Using X-Ray Computed Tomography to Quantify Damage of Hot-Mix Asphalt in the Dynamic Complex Modulus and Flow Number Tests

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USING X-RAY COMPUTED TOMOGRAPHY TO QUANTIFY DAMAGE OF HOT-MIX ASPHALT IN THE DYNAMIC COMPLEX MODULUS AND FLOW NUMBER TESTS

A Thesis
Submitted to the Graduate Faculty of the
Louisiana State University and
Agricultural and Mechanical College
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requirements for the degree of

Master of Science in Civil Engineering

in

The Department of Civil and Environmental Engineering

By
Hao Ying
B.S., China University of Geosciences, China, 2008
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VITA
ABSTRACT

The dynamic modulus test is conducted based on the assumption that no damage is induced during the testing process and that measurements are conducted within the linear viscoelastic region. In addition, significant damage is expected to occur during the flow number test process. While these assumptions are well-defined, the levels of damage taking place during these tests have not been quantified. The objectives of this study were to quantify the levels of damage in the dynamic complex modulus and flow number tests using x-ray computed tomography (CT) and to characterize the microstructural properties of asphalt mixtures under loading. Four Superpave mixtures including one conventional hot-mix asphalt (HMA) and three warm-mix asphalt (WMA) mixes were evaluated in this study. Two WMA processes (i.e., water foaming and Rediset™ additive) were used in the preparation of the WMA mixes. In addition, reclaimed asphalt pavement (RAP) was used in the preparation of the mixes at a content ranging from 15 to 30%. Results of the experimental program indicated that the damage taking place in the dynamic modulus test is minimal and homogeneous while the damage taking place in the flow number test is significant and heterogeneous. In addition, specimen preparation may significantly influence the three-dimensional air voids distribution in HMA.
CHAPTER 1 – INTRODUCTION

1.1 Background

The Mechanistic-Empirical Pavement Design Guide (MEPDG) was introduced in 2004 with new concepts and methodologies to consider advanced characterization of pavement materials, loads and traffic spectra, and other variables not incorporated in traditional design practices (Khanum et al. 2008). In the new design guide, time and temperature dependencies of Hot-Mix Asphalt (HMA) are described through the dynamic modulus test (ARA 2004). The dynamic modulus test was introduced in NCHRP 9-19 as a simple performance test (SPT) that can be used to predict field performance of asphalt mixes (Witczak et al. 2002). Research activities in NCHRP 9-19 also recommended using the Flow Number (FN), from the triaxial repeated load test, as a performance indicator of mix resistance to permanent deformation.

The standard test method for determining the dynamic modulus of asphalt mixes, AASHTO TP 62-07, assumes that measurements are conducted within the linear viscoelastic region, in which the dynamic modulus is independent of stress or strain amplitude. To ensure that the relationship between stress and strain is linear, the axial strains in the specimen should range between 50 and 150 microstrains. Calculation of the dynamic modulus also assumes proportionality of the induced strain to the applied stress on the specimen. Therefore, no damage should be induced in the specimen in order to ensure validity of the measurements and the calculations. In contrast, the flow number test is continued until 10,000 cycles or until the specimen fails through excessive tertiary permanent deformation. Therefore, substantial damage is expected to occur during the test process. Although the assumptions made in the dynamic modulus test and the flow number test are well defined, the levels of damage taking place during testing have not been quantified.
1.2 Problem Statement

The dynamic modulus test assumes that no damage is induced in the specimen during the test. Measurements and calculations are based on the validity of this assumption. In contrast, the flow number test is continued until 10,000 cycles or until the specimen fails through excessive tertiary permanent deformation. Therefore, substantial damage is expected to occur during the test process. Although the assumptions made in the dynamic modulus test and the flow number test are well defined, the levels of damage taking place during testing have not been quantified.

1.3 Research Objectives

The objectives of this study are to quantify the levels of damage in the dynamic complex modulus and the flow number tests, using x-ray computed tomography (CT) and digital image analysis. Results of the experimental program were used to provide insight into three-dimensional (3D) air voids distribution in asphalt mixes, prior to and after loading. Measurements were also used to characterize the effects of compaction on 3D air voids distributions and the microstructural properties of asphalt mixtures under loading.

1.4 Research Approach

The research approach adopted in this study consisted of completing the following four main tasks:

Task 1: Literature Review

A comprehensive literature review was conducted to review the following topics: 1) the dynamic modulus and flow number tests; 2) x-ray CT imaging analysis of HMA; and 3) damage quantification for HMA.
Task 2: Experimental Program

Four Superpave mixture types including one conventional HMA and three warm-mix asphalt (WMA) mixes were evaluated. Two WMA processes (i.e., water foaming and Rediset™ additive) were used in the preparation of the WMA mixes. In addition, reclaimed asphalt pavement (RAP) was used in the preparation of the mixes at a content ranging from 15 to 30%. The dynamic modulus test was conducted in accordance with AASHTO TP 62. The test was conducted by applying a uniaxial sinusoidal (i.e., haversine) compressive stress to an unconfined cylindrical test specimen. The test was conducted at four temperatures (4.4, 20, 37.8, and 54.4°C) and six loading frequencies of 0.1, 0.5, 1.0, 5, 10, and 25 Hz at each temperature to allow for the development of master curves that can be used to assess and to compare the viscoelastic characteristics of the different mixes. The flow number test was used to assess the permanent deformation characteristics of paving materials by applying a repeated dynamic load for several thousand repetitions on a cylindrical asphalt sample (Bonaquist et al. 2003). The “Flow Number” is defined as the starting point, or cycle number, at which tertiary flow occurs on a cumulative permanent strain curve, obtained during the test.

Task 3: Capture Digital Images Using X-Ray Computed Tomography

The objective of this task was to characterize the three-dimensional internal structure of laboratory-compacted HMA samples, using x-ray computed tomography imaging techniques. The imaging of 2D horizontal X-Ray CT was obtained along the height of the test samples at a close interval before and after loading. The captured 2D images were then used to determine the 3D air voids distribution in the compacted specimens.
Task 4: Damage Analysis

Air voids distribution was compared before and after testing in the dynamic complex test modulus and flow number test. A damage variable was introduced to quantify the levels of damage taking place during the complex modulus and flow number tests.

1.5 Scope

This thesis consists of 5 chapters. Chapter 2 presents a literature review of the subject. This includes dynamic modulus and flow number test, as well as x-ray CT imaging analysis and damage quantification of asphalt mixtures. The testing process and digital image analysis are introduced in Chapter 3. Chapter 4 presents the evaluation of data from dynamic modulus and flow number test. Master curves were constructed to compare the rheological properties of WMA to the HMA mixtures. Chapter 5 presents the conclusions of this study and the recommendations for future research.
CHAPTER 2 – LITERATURE REVIEW

2.1 Introduction

Hot-mix asphalt (HMA) is a viscoelastic and thermoplastic material that is characterized by a certain level of rigidity of an elastic solid body, but at the same time, flows and dissipates energy by frictional losses as a viscous fluid (Elseifi et al. 2003). As with any viscoelastic material, the HMA’s response to stress is dependent on both temperature and loading time. At high temperatures or under slow moving loads, HMA may exhibit a purely viscous flow. This behavior results in load-associated problems, such as rutting due to its low shear modulus and inability to preserve its shape. However, at low service temperatures or rapid applied loading, HMA becomes progressively harder and eventually brittle, which makes it vulnerable to non-load-associated problems such as low-temperature cracking. Susceptibility to fatigue-associated problems is more critical at normal (intermediate) temperatures, because a significant part of the traffic loads are applied at these temperatures.

One major distress that is considered in pavement design is rutting of HMA. Rutting occurs in the pavement as a result of traffic loads. Rutting generally develops during hot weather when the asphalt is soft and viscous as the result of traffic load. It can be identified by a surface depression in the wheel path. At high temperature, rutting can also lead to bleeding, which is a phenomenon in which asphalt binder rises to the pavement surface (Panoskaltsis 2005). Rutting also increases the chances of fatigue failure of flexible pavements (Panoskaltsis 2005). An excessive level of rutting at the pavement surface is considered a serious safety issue.

2.2 Laboratory Characterization of HMA

Laboratory performance tests are used to relate asphalt mix design to the performance in the field. Pavement performance can be characterized in the laboratory by measuring permanent
deformation resistance, fatigue life, tensile strength, stiffness, and moisture susceptibility. The common laboratory test methods for evaluating HMA are presented in Table 2.1.

**Table 2.1 Laboratory test methods**

<table>
<thead>
<tr>
<th>Performance Indicator</th>
<th>Test Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent Deformation</td>
<td>Static Creep tests</td>
</tr>
<tr>
<td></td>
<td>Repeated load tests</td>
</tr>
<tr>
<td></td>
<td>Dynamic modulus tests</td>
</tr>
<tr>
<td></td>
<td>Hveem and Marshall mix design tests</td>
</tr>
<tr>
<td></td>
<td>Simulative tests</td>
</tr>
<tr>
<td>Fatigue Life</td>
<td>Flexural test</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>Indirect Tension test</td>
</tr>
<tr>
<td></td>
<td>Thermal Cracking test</td>
</tr>
<tr>
<td>Stiffness</td>
<td>Stiffness test</td>
</tr>
<tr>
<td>Moisture Susceptibility</td>
<td>Modified Lottman test</td>
</tr>
</tbody>
</table>

2.3 **Dynamic Modulus and Flow Number Test of Asphalt Mixtures**

Two laboratory tests were conducted in this study to characterize the strength and rutting resistance of HMA: the dynamic complex modulus (|E*|) test and the flow number test. The dynamic complex modulus (|E*|) is an important design parameter for predicting rutting and fatigue cracking. Outputs from the dynamic modulus test may be used to create master curve, which tends to explain the behavior of mixtures over a wide range of temperatures and rates of loading. The dynamic modulus is also a required input parameter in the Mechanistic-Empirical Design Guide (MEPDG) software for pavement design.
The flow number is the number of test cycles required until tertiary flow starts in the mixture. The higher the flow number, the longer the time until the tertiary flow in the mixture starts. The flow number varies with the change in the asphalt content and percentage of air voids in the HMA (Vassilis 2005).

2.2.1 Dynamic Modulus of Asphalt Mixtures

In the linear viscoelastic region, the stress-strain relationship under a continuous sinusoidal loading is defined by its complex modulus (E*). E* is a complex number that relates stress to strain for linear viscoelastic materials subjected to sinusoidal loading in the frequency domain, see Figure 2.1. The complex modulus is defined as the ratio of the amplitude of the sinusoidal stress at any given time, t, and the amplitude of the sinusoidal strain at the same time and frequency, resulting in a steady state response (Dougan et al. 2003):

\[
E^* = \frac{\sigma_0}{\varepsilon_0} = \frac{\sigma_0 e^{i\omega t}}{\varepsilon_0 e^{i(\omega t - \theta)}} = \frac{\sigma_0 \sin \omega t}{\varepsilon_0 \sin(\omega t - \theta)}
\]

(1)

\[
\phi = \frac{T_l}{T_p}
\]

(2)

where,
\(\sigma_0\) = peak (maximum) stress;
\(\varepsilon_0\) = peak (maximum) strain;
\(\phi\) = phase angle, degrees;
\(\omega\) = angular velocity;
\(t\) = time, seconds;
i = imaginary component of the complex modulus;
\(T_l\) = time lag (sec); and
\(T_p\) = period of sinusoidal loading (sec).

Mathematically, the dynamic modulus is defined as the absolute value of the complex modulus:

\[
|E^*| = \frac{\sigma_0}{\varepsilon_0}
\]

(3)

Two dynamic modulus test procedures are recommended by NCHRP Project 9-19 (Witczak et al. 2002): one for permanent deformation at high temperature and one for fatigue.
cracking at an intermediate temperature. The dynamic modulus test results matched well with rutting resistance of mixtures used in experimental sections at MnRoad, Wes-Track, and the FHWA Pavement Testing Facility (Witczak et al. 2002). Only a fair correlation was found between cracking observed in the experimental sections and the dynamic modulus measured at 4.4 and 21.1°C.

**Figure 2.1 Dynamic (Complex) Modulus Test**

In NCHRP report 513, two simple performance testers (SPTs) from Interlaken Technology Corporation (ITC) and Shedworks, Inc (IPC) were evaluated with the purpose of finding an economical tester for use in Superpave mix design. The dynamic modulus tests were conducted at three conditions: 25°C unconfined for evaluating mixtures with fatigue cracking potential, 45°C confined for evaluating mixtures with rutting potential, and 45°C unconfined for evaluating open- or gap-graded mixtures for rutting potential. For each condition, dynamic moduli and phase angles were measured at frequencies of 25, 10, 5, 1, 0.5, and 0.1 Hz. The
testing used a 138 kPa confining pressure was used in the testing. Stress levels were varied until the target strain reached a level of 100 μstrain. The data analysis consisted of four steps: First, construct plots based on the modulus and phase angle to observe general data trends. Second, analyze the equality of variances between the various cells of the experiment. Third, analyze the differences in mean response for the two devices and laboratories. Fourth, determine the overall levels of variability for the dynamic modulus test and recommend limits for the quality indicators to be included in the test protocol.

