Development of a Flexible Pavement Database for Local Calibration of the MEPDG

Part 2 Evaluation of ODOT SMA Mixtures

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There has been some reluctance on the part of some in Oklahoma to use SMA mixtures. There are several factors that could be involved in the slow acceptance of SMA mixtures in Oklahoma. These factors are 1) the extra expense associated with the higher binder contents and better quality aggregates required, 2) a lack of data indicating that SMA mixtures perform substantially better than conventional Superpave mixtures and 3) a lack guidance on thickness design benefits, including appropriate input parameters for the MEPDG. The objectives this study are to evaluate the performance of SMA mixes compared to S-4 mixes and to determine the performance benefits. Testing included Hamburg Rut Tests and dynamic modulus testing.

Hamburg rut testing indicated that SMA resists permanent deformation better than ODOT S-4 mixes made with the same source and grade of asphalt cement. Both measured and predicted dynamic modulus of SMA was less than ODOT S-4 mixes. The Asphalt Institutes fatigue equation indicated longer fatigue life for SMA compared to S-4 mixes. MEPDG prediction models contradict these findings.

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(MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS				
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
LENGTH				
in	Inches	25.4	millimeters	mm
ft	Feet	0.305	meters	m
yd	Yards	0.914	meters	m
mi	Miles	1.61	kilometers	km
AREA				
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	Acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	Gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
TEMPERATURE (exact degrees)				
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
FORCE and PRESSURE or STRESS				
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS				
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	Liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	Lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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CHAPTER 1

INTRODUCTION

PROBLEM STATEMENT

This report is the second part of a two part study by the University of Oklahoma (OU) and Oklahoma State University (OSU). The goal of the OSU portion of this study, reported herein, is to evaluate the performance of stone matrix asphalt (SMA) compared to conventional ODOT S-4 mixes.

There has been some reluctance on the part of some in Oklahoma to use SMA mixtures. There are several factors that could be involved in the slow acceptance of SMA mixtures in Oklahoma. These factors are 1) the extra expense associated with the higher binder contents and better quality aggregates required, 2) a lack of data indicating that SMA mixtures perform substantially better than conventional Superpave mixtures and 3) a lack guidance on thickness design benefits, including appropriate input parameters for the Mechanistic-Empirical Pavement Design Guide (MEPDG).

The goal of this combined OU OSU project is to develop a flexible pavement database and to populate this database with data required for calibration of the new MEPDG) design criteria. Design of pavements in accordance with the new MEPDG requires several types of data, namely material (soil, aggregate, asphalt) properties, pavement structural characteristics, traffic data and environmental conditions. Successful implementation of the new MEPDG will require a comprehensive and user friendly database and an assessment of the database through local material calibration activities. The results from this project are expected to provide pavement design professionals with appropriate tools and a better understanding of how the new MEPDG will allow for optimization of materials, evaluate and incorporate new materials into designs, and evaluate the impacts of anticipated heavier loads and new axle configurations on pavement performance in Oklahoma. Because of limited scope, this proposal addresses only material (asphalt, soil and aggregate) properties. Traffic and environmental aspects (e.g., changes in resilient modulus due to seasonal variations in in-situ moisture, parameters for soil-water characteristic curves) may be addressed in future under a separate study.

The MEPDG uses a hierarchical approach for materials characterization (1). The first level of material characterization provides the highest design reliability with each succeeding level being a drop in design reliability. The first or highest level entails measured material properties. For hot-mix asphalt it is dynamic modulus and for the binder it is shear modulus and phase angle. Unbound soils and aggregate base use resilient modulus, Poisson’s ratio, unit weight, and lateral earth pressure coefficient. In addition, hydraulic conductivity is needed for climatic condition modeling (2). Design involving stabilized subgrade and aggregate base generally use resilient modulus (or elastic modulus), Poisson’s ratio and unit weight. The second level generally entails calculation of the above material properties from index properties such as gradation and void content. The third or lowest level typically uses default values.

This study is a collaborative project between OSU and OU. The OU team will be responsible for the soil, aggregate, and asphalt binder part of this study and the OSU team will be primarily responsible for the asphalt mix part.

SCOPE

As noted above, this study is limited to material (soil, aggregate and asphalt) properties only. Traffic data and environmental conditions are not addressed. As discussed subsequently, while most material property data are already available from previous studies at OSU and OU, laboratory tests on selected binders and asphalt mixes will be performed for enhanced application of the database.

This project will be pursued in two phases. The first phase will involve development of a database populated with material (soil, aggregate and asphalt) properties. Calibration of local materials and actual design applications using the new MEPGD will be pursued in Phase II. Funding for Phase II may be sought jointly from ODOT and the Oklahoma Transportation Center.

OBJECTIVES

The objective of this project is to develop a flexible pavement database and to populate this database with local data required for local calibration of new MEPDG design criteria. By providing local material properties and local calibration, ODOT would be able to obtain near level 1 reliability for level 2 or 3 material input costs. A second, and primary objective of the OSU study, is to evaluate the performance of SMA mixes compared to S-4 mixes and to determine performance benefits. The objective of this project would be met by carrying out the following tasks.

STUDY TASKS

Task 4a HMA MEPDG Input Properties

Oklahoma State has an existing database of dynamic modulus data from ODOT S-3 and S-4 mixtures (3). The database will be populated with the existing data. However, S-2 and SMA mixtures need to be evaluated. Aggregate was shown to not have a significant effect on dynamic modulus; therefore, limited testing is all that is necessary to populate the data base.

Up to four additional S-2 mixtures would be sampled and tested for E* using PG64-22 asphalt cement. The results would be combined with the data from the previous study (3) and default E* values would be developed for ODOT S-2 mixtures. ODOT decided that this task would not be necessary as S-2 mixes were rarely used.

Task 4b SMA Mixtures

There has been some reluctance on the part of some in Oklahoma to use SMA mixtures. There are several factors that could be involved in the slow acceptance of SMA mixtures in Oklahoma. These factors are 1) the extra expense associated with the higher binder contents and better quality aggregates required, 2) a lack of data indicating that SMA mixtures perform substantially better than conventional Superpave mixtures and 3) a lack guidance on thickness design benefits, including appropriate input parameters for the MEPDG.

To overcome this reluctance to embrace SMA, the following would be performed under this subtask. SMA mixtures would be made from high quality aggregates from Oklahoma, possibly rhyolite, limestone and granite. S-4 mixtures would be made with the same aggregates or selected from previously available data base for comparison. All mixture would be made with PG 76-28 asphalt cement. To evaluate the performance properties of the mixtures, samples would be tested for dynamic modulus and Hamburg rut resistance.

CHAPTER 2

BACKGROUND

NEED FOR THE MEPDG

The various editions of the *AASHTO Guide for Design of Pavement Structures* have served well for several decades; nevertheless, many serious limitations exist for their continued use as the nation's primary pavement design procedures. Listed below are some of the major deficiencies of the existing design guide (4):

- Traffic loading deficiencies
- Rehabilitation deficiencies
- Climatic effects deficiencies
- Subgrade deficiencies
- Surface materials deficiencies
- Base course deficiencies
- Truck characterization deficiencies
- Construction and drainage deficiencies
- Design life deficiencies
- Performance deficiencies
- Reliability deficiencies

GENERAL INPUT REQUIREMENTS

The guide for the Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures (referred to hereinafter as MEPDG) was developed to provide the highway community with a state-of-the-practice tool for design of new and rehabilitated pavement structures. The MEPDG is a result of a large study sponsored by AASHTO in cooperation with the Federal Highway Administration and was conducted through the National Cooperative Highway Research Program (NCHRP) [NCHRP-1-37A]. The final product is design software and a user guide. The MEPDG is based on comprehensive pavement design procedures that use existing mechanistic-empirical technologies. MEPDG software is temporarily available for trial use on the web. The software can be downloaded from www.trb.org/mepdg. The software is described as a user oriented computational software package and contains documentation based on MEPDG procedures (4). The MEPDG employs common design parameters for traffic, subgrade, environment, and reliability for all pavement types (4).

Input parameters for the MEPDG are grouped into five areas: project information, design information, traffic loadings, climatic data and structural data. The structural data is separated into two sections, one on structural layers and one on thermal cracking (2). The focus of this study is on the input data required in the *Layers* section for HMA mixtures.

Analysis

The analysis parameters are performance criteria which the pavement under consideration is expected to fulfill. The parameters (distresses) listed on the guide for flexible pavements are terminal IRI, HMA surface down cracking (longitudinal cracking), HMA bottom up cracking (alligator cracking), HMA thermal fracture, chemically stabilized layer fatigue fracture, permanent deformation for total pavement and permanent deformation for AC only. The user can use the default values or may enter limiting values for these parameters. MEPDG predicts the values for the aforementioned analysis parameters at the end of the design period and compares them with the limiting values. If the predicted values are less than the limiting values, the design is considered as ‘’pass’’ if not ‘’fail’’.

Traffic

Traffic data includes initial two-way Annual Average Daily Truck Traffic (AADTT), number of lanes in design direction, percent of trucks in design direction, percent of trucks in design lane and operational speed. Other traffic inputs required, such as traffic volume adjustment factors, axle load distribution factors and general traffic inputs are also incorporated in the traffic part of the guide.

Climate

The climate part of the MEPDG guide has a sophisticated climatic modeling tool called the Enhanced Integrated Climatic Model (EICM). This modeling tool is used to model temperature and moisture within each pavement layer and the subgrade. The EICM model considers hourly climatic data from weather stations across the country (temperature, precipitation, solar radiation, cloud cover, and wind speed).The pavement layer temperature and moisture predictions from the EICM are calculated hourly over the design period and used in various ways to estimate material properties for the foundation and pavement layers throughout the design life.

Layers

The input requirement for HMA layers uses a hierarchical approach with three levels of materials characterization. The first level provides the highest design reliability and each succeeding level is a drop in design reliability. Within each level there are three input screens, *Asphalt Mix, Asphalt Binder* and *Asphalt General*. Any level of reliability may be used with any layer in the pavement system. However, the same level of reliability is required for each input screen within a pavement layer (4).

Asphalt Mix Screen

The *Asphalt Mix* screen allows three levels of reliability; however, the required inputs are the same for reliability levels 2 and 3. For level 1 reliability, dynamic modulus was originally required at a minimum of three temperatures and three frequencies. The laters version of the software requires dynamic modulus at five temperatures and six frequencies. One of the temperatures must be greater than 51.7^{o}C (125^{o}F). For level 2 and 3 reliability, the dynamic modulus is calculated using a predictive equation based on mix properties. The required mix properties for the *Asphalt Mix* screen are the aggregate percent retained on the 3/4 inch, 3/8 inch and No. 4 sieves and the percent passing the No. 200 sieve (4).

Asphalt Binder Screen

The *Asphalt Binder* screen allows three levels of reliability; however, the required inputs are the same for reliability levels 1 and 2. For level 1 or 2 reliability, the shear modulus (G*) and phase angle (δ) for the binder are required from the dynamic shear rheometer (DSR) test. The DSR parameters are required at a minimum of three temperatures. For level 3 reliability the grading of the asphalt binder is all that is required. The MEPDG allows the use of PG graded binders, viscosity (AC) graded binders or penetration graded binders (4).

Asphalt General Screen

The *Asphalt General* screen allows three levels of reliability; however, the required inputs are the same for all three reliability levels. The *Asphalt General* screen is separated into four sections: *General, Poisson's Ratio, As Built Volumetric Properties* and *Thermal Properties*. The *General* section requires the reference temperature for development of master curves for dynamic modulus. The default value is 70^{o}F but other temperatures may be entered. The *Poisson's Ratio* section allows the user to select the default value of 0.35 for HMA, enter a user defined value or allow the software to calculate Poisson's ratio using a predictive equation. *As Built Volumetric Properties* include volume binder effective (Vbe), air voids and compacted unit weight. Default values are 11.0%, 8.5% and 148 pcf, respectively. Required *Thermal Properties* are thermal conductivity and heat capacity. Either user defined or default values may be entered. Default values are 0.67 BTU/hr-ft-^{o}F for thermal conductivity and 0.23 BTU/lb-^{o}F for heat capacity (4).

MASTER CURVES

To perform a level 1 analysis using the MEPDG, dynamic modulus at a minimum of three test temperatures and three frequencies are required (4). AASHTO TP 62-03 recommends six frequencies and five test temperatures. The dynamic modulus values at different frequencies are used by the MEPDG to develop master curves. According to the user manual for the MEPDG (4), the stiffness of HMA at all levels of temperature and time rate of load is determined from a master curve constructed at a reference temperature (generally taken as 70°F). Master curves are constructed using the principle of time-temperature superposition. The data at various temperatures are shifted with respect to time until the curves merge into a single smooth function. The master curve of dynamic modulus as a function of time formed in this manner describes the time dependency of the material. The amount of shifting at each temperature required to form the master curve describes the temperature dependency of the material. The greater the shift factor, the greater the temperature dependency (temperature susceptibility) of the mixture. Figure 1 shows the results of a dynamic modulus test on an HMA sample and how the data at each temperature can be shifted to form a smooth curve. Figure 2 shows the resultant master curve at a reference temperature of 70^{o} F (21.1^{o} C).

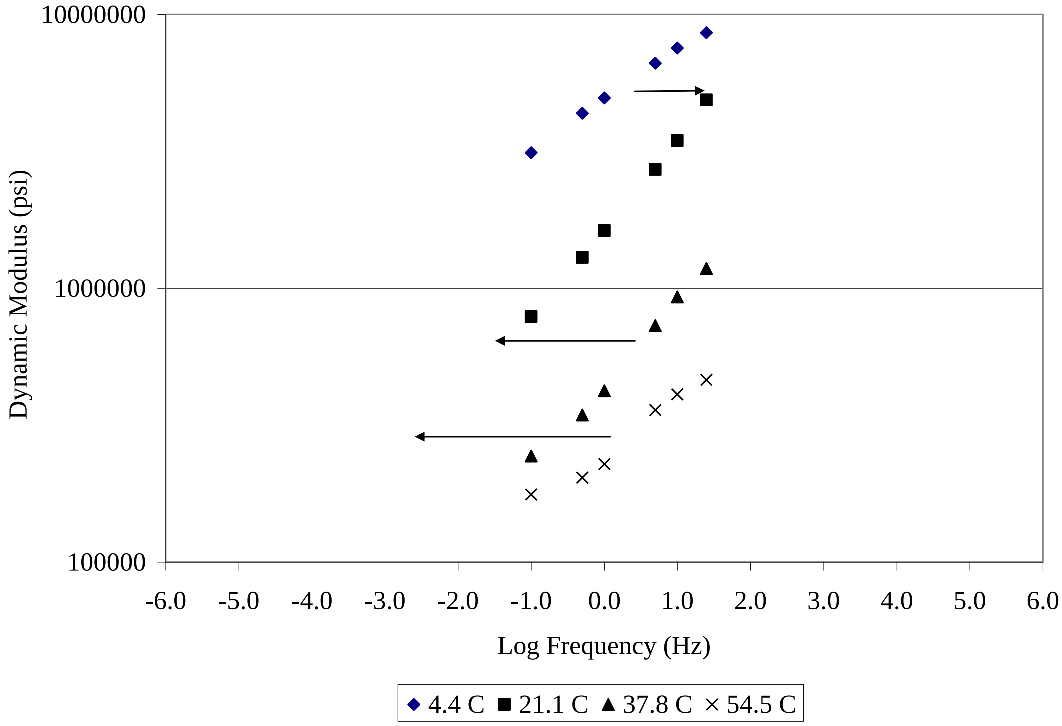

Figure 1. Results of dynamic modulus test on HMA sample.

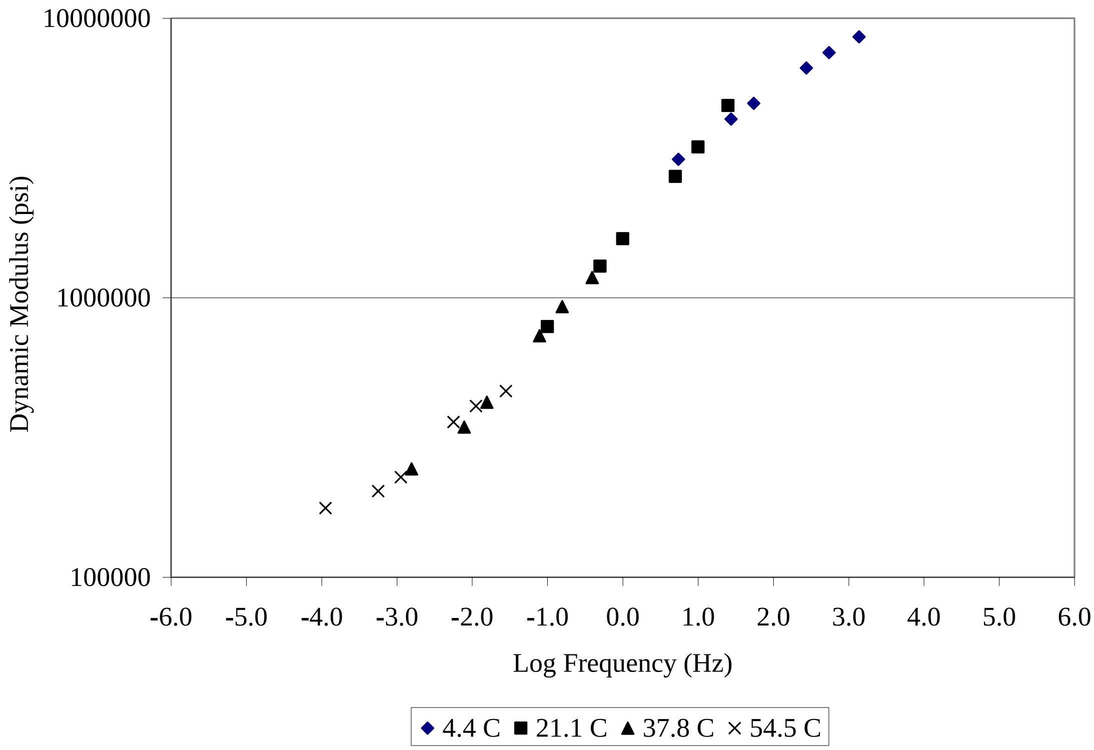

Figure 2. Test data shifted to form master curve.

According to the MEPDG (4), the master modulus curve can be mathematically modeled by a sigmoidal function described as:

$$\log\ E^* = \delta + \frac{\alpha}{1+e^{\beta+\gamma\ \log t_r}} \qquad [1]$$

Where,

t_r = reduced time of loading at reference temperature
δ = minimum value of E*
δ + α = maximum value of E*
β, γ = parameters describing the shape of the sigmoidal function.

The shift factor can be shown in the following form:

$$a(T) = t / t_r \qquad [2]$$

Where,
a(T) = shift factor as a function of temperature
t = time of loading at desired temperature
t_r = reduced time of loading at reference temperature
T = temperature of interest .

For precision, a second order polynomial relationship between logarithm of the shift factor i.e. log a (Ti) and temperature in degrees Fahrenheit is used. The relationship can be expressed as follows:

$$Log\,a\ Ti = aTi^2 + bTi + c \qquad [3]$$

Where,
a(Ti) = shift factor as a function of temperature Ti
Ti = temperature of interest, °F
a, b and c = coefficients of the second order polynomial.

The time-temperature superposition is performed by simultaneously solving for the four coefficients of the sigmoidal function (δ, α, β, and γ) as described in equation [1] and the three coefficients of the second order polynomial (a, b, and c) as described in equation [3]. A nonlinear optimization program for simultaneously solving these seven parameters is used for developing master curves.

