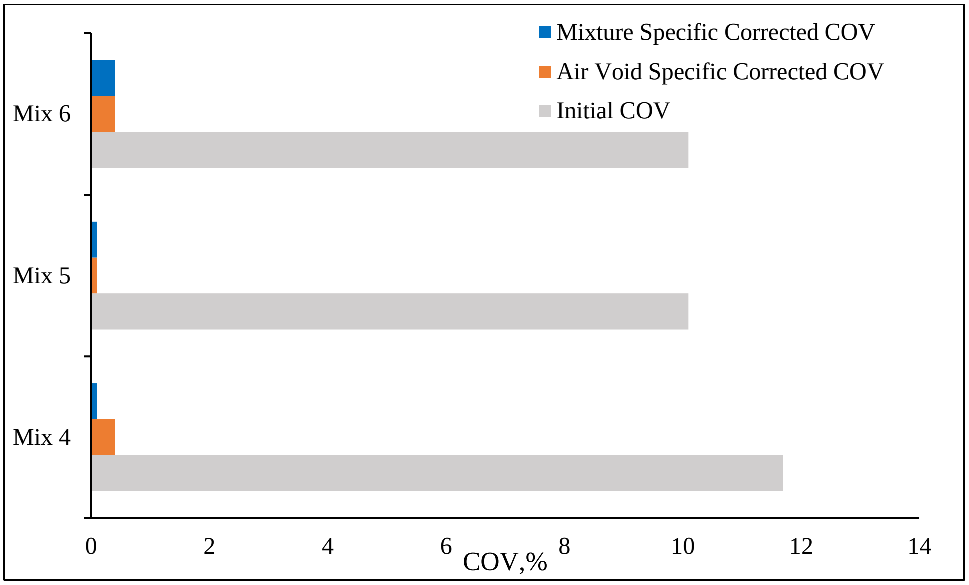


Figure B-1 IDEAL-CT results after corrections

Table B-3 HT-IDT corrections

Mixture	Air Voids	Lab 1 Load (KN)	Lab 2 Load (KN)	Ratio of Peak Load	Lab 1 Strength	Lab 2 Strength	Ratio of Tensile Strength	Corrected Lab 2 Strength (specific air voids)	Corrected Lab2 tensile Strength (Mix specific)
					(kPa)	(kPa)		(kPa)	(kPa)
Mix 4	7	4.4	6.9	0.6377	196	308	0.6364	196	195
	7.1	4.4	6.7	0.6507	195	299	0.6522	195	189
	7.2	4.2	6.9	0.6087	188	309	0.6084	188	195
Interlaboratory COV						22.5		0.0	0.0
Mix 5	7	4.2	7.2	0.5833	188	323	0.582	188	205
	7	4.2	7.1	0.5915	188	317	0.5931	188	202
	7.2	4.4	6	0.7333	196	269	0.7286	197	171
Interlaboratory COV						22.8		0.1	0.5
Mix 6	7.1	4.8	5.2	0.9246	214	233	0.9185	215	205
	7.2	4.6	4.9	0.9249	204	222	0.9189	205	196
	7.4	4	5.1	0.7945	180	227	0.793	180	200
Interlaboratory COV						6.6		0.3	0.3

Table B-4 Results of paired t-test

Mix	Initial Strength	Corrected Lab 2 Strength (specific air voids)	Corrected Lab 2 tensile Strength (Mix specific)
Mix 4	H1	H0	H0
Mix 5	H1	H0	H0
Mix 6	H1	H0	H0

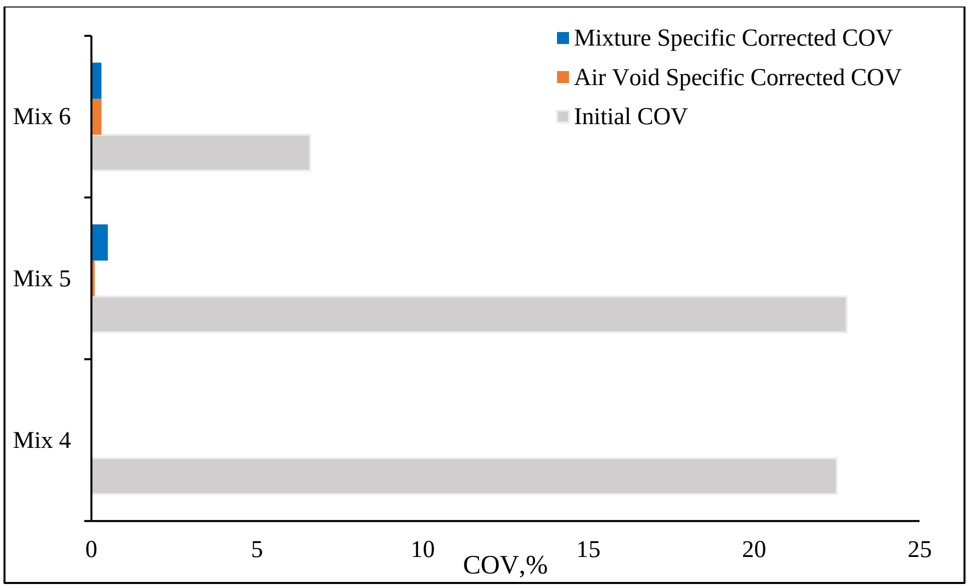


Figure B-2 HT-IDT results after corrections

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