![Image](image_url)

**Figure 2.2** Comparison of Dynamic Modulus Values Generated using the Interlaken (ITC) and Shedworks (IPC) Simple Performance Test Devices (Bonaquist et al. 2003)

According to the proposed “2002 Guide for the Design of Pavement Systems”, which was developed during THE National Cooperative Highway Research Program (NCHRP) Project 1-37A, the modulus of the asphalt concrete at all analysis levels of temperature and the rate of loading is determined from a master curve constructed at a reference temperature, generally 21°C. The master curve for asphalt concrete is a critical input for flexible pavement design in the
Mechanistic-Empirical Pavement Design Guide (MEPDG). The general master modulus curve may be mathematically described as (Witczak et al. 2002):

\[
\log|E^*| = \delta + \frac{\alpha}{1 + e^{\beta + \gamma (\log \omega_r)}}
\]  

(4)

where,
E* = dynamic modulus;
\( \omega_r \) = reduced frequency;
\( \delta \) = minimum value of E*;
\( \delta + \alpha \) = maximum value of E*; and
\( \beta, \gamma \) = parameters describing the shape of the sigmoidal function.

Dynamic modulus experimental data may be used to develop an extended master curve in accordance with the procedure suggested by Bonaquist and Christensen (2005). This procedure employs the sigmoidal function to describe the rate dependency of the complex modulus master curve (Bonaquist et al. 2005):

\[
\log(E^*) = \delta + \frac{(\text{Max} - \delta)}{1 + e^{\beta + \gamma \left\{ \frac{\log(t)}{9.14714} - \frac{\Delta E_a}{19.14714} \left( \frac{1}{T} \right) \left( \frac{1}{29525} \right) \right\} } 
\]  

(5)

where,
E* = complex modulus;
t = loading time;
T = temperature in ° Rankine;
\( \delta, \beta \) and \( \gamma \) = fitting parameters; and
Max = limiting maximum modulus.

Witczak et al (2002) developed an equation for predicting the dynamic modulus |E*| of HMA, as shown in Equation 6. This equation can estimate dynamic modulus of dense-graded HMA mixes over a range of temperature, as well as rates of loading and aging conditions from information, which is usually available from the conventional binder tests and the volumetric properties of HMA mixes (Gedata et al. 2010).
where, 

$E^* = \text{dynamic modulus, } 10 \text{ psi}$;  

$\eta = \text{binder viscosity at the age and temperature of interest, } 106 \text{ Poise}$;  

$f = \text{loading frequency, Hz}$  

$V_a = \% \text{air void content}$;  

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

$\rho_{34} = \text{cumulative } \% \text{ retained on } 3/4 \text{ in (19 mm) sieve}$;  

$\rho_{38} = \text{cumulative } \% \text{ retained on } 3/8 \text{ in (9.5 mm) sieve}$;  

$\rho_4 = \text{cumulative } \% \text{ retained on } #4 \text{ (4.76 mm) sieve}$; and  

$\rho_{200} = \% \text{ passing } #200 \text{ (0.075 mm) sieve}$.

Bari and Witczak (2006) revised the Witczak model, using 7400 data points from 346 HMA mixes. The new Witczak model uses dynamic shear modulus ($|G_b^*|$ ) and phase angle ($\delta_b$) of binder as input parameters. The new Witczak model is shown in Equation (7):

$$
\log|E^*| = -1.249937 + 0.02923(\rho_{200}) - 0.001767(\rho_{200})^2 - 0.002841(\rho_4) - \\
0.058097(V_a) - \frac{0.82208(V_{beff})}{V_{beff} + V_a} + \frac{3.871997 - 0.0021(\rho_4) + 0.0311(\rho_{38}) - 0.000017(\rho_{38})^2 - 0.00547(\rho_4)}{1 + e^{(-0.6033136 + 0.3143351 \log(f) + 0.393532 \log(\eta))}}
$$

(6)

where,  

$E^* = \text{dynamic modulus, } 10 \text{ psi}$;  

$\eta = \text{binder viscosity at the age and temperature of interest, } 106 \text{ Poise}$;  

$f = \text{loading frequency, Hz}$  

$V_a = \% \text{air void content}$;  

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

$\rho_{34} = \text{cumulative } \% \text{ retained on } 3/4 \text{ in (19 mm) sieve}$;  

$\rho_{38} = \text{cumulative } \% \text{ retained on } 3/8 \text{ in (9.5 mm) sieve}$;  

$\rho_4 = \text{cumulative } \% \text{ retained on } #4 \text{ (4.76 mm) sieve}$; and  

$\rho_{200} = \% \text{ passing } #200 \text{ (0.075 mm) sieve}$.

Christensen et al. (2006) modified the Hirsch model for predicting dynamic modulus of HMA mixes. The modified Hirsch model is shown in Equations (8) and (9):

$$
\log|E^*| = -0.349 + 0.754(|G_b^*|^{-0.0052}) * \left(6.65 - 0.0032\rho_{200} + 0.0027(\rho_{200})^2 + 0.011(\rho_4 - 0.0001\rho_4^2 + 0.006\rho_{38} - 0.00014\rho_{38}^2 - 0.08 V_a - \frac{1.06V_{beff}}{V_{beff} + V_a}) + \right.\\
\left.\frac{2.56 + \frac{0.71V_{beff}}{V_{beff} + V_a} + 0.012(\rho_{38}) - 0.0001(\rho_{38})^2 - 0.01(\rho_4)}{1 + e^{(-0.7814 - 0.5785 \log(|G_b^*|) + 0.8834 \log(\delta_b))}}\right)
$$

(7)

where,  

$E^* = \text{Dynamic modulus (psi)}$;  

$\rho_{34}, \rho_{38}, \rho_4, \rho_{200}, V_a, V_{beff}$ as previously defined;  

$|G_b^*| = \text{Dynamic shear modulus of asphalt binder (psi)}$; and  

$\delta_b = \text{Phase angle of asphalt binder (degree)}$.

Christensen et al. (2006) modified the Hirsch model for predicting dynamic modulus of HMA mixes. The modified Hirsch model is shown in Equations (8) and (9):
\[ |E^*|_m = P_c \left[ 4200000 \left( 1 - \frac{VMA}{100} \right) + 3|G^*_b| \left( \frac{(VMA)(VFA)}{100} \right) \right] + \frac{1-P_c}{\left( \frac{1-VMA}{100} \right)^{\frac{VMA}{4200000}} + \frac{3|G^*_b(VFA)|}{VMA}} \]  

(8)

\[
P_c = \frac{\left( 20 + \frac{3|G^*_b(VFA)|}{VMA} \right)^{0.58}}{65000 + \left( \frac{3|G^*_b(VFA)|}{VMA} \right)^{0.58}}
\]  

(9)

where,

\(|E^*|_m = \text{Absolute value of asphalt mixture dynamic modulus, (psi)}; \)

\(|G^*_b| = \text{Absolute value of asphalt binder complex shear modulus (psi)}; \)

VMA = Voids in mineral aggregates in compacted mixture (%) ; and

VFA = Voids filled with asphalt in compacted mixture (%).

A reasonable engineering estimate of the maximum shear modulus for asphalt binders is 145,000 psi (Christensen and Anderson 1992). Substituting this value into Equation (8) and (9) yields the recommended equation for estimating the limiting maximum modulus of asphalt concrete mixtures from volumetric data (Bonaquist and Christensen 2007):

\[ |E^*|_{max} = P_c \left[ 4200000 \left( 1 - \frac{VMA}{100} \right) + 435000 \left( \frac{(VMA)(VFA)}{100} \right) \right] + \frac{1-P_c}{\left( \frac{1-VMA}{100} \right)^{\frac{VMA}{4200000}} + \frac{435000(VFA)}{435000(VFA)}} \]  

(10)

where,

\[
P_c = \frac{\left( 20 + \frac{435000(VFA)}{VMA} \right)^{0.58}}{65000 + \left( \frac{435000(VFA)}{VMA} \right)^{0.58}}
\]  

(11)

\(|E^*|_{max} = \text{limiting maximum mixture dynamic modulus}; \)

VMA = Voids in mineral aggregates (%); and

VFA = Voids filled with asphalt (%).

In AASHTO Provisional Standard TP 62-03 “Standard Method of Test for Determining Dynamic Modulus of Hot-Mix Asphalt Concrete Mixtures,” at least two replicate specimens should be tested at five temperatures between 10 and 54.4°C and six frequencies between 0.1 and 25Hz. Bonaquist and co-workers (2005) presented an alternative method to test the dynamic
modulus at only three temperatures between 4 and 46.6°C and four rates of loading between 0.01 and 10 Hz. After substituting the limiting maximum modulus estimated from the predicted model, the MEPDG master curve equation is fit to this data, using the same numerical optimization techniques. A comparison of master curves using the reduced testing and the complete AASHTO TP62-03 data set for 22 mixtures yielded similar master curves, except in these cases which were extremely high, or where low moduli were measured at -9 °C.

Mohammad et al. (2005) employed the dynamic modulus test in a study to characterize the permanent deformation characteristics of HMA mixtures. In addition, they also evaluated the sensitivity of the dynamic modulus test results in a pavement rutting performance prediction using the MEPD software. Six plant-produced HMA mixtures were prepared and tested at five different temperatures (-10, 4.4, 25, 37.8, and 54.˚C) and six frequencies (0.1, 0.5, 1.0, 5, 10, and 25 Hz). The dynamic modulus test results, shown in Figure 2.3 (a to e), noted that as the temperature increases and the loading frequency decreases, the dynamic modulus value decreases. As shown in Figure 2.3 (a-e), the elastic properties of $|E^*|$ for an asphalt mixture is strongly temperature-dependent. Meanwhile, the observations show that the binder course mixtures have a better rut-resistance than the wearing course mixtures. The binder course performance at high temperatures may be attributed to the larger aggregate size (NMAS =25 mm) as well as possibly high reclaimed asphalt pavement contents. In addition, the large aggregates have a stronger stone-to-stone contact in the mixes, resulting in a high stiffness shown in the dynamic modulus test. In conclusion, the authors believed that the dynamic modulus test was sensitive to the nominal maximum size of aggregate in a mixture. Larger aggregates combined with RAP materials, could result in high $|E^*|$ values at high temperatures.
Bari et al. (2004) studied the effect of lime addition on the E* stiffness of HMA mixtures. The study revealed that the standard test and design methodologies of the new M-E pavement design guide can be effectively used for lime-modified HMA mixes. An E* test was conducted on six different HMA specimens at 10, 4.4, 21, 37.8 and 54.4°C using the frequencies of 25, 10, 5, 1, 0.5, and 0.1Hz. A uniaxial compression stress controlled mode was used in the test to produce 25 to 150 micro-strains. Bari and co-workers (2004) showed that hydrated lime can increase the E* of HMA mixtures by 17% to 65% across a range of mixtures, lime contents, and temperature. The Witczak prediction equation was applied to estimate E* and to generate master curves in their study. Figure 2.4 shows a comparison of master curves of the two mixtures with different lime contents. As shown in this figure, E* increased due to lime addition at all frequencies.

Kim and Lee (2006) developed a dynamic modulus testing procedure for cold in-place recycling foamed asphalt (CIR-foam) using the SuperPave simple performance tester (SPT). Seven CIR-foam mixtures made of seven different reclaimed asphalt pavements (RAP) materials were collected throughout the state of Iowa. The dynamic modulus was measured at three different temperatures (4.4, 21.1 and 37.8°C) and at six different loading frequencies (25, 10, 5, 1, 0.5 and 0.1Hz) for all RAP sources. Testing started at the lowest temperature and proceeded to the higher temperature in order to minimize potential damage to the test specimens. At a given temperature, testing began at the highest frequency of loading and proceeded to the lower frequency. An analysis procedure for constructing the master curves for the CIR-foam mixtures was presented in this study. Master curves were constructed, using the time-temperature correspondence principle. The following equivalency between frequency and temperature was applied for the range of dynamic moduli of asphalt mixtures:

\[
\log(f_r) - \log(f) = \log[\alpha(T)] \tag{12}
\]
where,
\(f_r\) = reduced frequency (Hz);
\(f\) = loading frequency (Hz); and
\(\alpha(T)\) = shifting factor.

Figure 2.3 Dynamic Modulus \(|E^*|\) test Results (Mohammad et al. 2005)
Figure 2.4 Master Curves of Asphalt Mixtures with Different Lime Contents (Bari et al. 2004)

The Williams-Landel-Ferry (WLF) equation (Williams et al. 1955) was used to determine the shift factor $\alpha (T)$:

$$\log(f_r) - \log(f) = \log[\alpha(T)] = \frac{C_1(T-T_{ref})}{C_2 + T - T_{ref}}$$ (13)

where,

$C_1, C_2$ = empirical constants.

The frequency $f_r$ is defined as:

$$f_r = \alpha(T) \times f$$ (14)

The impacts of RAP material characteristics on the dynamic modulus were also confirmed. The authors found that at high temperature, the dynamic modulus is influenced by residual binder characteristics, whereas at low temperatures, the dynamic modulus is affected by the amount of fines in the RAP materials.