E* PREDICTIVE EQUATION

The MEPDG uses laboratory E* data for Level 1 reliability designs, while it uses E* values from Witczak's E* predictive equation for Levels 2 and 3 reliability designs. There are two other E* predictive equations available, the Hirsch model (5) and the New

Revised Witczak E* Predictive Model (6). The current version of the Witczak's E* predictive model that is included in the MEPDG was based upon 2,750 test points and 205 different HMA mixtures (34 of which are modified). Most of the 205 HMA mixtures were dense-graded using unmodified asphalts. The current version of the E* predictive equation in the MEPDG, updated in 1999, is (4):

$$\log E^* = 1.249937 + 0.249937 + 0.02932\rho_{200} - 0.001767\ \rho_4^{\ 2} - 0.002841\rho_4 - 0.058097V_a$$
$$-0.802208\left(\frac{V_{beff}}{V_{beff}+V_a}\right) + \frac{3.871977 - 0.0021\rho_4 + 0.003958\rho_{38} - 0.000017\ \rho_{38}^{\ 2} + 0.005470\rho_{34}}{1+e^{-0.603313-0.313351\log\ f\ -0.393532\log\ \eta}} \qquad [4]$$

Where,

E* = dynamic modulus, 10^5 psi
η = asphalt viscosity at the age and temperature of interest, 106 Poise (use of RTFO aged viscosity is recommended for short-term oven aged lab blend mix)
f = loading frequency, Hz
Va = air void content, %
Vbeff = effective asphalt content, % by volume
ρ34 = cumulative % retained on 3/4 in (19 mm) sieve
ρ38 = cumulative % retained on 3/8 in (9.5 mm) sieve
ρ4 = cumulative % retained on #4 (4.76 mm) sieve
ρ200 = % passing #200 (0.075 mm) sieve.

The major difference between the current Witczak E* predictive model and the other two models is in how the asphalt viscosity is determined. In the Hirsh model (5) and the new revised Witczak model (6), the asphalt viscosity is determined directly in the model from the binder complex shear modulus (G*) and phase angle (δ), determined in accordance with AASHTO T 315 *Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR).* In the current E* predictive equation in the MEPDG, the asphalt viscosity must be calculated in a separate equation.

In the Witczak E* predictive equation [4], the asphalt viscosity (η) can be determined using equation [5] if the binder complex shear modulus (G*) and phase angle (δ), determined in accordance with AASHTO T 315 *Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)*, are known at a minimum of three test temperatures (6).

$$\eta = \frac{G^*}{10}\left(\frac{1}{\sin\delta}\right)^{4.8628} \qquad [5]$$

Where,

η = asphalt viscosity, cP
*G** = binder complex shear modulus, Pa
δ = binder phase angle, °.

Once the asphalt viscosity (η) is determined, the ASTM VTS parameters shown in equation [6] are found by linear regression of equation [6] after log-log transformation of the viscosity and log transformation of the temperature data (6).

$$\log\log\eta = A + VTS \log T_R \qquad [6]$$

Where,

η = asphalt viscosity, cP
A, VTS = regression parameters
T_R = temperature, ° Rankine.

If AASHTO T 315 test results are not available, default values for A and VTS, measures of asphalt's temperature susceptibility, are available in the MEPDG if the grade of the asphalt cement is known. The viscosity is calculated using the default A and VTS values and equation [6]. The viscosity at each test temperature is used with equation [4] to calculate the dynamic modulus (4). The default A and VTS values for the PG 76-28 asphalt binder used in this study is shown in Table 1.

Table 1. Default A and VTS Parameters from MEPDG

Parameters	PG 76-28
A	9.200
VTS	-3.024

Tran and Hall (7) compared measured dynamic modulus values to predicted values using the Witzack predictive equation found in the MEPDG for Arkansas HMA mixtures. The authors reported that there was no significant difference between measured and predicted dynamic modulus values, indicating that the Witzack predictive equation could be used to estimate dynamic modulus values of Arkansas mixes.

Birgisson et al. (8) compared measured dynamic modulus results from 28 Florida HMA mixtures to the results using the Witczak predictive equation. Results showed a bias in the results and a multiplier was recommended to correlate Florida mixtures to the predictive equation results. Birgisson et al. (8) reported that using binder viscosities from DSR testing were lower than measured values and that using binder viscosities from the Brookfield rotational viscometer resulted in slightly higher predicted modulus values compared to measured values.

EFFECT OF MIXTURE VARIABLES ON DYNAMIC MODULUS

King, et al. (9) studied the effects of mixture variables on dynamic modulus for different North Carolina mixes. Mixtures were prepared with different aggregate gradations, aggregate sources, binder sources, binder PG grades and asphalt contents. Master curves for each mix were prepared based on measured dynamic modulus values provided by the North Carolina DOT. The results of the study indicated that binder source, binder PG grade and asphalt content had a significant effect on dynamic modulus. However, aggregate source and gradation, within the same NCDOT mix classification, did not have a significant effect on dynamic modulus.

Tran and Hall (7) evaluated the sensitivity of measured dynamic modulus values of Arkansas HMA mixtures. Mix parameters evaluated included maximum nominal aggregate size (25 mm and 12.5 mm), void content (4.5% and 7.0%), and asphalt content (optimum and optimum ± 0.5%). The results indicated that aggregate size, air void content and asphalt content all had a significant effect on measured dynamic modulus.

Shah, McDaniel and Gallivan (10) summarized the results of dynamic modulus values obtained from 11 HMA mixtures from the North Central Superpave User Producer Group. Mixtures made with PG 58-28 binders were found to be statistically different from mixtures made with PG 70-28 binders. Superpave mixtures produced significantly different dynamic modulus values than Marshall mixtures, and Superpave mixtures had lower dynamic modulus values than stone mastic asphalt (SMA) mixtures.

SMA LITERATURE REVIEW

A comprehensive review of the literature on SMA was prepared by NCAT as a part of NCHRP 9-8 and can be found in the report *NCHRP 9-8 Designing Stone Matrix Asphalt Mixtures, Volume 1 - Literature Review* (11). Therefore, a comprehensive literature was not repeated.

Stone matrix or stone mastic asphalt (SMA) gained popularity in the United States after the European Study Tour of 1990 (12). The tour was arranged to exchange ideas and experience with highway agencies and the construction industry in Europe on design methods as well as production and placement of asphalt pavements. The group consisted of officials from American Association of State Highway and Transportation Officials (AASHTO), Federal Highway Administration (FHWA), National Asphalt Pavement Association (NAPA), Strategic Highway Research Program (SHRP), Asphalt Institute (AI) and the Transportation Research Board (TRB).

The group visited six European nations namely, Sweden, Denmark, Germany, France, United Kingdom and Italy because of similarities they share with the United States. They are all industrialized nations, have extensive highway and road systems and motor vehicles are increasingly relied upon for movement of people and goods. All the nations visited have modern, capable highway agencies and a mature construction industry. The tour participants found stone matrix asphalt (SMA) to be the most promising special-purpose mixture which could be used in the United States (12). Accordingly, four states (Wisconsin, Georgia, Michigan, and Missouri) constructed the first SMA projects in 1991 (12) and its use has been growing since that time.

German road contractors first used SMA, the English translation of "split mastix asphalt" in the 1960's (13). Its use is now prevalent in many European countries. The development of SMA was necessitated by the need for a high performance wearing surface that was capable of resisting rutting and abrasion under heavy traffic loads. SMA is composed of crushed stone aggregates, asphalt cement and a stabilizing additive, normally cellulose fiber or mineral fiber.

SMA is often considered as a premium mix because of higher initial costs due to increased asphalt contents and the use of more durable aggregate. However, this higher initial cost may be more than offset by improved performance for medium and high traffic loading situations. In addition to improved durability, fatigue and rutting resistance, other reported benefits include improved wet weather friction due to coarser surface texture and low tire noise. Reflective cracking in a SMA mixture is often not as severe as dense-graded mixtures since cracks have fewer tendencies to spall (12).

Purpose

SMA is a gap-graded HMA mixture that relies on a stable stone-on-stone contact to maximize rutting resistance and a rich mortar binder to improve durability. Because of this, these mixes are almost exclusively used for wearing surfaces on high volume interstates and U.S. highways. Special cases such as heavy, slow moving vehicles may warrant the use of SMA for intermediate layers (12).

Since it was first introduced in Europe, SMA has provided a rut resistant pavement surface that has resulted in about a 25-30% increase in the service life of such pavements (14). SMA differs from the traditional dense graded aggregate mixes in that it is a gap graded mixture which contains a large amount of coarse aggregate, i.e. aggregates with a minimum particle size of 4.75mm. The gap aggregate gradation is the reason for the rut resisting ability of SMA mixes because it provides stone-on-stone contact which forms a stone skeleton after compaction that is capable of resisting further densification under traffic loads and thus provide resistance against rutting.

Wolfgang et al. (13) listed some of the advantages of properly designed and produced SMA pavements as follows:

1) The stone skeleton gives the mix excellent shear resistance due to its high internal friction.
2) The voidless mastic, which is rich in binder, provides significant durability and adequate resistance to cracking.
3) The increased amount of large sized aggregates provides superior resistance to the wear of studded tires.
4) Good skid resistance and proper light reflection are enhanced in SMA mixes because of the rough surface texture of such mixes.

SMA mixes have been used in the United States (US) since 1990. Traffic rates on pavements with such mixes have been high and this has resulted in large amounts of traffic loadings on SMA pavements in a short period of time.

The FHWA, in association with various state Departments of Transportation, started a series of SMA trial pavements in five states in 1991 (14). Bukowski (14) listed some of the findings of the initial evaluation conducted in 1991 as follows:

1) The 4.75mm sieve controls the existence of appropriate stone-on-stone contact. The percentage of the coarse aggregate passing this sieve should not exceed 30%.
2) In order to maximize stone-on-stone contact, the amount of flat and elongated aggregates should be controlled by limiting the amount of coarse aggregates with a length to width ratio of 3 to 1 to about 20% of the total aggregate.

Brown et al. (15) stated that initial SMA design in the U.S. attempted to duplicate the techniques employed by European designers. However, because of differences in material properties and construction practices it proved to be unrealistic. The majority of SMA mixes currently in the U.S. were made using moderately stiff to stiff asphalts usually having a penetration in the 60 to 80 range (15). Temperature variations in this country require that the use of other grades of asphalt may be more appropriate for the various climatic regions in the US. Current practice is to use the appropriate PG grade for the climate and traffic level of the project.

Mixture Design

Wolfgang et al. (13) stated that the three principal conditions that must be satisfied during the design of SMA mixes are:

1) The coarse aggregates must be able to form a stone skeleton with firm contact between the aggregates.
2) The coarse particles should be held together by a voidless mastic such as asphalt cement.
3) The mastic should be stable enough to prevent drain down from the coarse particles during storage, haulage and placement of the mix.

The first condition implies that there must exist enough void space in the compacted mixture to accommodate the mastic and the required air voids in the compacted mix. Furthermore, the density of the coarse aggregate in the compacted mix should be nearly the same as the coarse aggregate compacted separately.

The voidless nature of the mastic suggested by the second condition is that the durability of the mastic is dependent on the degree of compaction and since the volume of mastic is less than the volume of aggregate, it is difficult to compact the mastic sufficiently.

The mastic is principally composed of asphalt cement, which is a visco elastic material with low viscosity at the mixing and compaction temperatures. The mastic will, therefore, drain off the coarse aggregate after it has been mixed. In Europe adding fibers to the mix has successfully stabilized the mastic (13). The type of stabilizer used has been of concern to both engineers and the general public. Initial SMA mixes developed in Germany were stabilized with asbestos fiber but strong public opinion against its use led to the search for alternative types of stabilizers which provided the mix with the same or better qualities than the asbestos fiber. In Sweden a cellulose fiber has been developed by a company known as NCC, and it is marketed under the trademark name "Viacotop" (16).

Mix Design Procedures

Superpave mix design procedures have been used to design SMA mixtures by making stone-on-stone contact of coarse aggregates and selection of high asphalt contents as the main criteria.

To ensure stone-on-stone contact of coarse aggregates in an SMA mixture, the voids in the coarse aggregate of the mix (VCA_{mix}) should be less than or equal to the voids in the coarse aggregate (VCA_{drc}). ODOT doesn’t consider this as a mix design requirement as recent changes in SMA gradation requirement ensure stone-on-stone contact.

SMA mixes are designed with the Superpave gyratory compactor by using 50 gyrations for $N_{des.}$ The minimum VMA requirement for this mix is set high (17% in design and 16.5 % in field) in order to ensure high optimum asphalt content. The minimum asphalt content in an SMA mixture is 6% and this asphalt content is adjusted to provide a 4% air void level.

After design of the mixture is completed, performance tests are usually conducted. The first test that should be performed on SMA Mix is a drain down test (AASHTO T 305). ODOT specifies a maximum of 0.2% drain down in these mixtures. Moisture sensitivity test (AASHTO T 283), which indicates the tensile strength ratio (TSR) of mixes, is the other recommended performance test. SMA mixes must have a minimum TSR value of 0.8 in design and 0.75 in field to meet ODOT's mix design requirement. Permeability (OHD L-44) and Rutting (OHD L-55) tests are the two additional performance tests which are required by ODOT. ODOT requires the permeability and the Hamburg rut depth of SMA mixes to be less than 12.5×10^{-5}cm/s and 12.5mm at 20,000 passes, respectively. ODOT's mix requirements for SMA are shown in Table 2.

Table 2. SMA Mix Properties (17)

Mix Property	**Design**	**Field**
VMA, min. %	17.0	16.5
Air Voids, %	4.0	4.0 ± 1.2
AASHTO T 283	0.80 min.	0.75 min.
Draindown, %	< 0.20	< 0.20
Permeability	< 12.5 X $10^{-5 cm/s}$	
Hamburg Rut Depth	< 12.5 mm @ 20,000 passes	

Determination of Stone-on-Stone Contact

Brown et al. (18) stated that satisfactory performance of SMA depends on adequate stone-on-stone contact. To determine the existence of stone-on-stone contact, the voids in the coarse aggregate fraction (+4.75mm) are determined using the dry-rodded technique in accordance with AASHTO T19. The dry rodded unit weight (γs) of the coarse aggregate is then substituted in the formula shown below to determine the voids in the coarse aggregate (VCA_{dry}) in the dry rodded condition.

$$VCA_{dry} = ((Gsb_{coarse} * \gamma w - \gamma s) / (Gsb_{coarse} * \gamma w)) * 100$$

Where:

γs = Unit weight of the coarse aggregate fraction in the dry rodded condition (kg/m^3)

γw = Unit weight of water (999 kg/m^3)

Gsb_{coarse} = bulk specific gravity of the coarse aggregate

The voids in the coarse aggregate of the compacted mix (VCA_{mix}) is determined from the bulk specific gravities of the mix (Gmb) and coarse aggregate (Gsb_{coarse}).

$$VCA_{mix} = 100 - (Gmb/Gsb_{coarse}) * Pca$$

Where:

Pca = Percentage of coarse aggregate in the mix.

Stone-on-stone contact exists when the VCA of the mix (VCA_{mix}) is less than or equal to the VCA of the coarse aggregate fraction (VCA_{dry}). Evaluation of stone-on-stone contact is not generally considered necessary as changes to SMA gradations limiting the percent passing the No. 4 sieve is thought to ensure stone-on-stone contact. Evaluation of stone-on-stone contact is not a part of ODOT's SMA mix design procedure.

Materials Specifications

The materials used to produce SMA include aggregate, mineral filler, asphalt cement and additives. SMA is a high quality mix which needs high quality materials. Cubical, low abrasion, crushed stone and manufactured sands are recommended because the mixture's rut resistance comes from the stone-on-stone aggregate skeleton. Aggregates should have 100 percent of the particles with one or more fractured faces. Where SMA is used as a surface course, the aggregates should also have a high polish values to retain good skid resistance. Natural sand should not be used in SMA mixtures.

SMA has a coarser gradation than a coarse graded Superpave mix. This mix has a low percentage passing at the No 4 sieve (22-30) to ensure stone-on-stone contact and to meet minimum VMA requirement and a high percentage passing the No 200 sieve (9-12) to adequately stiffen the binder so that the mixture is rut resistant. Mineral fillers are added to the mixture so that there are enough materials passing the No 200 sieve.

NCAT's (15) material specifications were the ones initially used by most state DOTs. They have been updated over the years, namely changing the gradation to ensure stone-on-stone contact. NCAT's and ODOT's gradation specifications for SMA (17) are shown in Table 3 and Table 4 shows ODOTS SMA aggregate requirements.

Table 3. Typical Gradation Requirements for SMA

	NCAT (15)	**ODOT (17)**
Sieve Size	**Percent Passing**	
¾ in.	100	100
½ in.	90-100	90-100
3/8 in.	65-80	65-80
No. 4	22-30	22-30
No. 8	16-24	16-24
No. 200	9-12	9-12

Table 4. Coarse Aggregate Specifications (17)

Test	Method	ODOT
LA Abrasion	AASHTO T 96	30% Max.
Fractured Faces	OHD L-18	100/95
Durability Index	AASHTO T 210	40 Min.
Insoluble Residue	OHD L-25	40% Min.
Micro-Deval	AASHTO TP 58	25% Max.
Flat & Elongated	ASTM D 4791	10% Max.
Nat'l Sand & Gravel		0% Max.
Clay balls, Friable	OHD L-9	0% Max.
Soft Particles	OHD L-38	5% Max.
Sticks or Roots	OHD L-9	0% Max.

Mineral Filler

Mineral filler used should consist of finely divided mineral matter such as rock or limestone dust, which must be sufficiently dry to flow freely and not contain any organic impurities. It must also have a Plasticity Index (PI) of not greater than 4 and should meet the requirements of AASHTO M17, (15).

Asphalt Cement

Asphalt cement used should meet the requirements of AASHTO M 226, Table 2 or AASHTO MP1. In most areas, it may be prudent to use one grade stiffer than is normally employed (15).The asphalt cement grade used in SMA is typically the same or slightly stiffer than that used for dense graded mixtures. Slightly higher asphalt content is used on this mix (typically 1-2%) as compared to conventional mixes to improve durability. ODOT requires PG 76-28.

Stabilizing Additive

To control draindown of excessive asphalt content, 0.3-0.4% by total mixture mass of stabilizing additives are used. Cellulose is the most widely used stabilizing fiber the other one being mineral fiber.

The stabilizer used may be cellulose fiber, mineral fiber, or polymer. It is added to the mixture to prevent the draining off of the asphalt cement from the coarse aggregate surfaces during mixing and compaction. Dosage rate for cellulose fiber is 0.3% by total mixture weight. For mineral fiber the dosage rate is 0.4% by total mixture weight. The amount of polymer added is the amount suggested by the manufacturer or determined from past experience. An allowable tolerance of fiber dosage is about +/- 10% of the required fiber weight (15). Most agencies currently require cellulose fibers.

Construction

SMA mixtures are difficult to work with because this mixes have a high coarse aggregate content, all crushed materials, and relatively stiff binders. Because of these reasons it is more difficult to construct good, dense, smooth longitudinal joints as compared to dense graded mix even if compaction and placement procedures are the same. However experience has shown that good joint can be built (19).

Early compaction, by keeping rollers right behind the paver, is needed because SMA mixtures tend to set up quickly. If they become cool they are very difficult to compact. Rubber tired rollers should not be used for compaction due to the mix sticking to the tires. Vibratory and static rollers should be used instead (20).

Performance

Since the first construction of SMA projects in U.S. in 1991, the performance history has shown good stability and good durability. SMA mixes can be expected to last longer than conventional mixes before reaching the same pavement condition level (21). The European experience shows that SMA mixes are generally expected to last up to 25% longer than conventional mixes (12).

The increase in cost for SMA is more than offset by the expected increase in pavement life. The life cycle cost, in terms of rehabilitation costs alone, are very favorable, but when combined with savings from fewer user delays the savings become truly significant (19).

One of the more comprehensive reviews of SMA performance in the US was performed by NCAT (22). NCAT reported that SMA was found to be a highly resistant to rutting, cracking and other pavement distresses when compared to conventional HMA. Of the over 90 SMA mixes evaluated, 90% had less than 4 mm of rutting and that cracking, with the exception of reflective cracking on a few pavements, was not a problem.