Singh and co-workers (2010) employed four empirical models: Witczak (2002), new Witczak (2006), Hirsch, and Al-Khateeb, to estimate the dynamic modulus of HMA mixes commonly used in Oklahoma. The tests were conducted for five different mixes prepared with a
variety of aggregate sources and sizes (limestone, granite, and rhyolite); binder grades; PG 64-22, PG 70-28, and PG 76-28; and four air voids levels of 6, 8, 10, and 12%. The dynamic modulus was measured and predicted using each empirical model. Goodness-of-fit statistics, comparison of measured and predicted values, and local bias statistics were applied to evaluate the performance of these models. The accuracy of each empirical model was evaluated at four different temperatures (4, 21, 40, and 55°C) and air void levels (6, 8, 10, and 12%). As shown in Figure 2.5, at the lower and intermediate temperatures the Witczak model (2006) showed significant errors in estimating the dynamic modulus, but performed well at high temperatures. The Hirsh model under-predicted the dynamic modulus at intermediate temperatures whereas the Al-Khateeb model over-estimated the dynamic modulus at high temperatures (Singh et al. 2010).

Figure 2.5 Average Errors Variation with Air Voids at (a) 4°C (b) 21°C (c) 40°C (d) 55°C (Singh et al. 2010)
The performance of each model also changed with air voids levels. The estimated error for each mixture is shown in Figure 2.6. The Witczak (1999) model performed well at lower air voids. However, the model prediction reduced at higher air voids levels. The performance of the Hirsch model had a similar trend to the Witczak (1999) model. The new Witczak (2006) model and the Al-Khateeb models performed poorly at all air voids levels (Singh et al. 2010).

![Figure 2.6 Estimated Error for each Prediction Model (Singh et al. 2010)](image)

Yang et al. (2011) developed a simplified predicting model for the dynamic modulus of HMA. A laboratory program was designed to evaluate the dynamic modulus of 20 selected Superpave asphalt concrete mixtures used in the state of Florida. Based on their test results and analyses, a simplified dynamic modulus model was proposed as follows:
\[ \log|E^*| = 2.312 + 0.01\rho_{200} + 0.1\rho_8 - 0.011\rho_{400} - 0.002\rho_3 + 0.023P_b - 0.042VFA \\
+ \frac{-1.34 - 0.019\rho_8 + 0.022\rho_4 + 0.004\rho_3 - 0.055P_b + 0.052VFA}{1 + e^{(-8.267-0.772\log f + 5.397\log T)}} \] (15)

where,

- \( T \): Test temperature, in °C
- \( f \): load frequency, in Hz
- \( P_b \): Percent weight of asphalt, % by weight.

Figure 2.7 presents a plot of the measured \( E^* \) stiffness data from the test results versus the predicted \( E^* \) stiffness data from the simplified predicting model under the same input conditions. As shown in this figure, all the individual data points are around the line of equality; and the linear fitting parameter is 0.9504, which indicates that there is no significant level of bias between the prediction results and the test results. The authors also compared the simplified predicting model with the classic Witczak model. The correlation was good, and the bias level was small.

**Figure 2.7** Measured Values versus Predicted \( E^* \) from Witczak Model (Yang et al. 2011)
2.2.2 Flow Number of Asphalt Mixtures

The “flow number” is defined as the starting point, or cycle number, at which tertiary flow occurs on a cumulative permanent strain curve obtained during the test. Commonly, the flow number is determined by locating the lowest point in the strain rate versus cycle number, or the minimum value of the strain rate (Witczak et al. 2002). Figure 2.8 shows a typical plot of the permanent strain versus loading cycle on a log–log scale. Research activities in NCHRP 9-19 recommended the use of the Flow Number (FN), from the triaxial repeated load test, as a performance indicator of mix resistance to permanent deformation. A load cycle consisting of a 0.1-s haversine pulse load and a 0.9-s dwell (rest) time is applied for the test duration—typically about 3 h or 10,000 loading cycles (Witczak et al. 2002). Results from the repeated load tests are typically presented in terms of the cumulative permanent strain versus the number of loading cycles. The cumulative permanent strain ($\varepsilon_p$) curve may be divided into three zones: primary, secondary, and tertiary. The cycle number at which tertiary flow starts is referred to as the “flow number.”

![Diagram showing the Flow Number (FN) on a cumulative permanent strain curve]

**Figure 2.8** Typical Relationships between Total Cumulative Plastic Strain and Loading Cycles
Kaloush and Witczak (2001) recommended a fundamentally-based laboratory procedure, simple for a permanent deformation evaluation of Superpave asphalt mixture. The Simple Performance Test (SPT) is intended to provide an accurate correlation to field rutting performance. Two tests were proposed: the repeated load permanent deformation test and the static creep/flow time test. Two tertiary flow laboratory response parameters were found to have an acceptable correlation with field rut depth data: the flow number from the repeated load test and the flow time from the static creep test. The two parameters were found to be repeatable and reliable in distinguishing between a wide range of asphalt mixtures, as well as sensitive to different testing variables (Kaloush et al. 2001). Both of these parameters were recommended for further evaluation and follow up validation testing. Kaloush and Witczak (2001) also identified some issues that should be evaluated in the SPT:

- Reliability to distinguish permanent deformation behavior between a wide range of mixtures;
- Repeatability of the test parameters;
- Simplicity (complexity) of the test procedure, cost of the equipment, and testing preparation requirements;
- Testing time needed to complete the testing procedure; and
- Technical level or experience required from the operator.

NCHRP Project 9-19 also verified that the machine compliance errors do not affect the computation of the flow time or flow number by comparing flow times and flow numbers measured with these three systems: the specimen-mounted deformation system, the radial deformation system, and the actuator-mounted deformation system (measuring the movement of the loading actuator). As shown in Figure 2.9, flow times and flow numbers from an actuator-mounted deformation system and the specimen-mounted deformation system were comparable.
Mohammad and co-workers (2005) conducted an unconfined flow number test to evaluate the permanent deformation characteristics of HMA mixtures. The effective temperature and design stress level selected were 54.4°C and 207 kPa, respectively. As shown in Figure 2.10, the flow number results had a fairly good correlation with the Hamburg rut depths for the six tested mixtures. It was also reported that both parameters are sensitive to the permanent deformation characteristics of HMA mixtures. The Hamburg rut depth decreased as the flow number increased.

**Figure 2.9** Comparison of Flow from Different Instrumentation Systems (Witczak et al. 2002)
The classical model for the determination of tertiary flow is presented in Equation 16. Biligiri and co-workers (2007) recommended a new, comprehensive, mathematical model (Equation 17) to accurately determine the FN:

\[
\varepsilon_p(N) = aN^b 
\]  

\[
\varepsilon_p(N) = aN^b + c(e^{dN} - 1) 
\]

where, 
\( \varepsilon_p(N) \) = permanent strain at ‘N’ cycles; 
\( N \) = number of cycles; and 
a, b, c, d = regression coefficients.

Figure 2.10 Relationship between Hamburg Rut Depth and Flow Number (Mohammad et al. 2005)

Current procedures for calculating the Flow Number depends on data smoothing techniques, which can yield very different values for the same specimen, if different moving average periods are used (Archilla 2007). Archilla and co-workers (2007) introduced a new methodology for determining FN by modeling the permanent strain versus load cycles curve as a continuous function:
\[ FN = \gamma [1 - e^{\frac{1}{\alpha} - 1}] \]  
(18)

\[ \varepsilon_{p, low} = \frac{1}{\beta} \left( 1 - \frac{1}{\alpha} \right)^{1/\alpha} \]  
(19)

where, 
\( \alpha, \beta \) and \( \gamma \) = model parameters.

Data from cold-mix replicate samples were analyzed and compared in this study. Archilla and co-workers reported that the proposed model could be useful for permanent deformation characterization. Zhou et al. (2004) introduced a three-stage model to determine the three stage (primary, secondary, and tertiary) deformation behavior in the flow number test. Zhou et al. (2004) selected the Power-law model to describe the deformation curve at the primary stage. In the primary stage, the curve was fitted using the Power-law model as shown below:

\[ \varepsilon_p = aN^b \]  
(20)

where, 
\( \varepsilon_p \) = accumulate permanent strain in the primary stage; 
\( N \) = number of load repetitions; and 
\( a, b \) = regression coefficients.

The calculated and the measured \( \varepsilon_p \) were both compared using the following equation:

\[ D_e = \frac{|\varepsilon_{p, measured} - \varepsilon_{p, predicted}|}{\varepsilon_{p, measured}} \]  
(21)

where, 
\( D_e \) = the deviation; 
\( \varepsilon_{p, measured} \) = measured permanent strain; and 
\( \varepsilon_{p, predicted} \) = calculated permanent strain.

The initiation of secondary stage was determined with \( D_e \) less than 3\%, and \( N \) determined at \( D_e \) < 3\% as a less than maximum cycle number. In the tertiary stage, the accumulated strain of the secondary stage was increased in linear function, and it may be described using a linear model:
\[ \varepsilon_p' = cN' + d \]  \hspace{1cm} (22)

where,
\( \varepsilon_p' \) = the accumulated strain;
\( N \) = the cycle number; and
\( c \) and \( d \) = regression coefficients.
The regression coefficients were calculated and compared with measured \( \varepsilon_p' \), using the equation below:

\[ R_d = \frac{d}{\varepsilon_p'} * 100\% \]  \hspace{1cm} (23)

where,
\( R_d \) = absolute ratio of \( d \) to current maximum \( \varepsilon_p' \)

The flow number (initiation of tertiary state) occurs when \( R_d < 1\% \) (or the D value > 0) and \( N' \) at \( R_d < 1\% \) is less than the maximum cycle number. Kvasnak and co-workers (2007) proposed the flow number as an additional test to complement the dynamic modulus within the MEPDG. Seventeen mixtures with unique mix parameters were prepared. The flow number of these mixtures was evaluated at different air voids and asphalt contents. A predictive equation (Equation 22) for flow number was developed. Six important factors were used to estimate the flow number as a function of the number of gyrations, bitumen viscosity, voids in the mineral aggregate, percent passing the 4.75-mm sieve, percent passing the 1.18-mm sieve, and percent passing the 0.075-mm sieve:

\[ \log FN = 2.886 + 0.00613 \times Gyr + 3.86 \times Visc - 0.072 \times VMA + 0.0282 \times P_{4.75} - 0.051 \times P_{1.18} + 0.075 \times P_{0.075} \]  \hspace{1cm} (24)

where,
\( Gyr \) = Gyrations; \( Visc \) = Bitumen Viscosity, \( 10^6 \) Poise;
\( VMA \) = Voids in the Mineral Aggregate;
P4.75 = Percent Passing the 4.75-mm Sieve;  
P1.18 = Percent Passing the 1.18-mm Sieve; and  
P0.075 = Percent Passing the 0.075-mm Sieve.

A high-precision model, known as GP/SA derived by Alavi and co-workers (2010), predicted the flow number of dense asphalt mixtures. A comprehensive experimental database was established upon a series of uniaxial, dynamic creep tests used for the development of the model. Generalized regression neural network and multiple regression-based analyses were performed to benchmark the GP/SA model. The contributions of the variables affecting the flow number were evaluated through a sensitivity analysis. A subsequent parametric study was also carried out, with the trends of the results confirmed by Alavi and co-workers (2010). Results indicated that the proposed GP/SA model is effective in evaluating the flow number of asphalt mixtures.

The FNest method (Bausano et al. 2007) defines the flow number as the maximum point at the curve of stiffness times versus cycles. In order to verify the applicability of the proposed approach, 123 flow number data were compared. In Goh and You study (2009), a new effectively simple, stepwise method for determining flow number was developed. This method assumed that the strain will only be maintained at the same point or increased over the load cycle number. Then, the flow number at the minimum point of new strain rate versus cycle number is determined. This stepwise method consists of three steps:

**Step 1:** Smooth the measured permanent deformation by re-allocating the measured results with an assumption of permanent strain. The result will either remain the same or increase over the load cycle number.

**Step 2:** Calculate the strain rate using the permanent deformation data set, which was modified in Step 1. The strain rate is calculated by the following equation:
Strain Rate \( = \frac{\varepsilon}{N} \) \hspace{1cm} (25)

where,
\( \varepsilon \) is permanent strain; \( N \) is loading cycle number at each cycle;

**Step 3:** locate the minimum point of strain rate versus load cycle curve to determine the flow number.

Figure 2.11 shows an example of determining the flow number using the stepwise method at 39.2°C.

![Figure 2.11 Sample Result of Flow Number (Goh and You 2009)](image)

The comparison plots of the stepwise and other three methods (cycles versus cycles method, FNest method, and three-stage method) are shown in Figure 2.12. The R-square > 0.98 was derived from these comparisons and indicated that these methods have shown an excellent correlation with the proposed stepwise method. Figure 12d shows that the comparison between the stepwise flow number and rate of deformation for all mixtures tested. It shows that an
excellent relationship ($R^2=0.962$) was found between rate of deformation and flow number. The results also indicated that the rate of deformation from the modified dataset using the stepwise approach can be used to compute the flow number.

(a) Comparisons of Stepwise and Cycles Versus Cycles Method (Goh and You 2009)

(b) Comparisons of Stepwise and FNest Method (Goh and You 2009)

Figure 2.12 The Comparison Plots of Four Methods (stepwise method, cycles versus cycle method, FNest method, and three-stage method)
Rodezno et al. (2009) found that there is no standard test protocol that addresses the required laboratory stress to be applied in the flow number test. The test can take several hours until tertiary flow is reached and in many cases, the sample may never fail. The authors stated
that developing a model capable of predicting or providing general guidance on the Flow Number characteristics of a mix is necessary. In this study, a Flow Number predictive model using HMA mixture volumetric properties and stress/temperature testing conditions as predictor variables was developed (Rodezno et al. 2009):

\[
\log FN = 2.174 + 0.649 \log V_1 + 0.101 P_{200} + 18.465 \log P + 0.0140 R_{04} - 0.084 V_a - 18.901 \log q - 0.872 R_{34} + 0.182 q - 0.193 p - 0.871 \log T
\]  

(26)

where,

- \(V_1\) = viscosity at testing temperature;
- \(P_{200}\) = % passing #200 (0.075 mm) sieve;
- \(R_{04}\) = cumulative % retained on #4 (4.76 mm) sieve; and
- \(R_{34}\) = cumulative % retained on 3/4 in (19 mm) sieve.

The accuracy evaluation of this model was done by using statistical method. The results are shown in Figure 2.13. The statistics had an \(R^2 = 0.62\) and \(Se/Sy = 0.60\), which are considered to be fair statistical measures of model accuracy. Rodezno and co-workers (2009) still addressed that the variability within replicates was relatively high, especially for the confined test conditions. It was reported that reduction in the test variability in future studies will help increase the prediction accuracy of model.

Zelelew and co-workers (2010) studied the laboratory permanent deformation resistance of WMA used in the Pennsylvania. Four types of WMA made with different WMA technologies (i.e., Advera®, Sasobit®, LEA, and Gencor) were considered in this study. Flow number and Hamburg tests were performed on these WMA specimens to evaluate the stiffness and permanent deformation resistance potential. The test results are shown in Figure 2.14. The WMA/Sasobit® specimens had higher permanent deformation resistance parameters compared to the other WMA technologies. The authors also reported that the WMA specimens were
susceptible to moisture damage, due to minor aging during production, particularly at low testing temperatures.

**Figure 2.13.** Predicted Versus Observed Log FN of the Developed FN Model (Rodezno et al. 2009)

(a) Flow Number Test Results; (b) Hamburg Test Results; (JMF: Job Mix Formula) (Zelelew et al. 2010)
2.4 X-Ray CT Imaging Analysis of HMA

X-ray Computed Tomography (CT) is a nondestructive imaging technology, capable of acquiring a 3D or 2D image of the internal structure of a solid object, such as asphalt concrete. The directing planar x-rays pass through the specimen, along several different paths and from different directions and are captured by the detector. The attenuations of x-rays within a specimen are recorded for calculating the linear attenuation coefficients, which may be used to represent the spatial locations of the different components of the specimen. After finishing the collection of attenuations for a full rotation of the specimen, it is vertically moved downward for scanning the next slice.

Braz and co-workers (1999) applied a computerized X-ray CT technique to study the evolution of crack in asphalt mixture during fatigue tests (Figure 2.15). The researchers found that the path of cracking is influenced by initial air voids in the mixture. By evaluating
tomographic images, the study noted that the propagation of cracking causes structure failure of the specimen. The crack initiates in the central region of the specimen and follow the direction of the applied load. Due to the limitations of X-ray CT at that time, the internal microstructure of asphalt mixture could not be detected.

X-ray CT was frequently applied in recent years to characterize cracks or any damage in asphalt mixtures by measuring the internal structure distribution of specimens, such as the locations of aggregates, as well as mastic and air voids. Image analysis techniques of X-ray CT images can be used to quantify damage parameters by measuring the cracking growth in mixtures under uniaxial loading.

In order to obtain more accurate internal structure distribution, 2D cross-sectional CT images are needed to reconstruct 3D visualization images of the specimens. 2D cross-sectional CT images were obtained to measure air void distribution and crack size at different depths with asphalt specimens. After capturing 2D cross-sectional images, the 3D visualization image of the sample can be reconstructed for importing to computer to simulate the performance of asphalt mixture under various loading and environmental conditions. Benefiting from this non-destructive technology, the intact sample may still be used for engineering properties tests such as the dynamic modulus test and the flow number test. Hence, X-ray CT is an effective technology to study the relationship between asphalt microstructure and engineering properties.

Masad and co-workers (1998) recommended that X-ray CT image analysis procedures be used to quantify the internal structure of asphalt concrete. Internal structure is quantified in terms of aggregate orientation, aggregate contacts and air void distribution in specimens compacted in the SGC and LKC. X-ray CT was used to obtain images from four specimens. All specimens were cut vertically into three vertical sections using a diamond saw. Two images were
captured subsequently from the two faces of each cut. A total of six images were acquired for each specimen. Before the analysis, air voids, asphalt binder and aggregates phases were defined by choosing threshold gray intensity. Due to the aggregates have different colors depending on their mineral composition; it is difficult to separate the aggregates from other two phases. In order to solve this issue and to isolate the air voids, the authors transformed the original image transformed into a binary image of black and white phases as shown in Figure 2.16. The white area represents the region of air void. The black area is the region of other two phases.

![Figure 2.15 The Tomographic Images of the Specimen after Blows (Braz and et al. 1999)](image)

Three-dimensional characterizations of the internal structure of asphalt mixtures may be achieved by using x-ray Computed Tomography (CT) imaging techniques (Masad et al. 2002). In this test setup, planar x-rays are passed through the specimen along several directional paths. Due to the difference in density of the constituents of the mix (i.e., aggregate, mastic, and air
voids), brighter regions corresponds to regions with higher densities. The specimen (usually a core) is then shifted vertically, and the entire procedure is repeated. A horizontal slice (image) is usually obtained at a close interval to cover the entire depth of the specimen. The captured images may then be used to determine the three-dimensional (3D) air void distribution in the compacted specimen or to create finite element meshes that describe the actual aggregate shape and distribution in the mix. These meshes may then be used in a micromechanical analysis to describe the heterogeneous behavior of the mix.

Wang and co-workers (2001) used X-Ray CT images and a virtual sectioning technique to reconstructed three-dimensional structures and to obtain cross-sectional images needed for the damage quantification. The specimen was moved downward for scanning slices along the height were obtained and stacked to produce the 3D volume structure. Figure 2.17 presents several cross-sectional images.

Figure 2.16 Image Threshold to Isolate Air Voids (white) (Masad et al. 1998)
Figure 2.17 Cross-sectional Images Produced by XRT Imaging (Wang 2002)

Figure 2.18 presents the 3D visualization of an asphalt concrete specimen, where 81 slices (sectional images) were stacked to produce the 3D volume rendering. In addition, Wang and co-workers (2001) mentioned that Tomography imaging carries a resolutions issue that in the transverse directions (i.e., horizontal and vertical), the resolutions are significantly different. This result is due to the spacing between adjacent image sections, which is much larger than the image resolution. In order to acquire smooth transitions and inherent connections between the images, an interpolation technique between the sectional images is recommended by the authors.

X-ray CT technique was applied to measure the internal structure in asphalt mixes by Masad and co-workers (2002). Experimental and analytical methods were employed together to quantify the structure air voids in their study. X-ray CT system and image analysis techniques were used to measure air void sizes at different depths within asphalt mixes. Horizontal Images of the internal structure of asphalt mix specimens were captured using an X-ray CT system.
Figure 2.19 shows an x-ray CT image of an asphalt concrete specimen with a diameter of 150 mm.

**Figure 2.18** Visualization in 3D of an Asphalt Concrete Specimen (Wang 2002)

**Figure 2.19** Horizontal Cross-Sectional X-ray CT image of Asphalt Concrete Specimen (Masad et al. 2002)
An abnormal model known as the Weibull distribution model (Weibull 1939) was used in their study to model the crack size distribution in asphalt mixes. The Weibull distribution described the statistical behavior in the breaking strength of materials. In the Masad and co-workers’ study, the Weibull model, in describing the air void distribution, quantified the effect of the compaction effort, method of compaction, and aggregate size distribution on air voids. The Weibull distribution involves two parameters: a scale parameter $\theta$ and a shape parameter $\beta$. The probability density function of this distribution is given by:

$$f(t) = \frac{\beta}{\theta} \left(\frac{t}{\theta}\right)^{\beta-1} e^{-(t/\theta)^\beta}; \ t > 0$$  \hspace{1cm} (27)

Its cumulative distribution function is given in Equation 29, whereas the exponential distribution can be obtained by letting $\beta=1$ in the above two equations.

$$F(t) = 1 - e^{-(t/\theta)^\beta}; \ t > 0$$  \hspace{1cm} (28)

As Masad et al. (2002) noted that the air void size data displayed three factors: compaction effort, method of compaction, and aggregate size distribution. There are $L$ factor levels for each factor, and $X_{ijk}$ denotes the area of the $k$th air void at the $j$th depth for the $i$th level of the factor. For the $i$th factor level, $D_i$ is the number of depths considered and $N_{ij}$ is the number of air voids at $j$th depth. Assumed the scale parameter depend on the factor level $i$ and depth, then write the Weibull density function for $X_{ijk}$ as:

$$f(X_{ijk}) = \frac{\beta}{\theta_{ij}} \left(\frac{X_{ijk}}{\theta_{ij}}\right)^{\beta-1} e^{-(X_{ijk}/\theta_{ij})^\beta}; \ X_{ijk} > 0$$  \hspace{1cm} (29)

where $k=1…N_{ij}; \ j=1…D_i; \ i=1…L.$
The air void size distribution curves of Superpave gyratory compacted specimens show a “bathtub” shape in Figure 2.20, where larger voids were present at both the top and bottom parts of a specimen. This type of shape becomes more marked at higher compaction.

![Figure 2.20: Median Size Distributions of Air Void Size with Depth at Different Compaction Efforts in Superpave Gyratory Compactor (Masad et al. 2002).]

The air void size distributions in specimens compacted by the Superpave gyratory compactor and the linear kneading compactor are different, as shown in Figure 2.21.

![Figure 2.21: Median Size Distribution of Air Void Size with Depth in Superpave Gyratory Compactor and Linear Kneading Compactor Compacted Specimens (Masad et al. 2002).]
The specimens compacted by the linear kneading compactor show an increasing trend in the size of air voids as the depth increases. Therefore the authors conclude that this method of compaction significantly affected the distribution of air void size. In addition, the Superpave gyratory compactor for preparing specimens with different aggregate sizes will cause markedly different air void sizes distribution in mixes.

In 2007, Adhikari and You used X-Ray Computed Tomography (CT) images technology to characterize the aggregate orientation, aggregate gradation, mastic distribution, and air void distribution in an asphalt mixture. Dynamic modulus of sand mastic and asphalt mixtures were measured with uniaxial compression tests in the laboratory, using three temperatures and six loading frequencies. From the X-ray CT images, the locations of aggregate, air voids, and mastic were predicted based on their grayscale intensities which ranged from 0 to 255. Corresponding to different densities of these three phases (aggregate, air voids, and mastic) within the asphalt mix, the X-ray images consist of 256 levels of gray intensity. The air void threshold value was chosen from 0-124; the mastic threshold value was from 125–203; and the aggregate threshold value was 126–255. Figure 2.22 shows the color segmentation applied to separate air voids, aggregate and mastic drawn from the gray intensity of images.

The study simulated three replicates of 3D Distinct Element Modeling (DEM) and six replicates of 2D DEM. For the purpose of acquiring accurate air void distribution, 2D images were visualized through vertical orientation. The aggregate elastic modulus and mastic properties were used as input parameters in this simulation. By recording the responses of the asphalt mastic, and mixture modules under a compressive load strain, this study calculated the dynamic moduli. The results from experimental measurements compared with the 2D and 3D predictions. Adhikari and You found that the 3D distinct element models have the ability to predict the
mixture moduli during a range of temperatures and loading frequencies. The 3D models prediction is much better superior to that of the 2D models. The modeling of asphalt mixture from 2D to 3D approach demonstrated significant progress.

Figure 2.22 Color segmentation on X-ray CT gray scale images (Adhikari and You 2007)

Wang and co-workers (2007) reported that the relative lower stiffness differences between aggregates and mastics are the reason that permanent deformation of HMA is mainly localized in the soft mastics. They also concluded that studies on the micro response of the aggregates and mastics could help understand the macro behavior of asphalt mixtures. In their study, a method using X-ray CT imaging system to measure the deformations in the mastics was
introduced. Due to the energy limitation of the X-ray system, the original specimen was cut perpendicular to the long axis of the original specimen into three pieces in order to acquire 0.15 mm/pixel resolution. In order to avoid boundary effect to the measurements, specimens were scanned at 5mm from one end of the specimen before and after APA tests (Figure 2.23). To avoid the basis for the separation of the mixture phases, Image Pro Plus was used for processing the X-ray images. Aggregates were separated from other phases by adjusting the threshold value. Figure 2.24a shows a gray-scale image, and Figure 2.24 b shows the image after processing. Based on the morphological characteristics of the particles, Wang and co-workers (2007) applied a pattern recognition algorithm to identify the particles in the images between adjacent slices as well as before and after testing. By tracking the particles, the motions of the particles such as rotation and translation can be quantified and the strains can be further estimated. They concluded that the translations of the particles are an important factor that affects the macro behavior of the mixture.