Wisconsin's (23) evaluation of their initial SMA mixes indicated that SMA reduced cracking compared to conventional HMA by 50% and that SMA showed significant improvement in frictional characteristics.

In a review of SMA for airfield pavements, Prowel et al. (24) reported that SMA performed superior to dense graded HMA (P401) and was observed to have better resistance to fuel, deicer chemicals, rutting, cracking and moisture damage.

Virginia (25) performed a life-cycle cost comparison and found SMA to be the most cost effective HMA for pavement maintenance purposes on the Virginia interstate system. This mix was observed to outperform the conventional dense graded mixes when placed under the same conditions and the high cost associated with this mix was justified by the increased predicted performance.

HAMBURG RUT TESTING

Performance tests were not included when Superpave mix design procedures were first adapted and there are a lot of ongoing researches to develop and incorporate a nationally adapted performance tests on Superpave mix designs. In the mean time, many department of transportation's have been using other existing tests. From these tests, wheel tracking tests are the most widely adapted tests.

Hamburg and asphalt pavement analyzer (APA) tests are the two types of wheel tracking tests that have been used by many DOTs. On this study, Hamburg test was used for performance testing of the mixtures.

Hamburg test was originated in the 1970s by Esso A.G. in Hamburg, Germany. The Hamburg rut tester is used mainly to test the rutting susceptibility of hot-mix asphalt (HMA). Because this test is conducted under water, it also has a potential to evaluate striping of asphalt mixtures (20).

OHD L-55 (26) is the test procedure for the Hamburg wheel-track testing of compacted HMA. Four 150mm diameter by 60 ± 2 mm tall Superpave gyratory specimens are required for each test. These specimens are then subjected to 20,000 load cycles of a loaded steel wheels or until the rut depth exceeds 20 mm.

The rut depth versus number of wheel cycles is plotted as shown in Figure 3. The striping inflection point (SIP) in the graph indicates the number of passes at which a sudden increase in rut depth occurs due to stripping of the binder from the aggregates.

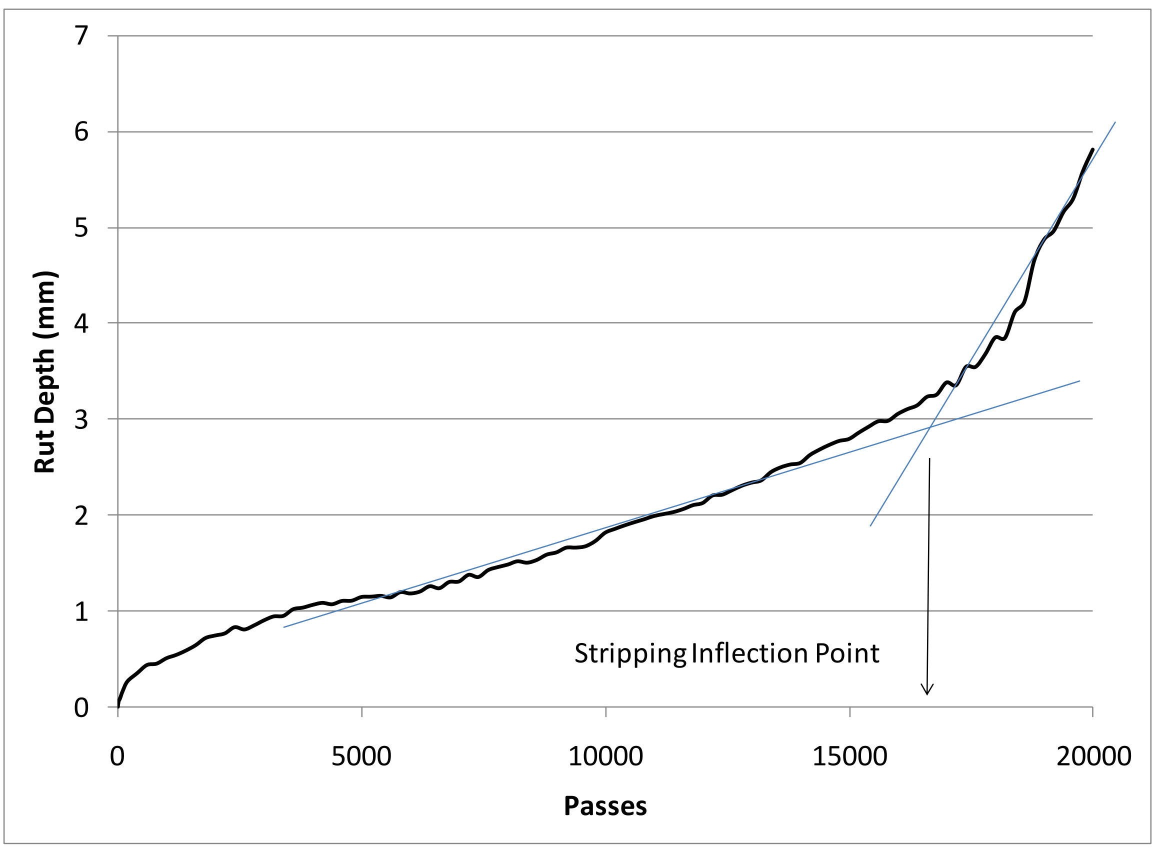

Figure 3. Typical Hamburg rut trace showing possible stripping inflection point.

CHAPTER 3

MIXTURES EVALUATED

MIXTURES

The primary objective of the OSU study was to evaluate the performance of SMA mixes compared to S-4 mixes and to determine performance benefits. To meet this objective, samples of mixtures produced for ODOT projects were collected over a two-year period. Mixtures were obtained by either contacting contractors directly or by contacting ODOT personnel to obtain mix samples. Ten mixtures, six SMA and four S-4, were sampled. Mix design information on each mix was obtained from either the contractor or ODOT.

All four S-4 mix samples were cold feed belt samples obtained after aggregate blending but prior to entering the drum. SMA mixtures are not routinely used in Oklahoma. Two of the six SMA mixtures were cold feed belt samples obtained from produced SMA mixtures. The four remaining SMA mixtures were not being produced; therefore, they were reproduced by sampling the individual components for each mix. Table 5 shows the mixtures sampled.

ASPHALT CEMENT

Regardless of the asphalt cement utilized with the field mixes, all mixtures were mixed with the same source of asphalt cement, Valero PG 76-28 OK.

MIXTURE VERIFICATION

The objective of this study was to evaluate ODOT SMA and S-4 mixtures, not to exactly reproduce field mixtures. Therefore, each mixture sampled was evaluated to determine optimum asphalt content and void properties. For mixtures sampled from the cold feed belt, the aggregates were oven dried at 230^{o} F and then the entire amount was sieved over a 1-inch sieve through No. 50 sieve, inclusive, and the material separated into sizes for batching. For the remaining mixes, the individual aggregate sources were oven dried at 230^{o} F and then combined to the mix design percentages (ODOT batching option 1) and then the entire amount was sieved over a 1-inch sieve through No. 50 sieve, inclusive, and the material separated into sizes for batching. Mineral filler and agricultural lime were added as a separate material. Sources of materials for the mixtures evaluated are shown in Tables 6 and 7 for SMA and S-4 mixtures, respectively.

After separating the combined aggregates by size four 4,700 g samples were prepared to the as received gradation and the samples mixed with two different percentages of PG 76-28 asphalt to bracket the JMF asphalt content. For SMA samples, the samples were compacted to the mix design N_{design} number of gyrations (50) in accordance with AASHTO T 312. SMA is a high traffic mix and should be compared to high traffic S-4 mixtures. For S-4 mix samples, the samples were compacted to the mix design N_{design} number of gyrations or a minimum of 100 gyrations, whichever was higher. After compaction, the samples were tested for bulk specific gravity in accordance with AASHTO T 166.

The optimum asphalt content was selected to produce 4.0% VTM if possible and a voids analysis at optimum asphalt content was performed to determine if the mixes met ODOT mix requirements. Mix gradations, N_{design} compaction level and void properties at optimum asphalt content for each mix are shown in Tables 8 and 9 for SMA and S-4 mixtures, respectively.

Table 5. Summary of Mixtures Sampled and Tested

Mix Type	Producer	Design No.	Design Traffic	Ndes	Mix ID Code
SMA	PMI-Silver Star	M2PV0160702600	10M+	50	SS
SMA	Cornell Const. Co.	M2PV0160600100	30M+	50	CL-1
SMA	Cornell Const. Co.	M2PV0110700100	30M+	50	CL-2
SMA	Haskell Lemon Const. Co.	M2QC0130702700	3M+	50	HL-1
SMA	Haskell Lemon Const. Co.	M2QC0130600101	10M+	50	HL-2
SMA	Cummins Const. Co.	M2QC0101004010	.	50	CU
S-4	T.J. Campbell Const. Co.	S4QC0190900600	3M+	100	TJC
S-4	Cornell Const. Co.	S4PV0110902000	30M+	125	CL-3
S-4	APAC-Oklahoma	S4QC0061003500	3M+	100	APAC
S-4	Haskell Lemon Const. Co.	S4QC0130902000	3M+	75	HL-3

Table 6. SMA Mix Aggregate Sources

Mix Code	Aggregate	Supplier	Source	Pit	% Used
	5/8 Chips	Hanson	Davis	5080	34
	5/8 Chips	Martin-Marietta	Davis	5005	15
SS	3/8 Chips	Martin-Marietta	Davis	5005	32
	Screenings	Falcon	Bowlegs	6709	8
	Agg. Lime	Dolese	Davis	5002	11
	5/8" Chips	Dolese	Cooperton	3801	35
	D Rock	Martin-Marietta	Snyder	3802	15
CL-1	Shot	Dolese	Cooperton	3801	27
	Screenings	Dolese	Cooperton	3801	18
	Agg. Lime	Dolese	Davis	5002	5
	3/4" Chips	Dolese	Cooperton	3801	17
	5/8" Chips	Martin-Marietta	Snyder	3802	56
CL-2	#4 Screenings	Dolese	Cyril	801	10
	Shot	Dolese	Cooperton	3801	10
	Mineral Filler	Dolese	Davis	5002	7
	3/4" Chips	Dolese	Cooperton	3801	15
	5/8" Chips	Hanson	Davis	5080	55
HL-1	Screenings	Martin-Marietta	Troy	3506	10
	Shot	Martin-Marietta	Mill Creek	3502	12
	Mineral Filler	Dolese	Davis	5002	8
	3/4" Chips	Dolese	Davis	5002	15
	5/8" Chips	Martin-Marietta	Snyder	3802	55
HL-2	#4 Screenings	Dolese	Cyril	801	11
	Shot	Dolese	Davis	5002	12
	Mineral Filler	Dolese	Davis	5002	7
	3/4" Chips	Dolese	Coleman	302	13
	5/8" Chips	Dolese	Coleman	302	45
CU	3/8" Chips	Dolese	Coleman	302	20
	Screenings	Dolese	Coleman	302	10
	Mineral Filler	Cummins	Plant Site		12

Table 7. S-4 Mix Aggregate Sources

Mix Code	Aggregate	Supplier	Source	Pit	% Used
TJC	5/8 Rock	Hanson	Davis	5008	19
	3/8 Chips	Martin-Marietta	Davis	5005	29
	Screenings	Hanson	Davis	5008	37
	Sand	GMI	Sooner Rd.	5514	15
CL-3	5/8" Chips	Martin-Marietta	Snyder	3802	30
	Shot	Dolese	Cooperton	3801	15
	Screenings	Dolese	Cooperton	3801	30
	C-33 Screenings	Martin-Marietta	Snyder	3802	10
	Sand	Mac Lemore Pit	Elk City		15
APAC	3/4" Chips	APAC-Oklahoma	Tulsa	7204	15
	Mine Chat		Tri-City Area		28
	Man. Sand	APAC-Oklahoma	Tulsa	7204	25
	Drag Sand		Tri-City Area		5
	Screenings	APAC-Oklahoma	Tulsa	7204	10
	Screenings	Holiday S&G	Bixby	7212	15
	Bag House Fines	APAC-Oklahoma	Tulsa	7204	2
HL-3	5/8" Chips	Martin-Marietta	Snyder	3802	34
	Stone Sand	Dolese	Cyril	801	26
	Man. Sand	Martin-Marietta	Davis	5005	15
	Screenings	Martin-Marietta	Mill Creek	3502	10
	Sand	GMI	OKC	1402	15

Table 8. Gradation and Mix Properties, SMA Mixes

Mix Code	SS	CL-1*	CL-2	HL-1	HL-2	CU	ODOT
Sieve							Spec.
Size	Percent Passing						
3/4"	100	100	100	100	100	100	100
1/2"	91	96	90	90	90	90	90-100
3/8"	75	73	68	65	69	71	65-80
No. 4	30	30	30	29	30	30	22-30
No. 8	21	21	17	21	19	20	16-24
No. 16	18	14	15	16	16	17	
No. 30	16	12	14	14	15	15	
No. 50	15	10	13	13	14	15	
No. 100	13	9	12	11	13	14	
No. 200	11.1	8.1*	9.6	9.9	9.7	11.0	9-12
% AC	6.0	6.6	6.5	6.2	6.3	6.0	min 6.0
% Fiber	0.3	0.3	0.3	0.3	0.3	0.3	0.3-0.4
Ndes	50	50	50	50	50	50	50
VTM	4.1	4.0	4.0	4.0	4.0	4.0	4
VMA	17.5	17.1	18.1	17.5	18.1	17.9	≥ 17.0
VFA	76.6	76.6	78	77.1	77.8	77.7	NR

*Produced under old SMA specification

NR = No requirement

Table 9. Gradation and Mix Properties, S-4 Mixes

Mix Code Sieve Size	TJC	CL-3	APAC	HL-3	ODOT Spec.
	Percent Passing				
3/4"	100	100	100	100	100
1/2"	97	96	95	97	90-100
3/8"	90	87	90	90	≤ 90
No. 4	52	69	63	70	
No. 8	36	47	39	47	34-58
No. 16	28	36	27	35	
No. 30	24	28	17	27	
No. 50	19	16	10	19	
No. 100	11	9	6	9	
No. 200	4.6	5.2	4.6	3.2	2-10
% AC	4.6	4.9	5.2	5.1	min. 4.6
Ndes	100	125	100	100	
% VTM	4.0	4.0	4.0	4.0	4.0
% VMA	14.4	14.7	14.6	14.8	≥ 14.0
% VFA	72.3	72.8	72.5	73.1	65-75

CHAPTER 4

HAMBURG RUT TESTING

HAMBURG TEST PROCEDURES

Hamburg rut depth testing was performed in general accordance with OHD L-55 (26). There are two major suppliers of Hamburg testing equipment. OSU has an ESRA that performs the “Hamburg” wheel tracking test slightly different than OHD L-55. In the ESRA, a test consists of two pills placed in a single mold, just as required in OHD L-55. However, the molds for the ESRA have the samples just touching, without sawing the ends of the samples. OHD L-55 requires that the samples be sawed so that they butt together and the loaded wheel travels across both samples without riding up on the mold. In the ESRA as the wheel leaves one sample it rides up on the mold and then back down on the other pill. The ESRA records the rut depth continuously, excluding the portion where the wheel rides up over the mold. The ESRA software looks at the maximum rut depth with each pass, which occurs at the center of each sample, and averages them for the recorded rut depth per pass. In the OHD L-55 procedure, the rut depth is recorded where the two samples butt together. Figure 4 shows samples in the ESRA mold and in the OHD L-55 mold.

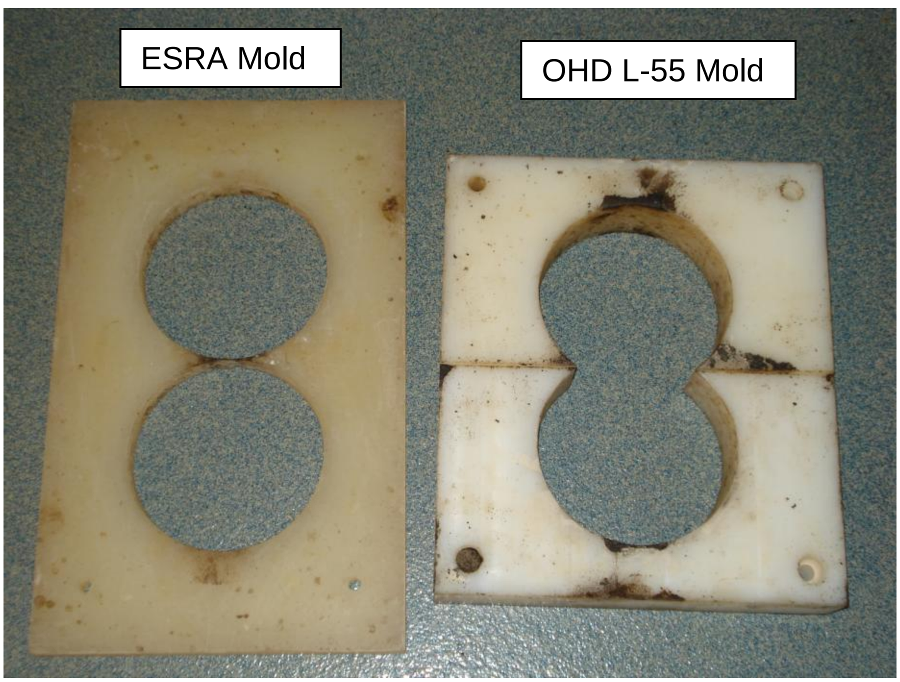

Figure 4. ESRA and OHD L-55 sample molds.

When this project started, Hamburg tests were performed using the ESRA. However, several research contracts were obtained during the time period of this project that required testing in strict accordance with OHD L-55. Toward the end of this project

OSU's ESRA was modified to meet the requirements of OHD L-55 and the software adjusted to measure the rut depth where the samples meet. The later samples tested in this study were tested in accordance with OHD L-55 whereas the earlier samples were tested using the ESRA.

Preparation of Hamburg Rut Test Samples

Samples for Hamburg rut testing were prepared in accordance with OHD L-55. All samples were mixed to the gradations and asphalt contents shown in Tables 6-9. Sample requirements are a 150 mm diameter sample compacted to 7 ± 1.0% voids total mix. Sample heights for samples evaluated using the ESRA were 75 ± 3 mm and 60 ± 2 mm for samples evaluated using OHD L-55. All samples were prepared using a single source of asphalt cement, Valero PG 76-28.

Mixing

All samples were mixed in a bucket mixer (Figure 5). The asphalt cement was stirred occasionally to prevent localized overheating while being heated to the mixing temperature of 325^{o} F. The aggregates were heated for a minimum of four hours at the mixing temperature of 325^{o} F. Approximately one hour before mixing, the compaction molds, spoons and spatulas were placed in the oven and brought to the mixing temperature. For mixing, the aggregates were placed in the bucket mixer and the desired amount of asphalt cement added. The mixture was mixed until well coated, approximately two minutes.

Figure 5. Bucket mixer used for mixing samples.

Compaction

After mixing, the mixture was oven-aged at the compaction temperature (300^{o} F) for two hours in accordance with AASHTO R 30. The samples were compacted in a 150 mm diameter mold to the required height using a Pine SGC.

HAMBURG TEST RESULTS

As previously stated, when this project started "Hamburg" tests were performed using the ESRA and toward the end of this project OSU's ESRA was modified to meet the requirements of OHD L-55. The later samples in this study tested were tested in accordance with OHD L-55 whereas the earlier samples in this study were tested using the ESRA. Test results for Hamburg rut testing are shown in Table 10. Individual traces of the rut depth versus wheel passes for each SMA mix are shown in Figures 6-11 and for the S-4 mixes in Figures 12-15, respectively.