**Figure 2.23** Reconstructed 3D Specimen before and after Testing (containing 200 sectional images) (Wang et al. 2007)
Kassem et al. (2008) employed X-ray CT and ground penetrating radar (GPR) to examine the quality of the compaction of thick asphalt layers within full-depth pavements in Texas. Air void distribution and uniformity in asphalt pavement cores from two different structural asphalt pavement sections were detected and analyzed. One incorporated a stone filled (SF) Superpave mix and the other incorporated a traditional dense graded Type B material. The X-ray CT images were separated into three phases: air voids, aggregate, and mastic by threshold. The thresholded images were analyzed to determine the average percent of air voids using macros that were developed in Image-Pro software (Media Cybernetics, Inc., MD). High percent air voids caused by the poor compaction were identified within the coarse SF Superpave mix. Equations 30-32 presented the determination of average percent air voids for a specimen (%AV), average percent air voids in an image (%AV_image), and average air void radius in an image (r), respectively (Kassem et al., 2008):

\[
%AV = \frac{1}{N} \sum_{i=1}^{N} %AV_{image} 
\]  

(30)
\[
\%AV_{\text{image}} = \frac{A_{TV}}{A_T}
\]  
(31)

\[
r = \sqrt{\frac{A_{TV}}{\pi n}}
\]  
(32)

where,
\(A_{TV}\) = total area of the air voids in a CT image;
\(A_T\) = total cross-sectional area of a CT image;
\(N\) = number of CT images; and
\(n\) = number of the air voids in a CT image.

The three dimensional air voids distribution within the SF mix, corresponding to the results in Figure 2.25(a) is shown visually in Figure 2.25(b). Figure 2.25(c) shows the GPR results about low density areas within the SF layer of the FW-01 section. Their GPR and X-ray CT results displayed great agreement between each other, showing that the use of GPR and X-ray CT to evaluate the compaction of asphalt pavement is effective.

You and co-workers (2009) predicted a dynamic modulus of asphalt mixture by applying both two dimensional (2D) and three-dimensional (3D) Distinct Element Method (DEM) generated from X-ray CT images. The accordance with the AASHTO TP62-03 standard, a uniaxial compression test was conducted to measure the dynamic modulus of sand mastic and asphalt mixtures at different temperatures and loading frequencies. Cylindrical HMA specimens were loaded by haversine compressive stress for a range of temperatures (4, -6, and -18 °C) and loading frequencies (0.1, 0.5, 1, 5, 10, and 25 Hz). The dynamic modulus (\(|E^*|\)) and phase angle were calculated by measuring recoverable axial strain. Figure 2.26 shows an example of the applied axial stress and the measured strain response versus time plots for an asphalt mixture specimen, tested at a temperature of 4°C and a frequency of 5 Hz. Figure 2.27 shows the dynamic moduli of HMA mixtures measured for this study across a range of temperatures and loading frequencies. The dynamic modules were used as input parameters to predict the asphalt
mixture dynamic modulus in 2D and 3D DEM simulations and compared with both the experimental measurements results. You and co-workers (2009) reported that the 3D discrete element models can successfully predict the asphalt mixture dynamic modulus over a range of temperatures and loading frequencies. In contrast, 2D discrete element models under predicted the asphalt mixture dynamic modulus.

![Figure 2.25 Three-Dimensional Images of Air Void and the percentage Air Voids Distribution plot (a), and GPR Data (b) (Kassem et al. 2008)](image-url)
(Figure 2.26 continues)

**Figure 2.27** Axial Stress and Axial Strain Response versus Time (loading frequency 5 Hz) for the Specimen at 4°C of the Mixture Specimen (You et al. 2009)
Figure 2.28 Dynamic Moduli of Asphalt Mixture Specimens for a Range of Loading Frequencies (0.1, 0.5, 1, 5, 10 and 25 Hz) and Test Temperatures (4, -6, and -18°C) (You et al. 2009)

2.5 Damage Quantification of HMA

Microstructure damage is considered to be an important aspect that affecting the behavior of geomaterials, such as asphalt mixes. The growth and coalescence of air voids and cracks within the microstructure control the deformation process and decrease the load-carrying capacity of the material, thus leading ultimately to failure (Khaleel et al. 2001; Taylor et al. 2002; Chiarelli et al. 2003; Bonora et al. 2004; Aubertin and Li, 2004). Many researchers proposed the importance of damage and its evolution in proper modeling of AC (e.g. Schapery, 1982; Sousa et al. 1993; Park et al. 1996; Kim et al. 1997; Scarpas et al. 1997; Lee et al., 2000). Park et al. (1996) addressed that any constitutive model describing the behavior of AC also must account for the effect of damage growth. Sousa et al. (1993) noted that adding damage constituents to a permanent deformation model can significantly improve its ability to predict the experimental measurements. Therefore, an appropriate damage prediction model at
microstructure level importantly predicts the macroscopic response of the material (Bonora et al. 2004). Asphalt concrete as a composite geomaterials which consists of asphalt binder, air voids with various sizes and shapes; aggregates with various sizes and shapes, and surface textures. Further due to imperfect bonding between aggregates and asphalt binder, various micro-damages can exist in asphalt concrete in the form of air voids and the cracks. Therefore, asphalt concrete may be considered a continuum with distributed damage.

Paris and Erdogan (1963) applied the damage evolution law to express the crack growth rate in the following equation:

\[
\frac{d_c}{dN} = AK^n
\]  

(33)

where,
- \( c \) = crack length;
- \( N \) = number of loading repetitions;
- \( A,n \) = parameters dependent on the material and on the experimental conditions; and
- \( K \) = stress intensity factor.

Perzyna (1984) shown in his study that the rate of damage in solid material is a function of three variances: viscoplastic energy, confinement pressure, and effective viscoplastic strain.

\[
\dot{\xi} = f(W_{vp}, \dot{I}_1, \dot{\varepsilon}_{vp})
\]  

(34)

where,
- \( W_{vp} \) = the rate of viscoplastic energy;
- \( \dot{I}_1 \) = the rate of change in the first stress invariant; and
- \( \dot{\varepsilon}_{vp} \) = the effective viscoplastic strain rate;
Perzyna made an assumption for elastic-viscoelastic solid material that the internal imperfections are generated from the nucleation, growth and transport of voids as follows (Perzyna, 1984):

\[
\dot{\xi} = (\dot{\xi})_{\text{nucleation}} + (\dot{\xi})_{\text{growth}} + (\dot{\xi})_{\text{transport}}
\]  

(35)

\[
(\dot{\xi})_{\text{nucleation}} = \frac{h}{1 - \xi} \text{tr} (\sigma D^p) + l J_1
\]  

(36)

\[
(\dot{\xi})_{\text{growth}} = (1 - \xi) \text{tr} (\Xi D^p)
\]  

(37)

\[
(\dot{\xi})_{\text{transport}} = (\dot{\xi})_{\text{diffusion}} = D_0 \nabla^2 \xi(x, t)
\]  

(38)

where,
\( h, l = \) nucleation material function;
\( J_1 = \) the first invariant of the Cauchy stress tensor;
\( D^p = \) rate of permanent deformation tensor;
\( \Xi = \) the matrix of the material function;
\( D_0 = \) diffusion constant for constant temperature; and
\( \nabla^2 = \) Laplasian operator.

With the purpose of studying the behavior of asphalt concrete, based on Perzyna’s (1984) approach, Tashman et al. (2004) proposed an empirical power law for damage evolution law.

\[
\xi(W_{vp}, \dot{I}_1, \dot{\varepsilon}_{vp}) = \xi_0 + H_c (W_{vp})^{H_p} + L_c (I_1)^{L_p} + E_c (\varepsilon_{vp})^{E_p}
\]  

(39)

where,
\( \xi = \) damage parameter;
\( \xi_0 = \) initial damage value;
\( W_{vp} = \) viscoplastic energy;
\( W_{vp} = \) viscoplastic energy;
\( I_1 = \) the first invariant of the stress tensor;
\( \varepsilon_{vp} = \) effective viscoplastic strain; and
\( H_c, H_p, L_c, L_p, E_c, \) and \( E_p \) = fitting coefficients.
Schapery (1990) introduced a methodology of thermodynamics of irreversible processes by developing a theory for describing the mechanical behavior of elastic media with growing damage and other structural changes. Based on this approach and continuum damage theory, Park and co-workers (1996) employed a viscoelastic continuum damage model to describe the damage growth within the asphalt concrete body under uniaxial load. The continuum damage model consists of the following three parts: 1) pseudo strain energy density function, 2) stress-strain relationship, and 3) damage evolution law:

\[ W^R = W^R (\varepsilon^R, S_m) = \frac{1}{2} C(s)(\varepsilon^R)^2 \]  \quad (40)

\[ \sigma = \frac{\partial W^R}{\partial \varepsilon^R} = C(s)\varepsilon^R \]  \quad (41)

\[ \dot{S}_m = \left( -\frac{\partial W^R}{\partial S_m} \right) \alpha_m \]  \quad (42)

substituting Equation (41) into Equation (42), and integrate the equation to obtain an implicit one-to-one functional relationship between S and a new damage parameter \( S^* \) (Schapery, 1990):

\[ S^* = k \left[ \int_0^S \frac{ds}{(-0.5 \cdot \frac{dC}{ds})^{1/2} \alpha_m} \right]^{1/2} \alpha_m \]  \quad (43)

where,
- \( W^R = \) pseudo strain energy;
- \( \varepsilon^R = \) pseudo strain;
- \( S_m = \) internal state variables (or damage parameters);
- \( \dot{S}_m = \) damage evolution rate;
- \( C(s) = \) a function of a single damage parameter \( S \); and
- \( \alpha_m = \) material constant, and \( k = \) numerical factor.
Park et al. (1996) assumed that the maximum values of S and $S^*$ are numerically for ease in plotting results. Thus, Equation (44) can be rewritten as:

$$\sigma = C(S^*)\varepsilon^R$$

(44)

Based on Park’s approach, Lee and co-workers (1998) adopted an elastic continuum damage model to predict the damage growth and recovery in asphalt concrete under various levels of uniaxial loading and loading rates, and under both strain control and stress control test modes. Additionally, pseudo stiffness is an indicator which can be used to characterize damage in the viscoelasticity material. Pseudo stiffness defined as follows:

$$S^R = \frac{\sigma_m}{\varepsilon^R_m}$$

(45)

where,

$S^R = \text{pseudo stiffness};$

$\varepsilon^R_m = \text{peak pseudo strain in each stress-pseudo strain cycle};$ and

$\sigma_m = \text{stress corresponding to } \varepsilon^R_m.$

Lee et al. (1998) assumed that the normalized damage parameter S is sufficient to describe the change in pseudo stiffness due to fatigue damage growth. S was obtained from the following numerical:

$$S \cong \sum_{i=1}^{N} \left[ \frac{1}{2} \left( \varepsilon^{R}_{m,i} \right)^2 (C_{i-1} - C_i) \right]^{1+\alpha} (t_i - t_{i-1})^{1/(1+\alpha)}$$

(46)

where,

$S = \text{the damage parameter at each discrete cycle};$

$\varepsilon^{R}_{m,i} = \text{peak pseudo strain in each stress-pseudostrain cycle};$

$C_i = \text{the pseudo stiffness};$ and

$t_i = \text{the corresponding time}; \alpha \text{ is material constant.}$
Kim et al. (2002) applied the damage evolution law to study the fatigue and healing potential of asphalt binders in sand asphalt mixtures. The researchers used the equations by replacing $\gamma^R_{m,i}$ with $\varepsilon^R_{m,i}$ for torsional strain-controlled fatigue tests, as shown in Equations 47 and 49:

$$S \equiv \sum_{i=1}^{N} \left[ \frac{l}{2} \left( \gamma^R_{m,i} \right)^2 (C_{i-1} - C_i) \right]^{\frac{\alpha}{1+\alpha}} (t_i - t_{i-1})^{1/(1+\alpha)} \quad (47)$$

$$W^R = \frac{1}{2} C(s)(\varepsilon^R_{m})^2 \quad (48)$$

The damage of a material can be characterized mainly by a decrease in the effective load-carrying area, caused by the growth of air voids (Murakami, 1988). The theory postulates that a damaged material, subjected to a state of stress, may be represented by an undamaged material subjected to an imaginary effective stress, as shown in Figure 2.28. The damage parameter is defined as the following (Murakami, 1988):

$$A_{lcrey} = S \left( 1 - \xi \right) \quad (49)$$

$$\xi = \frac{S_a}{S} \quad \text{where} \quad 0 \leq \xi \leq 1 \quad (50)$$

$$\sigma_e = \frac{\sigma}{1-\xi} \quad (51)$$

where,
$\sigma_e$ = imaginary effective stress applied to the assumed undamaged specimen;
$A_{lcrey}$= effective load-carrying area;
$S$= total cross-sectional area of the specimen; and
$S_a$= the area of air voids.
The test stress (σ) is the load divided by the total cross section area of sample, because of the decrease in effective load-carrying area (A_{loc}^c), the effective stress applied to the assumed undamaged sample is equal to the stress applied to the damaged specimen (σ), multiplied by 1/1 - ξ, where ξ is the damage parameter.

![Diagram showing stress and damage](image)

**Current Damage State**  
**Fictitious Undamaged State**  
*Figure 2.29* Damage-Effective Stress Theory (Murakami, 1988)

Wang et al. (2001) presented that damage in materials may be described as the following specific void or crack surfaces, the spacing between cracks, scalar representation of damage and or a general tensorial representation of damage. The paper developed methods to quantify the specific damaged surface area, the specific damaged surface area tensor, the damage tensor, the mean solid path among damaged surfaces, as well as the mean solid path tensor. Although the distribution of the damaged surface areas around the orientation is not uniform; it can be approximated by a second order fabric tensor (Wang et al., 2001):

$$ S (n) = \frac{S_w}{4\pi} (1 + S_{ij} n_i n_j) \quad (52) $$

where,

$ S (n) $ = the distribution density function of the specific damaged surface area;
\( n = \text{the unit normal to a unit sphere, and } n_i \text{ and } n_j \text{ are the directional cosines; } \)
\( S_v = \text{the specific damaged surface area; and } \)
\( S_{ij} = \text{the second order fabric tensor representing the orientational variation of the distribution.} \)

The distribution of the average spacing among the damaged surfaces is usually anisotropy. It can be represented by a second order fabric tensor (Wang et al., 2001):

\[
\lambda(n) = \lambda(1 + \lambda_{ij}n_in_j)
\]

where,
\( \lambda(n) = \text{function representing the orientational variations of the mean solid path among the damaged surfaces; } \)
\( n = \text{the unit normal to a unit sphere; } \)
\( n_i \text{ and } n_j = \text{the directional cosines; } \)
\( \lambda = \text{the mean solid path in all orientations among the damaged surfaces; and } \)
\( \lambda_{ij} = \text{the second order mean solid path tensor representing the orientational variations of the mean solid path.} \)

The damage tensor is symmetric in shape with six independent variables. For the quantification of the damage tensor, the area fractions of voids or cracks in six different orientations must to be evaluated and fitted into the following equation (Wang et al. 2001):

\[
\varphi_{ij} n_j = f_i
\]

where,
\( \varphi_{ij} = \text{the damage tensor; } \)
\( n_j = \text{orientational cosines; and } \)
\( f_i = \text{the observed area fractions in six orientations which are } (1,0,0), (0,1,0), (0,0,1), (1,1,1), (1, -1,1), (-1, -1, 1). \)

The paper also presented the relationship between the quantified damage parameters and subsequent applications in mechanical modeling. By obtaining quantifiable damage parameters of the three asphalt concrete mixes with known performance, Wang et al. indicated that the average spacing among the damaged surface, the comprehensive damage tensor quantity, and the
spacing size ratio might represent the damage state quite well. However, the specifically damaged surface area may not well represent the behavior of damaged materials.
CHAPTER 3 – METHODOLOGY

3.1 Introduction

The experimental program conducted in this study was designed to quantify the levels of damage in the dynamic complex modulus and the flow number tests and to characterize the microstructural properties of asphalt mixtures under loading. X-ray Computed Tomography (CT) was used to characterize the internal structure of the prepared asphalt specimens. Two conventional hot-mix asphalt (HMA) and six warm-mix asphalt (WMA) mixes were evaluated in this study.

3.2 Test Materials Preparation

A two-step process was adopted to prepare the specimens: (1) prepare HMA and WMA specimens according to the job mix formula, and (2) cut prepared specimens to the specified test dimensions.

3.2.1 Asphalt Mixtures Preparation

Four Superpave mixtures, inclusive of one conventional HMA and three WMA mixes were evaluated. Two processes (i.e., water foaming and Rediset™ additive) were used in the preparation of the WMA mixes. Rediset™ was added to the mixtures to reduce mixing temperatures and enhance the moisture resistance of the mix. Walker (2009) reported that this additive can lower mixing temperature by up to 33°C versus regular HMA. Water foaming is a method that separates the binder into two distinct components: a soft binder and a hard binder in the form of foam. In the first stage, the soft binder component, when mixed with the aggregates at approximately 110°C achieves full aggregate coverage (Walker 2009). The hard binder component then is mixed with the pre-coated aggregates in the form of foam. In the second
stage, cold water is injected into the heated, hard binder to produce large volume of foam in the mixtures. Reclaimed asphalt pavement (RAP), also used in the preparation of the mixes, had a content ranging from 15 to 30%. Table 3.1 presents the mixture proportions for the four mixes evaluated in this study; Table 3.2 provides a description of the eight test specimens.

3.2.2 Specimens Compaction

All cylindrical specimens for this study were compacted with the Superpave Gyratory Compactor (Figure 3.1). For the dynamic modulus specimens, loose mixtures were used as samples from plant-produced materials for a project located on US 171 in Louisiana. The samples were compacted in an on-site mobile laboratory, using a Superpave pneumatic gyratory compactor with no reheating, target to an air voids content of 7.0%. Dynamic modulus specimens were compacted to a 165-mm height and a 150-mm diameter. Test specimens were then cored and cut from the center of the gyratory specimens to result in 100-mm diameter by 150-mm high specimens. For the flow number specimens, loose asphalt mixtures, as samples from the same project were transported to the Louisiana Transportation Research Center (LTRC) laboratory, reheated, and compacted using a pneumatic Superpave gyratory compactor target for an air voids content of 7.0%. Flow number specimens were compacted to a 165-mm height and a 100-mm diameter. Test specimens were cut to result in 100-mm diameter by 150-mm high specimens.

3.3 The Dynamic Modulus Test

The dynamic modulus test requires an understanding of linear viscoelastic concepts. The following equation represents the continuous sinusoidal stress for the one-dimensional case (Ferry 1980):
\[ \sigma = \sigma_0 \sin(\omega t) \]  
(55)

\[ \omega = 2\pi f \]  
(56)

where,
\( \sigma_0 \) = the stress amplitude;
\( \omega \) = the angular velocity; and
\( f \) = the frequency.

The steady state strain can be written as:

\[ \varepsilon = \varepsilon_0 \sin(\omega t - \phi) \]  
(57)

\[ \phi = \frac{T_i}{T_p} \times 360^\circ \]  
(58)

where,
\( \varepsilon_0 \) = the strain amplitude;
\( \phi \) = the phase angle;
i = imaginary component of the complex modulus;
\( T_i \) = time lag (sec); and
\( T_p \) = period of sinusoidal loading (sec).

The phase angle relates to the time lags between the stress and strain. It is an indicator of the viscous and elastic properties of the material, for a pure elastic material, \( \phi = 0^\circ \)C, and for a pure viscous material, \( \phi = 90^\circ \)C. The complex modulus, \( E^* \), is defined mathematically as:

\[ E^* = E' + iE'' = \frac{\sigma_0 \cos \phi}{\varepsilon_0} + i \frac{\sigma_0 \sin \phi}{\varepsilon_0} \]  
(59)

where,
\( E' \) = storage modulus; and
\( E'' \) = loss modulus.

The dynamic modulus, \(|E^*|\), is the absolute value of the complex modulus. It is defined as the maximum (i.e., peak) dynamic stress \((\sigma_0)\), divided by the peak recoverable axial strain \((\varepsilon_0)\):
The dynamic modulus test was conducted in accordance with AASHTO TP 62. Four types of asphalt mixtures were tested in this study, using the Simple Performance Tester (SPT) (Figure 3.2). The test was conducted by applying a uniaxial sinusoidal (i.e., haversine) compressive stress to an unconfined cylindrical specimen. The haversine compressive stress was applied on each specimen to achieve a target vertical strain level of 100 microns in an unconfined test mode. The dynamic modulus test was conducted at four temperatures (4.4, 20.0, 37.8, and 54.4°C) and at six loading frequencies (0.1, 0.5, 1.0, 5.0, 10.0, and 25.0 Hz). Twenty-four temperature and frequency combinations were used to characterize the viscoelastic properties of the test specimens. The testing temperature was increased from the lowest to the highest level. At each temperature, testing began with the highest frequency of loading and proceeded to the lowest one. The rationale for selecting this frequency and temperature sequence is to minimize the damage to the specimen during testing. Measuring the dynamic modulus at various frequencies and temperatures allows one to evaluate the rutting and fatigue resistances of mixtures under different traffic conditions from high speed (25 Hz) to low speed (0.1 Hz).

Dynamic modulus at each temperature-frequency combination was recorded to allow for the development of master curves, used to assess and to compare the viscoelastic characteristics of the different mixes. Complex modulus experimental data were used to develop an extended master curve, in accordance with the procedure suggested by Bonaquist and Christensen (2005). This procedure employs the sigmoidal function to describe the rate dependency of the complex modulus master curve:

\[ |E^*| = \frac{\sigma_0}{\varepsilon_0} \]
\[
\log(E^*) = \delta + \frac{(Max - \delta)}{1 + e^{\beta + \gamma \left( \log(t) - \frac{\Delta E_o}{19.14714 \left( \frac{1}{T} - \frac{1}{29525} \right)} \right)}}
\]

(61)

where,
\(E^*\) = the complex modulus;
\(t\) = the loading time;
\(T\) = temperature in ° Rankine;
\(\delta, \beta\) and \(\gamma\) = fitting parameters; and
\(\text{and Max}\) = the limiting maximum modulus.

A typical instrumental setup for the dynamic modulus test used in this study is shown in Figure 3.3. Three Linear Variable Differential Transducers (LVDTs) with a range of 1mm were placed at 120° degrees on the sample surface to measure the vertical deformation. A vertical gauge length of 70 mm was maintained between two studs (gauge points). After a specimen was conditioned to the test temperature in the environmental chamber, a compressive stress (haversine) was applied on the sample to reach a targeted vertical strain level of 100 microstrains. The data of the last six cycles were collected and the dynamic modulus and phase angle were calculated, using the STP software.

### Table 3.1 Descriptions of the Test Specimens

<table>
<thead>
<tr>
<th>Mix Type</th>
<th>Specimen ID</th>
<th>Test</th>
<th>VTM (%)</th>
<th>Before Testing</th>
<th>After Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>WMA 15% RAP Foamed</td>
<td>1</td>
<td>Dynamic Modulus</td>
<td>7.3</td>
<td>7.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Flow Number</td>
<td>6.3</td>
<td>9.5</td>
<td></td>
</tr>
<tr>
<td>WMA 30% RAP Foamed</td>
<td>3</td>
<td>Dynamic Modulus</td>
<td>5.5</td>
<td>9.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Flow Number</td>
<td>5.8</td>
<td>5.8</td>
<td></td>
</tr>
<tr>
<td>Conventional 15% RAP</td>
<td>5</td>
<td>Dynamic Modulus</td>
<td>7.1</td>
<td>9.9</td>
<td></td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>Flow Number</td>
<td>7.0</td>
<td>7.5</td>
<td></td>
</tr>
<tr>
<td>WMA 15% RAP Rediset™</td>
<td>7</td>
<td>Dynamic Modulus</td>
<td>7.8</td>
<td>8.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>Flow Number</td>
<td>7.4</td>
<td>10.2</td>
<td></td>
</tr>
</tbody>
</table>
### Table 3.2 Job Mix Formula for the Four Mix Types