Table 10. Hamburg Mix Properties and Test Results

Mix ID	Mix Type	Sample	VTM	Test Configuration	Rut Depth	Average Rut Depth
					20,000 Passes	
			(%)		(mm)	(mm)
SS	SMA	1	6.1	ESRA	8.96	8.70
		2	6.7		8.43	
CL-1	SMA	1	6.5	ESRA	6.68	6.68
		2				
CL-2	SMA	1	6.5	ESRA	5.31	5.43
		2	6.3		5.55	
HL-1	SMA	1	7.0	ESRA	7.81	7.41
		2	6.6		7.01	
HL-2	SMA	1	6.5	OHD L-55	4.46	4.58
		2	6.4		4.69	
CU	SMA	1	6.5	OHD L-55	5.22	4.55
		2	7.3		3.87	
TJC	S-4	1	6.8	ESRA	12.38	12.38
		2	6.4		12.5+	
CL-3	S-4	1	7.0	ESRA	9.13	10.32
		2	6.6		11.50	
APAC	S-4	1	7.2	OHD L-55	5.81	4.07
		2	7.2		2.32	
HL-3	S-4	1	6.8	OHD L-55	5.63	5.58
		2	7.3		5.52	

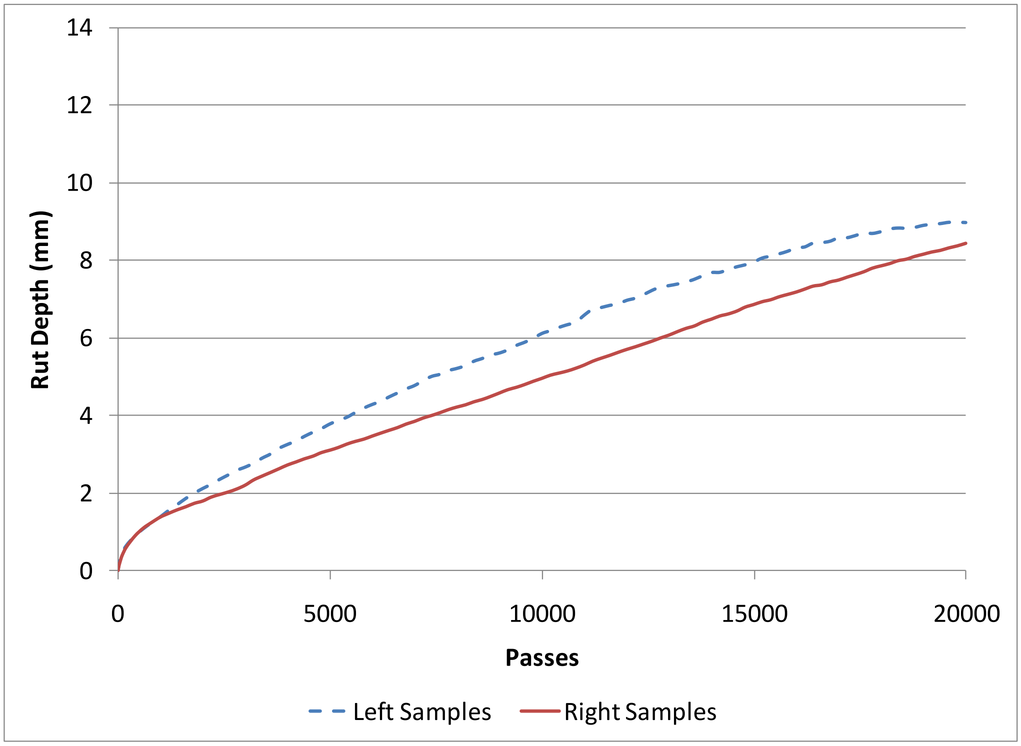

Figure 6. Hamburg rut depth vs. wheel passes, SS SMA mix.

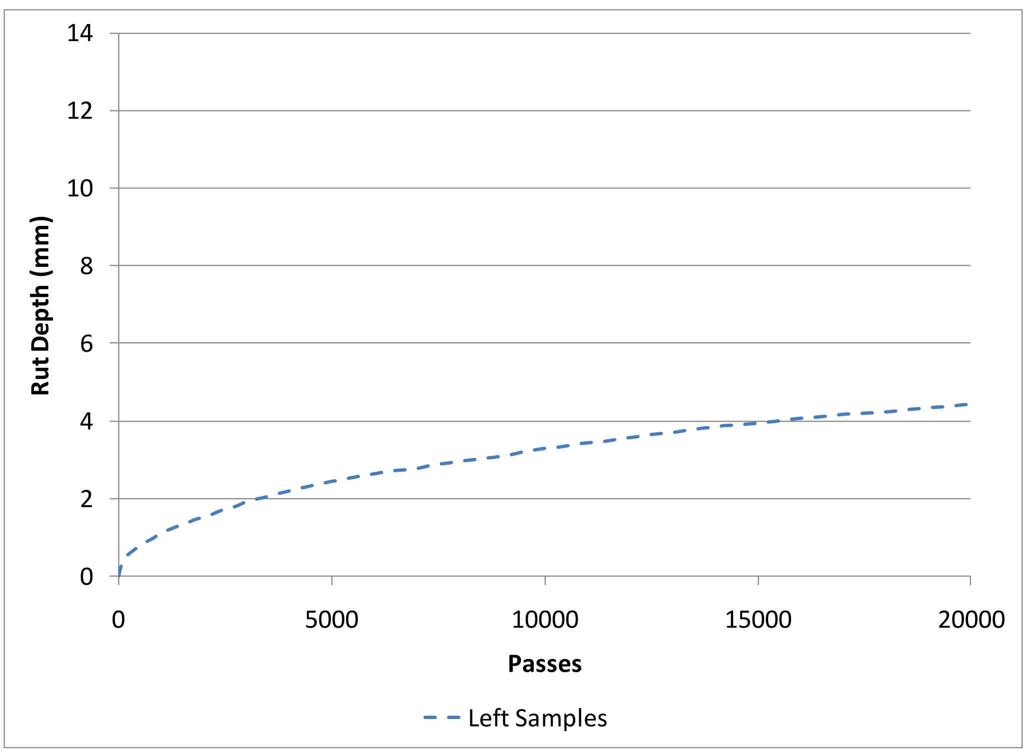

Figure 7. Hamburg rut depth vs. wheel passes, CL-1 SMA mix.

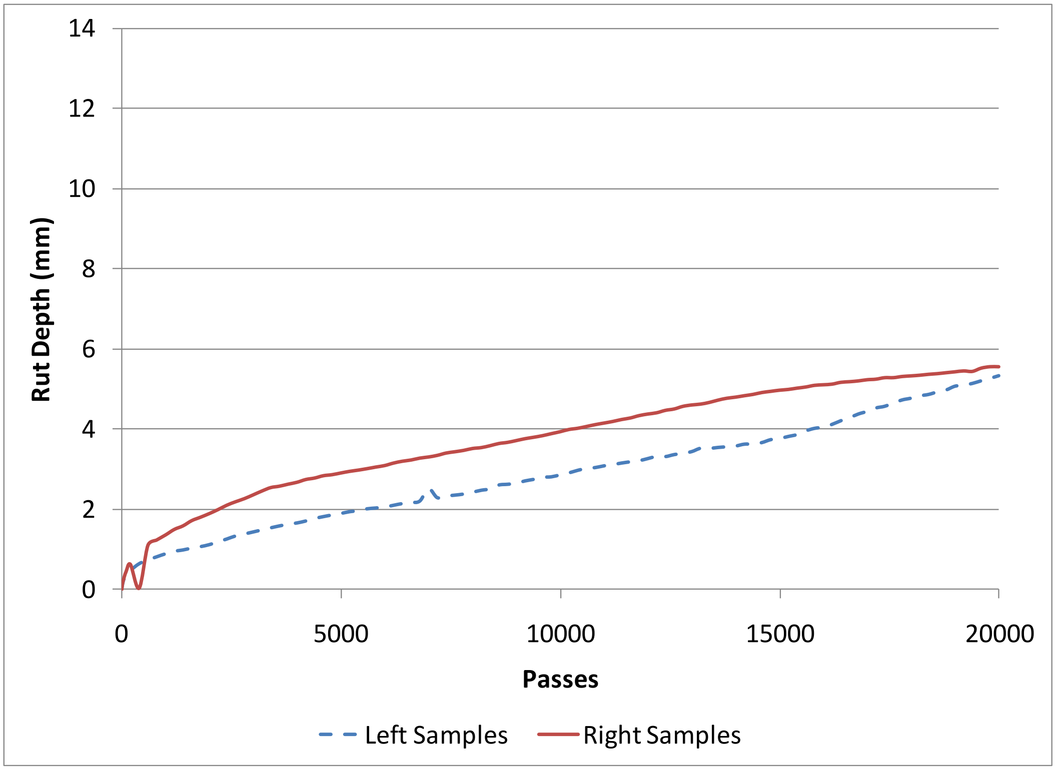

Figure 8. Hamburg rut depth vs. wheel passes, CL-2 SMA mix.

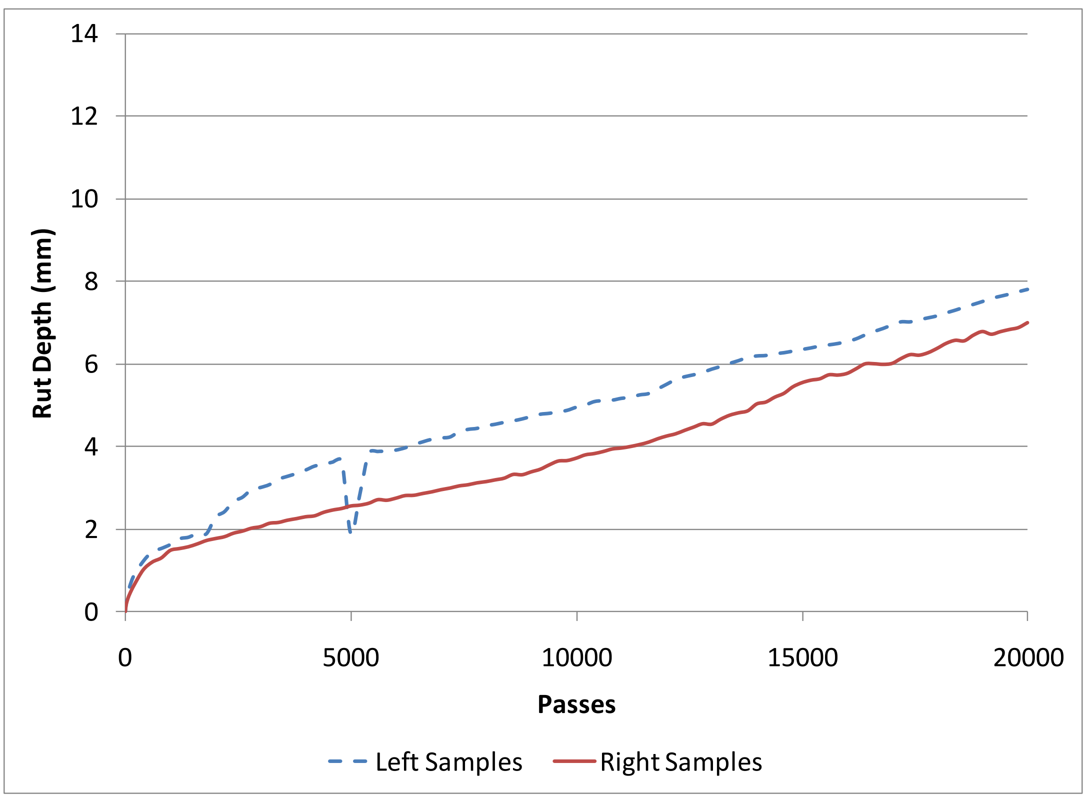

Figure 9. Hamburg rut depth vs. wheel passes, HL-1 SMA mix.

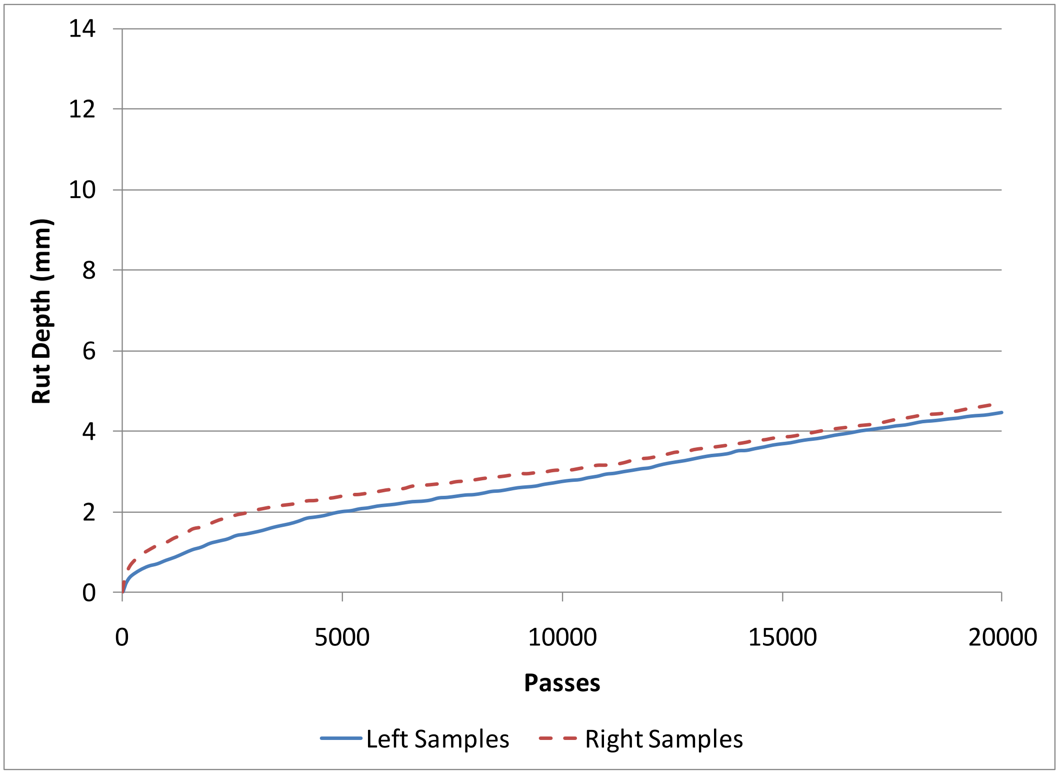

Figure 10. Hamburg rut depth vs. wheel passes, HL-2 SMA mix.

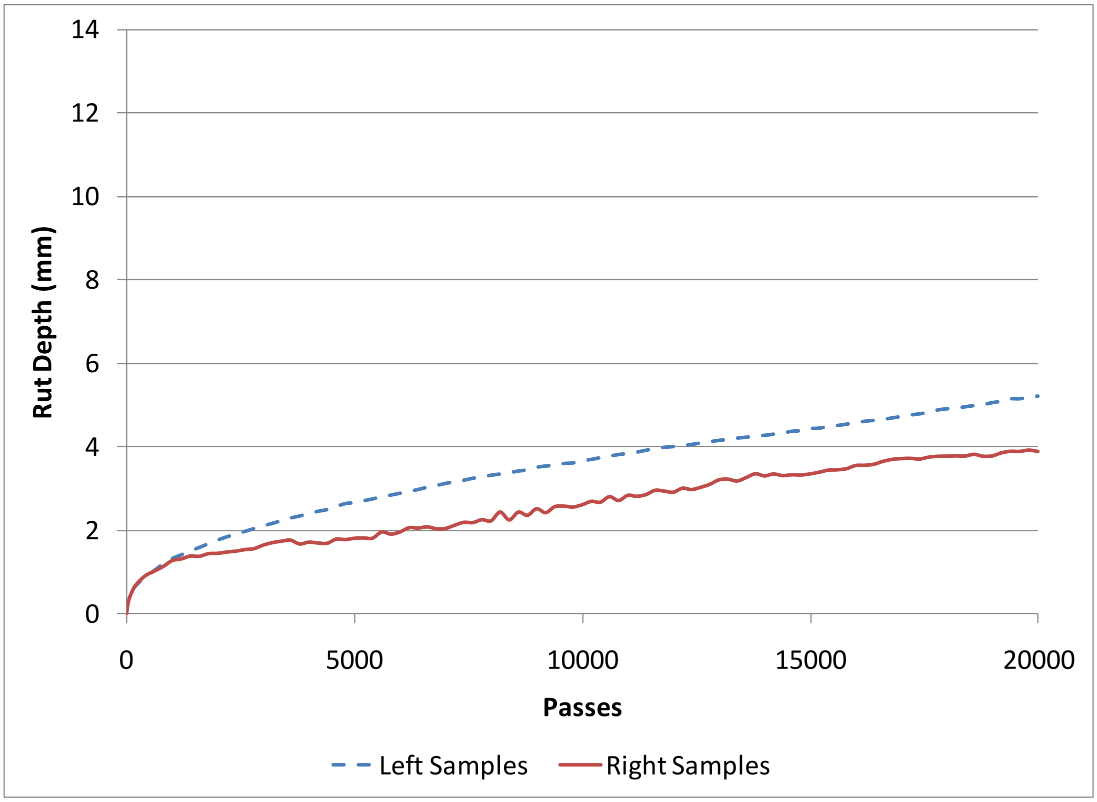

Figure 11. Hamburg rut depth vs. wheel passes, CU SMA mix.

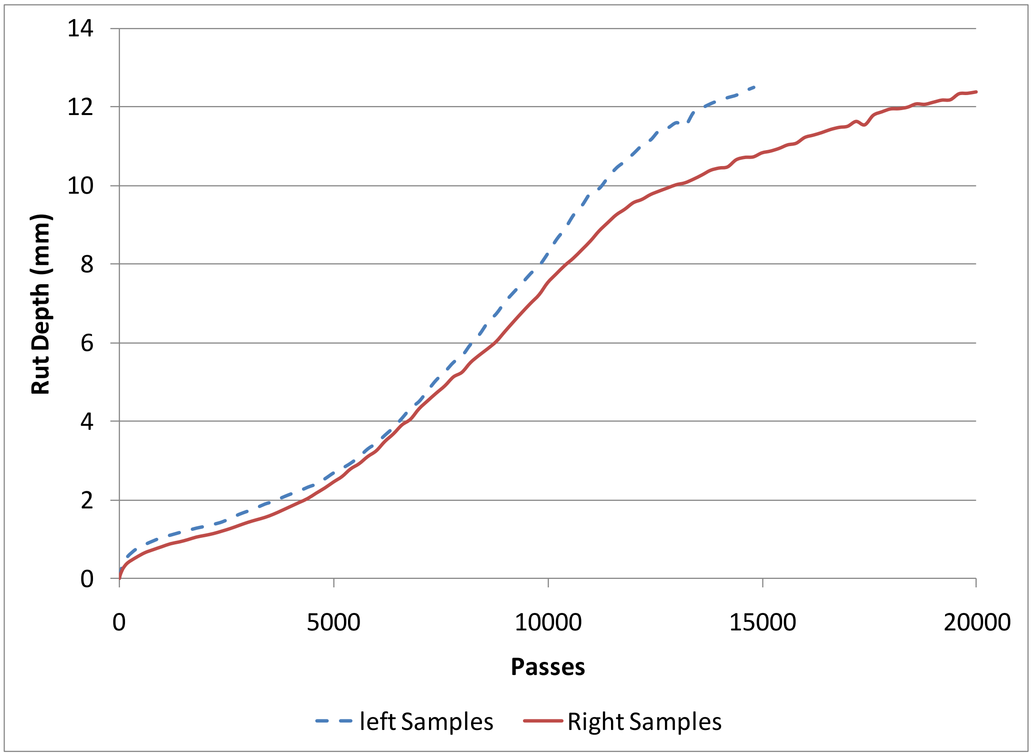

Figure 12. Hamburg rut depth vs. wheel passes, TJC S-4 mix.

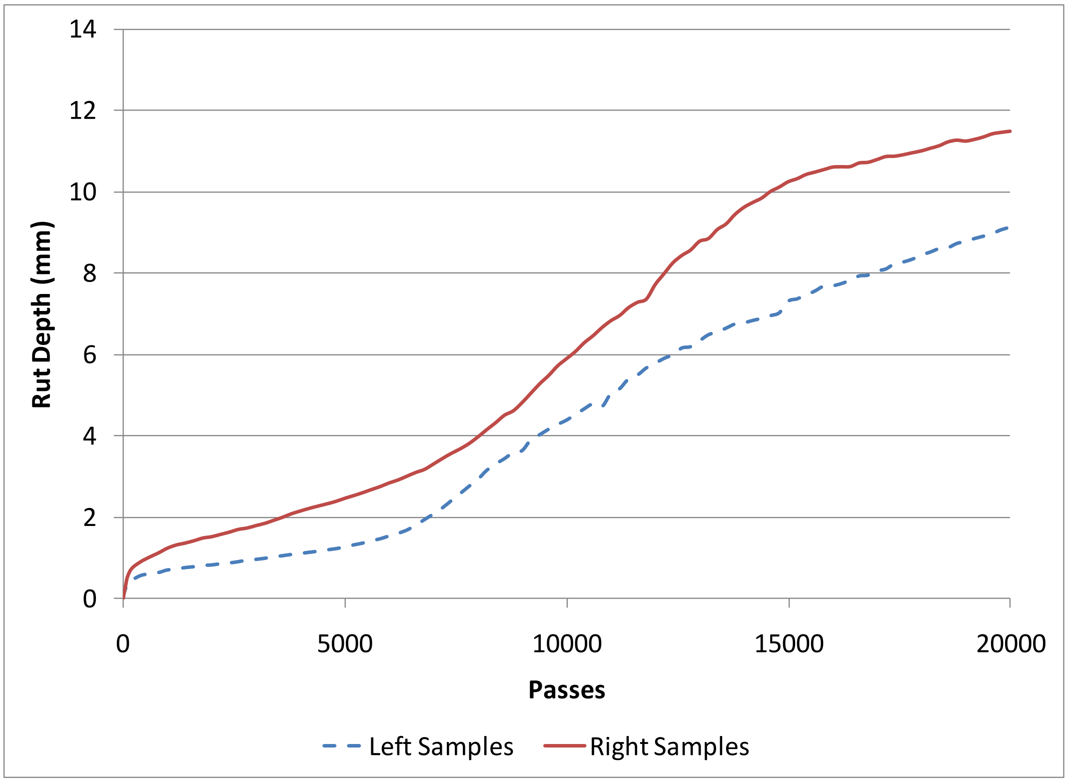

Figure 13. Hamburg rut depth vs. wheel passes, CL-3 S-4 mix.

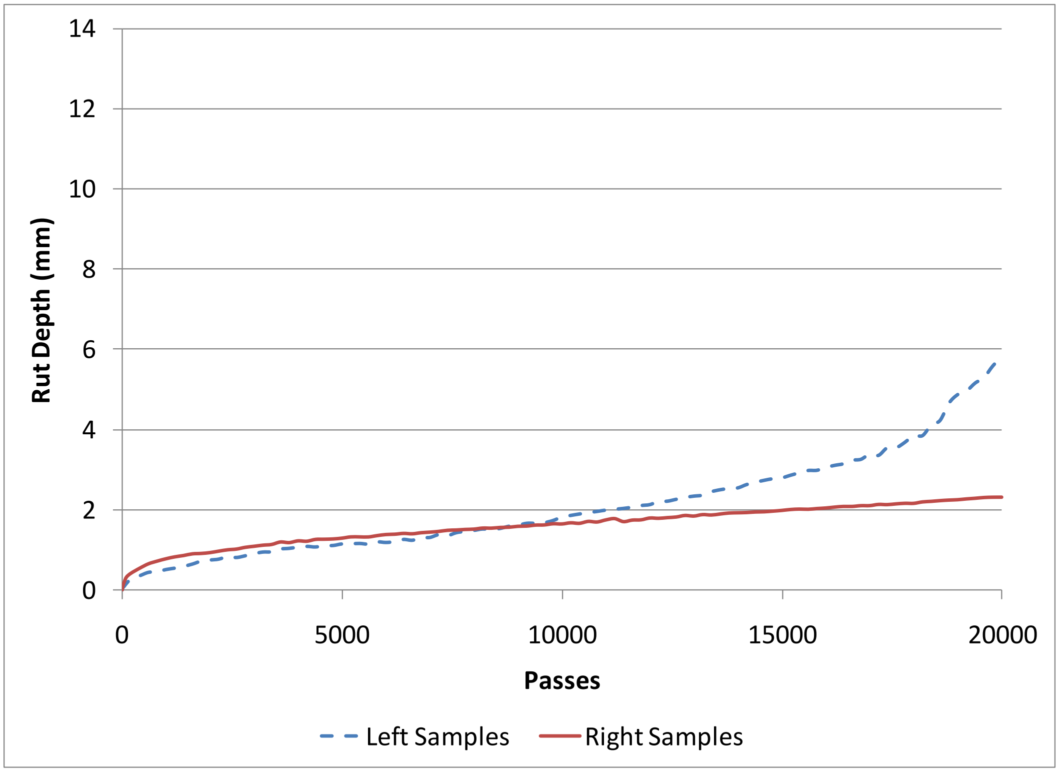

Figure 14. Hamburg rut depth vs. wheel passes, APAC S-4 mix.

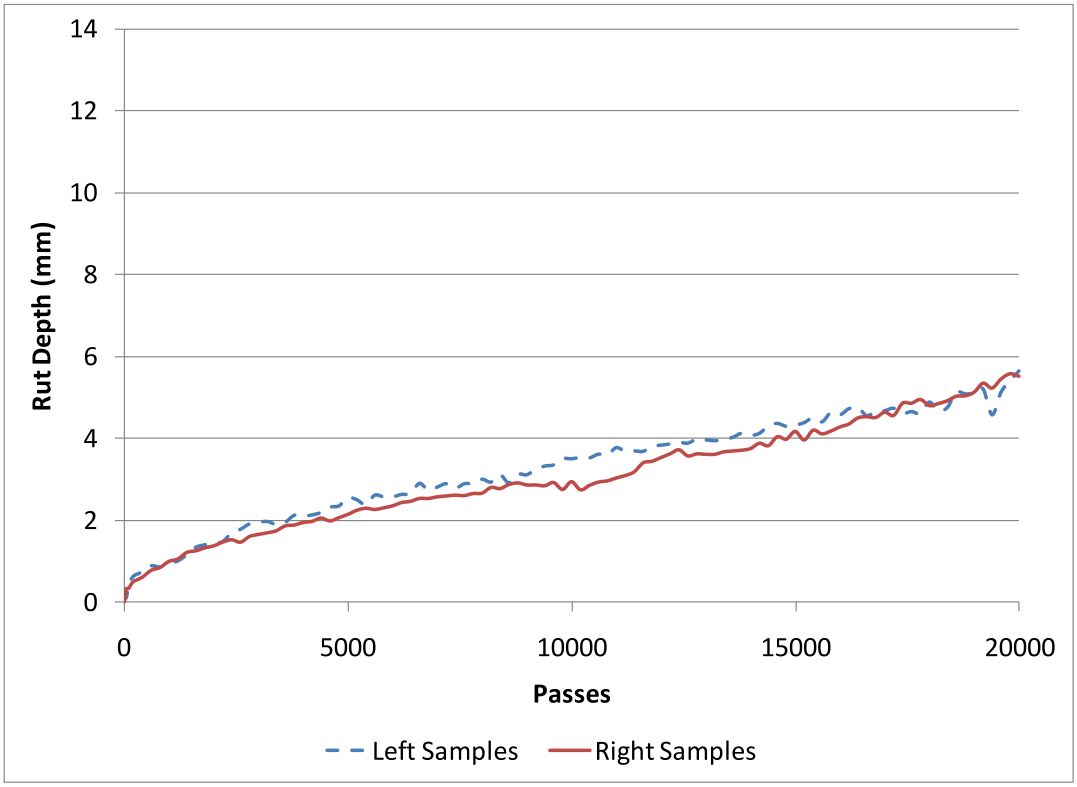

Figure 15. Hamburg rut depth vs. wheel passes, HL-3 S-4 mix.

ANALYSIS OF HAMBURG RUT TEST DATA

As previously stated, Hamburg rut depth testing was performed in general accordance with OHD L-55. Toward the end of this project OSU's ESRA had to be modified to meet the requirements of OHD L-55 and the software adjusted to measure the rut depth where the samples meet. The later samples in this study tested were tested in accordance with OHD L-55 whereas the earlier samples in this study were tested using the ESRA. Test results for Hamburg rut testing were shown in Table 10. Figure 16 is a plot of mean Hamburg rut depths, by mix type and test method. As shown in Figure 16, it does appear that the raising and lowering of the loaded wheel upon the samples in the ESRA results in larger rut depths than in OHD L-55. However, replicate samples were not tested under each configuration so this is just an observation that would need verification.

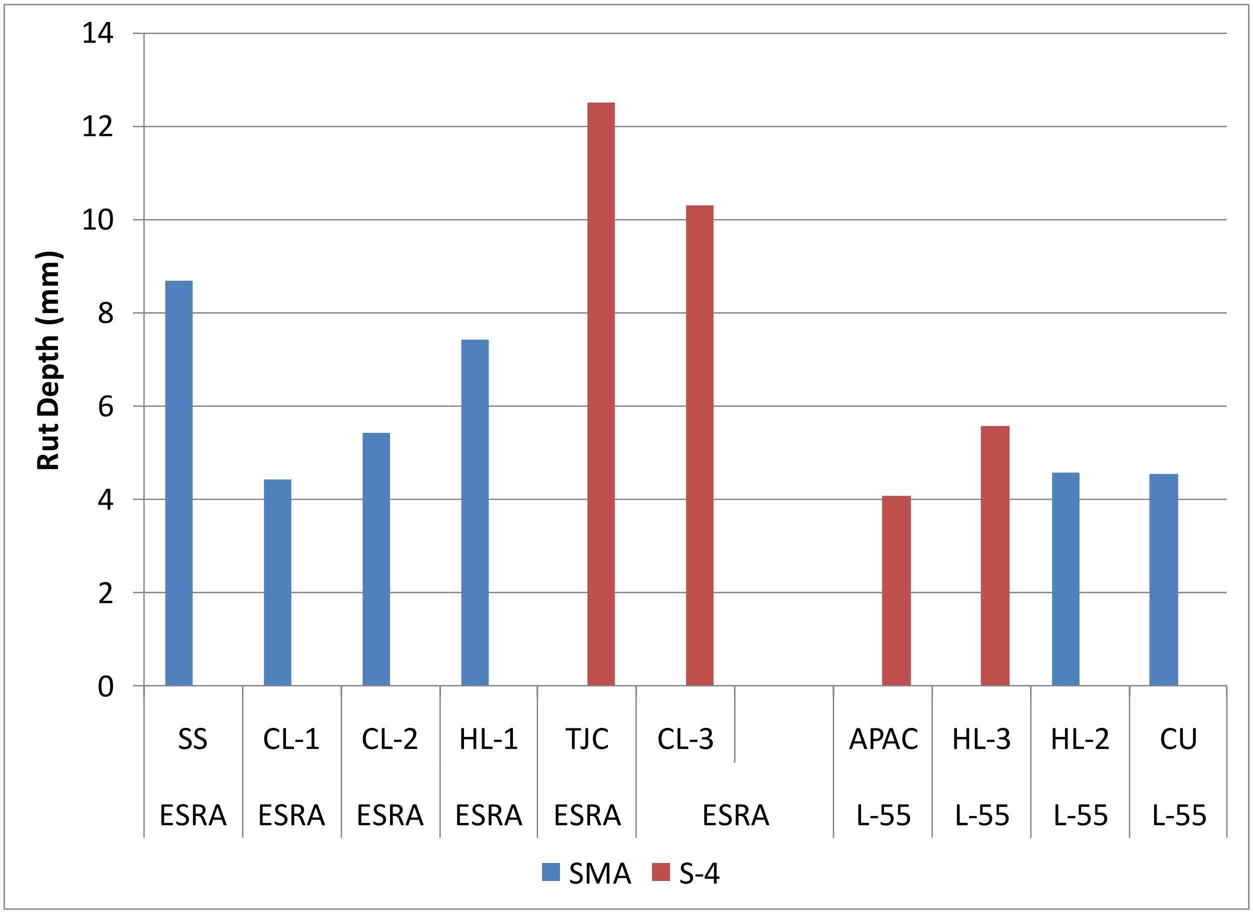

Figure 16. ESRA and OHD L-55 Hamburg results.

Due to the fact that samples were tested using two different configurations, an analysis of variance (ANOVA) was performed on the rut depth data by test configuration and mix type. The results are shown in Table 11.

Table 11. ANOVA of Hamburg Rut Depths

Source	Degrees of Freedom	Sum Squares	Mean Square	F Ratio	Prob. > F
Mix	1	41.870	41.870	14.06	0.0019
Test	1	73.870	73.870	24.80	0.0002
Mix * Test	1	27.502	27.502	9.23	0.0083
Error	15	44.684	2.979		
Total	18	187.926			

The results in Table 11 indicate that there is a significant difference in Hamburg rut depths by mix type, by test configuration and that there was a significant interaction between mix type and test configuration. For all mixes, regardless of test configuration, S-4 mixes had a mean rut depth of 8.41 mm and SMA mixes a mean rut depth of 5.98 mm. Mixes tested in the ESRA had a mean rut depth of 8.68 mm and using the OHD L-55 configuration, 4.69 mm. The differences in mean rut depths were significant at a level of significance exceeding 99 percent (alpha = 0.01). However, one should be careful interpreting the results. The same mixes were not tested using both test configurations. Therefore, one can only say that the mixes tested in the ESRA were from a different population than those tested using OHD L-55. However, a comparison of the methods does appear warranted.

There was a significant interaction between mix type and test configuration. To investigate this interaction a 1-way ANOVA was performed on the rut depths for mix type by test configuration. The results are shown in Table 12.

Table 12. ANOVA on Hamburg Rut Depth, By Test Method

Source	Degrees of Freedom	Sum Squares	Mean Square	F Ratio	Prob. > F
		ESRA			
Mix	1	69.236	69.236	17.62	0.0023
Error	9	35.369	3.930		
Total	10	104.605			
		OHD L-55			
Mix	1	0.135	0.135	0.09	0.7779
Error	6	9.315	1.553		
Total	7	9.450			

As shown in Table 12, there is a statistically significant difference in mean rut depths between SMA and S-4 mixes for the mixes tested using the ESRA configuration at a level of significance of 95 percent (alpha = 0.05); however, there was no significant difference in mean rut depths for the mixes tested using the OHD L-55 arrangement. To illustrate this difference, the results of Duncan's multiple range test are shown in Table 13. Means with the same letter are not significantly different.

Table 13. Duncan's Multiple Range Test, by Method

Grouping*	Mean Rut Depth	N	Mix
	ESRA		
A	12.00	4	S-4
B	6.79	7	SMA
	OHD L-55		
A	4.82	4	S-4
A	4.56	4	SMA

* Means with the same letter not significantly different.

To further investigate the results, an ANOVA was performed on Hamburg rut depths of the individual mixes. The results are shown in Table 14. The results show that there is a statistically significant difference in Hamburg rut depths for the mixes evaluated. To determine which mixes were statistically different, Duncan's multiple range test was performed. The results are shown in Table 15. Means with the same letter are not significantly different at a level of significance of 95 percent (alpha = 0.05). The results were presented graphically in Figure 16.

Table 14. ANOVA on Mix ID

Source	Degrees of Freedom	Sum Squares	Mean Square	F Ratio	Prob. > F
ID	9	174.161	19.351	12.65	0.0004
Error	9	13.764	1.529		
Total	18	187.925			

Table 15. Duncan's Multiple Range Test, by Mix

Grouping*		Mean Rut Depth	N	ID	Mix
	A	13.69	2	TJC	S-4
	B	10.32	2	CL-3	S-4
	B	8.70	2	SS	SMA
C	B	7.41	2	HL-1	SMA
C	D	5.58	2	HL-3	S-4
C	D	5.43	2	CL-2	SMA
C	D	4.58	2	HL-2	SMA
C	D	4.55	2	CU	SMA
C	D	4.44	1	CL-1	SMA
	D	4.07	2	APAC	S-4

* Means with the same letter not significantly different

Findings

As shown in Table 15 and Figure 16, one can make an S-4 mix that will compare favorably with an SMA mix. In fact, the best performing mix was an S-4 mix. However, the next four mixes with the lowest rut depths were SMA mixes and the two worst performing mixes were S-4 mixes. It is possible to make an S-4 mix that will resist rutting as well as an SMA mix but overall, SMA mixes had statistically significant lower Hamburg rut depths than S-4 mixes.

CHAPTER 5

DYNAMIC MODULUS TEST PROCEDURES & RESULTS

A second objective of this project was to obtain typical dynamic modulus values for Oklahoma SMA mixtures for use in the M-EPDG. Aggregates were obtained from SMA mixtures across the state and the mixtures reproduced using a single source of asphalt cement, PG 76-28.

DYNAMIC MODULUS TEST PROCEDURES

Preparation of Dynamic Modulus Test Specimen

Samples for dynamic modulus testing were prepared by mixing the aggregates with a single source of PG 76-28 OK asphalt cement from Valero. All samples were mixed to the gradations and asphalt contents shown in Tables 6-9. Test samples were prepared in accordance with the requirements of AASHTO TP 62-03 (27) and NCHRP 9-29: PP 01 (28).

Sample Requirements

The AASHTO TP 62 requirements for dynamic modulus test samples are provided in Table 16. They are similar to the NCHRP PP 01 requirements. Dynamic modulus testing requires a 150 mm high by 100 mm diameter sample, of a target air void content, be cored from 175 mm high by 150 mm diameter sample. There is no simple conversion factor for compaction of a 175 mm high, 150 mm diameter SGC compacted sample to a cored dynamic modulus (E*) sample with a given target air void content. The two samples will not have the same VTM due to a density gradient present in SGC compacted samples. A trial and error procedure is required to determine the density or void content of the larger sample required to produce a cored and sawed test sample of the intended void content. Recommended target air void contents for HMA samples are 4-7%. For this project, the HMA test samples were compacted to a void content of 5.0 ± 1 % VTM.

Batching

A 5,700 to 6,300 gram batch of aggregate, batched to the desired gradation, was required to produce a 175 mm high by 150 mm diameter test specimen that when cored to 100 mm diameter and sawed to the required sample height of 150 mm, would produce the required target void content of 5.0 ± 1% VTM.

Mixing and Compaction

All samples were mixed in a bucket mixer, cured in accordance with AASHTO R 30 and compacted in a Pine SGC as previously described in chapter 4.

Table 16. Criteria for Acceptance of Dynamic Modulus Test Specimens (27)

Criterion Items	Requirements
Size	Average diameter between 100 mm and 104 mm Average height between 147.5 mm and 152.5 mm
Gyratory Specimens	Prepare 175 mm high specimens to required air void content (AASHTO T 312)
Coring	Core the nominal 100 mm diameter test specimens from the center of the gyratory specimen Check the test specimen is cylindrical with sides that are smooth parallel and free from steps, ridges and grooves
Diameter	The standard deviation should not be greater than 2.5 mm
End Preparation	The specimen ends shall have a cut surface waviness height within a tolerance of ± 0.05 mm across diameter The specimen end shall not depart from perpendicular to the axis of the specimen by more than 1 degree
Air Void Content	The test specimen should be within ± 1.0 percent of the target air voids
Replicates	For three LVDT's, two replicates with a estimated limit of accuracy of 13.1 percent
Sample Storage	Wrap specimens in polyethylene and store in environmentally protected storage between 5 and 26.7° C (40 and 80° F) and be stored no more than two weeks prior to testing

Coring & Sawing

After compaction, the samples were extruded from the compaction molds, labeled and allowed to cool to room temperature. Next, the compacted samples were cored and sawed to obtain a 150 mm tall by 100 mm diameter test sample with 5.0 ± 1 % air voids. The samples were cored using a diamond studded core barrel to obtain the required diameter of 100 mm (Figure 17). The cored samples were then sawed to obtain the required 150 mm height (Figure 18). The cored and sawed samples were washed to eliminate all loose debris. After cleaning, the samples were tested for bulk specific gravity in accordance with AASHTO T 166. The dry mass was determined by using the

CoreDry™ apparatus (ASTM D 7227). From the bulk specific gravity and Gmm for each mix, the air void content was determined.