<table>
<thead>
<tr>
<th>Mixture Designation</th>
<th>Conventional 15% RAP</th>
<th>WMA 15% RAP Foamed</th>
<th>WMA 30% RAP Foamed</th>
<th>WMA 15% RAP Rediset&lt;br&gt;TM</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aggregate Blend (by mass)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5/8&quot; Stone</td>
<td>10%</td>
<td>11%</td>
<td>10%</td>
<td>11%</td>
</tr>
<tr>
<td>1/2&quot; Stone</td>
<td>52%</td>
<td>46%</td>
<td>38%</td>
<td>46%</td>
</tr>
<tr>
<td>RAP</td>
<td>15%</td>
<td>15%</td>
<td>30%</td>
<td>15%</td>
</tr>
<tr>
<td>Screens</td>
<td>10%</td>
<td>15%</td>
<td>15%</td>
<td>15%</td>
</tr>
<tr>
<td>Coarse Sand</td>
<td>7%</td>
<td>13%</td>
<td>7%</td>
<td>13%</td>
</tr>
<tr>
<td>Fine Sand</td>
<td>6%</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Additives</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2% (AZKO-NOBEL)</td>
</tr>
<tr>
<td>Binder Type</td>
<td>PG 70-22M</td>
<td>PG 70-22M</td>
<td>PG 70-22M</td>
<td>PG 70-22M</td>
</tr>
<tr>
<td>Asphalt</td>
<td>4.2</td>
<td>4.3</td>
<td>4</td>
<td>4.3</td>
</tr>
<tr>
<td>from RAP</td>
<td>0.8</td>
<td>0.7</td>
<td>1.4</td>
<td>0.7</td>
</tr>
<tr>
<td>Total (Design)</td>
<td>5.0</td>
<td>5.0</td>
<td>5.4</td>
<td>5.0</td>
</tr>
<tr>
<td>Anti-Strip</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>Design air void, %</td>
<td>3.4</td>
<td>3.3</td>
<td>3.6</td>
<td>3.4</td>
</tr>
<tr>
<td>VMA, %</td>
<td>14.5</td>
<td>14.5</td>
<td>14</td>
<td>14</td>
</tr>
<tr>
<td>VFA, %</td>
<td>78</td>
<td>78</td>
<td>75</td>
<td>76</td>
</tr>
<tr>
<td>Metric (U. S.) Sieve Blend Gradation</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>37.5 mm (1½ in)</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>25 mm (1 in)</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>19 mm (¾ in)</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>12.5 mm (½ in)</td>
<td>93</td>
<td>93</td>
<td>93</td>
<td>94</td>
</tr>
<tr>
<td>9.5 mm (⅛ in)</td>
<td>82</td>
<td>82</td>
<td>82</td>
<td>82</td>
</tr>
<tr>
<td>4.75 mm (No. 4)</td>
<td>50</td>
<td>50</td>
<td>53</td>
<td>54</td>
</tr>
<tr>
<td>2.36 mm (No. 8)</td>
<td>34</td>
<td>34</td>
<td>38</td>
<td>40</td>
</tr>
<tr>
<td>1.18 mm (No. 16)</td>
<td>27</td>
<td>27</td>
<td>28</td>
<td>29</td>
</tr>
<tr>
<td>0.6 mm (No. 30)</td>
<td>23</td>
<td>23</td>
<td>22</td>
<td>24</td>
</tr>
<tr>
<td>0.3 mm (No. 50)</td>
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<td>18</td>
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<td>18</td>
</tr>
<tr>
<td>0.15 mm (No. 100)</td>
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<td>8</td>
<td>10</td>
<td>9</td>
</tr>
<tr>
<td>0.075 mm (No. 200)</td>
<td>5</td>
<td>5</td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>G&lt;sub&gt;sb&lt;/sub&gt; Aggregate</td>
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<td>2.642</td>
<td>2.651</td>
<td>2.642</td>
</tr>
<tr>
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<td>2.386</td>
<td>2.397</td>
<td>2.386</td>
</tr>
<tr>
<td>G&lt;sub&gt;se&lt;/sub&gt;</td>
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<td>2.666</td>
<td>2.703</td>
<td>2.666</td>
</tr>
<tr>
<td>P absorb</td>
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<td>0.35</td>
<td>0.75</td>
<td>0.35</td>
</tr>
<tr>
<td>Dust/P eff</td>
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<td>1.09</td>
<td>1.22</td>
<td>1.07</td>
</tr>
</tbody>
</table>
Figure 3.1 Superpave Gyratory Compactor

Figure 3.2 Simple Performance Tester Used in This Study
3.4 The Flow Number Test

The flow number test was used to assess the permanent deformation characteristics of paving materials by applying a repeated dynamic load for several thousand repetitions on a cylindrical asphalt specimen (Bonaquist et al. 2003). In this study, the test was conducted according to the NCHRP Report 513 “Simple Performance Tester for Superpave Mix Design: First Article Development and Evaluation” (Bonaquist et al. 2003) at an effective temperature ($T_{\text{eff}}$) of 54°C and a stress level and 207 kPa. The Simple Performance Tester (SPT) was employed to test the prepared specimens for each mixture type (Conventional 15% RAP, WMA 15% RAP Foamed, WMA 30% RAP Foamed and WMA 15% RAP Rediset™). A repeated
dynamic loading cycle of 1.0 second in duration, consisting of a 0.1-second haversine load followed by 0.9-second rest period was applied on the cylindrical asphalt specimen with 100 mm in diameter and 150 mm in height. See Figure 3.4. Permanent axial strains are recorded throughout the test. However the flow number test is a destructive test, where a compressive stress was applied until the sample failed. Figure 3.5 shows the deformed sample after the test.

The “flow number” is defined as the starting point, or cycle number, at which tertiary flow occurs on a cumulative permanent strain curve obtained during the test. The cumulative permanent deformation curve can be divided into three regions: primary, secondary and tertiary (Figure 3.6). In the primary zone permanent deformation accumulates rapidly, while in the secondary zone the incremental deformations decrease, reaching a constant value. The permanent deformations again increase and accumulate rapidly in the tertiary zone. Figure 3.7 shows a typical plot of the rate of change in permanent strain versus loading cycle on a log–log scale.

Commonly, the flow number is determined by locating the lowest point in the strain rate versus cycle number, or the minimum value of the strain rate (Witczak et al. 2002). The procedure utilized to compute flow number and flow time consisted of three major steps: (1) numerical calculation of creep rate; (2) smoothing of the creep rate data; and (3) identifying the lowest point in the creep rate versus cycle number.

(1) Numerical calculation of creep rate was employed according to the procedure described in NCHRP Report 513. The derivative of the axial strain, with respect to cycle, was determined using the following equation:

\[
\frac{d\varepsilon_i}{dc} = \frac{\varepsilon_{i+\Delta c} - \varepsilon_i}{\Delta c}
\]  

(62)
where,
\[
\frac{d\varepsilon_i}{dc} = \text{Rate of change of axial strain with respect to cycle;}
\]
\[
\varepsilon_i = \text{Strain at } i \text{ cycle;}
\]
\[
\varepsilon_{i+\Delta c} = \text{Strain at } i + \Delta c \text{ cycle; and}
\]
\[
\Delta c = \text{Sampling interval.}
\]

(2) The values calculated from Equation 8 were plotted against the number of cycles. As shown in Figure 3.8, cycle numbers were plotted on the x-axis and derivatives of the axial strain were plotted on the y-axis. A 2\textsuperscript{nd} order polynomial model was fitted, using Microsoft Excel to smooth the rate of change of strain versus the numbers of cycles curve as shown in Figure 3.8.

(3) The flow number is the resultant cycle number at which the minimum value of the smoothed creep rate occurs. In this study, flow number was reported as the nearest cycle number corresponding to the lowest value of the rate of change of axial strain. The polynomial fitted curve shown in Figure 3.9 is mathematically represented by the following equation:

\[
y = A + Bx + Cx^2
\]

(63)

By differentiating Equation 63 with respect to \(x\), one may get:

\[
\frac{dy}{dx} = B + 2Cx
\]

(64)

The cycle number of the lowest strain rate (Flow Number) was obtained by setting Equation 64 equals to:

\[
\frac{dy}{dx} = B + 2Cx = 0
\]

(65)
Figure 3.4 General Description of the Flow Number Test

Figure 3.5 Deformed Specimen after the Flow Number Test
**Figure 3.6** Relationship between Total Cumulative Plastic Strain and Loading Cycles

**Figure 3.7** Typical Plot of the Rate of Change in Permanent Strain versus Loading Cycles for the Repeated Load Test
3.5 X-Ray Computed Tomography

X-ray computed tomography (CT) is an innovative, nondestructive technique used for obtaining digital information on the three-dimensional internal microstructure of solid materials. Different phases in solid materials may be distinguished using x-ray CT. In recent years, this imaging technique has been utilized to characterize the microstructure of asphalt mixes (Shashidhar 1999; Wang et al. 2001; Masad et al. 2002, 2004).

Figure 3.9 shows the components of x-ray CT. It consists of a source, a detector and a stage between in which test specimen is placed. The planar x-ray beams are shot to the specimen by the high energy source. The specimen stage rotates a full rotation with a constant speed during the scanning, and moves down at a specific distance. The interval between two
subsequent x-ray CT images is typically 1 mm and scanning time of each x-ray CT images is 2 minutes (see Figure 3.10). The intensity of the x-ray beams changes after going through the test specimen due to the difference in density of the constituents of the mix (i.e., air voids, aggregate, and mastic). The higher density material will result in a higher attenuation of x-rays. The linear attenuations of a substance in specimen along various directional paths are recorded by the detector. In an x-ray CT image, the low density material is represented by means of a darker color, while the high density material is represented by a brighter color (on a 256 gray level scale).

In this study, the x-ray system in the Advanced Characterization of Infrastructure Materials (ACIM) laboratory at Texas A&M University was used to scan the test specimens (Figure 3.11). Eight asphalt specimens were scanned before and after tests. 145 cross-sectional images per specimen obtained as shown in Figure 3.10. The setup includes two separate x-ray systems; the micro-focus and the mini-focus. The micro-focus system consists of a 225 kV x-ray source and an image intensifier detector, while the mini-focus has a 350 kV x-ray source and a linear detector. Due to the limited power of the micro-focus system, it is more applicable to scan small asphalt mixture specimens with better resolution. The mini-focus is more applicable to scan large asphalt mixture specimens with an adequate resolution of 0.17 mm/pixel. In this study, the mini-focus was used to scan the test specimens. The three example cross-sectional images from each depth (top third, middle third and bottom third) of all the specimens are presented in Appendix C.
**Figure 3.9.** Components of X-ray CT System (Masad et al. 2002)

**Figure 3.10** X-ray Multi-Depth Testig of HMA Specimen
Digital image processing is an effective approach for gathering information using a computer-based system (Elseifi et al. 2008). This method processes the digital images to enhance some desired features and then analyzes the image based on the differences in color of the features. Masad et al. (1999) used digital image analysis to quantify aggregate orientation, gradation, and aggregate segregation in HMA. In addition, the researchers employed this methodology to reveal the influence of compaction levels in the SuperPave Gyratory Compactor as well as the field on the internal structure of HMA.

As shown in Figure 3.12, digital image analysis consists of three major steps in this study: 1) image acquisition, 2) image processing and 3) image analysis. 2D digital images were captured at different depths within the test specimen to determine the air void distributions along
the sample. Macros that were developed in the Image-Pro® software (1999) were used to conduct the analysis. Image-Pro Plus is a powerful software program that includes extensive enhancement and measurement tools and allows writing application-specific macros to automate the processing and analysis procedures (Elseifi et al. 2008).

X-ray presents the different constituents within the specimen in 256 distinct levels of gray intensity. In order to select the proper gray level threshold for a given specimen, a trial and error process was employed. First, an arbitrary threshold value was picked up and the air voids value was compared to the laboratory measured value. For example, a threshold value was chosen at 100 and the software gives 8% air voids, based on that value. Meanwhile, the laboratory measured percent air void is 6%, which is less than the predicted value (8%). Therefore, the threshold was reduced until the measured and the predicted values matched. By choosing the proper gray level, the air voids phase was thresholded from the other constituents (Mastic and aggregates).

For HMA, it is difficult to isolate aggregates from mastic by choosing a threshold value. The reason is that the aggregates within the specimen have various colors depending on their mineral composition (Masad et al. 1999). Therefore, many aggregates will have very similar gray color with the mastic. To avoid this problem, the threshold value for aggregates and mastic were chosen as the same (125-255) to provide the necessary contrast to identify air voids. As shown in Figure 3.13 and by using this method, the original X-ray image was converted to an image, which only had black (air void) and white phases (aggregates and mastic).

Using the processed images similar to the one shown in Figure 3.13b, the total area of the air voids in the image was calculated by using a macro developed in Image-Pro® software. Determining the percentage air voids in an asphalt specimen relies on matching the calculated
percent air void using Equation 12 with the measured laboratory percent air voids. The total percent of air voids (\(\%AV_t\)) in a test specimen was calculated using Equation 66, while the average percent air void in an image (\(\%AV_i\)) and the average radius of air voids (\(R\)) were calculated as given in Equations 67, and 68, respectively (Masad et al. 2002):

\[
%AV_t = \frac{1}{N} \sum_{1}^{N} %AV_i \quad (66)
\]

\[
%AV_i = \frac{A_{iv}}{A_t} \quad (67)
\]

\[
R = \sqrt[10]{\frac{A_{iv}}{\pi n}} \quad (68)
\]

where,

\(N\) = the number of images;

\(A_{iv}\) = the total area of the air voids an image;

\(A_t\) = the total cross-sectional area of an image; and

\(n\) = the number of the air voids in an image.

A scalar variable \(\xi\) was defined to quantify the levels of irreversible damage in HMA (Tashman et al. 2005, Kachanov 1986):

\[
\xi = 1 - \frac{\overline{A}}{A_o} \quad (69)
\]

where,

\(A_o\) = the initial area of the undamaged section; and

\(\overline{A}\) = the effective cross-sectional area in the current damaged state.

The variable \(\xi\) describes a positively monotonically increasing function, which is equal to 0 for the undamaged material and equal to 1 for the totally damaged material. The effective cross-sectional area of the specimen was determined by calculating the total area of air voids in a CT image, using image analysis and then subtracting the areas of air voids from the total cross-sectional area of the CT image.
Figure 3.12 Three Major Steps in Image Analysis Technique
**Figure 3.13** (a) Raw X-ray Image and (b) Thresholded X-ray Image
CHAPTER 4 – RESULTS AND ANALYSIS

4.1 Introduction

In this chapter, results obtained from mechanical and x-ray tests of HMA and WMA mixtures are presented and analyzed. The dynamic modulus and flow number (FN) test results were used to construct master curves and to evaluate the mix resistance to permanent deformation, respectively. Air voids distribution along the depth of the eight specimens before and after testing were measured and analyzed with the purpose of quantifying the damage taking place during mechanical testing. Specimen description was previously presented in Table 3.1.