Figure 17. Sample being cored to required test diameter.

Figure 18. Sample being sawed to obtain parallel faces.

The test samples were next checked for conformance to the sample requirements of AASHTO TP 62-03 and NCHRP 9-27 PP 01. The criterion for acceptance of the samples is listed in the Table 16. Samples which met all criteria were fixed with six steel studs to hold three linear variable displacement transducers (LVDTs). The LVDT have a gauge length of 4 inches. Care was taken to precisely position the studs 4 inches apart and 2 inches from the center of the sample. Once the epoxy was dry and the studs were firmly attached to the sample, they were ready for testing. Figure 19 shows a sample prepared for dynamic modulus testing.

Figure 19. Test specimens for dynamic modulus testing.

Testing

There are slight differences in test procedures for dynamic modulus between AASHTO TP 62-03 (27) and NCHRP 9-29: PP 02 (28). The NCHRP procedure is specifically designed for the Simple Performance Test System or AMPT (Asphalt Mixture Performance Tester). The test parameters common to both are shown in Table 17. Table 18 shows the slight differences in the two test procedures and the procedures followed in this study.

AASHTO TP 62 (27) originally required testing at -10° C (14°F). With most test set ups (OSU's included) samples cannot be easily tested at -10° C (14°F) due to accumulation of frost in the test chamber. Testing at temperatures below 0° C (32°F) is no longer required. At the high TP 62 test temperature, 54.4°C (130°F), and /or the highest recommended test frequency, problems are often encountered with repeatability of the strain measurements and damage to the test sample is possible. In a previous study (3) several test samples were damaged due to excessive strain. For this study the procedures of NCHRP 9-29 PP 02 were followed with the exception of additional test frequencies. Figure 20 shows a sample ready for testing. Teflon end caps were used in this study as allowed by NCHRP 9-29 PP 02 in lieu of the rubber membranes shown.

Table 17. Common Test Parameters for Dynamic Modulus Test (27)

Test Parameters	Values		
Equilibrium Times	Specimen Temperature C (F)	Time From Room Temperature (hrs)	Time From Previous Test Temperature (hrs)
	4.4 (40)	Overnight	min. 4
	21.1 (70)	1	3
	37.8 (100)	2	2
	54.4 (130)	3	1
Contact Load	5 % of test load		
Axial Strains	Between 50 and 150 microstrain		
Approximate Load at Test Frequency	At 4.4: 100 -200 psi At 21.1: 50-100 psi At 37.8: 20-50 psi At 54.4: 5-10 psi		
Preconditioning Cycles	At 25 Hz: 200 cycles At 10 Hz: 200 cycles At 5 Hz: 100 cycles At 1 Hz: 20 cycles At 0.5 Hz: 15 cycles At 0.1 Hz: 15 cycles At 0.01 Hz: 15 cycles		

Table 18. Test Parameters for Dynamic Modulus Test (27)

Test Paremeters	AASHTO TP 62	NCHRP 9-29 PP 02	Used
Load Frequency (Hz)	25, 10, 5, 1, 0.5, 0.1	10, 1, 0.1, 0.01	10, 5, 1, 0.1, 0.01
Test Temperatures C	4.4, 21.1, 37.8, 54.4	4, 20, 45	4, 20, 45

Figure 20. HMA sample ready for dynamic modulus testing.

DYNAMIC MODULUS TEST RESULTS

The results of the dynamic modulus testing performed as described above are shown in Tables 19-23 for the SMA mixtures and Tables 24 and 25 for S-4 mixtures, respectively. Data from S-4 mixtures made with the same source of PG 76-28 asphalt from a previous study by the author (3) were used to supplement the S-4 mixtures for this study.

Table 19. Dynamic Modulus Results, SS SMA

Temperature (C)	Frequency (Hz)	Dynamic Modulus (psi) Sample 1	Sample 2	Avergae
4	10	2,097,329	2,205,382	2,151,356
	5	1,953,732	1,860,755	1,907,244
	1	1,443,091	1,340,384	1,391,738
	0.5	1,331,824	1,173,235	1,252,530
	0.1	989,063	778,884	883,974
20	10	776,428	629,952	703,190
	5	605,603	528,256	566,930
	1	362,287	300,596	331,442
	0.5	287,622	228,955	258,289
	0.1	164,149	138,334	151,242
45	10	190,844	216,328	203,586
	5	177,593	219,758	198,676
	1	87,546	130,959	109,253
	0.5	75,862	105,616	90,739
	0.1	62,922	65,360	64,141

Table 20. Dynamic Modulus Results, CL-1 SMA

Temperature (C)	Frequency (Hz)	Dynamic Modulus (psi) Sample 1	Sample 2	Avergae
4.4	10	2,476,284	2,081,857	2,279,071
	5	1,972,183	1,727,844	1,850,014
	1	1,051,517	968,831	1,010,174
	0.5	824,842	784,366	804,604
	0.1	518,753	510,413	514,583
21.1	10	564,557	752,170	658,364
	5	443,455	545,339	494,397
	1	270,070	303,962	287,016
	0.5	219,701	234,744	227,223
	0.1	145,706	151,108	148,407
37.8	10	219,300	261,306	240,303
	5	184,768	228,710	206,739
	1	107,895	143,357	125,626
	0.5	90,737	123,337	107,037
	0.1	68,979	98,507	83,743
54.4	10	189,724	264,057	226,891
	5	150,775	210,098	180,436
	1	101,254	132,051	116,653
	0.5	87,352	113,811	100,581
	0.1	71,777	85,841	78,809

Table 21. Dynamic Modulus Results, CL-2 SMA

Temperature (C)	Frequency (Hz)	Dynamic Modulus (psi)		
		Sample 1	Sample 2	Average
4	10	1,645,178	1,693,992	1,669,585
	5	1,398,260	1,512,718	1,455,489
	1	1,005,142	947,846	976,494
	0.5	973,056	737,011	855,034
	0.1	575,527	635,206	605,367
20	10	739,455	810,446	774,951
	5	611,249	642,434	626,842
	1	312,625	442,947	377,786
	0.5	.	334,184	334,184
	0.1	.	226,596	226,596
45	10	216,482	199,255	207,869
	5	159,429	180,088	169,759
	1	.	106,424	106,424
	0.5	.	69,008	69,008
	0.1	.	52,236	52,236

. Sample damaged during testing

Table 22. Dynamic Modulus Results, HL-1 SMA

Temperature (C)	Frequency (Hz)	Dynamic Modulus (psi)		
		Sample 1	Sample 2	Average
4	10	1,933,869	1,982,620	1,958,245
	5	1,700,392	1,754,267	1,727,330
	1	1,197,119	1,275,723	1,236,421
	0.5	1,003,570	1,087,380	1,045,475
	0.1	632,566	659,225	645,896
20	10	763,872	673,589	718,731
	5	582,896	575,112	579,004
	1	469,321	390,900	430,111
	0.5	334,184	394,363	364,274
	0.1	282,826	248,466	265,646
45	10	221,829	184,108	202,969
	5	119,168	138,353	128,761
	1	144,775	136,107	140,441
	0.5	84,613	116,085	100,349
	0.1	46,526	47,514	47,020

Table 23. Dynamic Modulus Results, HL-2 SMA

Temperature (C)	Frequency (Hz)	Dynamic Modulus (psi) Sample 1	Sample 2	Average
4	10	2,282,271	1,652,080	1,967,176
	5	1,923,857	1,486,485	1,705,171
	1	1,356,475	1,084,830	1,220,653
	0.5	1,100,974	879,518	990,246
	0.1	687,127	592,623	639,875
20	10	542,972	706,418	624,695
	5	446,216	472,720	459,468
	1	461,825	379,949	420,887
	0.5	343,396	284,064	313,730
	0.1	251,662	271,510	261,586
45	10	239,447	188,340	213,894
	5	234,205	184,776	209,491
	1	137,065	118,580	127,823
	0.5	123,023	83,155	103,089
	0.1	49,444	69,417	59,431

Table 24. Dynamic Modulus Results, TJC S-4

Temperature (C)	Frequency (Hz)	Dynamic Modulus (psi) Sample 1	Sample 2	Smaple 3	Average
4	10	2,370,813	2,735,110	2,604,007	2,669,559
	5	2,117,981	2,612,153	2,333,461	2,472,807
	1	1,579,826	1,869,208	1,710,724	1,789,966
	0.5	1,393,672	1,656,723	1,485,198	1,570,961
	0.1	1,037,074	1,180,098	1,059,636	1,119,867
20	10	959,120	1,052,386	1,009,044	1,030,715
	5	798,441	833,419	709,877	771,648
	1	690,867	594,749	699,187	646,968
	0.5	547,399	474,524	574,495	524,510
	0.1	353,407	315,435	380,662	348,049
45	10	278,994	263,092	311,880	287,486
	5	243,536	246,953	243,724	245,339
	1	168,831	184,621	221,550	203,086
	0.5	153,866	172,418	169,482	170,950
	0.1	117,730	131,732	123,471	127,602

Table 25. Dynamic Modulus Results, CL-3 S-4

Temperature (C)	Frequency (Hz)	Dynamic Modulus (psi) Sample 1	Sample 2	Average
4	10	1,830,717	2,638,793	2,234,755
	5	1,584,215	2,499,956	2,042,086
	1	1,106,565	1,789,653	1,448,109
	0.5	950,746	1,528,538	1,239,642
	0.1	596,049	1,030,639	813,344
20	10	1,156,935	946,091	1,051,513
	5	847,946	719,627	783,787
	1	572,986	624,391	598,689
	0.5	422,756	462,543	442,650
	0.1	340,208	298,184	319,196
45	10	221,146	265,271	243,209
	5	207,470	236,246	221,858
	1	134,041	155,323	144,682
	0.5	103,927	88,516	96,222
	0.1	83,340	72,762	78,051

CHAPTER 6

ANALYSIS OF DYNAMIC MODULUS TEST RESULTS

LABORATORY DYNAMIC MODULUS

AASHTO TP 62 (27) and NCHRP 9-29: PP 02 (29) requires dynamic modulus testing at different frequencies and test temperatures because temperature and frequency have a significant effect on dynamic modulus. A review of the test data indicated that frequency had a consistent effect on dynamic modulus, showing an increase in dynamic modulus with an increase in frequency. Therefore, in order to simplify the analysis, a two-way analysis of variance (ANOVA) was performed to determine if there is a statistical difference in dynamic modulus between SMA mixes and test temperature by using a single frequency. The middle frequency (1 Hz) was selected since all the frequencies showed similar trends.

The results of the ANOVA, shown in Table 26, indicate that SMA mixes and test temperature had a significant effect on measured dynamic modulus. The interaction between SMA mixes and test temperature had a significant effect as well, at a confidence limit of 95% ($\alpha = 0.05$).

Table 26. ANOVA for Measured Dynamic Modulus

Source	Degrees of Freedom	Sum Squares (X 10^9)	Mean Square (X 10^9)	F Ratio	Prob. > F
Mix	4	110.14	27.54	5.59	0.0067
Temperature	2	5765.46	2882.73	585.16	<0.0001
Mix * Temp	8	158.63	19.83	4.02	0.0112
Error	14	68.97	4.93		
Total	28	6103.20			

To show which mixes and test temperatures were significantly different from each other, Duncan's multiple range test was performed. The results are shown in Table 27. The results in Table 27 indicate that dynamic modulus results were significantly different at each test temperature. For the SMA mixes evaluated, the lower the test temperature the higher the dynamic modulus, as expected. Table 28 shows the results of Duncan's multiple range test on the individual SMA mixes. The mean dynamic modulus values are for all test temperatures, not by individual test temperature. The results show that CL-1 has a significantly different average dynamic modulus from the other mixes evaluated. It is interesting to note that CL-1 was made under the old ODOT SMA specification.

Table 27. Duncan's Multiple Range Test for Test Temperature

Grouping*	Mean E (psi)	N	Temperature
A	1,167,096	10	4 C
B	369,480	10	20 C
C	123,003	9	45 C

* Means with the same letter not significantly different.

Table 28. Duncan's Multiple Range Test for Mix ID

Grouping*	Mean E (psi)	N	Mix
A	610,811	6	SS
A	602,324	6	HL 1
A	589,787	6	HL 2
A	563,061	5	CL 2
B	473,325	6	CL 1

* Means with the same letter not significantly different.

Because there was an interaction effect, Duncan's multiple range test was performed by test temperature. Duncan's multiple range test indicates which means are significantly different at a confidence limit of 95% ($\alpha = 0.05$). The results of Duncan's multiple range test at 4 C (40°F), 20° C (68°F) and 45° C (113°F) are shown in Table 29. Means with the same letter not significantly different at a confidence limit of 95% (alpha = 0.05).

It is significant to note that the only test temperature where there was a significant difference in dynamic modulus was at 4^{o}C. SMA is used to resist fatigue cracking and rutting. Fatigue cracking and rutting are evaluated at intermediate and high pavement temperatures, not cold temperatures. At these intermediate and high pavement temperatures there was no significant difference in dynamic modulus for the SMA mixtures evaluated. Therefore, when using the M-EPDG, different SMA mixes should not impact predicted pavement performance.

Table 29. Results of Duncan's Multiple Range Test for Mix ID, by Test Temperature

Grouping*		E (psi)	N	ID
		4 C		
A		1,391,738	2	SS
A	B	1,236,421	2	HL-1
A	B	1,220,653	2	HL-2
	B	1,010,174	2	CL-1
	B	976,494	2	CL-2
		20 C		
A		430,111	2	HL-1
A		420,887	2	HL-2
A		377,947	2	CL-2
A		331,442	2	SS
A		287,016	2	CL-1
		45 C		
A		140,441	2	HL-1
A		127,823	2	HL-2
A		122,786	2	CL-1
A		109,253	2	SS
A		106,424	1	CL-2

* Means with the same letter not significantly different

MASTER CURVES

To perform a level 1 analysis using the M-EPDG, the dynamic modulus at five test temperatures, -10, 4.4, 21.1, 37.8 and 54.4°C and six frequencies, 25, 10, 5, 1, 0.1 and 0.01 Hz. are required (4). According to the user manual for the M-E PDG (4), the stiffness of HMA at all levels of temperature and time rate of load is determined from a master curve constructed at a reference temperature (generally taken as 70°F). Master curves are constructed using the principle of time-temperature superposition. The data at various temperatures are shifted with respect to time until the curves merge into a single smooth function. The procedure was described in Chapter 2 .The master curve of dynamic modulus as a function of time formed in this manner describes the time dependency of the material. The amount of shifting at each temperature required to form the master curve describes the temperature dependency of the material. The greater the shift factor, the greater the temperature dependency (temperature susceptibility) of the mixture.

Table 30 shows the dynamic modulus at each temperature and frequency required for the MEPDG. Figures 21–25 show the complete master curves.

Table 30. Average Measured Dynamic Modulus Master Curves for SMA Mixes

Temperature (C)	Frequency (Hz)	Dynamic Modulus (psi) SS	CL-1	CL-2	HL-1	HL-2
-10	25	2,934,293	2,808,822	2,224,722	2,370,881	2,420,907
	10	2,852,598	2,708,167	2,110,180	2,253,487	2,303,599
	5	2,775,858	2,615,175	2,017,566	2,157,066	2,206,090
	1	2,537,305	2,334,984	1,784,727	1,909,647	1,952,129
	0.5	2,405,149	2,185,194	1,678,053	1,794,306	1,832,346
	0.1	2,028,163	1,777,846	1,421,027	1,512,699	1,537,742
4.4	25	2,219,566	1,981,051	1,544,691	1,648,740	1,680,334
	10	1,988,691	1,736,847	1,396,828	1,485,985	1,509,711
	5	1,797,713	1,542,778	1,284,455	1,361,647	1,379,218
	1	1,330,104	1,096,960	1,028,614	1,077,918	1,082,433
	0.5	1,134,044	922,122	923,393	961,563	961,696
	0.1	737,331	589,375	698,277	715,011	709,243
21.1	25	876,221	702,745	780,384	804,426	800,156
	10	675,007	539,554	659,521	673,072	666,902
	5	548,997	440,707	575,880	583,271	576,975
	1	339,496	281,190	409,822	408,676	405,498
	0.5	279,257	236,080	350,620	347,956	347,065
	0.1	187,478	167,295	240,094	237,371	242,412
37.8	25	246,388	211,499	314,566	311,456	312,268
	10	197,060	174,513	253,302	250,372	254,598
	5	169,812	153,940	214,302	212,165	218,875
	1	128,771	122,532	144,672	145,406	156,998
	0.5	117,519	113,776	122,227	124,304	137,523
	0.1	100,249	100,143	83,414	88,258	104,197
54.4	25	117,210	113,534	121,582	123,700	136,966
	10	106,248	104,910	97,617	101,388	116,360
	5	100,077	100,005	82,993	87,869	103,837
	1	90,457	92,266	57,971	64,841	82,346
	0.5	87,713	90,303	50,148	57,641	75,557
	0.1	83,367	86,463	36,763	45,266	63,763

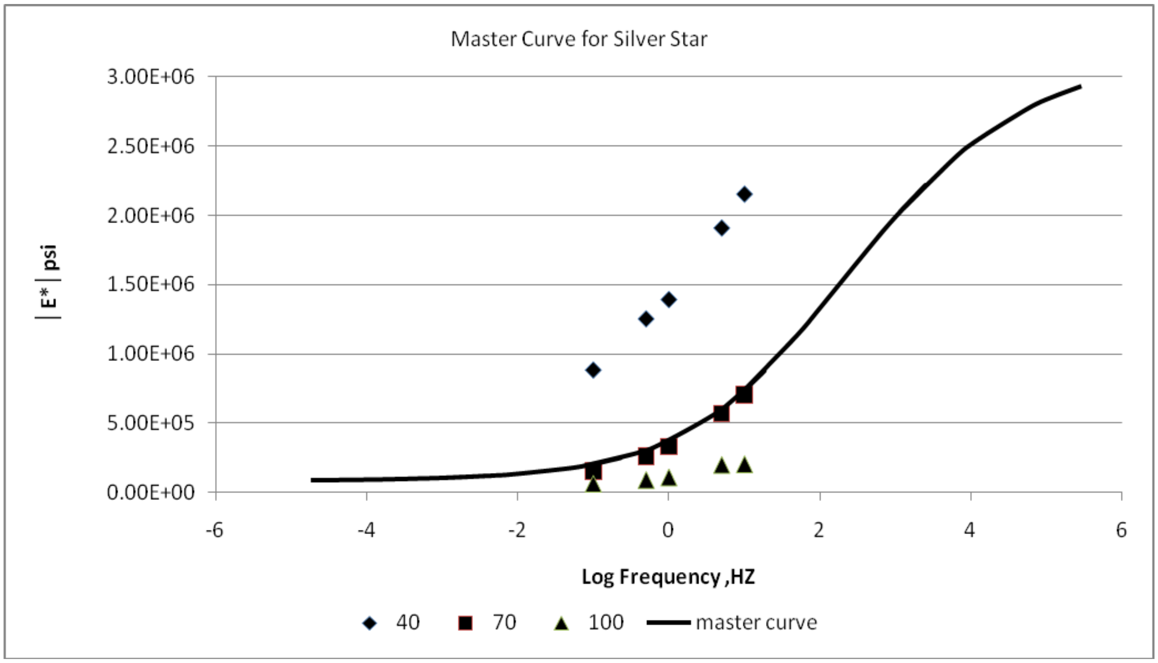

Figure 21. Master curve for SS, SMA mix.

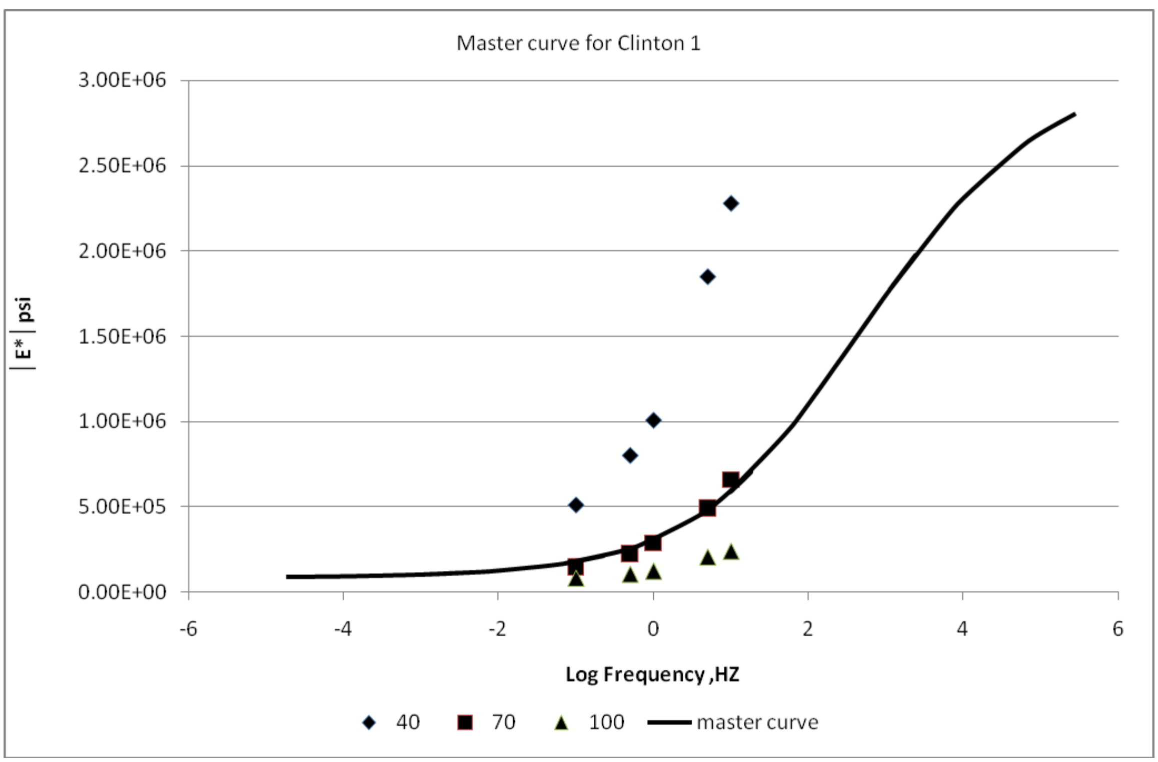

Figure 22. Master curve for CL-1, SMA mix.

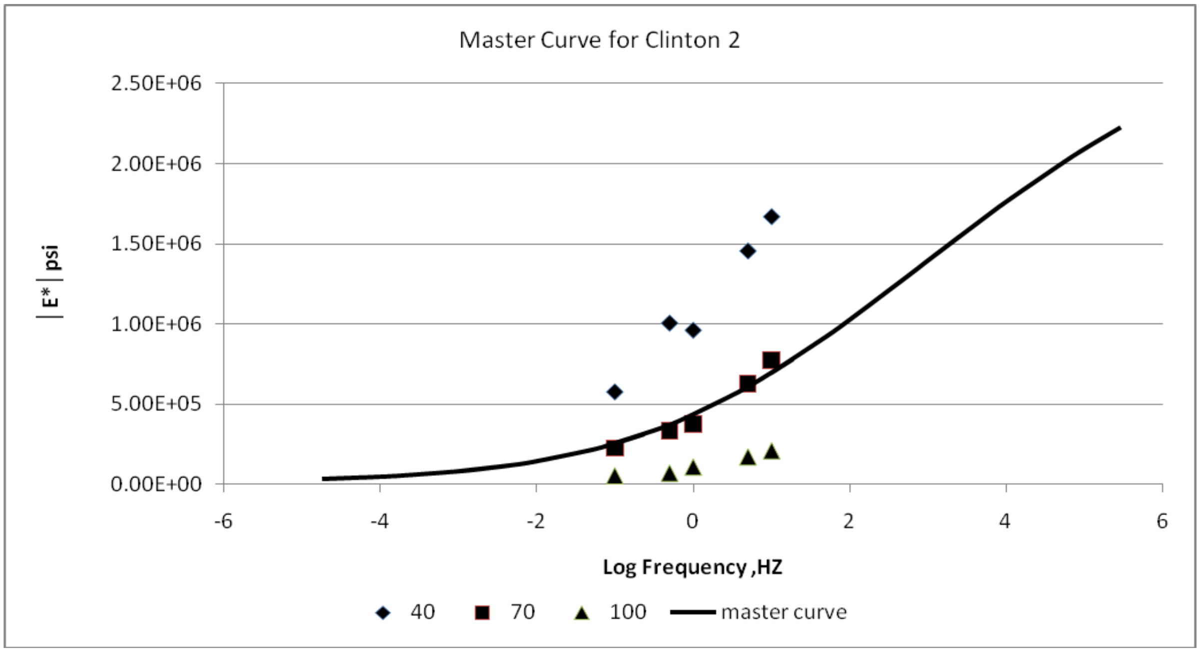

Figure 23. Master curve for CL-2, SMA mix.

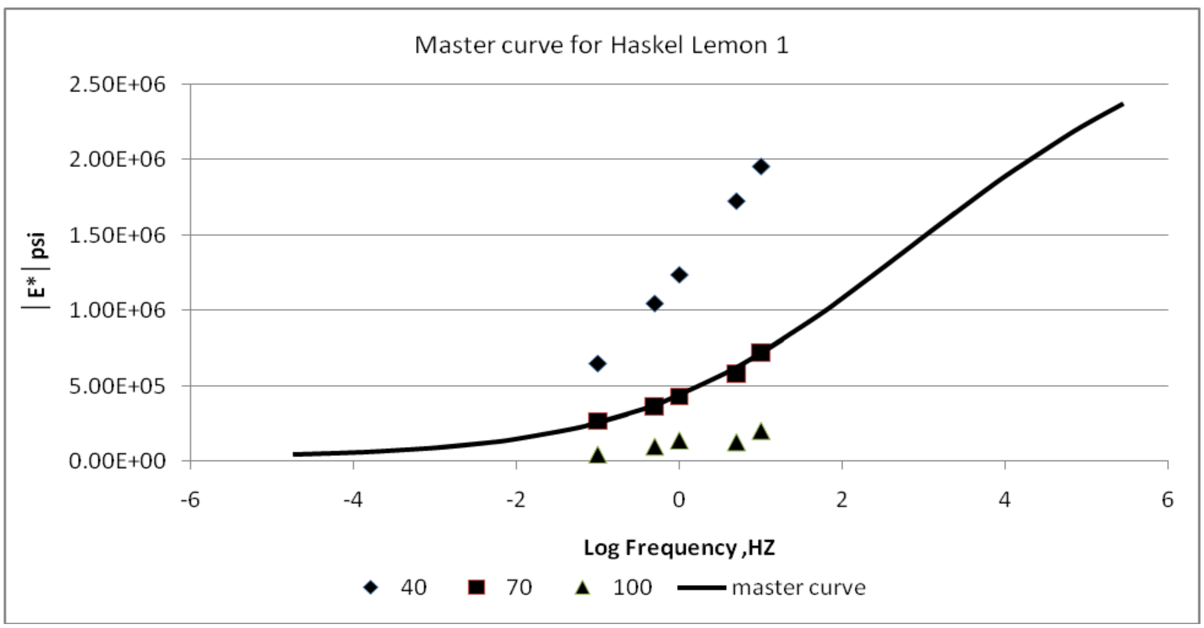

Figure 24. Master curve for HL-1, SMA mix.

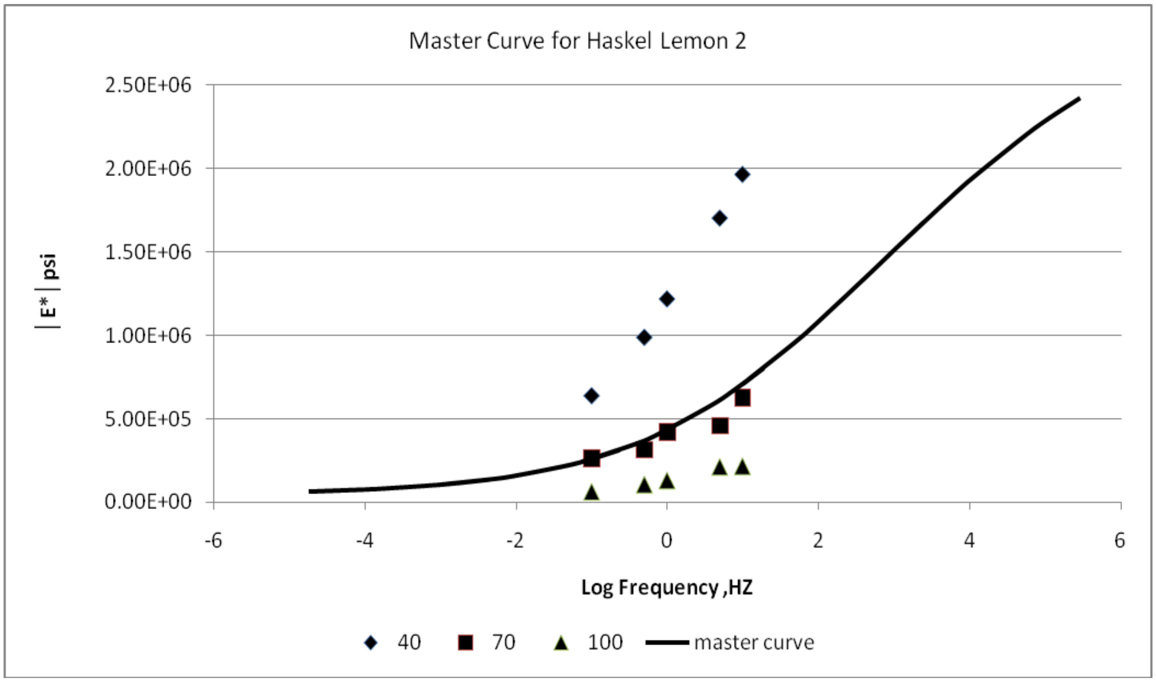

Figure 25. Master curve for HL-2, SMA mix.

COMPARISONS WITH HMA

The average dynamic modulus values of the SMA mixtures were compared to dynamic modulus values of the S-4 mix from a previous study (3) of Oklahoma HMA mixtures. Only S-4 mixtures made with Valero PG 76-28 binder were utilized. The E* values from the previous study are shown in Table 31. The comparisons can be made by master curve, which would show the effect of both temperature and frequency. However, frequency has a consistent effect on dynamic modulus and making the comparisons at one frequency simplifies the analysis. The comparisons between SMA and S-4 mixtures at a frequency of 1 Hz are shown in Figure 26.

The comparisons between SMA and S-4 HMA mixtures at 1 Hz in Figure 26 show that SMA mixes were not as stiff as S-4 HMA mixes at any of the temperatures evaluated. The S-4 mix was 30 to 70 percent stiffer than the SMA mix over the range of temperatures and frequencies tested.

Table 31. Average Measured Dynamic Modulus

Temperature (C)	Frequency (Hz)	Dynamic Modulus (psi) SMA	S4 *
-10	25	2,551,925	3,153,904
	10	2,445,606	3,089,316
	5	2,354,351	3,030,241
	1	2,103,758	2,851,853
	0.5	1,979,010	2,754,430
	0.1	1,655,495	2,473,893
4.4	25	1,814,876	2,617,325
	10	1,623,613	2,443,901
	5	1,473,162	2,296,035
	1	1,123,206	1,904,617
	0.5	980,564	1,721,328
	0.1	689,847	1,289,845
21.1	25	792,786	1,452,786
	10	642,811	1,211,151
	5	545,166	1,038,528
	1	368,936	693,635
	0.5	312,196	574,288
	0.1	214,930	365,686
37.8	25	279,235	503,765
	10	225,969	389,281
	5	193,819	320,928
	1	139,676	210,345
	0.5	123,070	178,433
	0.1	95,252	128,253
54.4	25	122,598	177,545
	10	105,305	145,839
	5	94,956	127,746
	1	77,576	99,155
	0.5	72,272	90,873
	0.1	63,124	77,517

* from previous study

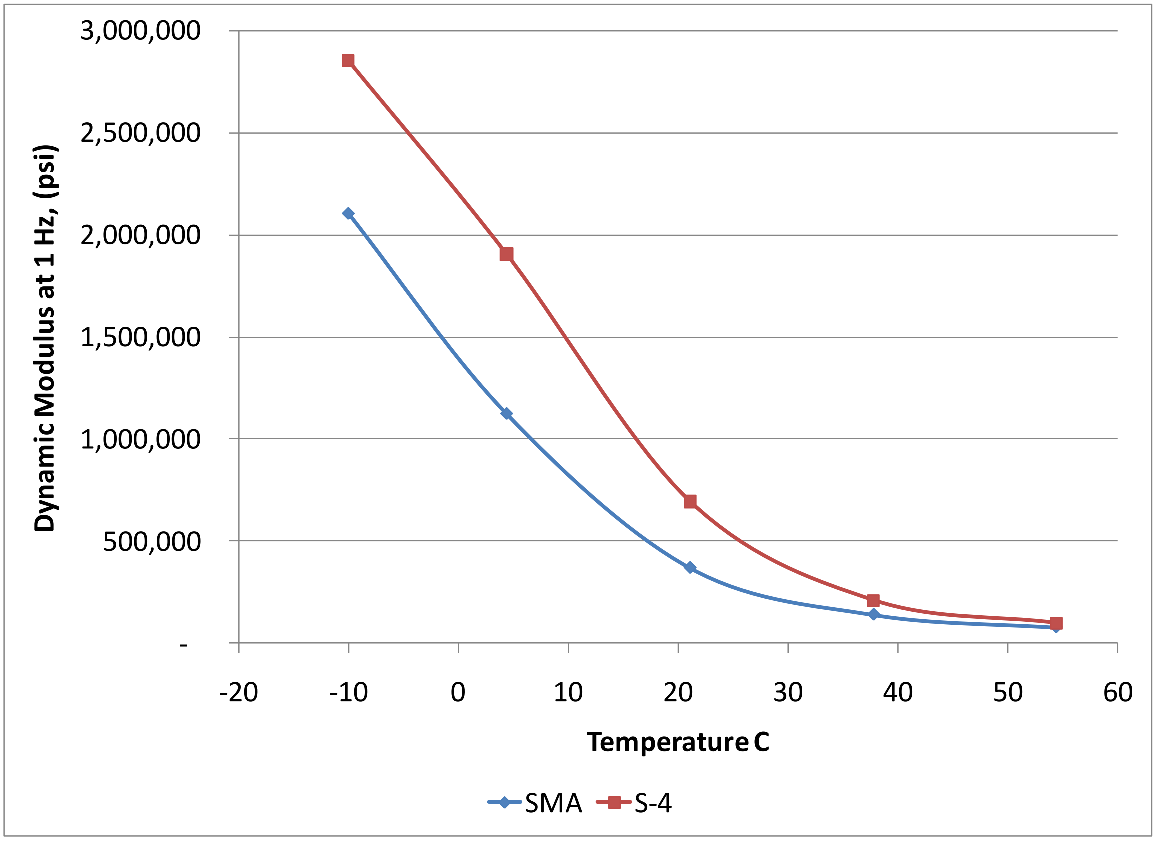

Figure 26. Average SMA and S-4 dynamic modulus, 1 Hz.

PREDICTIVE EQUATION

One of the objectives of this study was to compare experimental dynamic modulus data to predicted values using Witczak's equation. The new M-EPDG uses the laboratory dynamic modulus data for input Level 1, while it uses dynamic modulus values from Witczak's predictive equation for input Levels 2 and 3. The Witczak predictive model was based upon 2,750 test points and 205 different HMA mixtures (34 of which are modified). Most of the 205 HMA mixtures were dense-graded using unmodified asphalts. SMA is not usually considered as a dense-graded mix. The literature did not state if SMA mixtures were used.

The predicted dynamic modulus for each of the samples tested was calculated using Witczak's equation [4], as described in Chapter 2. Volumetric properties used to determine predicted dynamic modulus for each sample are listed in Table 32. The predicted dynamic modulus data for each temperature and frequency evaluated are provided in Table 33.

Table 32. Mix Properties for Calculation of Dynamic Modulus

Mixes	% Reatined			% Pass.	Va (%)	Vbeff(%)
	3/4 "	3/8 "	No. 4	No. 200		
			SMA Mixtures			
SS	0	25	70	11	5.0	13.3
CL-1	0	27	70	8.1	5.0	13.6
CL-2	0	32	70	10	5.5	13.9
HL-1	0	31	71	10	5.2	13.3
HL-2	0	35	71	10	5.4	13.9
Average	0.0	30.0	70.4	9.8	5.2	13.6
Std. Dev.	0.0	4.0	0.5	1.1	0.2	0.3
			S-4 Mixtures *			
Average	0	12.3	35.1	5.28	4.33	9.1
Std. Dev.	0	2.1	9.1	1.4	0.61	0.57

***** From previous study (3)

Table 33. Predicted Dynamic Modulus for SMA Mixes

Temperature (C)	Frequency (Hz)	SS	CL-1	CL-2	HL-1	HL-2
		Dynamic Modulus (psi)				
-10	25	2,575,577	2,439,615	2,697,608	2,664,707	2,697,608
	10	2,438,989	2,310,412	2,553,553	2,522,818	2,553,553
	5	2,330,671	2,207,943	2,439,354	2,410,319	2,439,354
	1	2,064,869	1,956,465	2,159,286	2,134,354	2,159,286
	0.5	1,945,472	1,843,488	2,033,562	2,010,437	2,033,562
	0.1	1,661,657	1,574,895	1,734,938	1,716,014	1,734,938
4.4	25	1,659,219	1,572,588	1,732,375	1,713,486	1,732,375
	10	1,496,562	1,418,626	1,561,393	1,544,843	1,561,393
	5	1,374,629	1,303,196	1,433,306	1,418,472	1,433,306
	1	1,100,870	1,043,984	1,146,040	1,134,928	1,146,040
	0.5	989,233	938,254	1,029,035	1,019,382	1,029,035
	0.1	750,643	712,231	779,304	772,628	779,304
21.1	25	805,956	764,639	837,155	829,808	837,155
	10	680,390	645,661	705,872	700,030	705,872
	5	593,389	563,207	615,011	610,168	615,011
	1	419,611	398,459	433,824	430,852	433,824
	0.5	357,069	339,145	368,737	366,387	368,737
	0.1	239,059	227,185	246,164	244,887	246,164
37.8	25	343,040	325,839	354,148	351,933	354,148
	10	273,588	259,950	281,990	280,415	281,990
	5	228,823	217,471	235,549	234,358	235,549
	1	147,812	140,564	151,683	151,112	151,683
	0.5	121,469	115,544	124,475	124,079	124,475
	0.1	75,981	72,321	77,600	77,461	77,600
54.4	25	145,793	138,646	149,595	149,039	149,595
	10	112,334	106,867	115,050	114,710	115,050
	5	91,842	87,396	93,926	93,706	93,926
	1	57,023	54,298	58,118	58,064	58,118
	0.5	46,360	44,158	47,181	47,166	47,181
	0.1	28,743	27,396	29,152	29,185	29,152

COMPARISON OF SMA MASTER CURVES

The predicted dynamic modulus values of the SMA mixtures were compared to the measured dynamic modulus values. The comparisons can be made by master curve, which would show the effect of both temperature and frequency. However, frequency has a consistent effect on dynamic modulus and making the comparison at one frequency simplifies the analysis. The comparisons between the predicted and measured dynamic modulus values at a frequency of 1 Hz are shown graphically in Figures 27-31.