4.2 Mechanical Responses of Evaluated Mixes

4.2.1 Dynamic Modulus Test Results

Figures 4.1 (a to d) present the variation of the dynamic modulus with temperatures and frequencies for the four mixtures evaluated in this study. As shown in these figures, the measured $E^*$ increased when the frequency increased and when the temperature decreased. The $E^*$ isotherms exhibited a straight-line shape at low temperatures (-10°C and 4°C) indicative of their elastic behavior at low temperatures. At intermediate and high temperatures (25°C, 37.8°C, and 54.4°C), the $E^*$ isotherms exhibited a curved shape, which is indicative of their viscoelastic behavior at intermediate and high temperatures. In addition, the $E^*$ isotherms for the four mixtures were similar, which indicates that WMA mixtures had similar characteristics to conventional HMA.

The phase angle is a fundamental indicator of the viscous properties of the mixtures. At high temperatures, a lower phase angle demonstrates that the behavior of the mixture is more elastic, and therefore the mix provides better resistance to rutting at high temperatures. Figures
4.2 (a to d) illustrate the phase angles for four mixtures at different test frequencies and temperatures. At low temperatures (-10°C and 4°C), the phase angles of all mixtures increased with the increase in temperature and the decrease in frequency. However at 54.4°C, the phase angles increased to a peak value first and then decreased with an increase in frequency. This general trend indicates that at low frequencies and high temperatures, all the WMA and HMA mixtures became more viscous and more energy was stored in plastic deformation, which caused deformation in the specimens.

Figure 4.1 Dynamic Modulus of the Four Mixtures at Different Temperatures and Frequencies
(Figure 4.2 continues)

(b)

(c)
Figure 4.4 Phase Angle of Four Mixtures at Different Temperatures and Frequencies
(Figure 4.5 continues)

(b) WMA 30 RAP Foamed (Specimen 4)

(c) WMA 15RAP Rediset (Specimen 7)
Complex modulus experimental data were used to develop an extended master curve in accordance with the procedure suggested by Bonaquist and Christensen (2005). This procedure employs the sigmoidal function to describe the rate dependency of the complex modulus master curve:

\[
\log(E^*) = \delta + \frac{\left(\text{Max} - \delta\right)}{1 + e^{\beta + \gamma \left(\frac{\log(t)}{\Delta E_a} - \frac{1}{T} + \frac{1}{29.525}\right)}}
\]

where,
E* = complex modulus;
t = loading time;
T = temperature in ° Rankine;
\(\delta, \beta, \gamma\) = fitting parameters; and
Max = limiting maximum modulus.
After fitting the sigmoidal model to the measured data, this model was used to extend the master curve to low and high frequencies not attainable in the test. The constructed master curves are presented in Figure 4.3. As shown in Figure 4.3, the master curves of the WMA and conventional mixtures prepared with 15% RAP were similar. On the other hand, the use of 30% RAP resulted in increased mix stiffness, when compared to conventional mixtures at high temperatures. Results presented in this figure indicate that WMA mixtures had similar viscoelastic characteristics to conventional HMA. In addition, the increase in RAP content improved the mix stiffness at high temperatures.

![Figure 4.7 Master Curves Comparison of the Evaluated Mixes](image)

**Figure 4.7** Master Curves Comparison of the Evaluated Mixes
4.2.2 Flow Number Test Results

The flow number is defined as the number of cycles at which tertiary flow occurs on a cumulative permanent strain versus number of cycles curve. The accumulation of permanent strain during the testing process is presented in Figure 4.4 for the four evaluated mixes.

![US171 - Flow Number @ 54.4°C](image)

**Figure 4.8** Strain Accumulation during the Flow Number Test

The calculation procedure of the flow number is described in Figures 4.5 and 4.6. Figure 4.5 shows the variation of the creep rate during the flow number testing process. The location of the point of minimum creep rate was determined by fitting a second order polynomial as shown in Figure 4.6. The minimum creep rate was then determined by solving for the root of the derivative of the fitted second order polynomial.
**Figure 4.9.** Variation of Creep Rate versus Loading Cycles in the Flow Number Test

**Figure 4.10** Identification of the Minimum Creep Rate Point in the Flow Number Test (*note: solid curve represents polynomial fitted curve*)
Figure 4.7 presents the flow number (FN) test results comparison for the four mixtures evaluated in this study. The greater the flow number the higher the mixture’s resistance to permanent deformation. The use of WMA technology and the increase in RAP content improved the mixture resistance to permanent deformation, compared to conventional HMA.

![Flow Number Comparison of Evaluated Mixes](image)

**Figure 4.11** Flow Number Comparison of Evaluated Mixes

4.3 Air Voids Distribution

Previous research has shown that the homogeneity of air voids distribution strongly influences the mechanical responses of HMA in the laboratory (Thyagarajan et al. 2010). Figure 4.8 and 4.9 present the air voids distribution along the depth of the eight specimens before and after testing. As shown in Figures 4.8 and 4.9, air voids distributions were distinctively different.
for the dynamic modulus and flow number specimens. For the dynamic modulus specimens, a relatively homogeneous air voids distribution was observed along the specimen height with higher air voids content in the top or bottom parts of the specimen. Although past research has reported a “C” shape for the air voids distribution (Masad et al. 2002), a partial C-shape was observed in this study, possibly due to the uneven cutting of the top and bottom parts of the specimen. To ensure uniformity of air voids, an equal length should be cut from both edges of the specimen. In contrast, the flow number specimens were highly heterogeneous with low air voids concentration in the bottom and the top thirds and a significant greater air voids concentration in the middle third of the specimen.

The discrepancy in air voids distribution is due to the different specimen sizes used in the compaction of the dynamic modulus and the flow number specimens and the reheating of the mixtures used in the preparation of the flow number specimens. Although both sets of specimens were compacted to a target air voids content of 7.0%, dynamic modulus specimens were compacted to a 150-mm diameter and flow number specimens were compacted to a 100-mm diameter. As shown in Figures 4.8 and 4.9, the gyratory compaction process results in a more uniform air voids distribution in the center of the specimen than around its circumference, which may explain the relatively more uniform air voids distribution observed in the dynamic modulus specimens. The effect of the foaming agent and WMA additive in facilitating the compaction process may have also been reduced upon reheating the asphalt mixtures.

The concentration of air voids in the top, middle, and bottom thirds of Specimens 5 and 6 is quantified in Figure 4.10 before and after testing. The values presented in this figure confirm the homogeneous air voids distribution in the dynamic modulus specimen and the heterogeneous air voids distribution in the flow number specimen. It also indicates that the air voids contents in
the dynamic modulus specimen remained relatively unchanged after testing, while a significant increase in air voids occurred in the flow number specimen after testing.

**Figure 4.12** Air Voids Distribution along the Depth for Specimens 1 to 4
Figure 4.13  Air Voids Distribution along the Depth for Specimens 5 to 8
Figure 4.14  Air Voids Content along Specimen Height for (a) Specimen 5 (dynamic modulus) and (b) Specimen 6 (flow number)
Figures 4.11 and 4.12 compare the three-dimensional air voids distributions for the WMA, prepared with foamed asphalt and 30% RAP (i.e., Specimens 3 and 4). As shown in these figures, while the flow number specimen experienced a significant increase in air voids during testing, the air voids distribution in the dynamic modulus specimen appeared identical before and after testing. This was confirmed by comparing the voids in the total mix (VTM) before and after testing as presented in Table 3.1 for the eight specimens and by the concentrations of air voids previously presented in Figure 4.10.

**Specimen 3 (Dynamic Modulus Test)**

![Three-Dimensional Air Voids Distribution along Specimen Depth](image)

(a) Before Testing  
(b) After Testing

**Figure 4.15** Three-Dimensional Air Voids Distribution along Specimen Depth
Specimen 4 (Flow Number)

(a) Before Testing  (b) After Testing

Figure 4.16 Three-Dimensional Air Voids Distribution along Specimen Depth

4.4 Damage Analysis

Figure 4.13 presents the visualization of the dynamic modulus and flow number specimens prior to and after testing (Specimens 5 and 6). The visualization of all the specimens was presented in Appendix A of this document. As shown in Figure 4.13, the general visualization of the dynamic modulus specimen before and after testing appears to be similar, indicating that little or no damage occurred during testing. In contrast, the specimen used in the flow number test experienced significant volume change especially in the middle third indicative of damage taking place during testing in a localized area of the specimen.
(a) Visualization of the Specimen Prior to and after Testing (Specimen 5 – dynamic modulus test)

(b) Visualization of the Specimen Prior to and after Testing (Specimen 6 – flow number)

**Figure 4.17** Visualization of Conventional 15% RAP Specimens Prior to and after Testing
Figure 4.14 and Figure 4.15 present the comparison of damage between Conventional HMA (15% RAP) and WMA (15% RAP) after dynamic modulus test and flow number test, respectively. Figure 4.14 evidenced that the little or no damage occurred in both Conventional HMA (15% RAP) and WMA (15% RAP) specimens during the dynamic modulus testing. In addition, as shown in Figure 4.15 the deformed area in specimen 2 and 6 are similar to each other. These specimens demonstrated that WMA mixtures have similar viscoelastic characteristics to conventional HMA.

![Figure 4.18 Visualization of the Specimens after Dynamic Modulus Testing (a: Specimen 1; b: Specimen 5)](image-url)
Figure 4.19  Visualization of the Specimens after Flow Number Testing (a: Specimen 2; b: Specimen 6)

The level of damage taking place during testing was quantified through the damage parameter, \( \xi \), previously defined in Equation 3.11. Figures 4.16 and 4.17 illustrate the calculated damage parameters for the dynamic modulus and flow number specimens after testing. As shown in Figures 4.16, little damage occurred during the dynamic modulus testing. In addition, the damage parameter was practically uniform throughout the height of the test specimens. In contrast, the damage experienced in the flow number test was not uniform with respect to the specimen height, with little to no damage occurring in the top and bottom thirds of the specimens and most of the damage taking place in the middle third of the specimens. These results confirm that the assumption of no damage taking place in the dynamic modulus test is valid. In contrast, the damage taking place in the flow number test is heterogeneous with most of the damage
occurring in the middle third of the specimen. These findings agree with Tashman et al. (2005), who reported that damage in HMA is a localized phenomenon occurring in a critical location in the specimen, due to the heterogeneity of the mix.

**Figure 4.20** Levels of damage in the dynamic modulus test
Figure 4.21 Levels of Damage in the Flow Number Test
CHAPTER 5 – CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary and Conclusions

This study quantified the level of damage in the dynamic complex modulus and flow number tests, using x-ray computed tomography and thereby characterized the microstructural properties of eight asphalt mixtures under loading. Among those eight mixtures two are WMA mixtures with 15% RAP content treated with water foaming, two are WMA mixtures with 30% RAP content treated with water foaming, two are WMA mixtures with 15% RAP content and Rediset™ additive added, the rest two are conventional HMA mixtures. The dynamic modulus test and flow number test were designed to evaluate the laboratory performance of these WMA and HMA mixtures. An x-ray CT test was performed on the prepared specimens before and after tests to characterize the internal structure. Based on the results of the experimental program, the following conclusions may be drawn:

- The damage taking place in the dynamic modulus test is minimal and homogeneous, while the damage taking place in the flow number test is significant and heterogeneous.
- Specimen preparation may significantly influence the air voids distribution in HMA. To achieve relatively homogeneous air voids distribution, 150-mm in diameter by 165-mm in height specimens should be prepared, cored, and evenly cut from both sides of the specimen.

5.2 Future Research and Recommendations

- In a future study, the micromechanical Finite Element (FE) method may be used to predict the behavior and failure mechanisms in asphalt mixtures. Based on the approach of digital image analysis presented in this study, FE models can may developed to simulate the three-
dimensional behavior and response of the mix to loading in both dynamic complex modulus
test and flow number test.

- The damage occurring within the asphalt mixtures during testing process was quantified by
  use of digital image analysis techniques in this study. The analysis of damage propagation in
  these mixtures during testing may be achieved by combining FE methods and digital image
  analysis techniques.

- WMA mixtures treated with other processes and different RAP contents should be evaluated by
  using the approaches presented in this study. The damage taking place in other laboratory test
  methods can also be quantified using the methodology presented in this study.
REFERENCES


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APPENDIX A: VISUALIZATION OF THE SPECIMEN PRIOR TO AND AFTER TESTING
Figure A1 Visualization of the Sample 3 Prior to and after Testing

Figure A2 Visualization of the Sample 4 Prior to and after Testing
Figure A3 Visualization of the Sample 7 Prior to and after Testing

Figure A4 Visualization of the Sample 8 Prior to and after Testing
APPENDIX B: X-RAY CT IMAGES OF SPECIMENS PRIOR TO AND AFTER TESTING
Figure B1 Horizontal cross-sectional x-ray CT image of Sample 1
Figure B2 Horizontal cross-sectional x-ray CT image of Sample 2
Figure B3 Horizontal cross-sectional x-ray CT image of Sample 3
(a) Before testing
(b) After testing

**Figure B4** Horizontal cross-sectional x-ray CT image of Sample 3
Figure B5 Horizontal cross-sectional x-ray CT image of Sample 6
Figure B6 Horizontal cross-sectional x-ray CT image of Sample 5
Figure B7 Horizontal cross-sectional x-ray CT image of Sample 7
Figure B8 Horizontal cross-sectional x-ray CT image of Sample 8
VITA

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