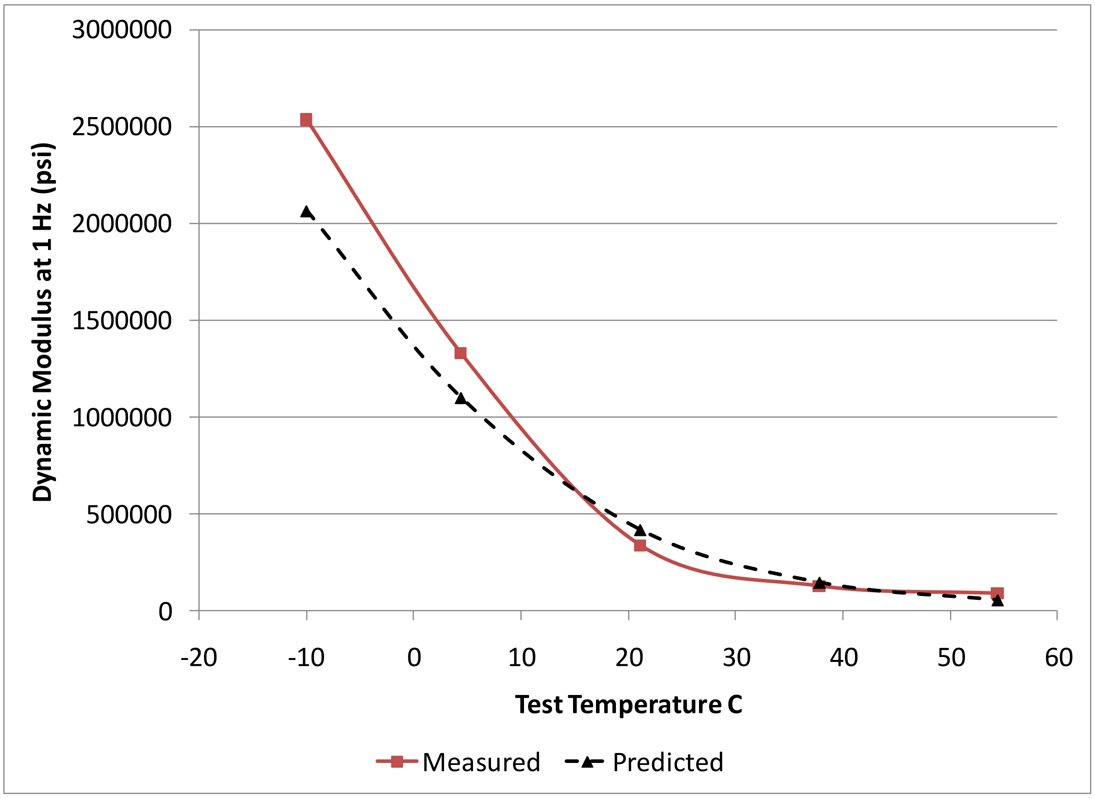

Figure 27. Measured vs. predicted dynamic modulus, SS mix.

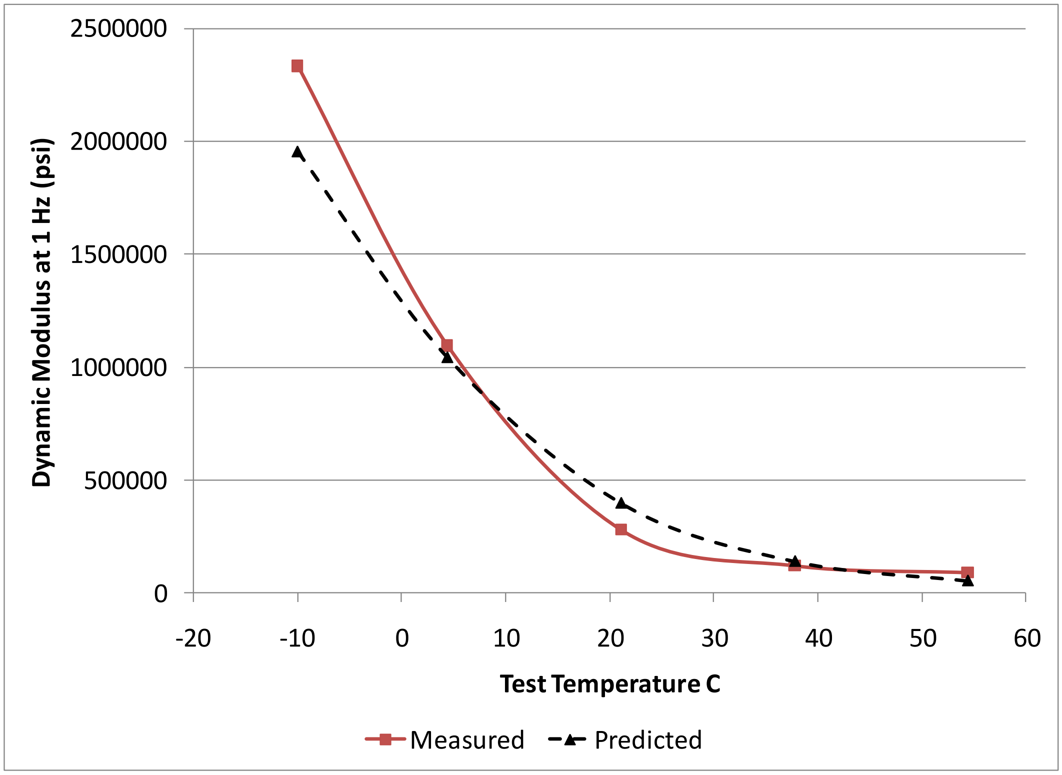

Figure 28. Measured vs. predicted dynamic modulus, CL-1 mix.

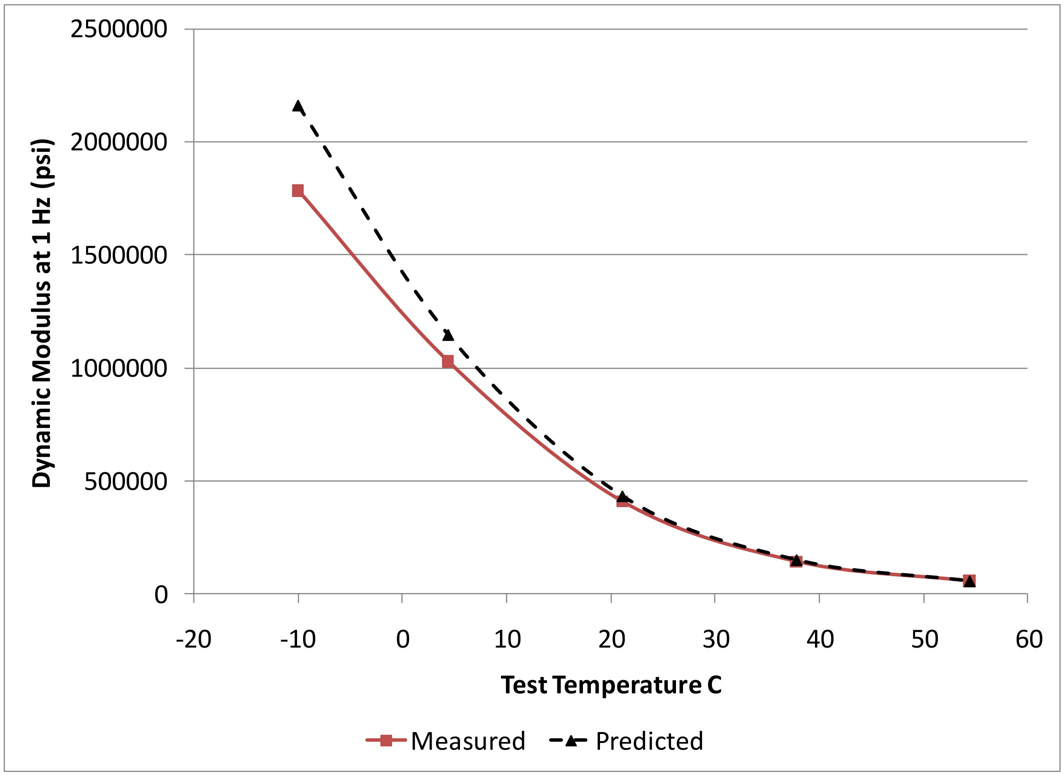

Figure 29. Measured vs. predicted dynamic modulus, CL-2 mix.

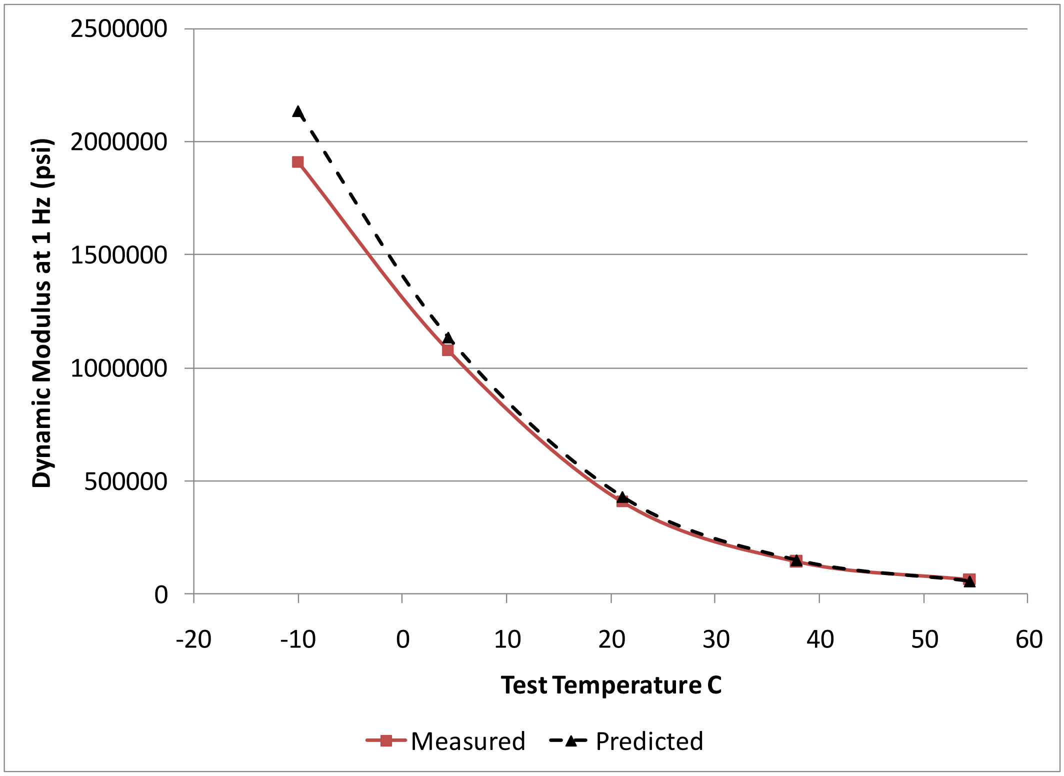

Figure 30. Measured vs. predicted dynamic modulus, HL-1 mix.

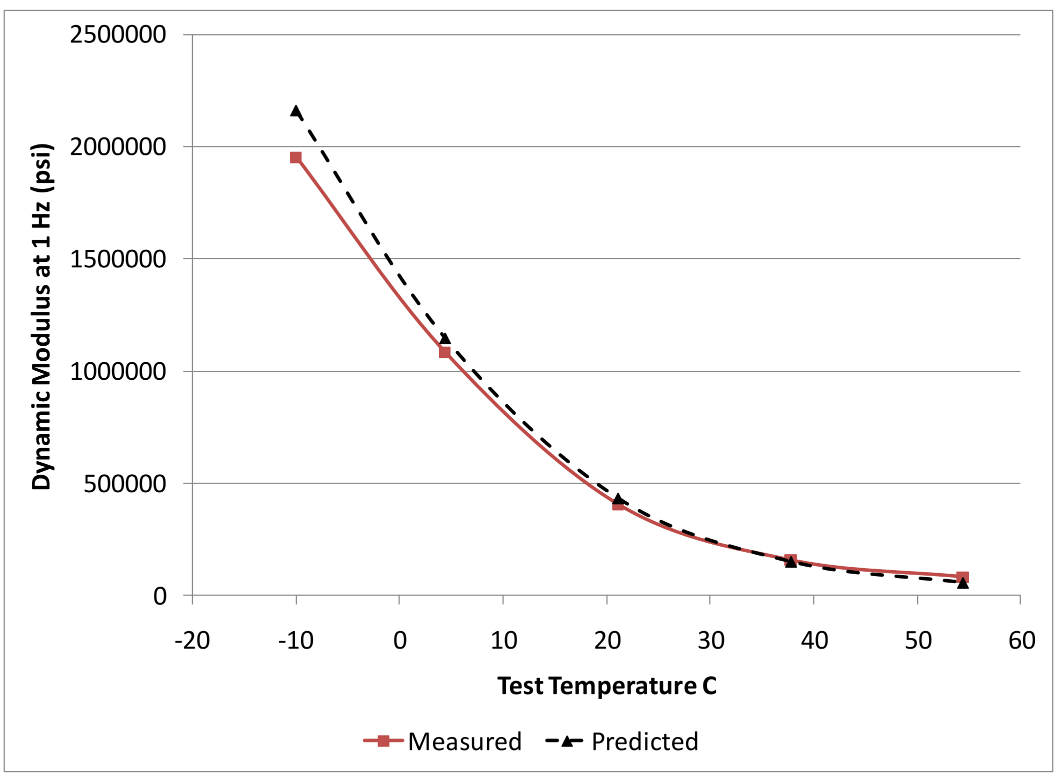

Figure 31. Measured vs. predicted dynamic modulus, HL-2 mix.

Figures 27–31 showed that the experimental and predicted dynamic modulus values are similar. The predictive equation seems to under predict dynamic modulus at higher test temperatures. Table 34 shows the average and predicted dynamic modulus values, the results are shown graphically in Figure 32. When comparing E* values, the differences in magnitudes with test temperatures can make visual comparisons difficult; therefore, Table 35 shows the percent increase in measured dynamic modulus compared to predicted dynamic modulus at 1 Hz. Table 35 shows the predictive equation slightly under predicts dynamic modulus with the discrepancy increasing with increasing temperature. The literature (6,30) has indicated close agreement between predictive equations and measured values when binder properties used in the predictive equations were from the same binders used in the measured values. However, Birgisson et al. (8) reported measured E* values considerably larger than predicted values. It appears that if the predictive equation in the M-EPDG is used to estimate dynamic modulus of SMA the values will be conservative compared to actual measured values of Oklahoma SMA mixtures.

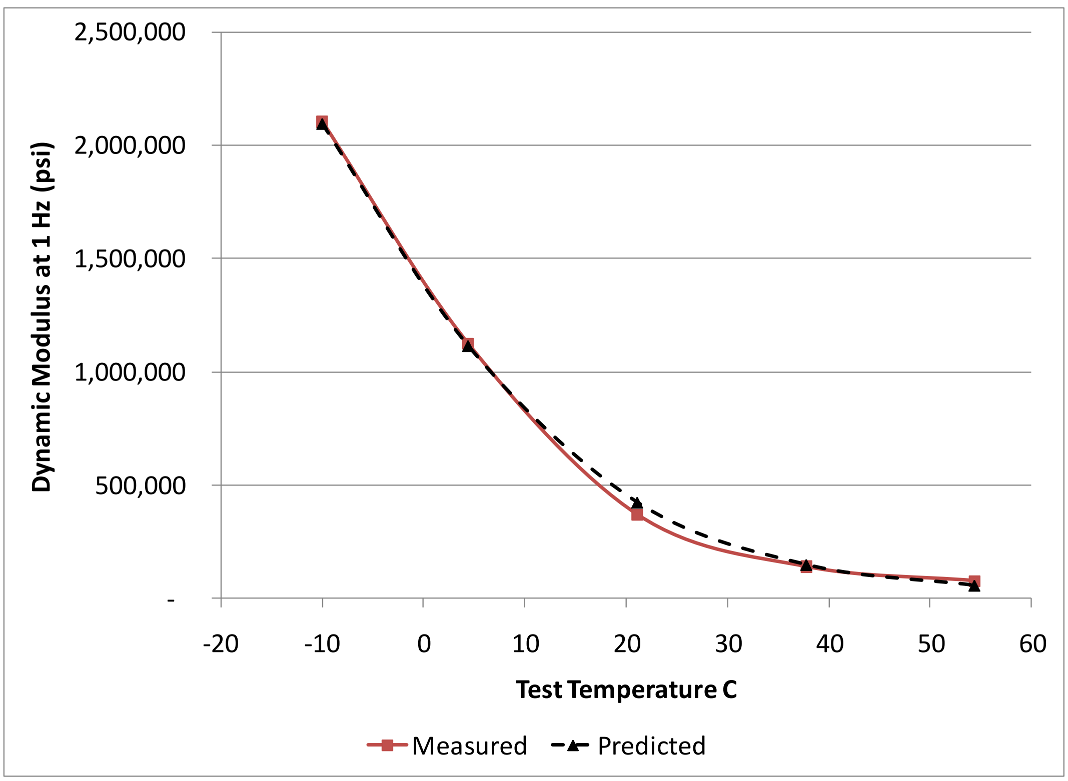

Figure 32. Average measured vs. predicted SMA dynamic modulus, 1 Hz.

Table 34. Average Measured and Predicted SMA Dynamic Modulus.

Temperature (C)	Frequency (Hz)	Dynamic Modulus (psi)	
		Measured	Predicted
-10	25	2,551,925	2,615,023
	10	2,445,606	2,475,865
	5	2,354,351	2,365,528
	1	2,103,758	2,094,852
	0.5	1,979,010	1,973,304
	0.1	1,655,495	1,684,488
4.4	25	1,814,876	1,682,009
	10	1,623,613	1,516,563
	5	1,473,162	1,392,582
	1	1,123,206	1,114,372
	0.5	980,564	1,000,988
	0.1	689,847	758,822
21.1	25	792,786	814,943
	10	642,811	687,565
	5	545,166	599,357
	1	368,936	423,314
	0.5	312,196	360,015
	0.1	214,930	240,692
37.8	25	279,235	345,822
	10	225,969	275,587
	5	193,819	230,350
	1	139,676	148,571
	0.5	123,070	122,008
	0.1	95,252	76,193
54.4	25	122,598	146,534
	10	105,305	112,802
	5	94,956	92,159
	1	77,576	57,124
	0.5	72,272	46,409
	0.1	63,124	28,726

Table 35. Percent Increase in Predicted E* Compared to Measured E*, 1 Hz

Test Temperature C	Percent Increase in Measured E* Compared to Predicted E*
-10	0.4%
4.4	0.8%
21.1	-12.8%
37.8	-6.0%
54.4	35.8%

CHAPTER 7

EVALUATION OF SMA USING MEPDG AND ASPHALT INSTITUTE METHODS

The final objective of this project was to compare the performance of SMA with S-4 mixes using the MEPGD design criteria. Because of conflicting results, SMA and S-4 mixtures were compared using the Asphalt Institute's fatigue equation as well.

MEPDG

The ANOVA showed a significant difference in dynamic modulus at 1 Hz between SMA and S-4 mixes. To determine if the differences in dynamic modulus by mix type would have an effect on pavement performance, the MEPDG was used. The analysis was performed using version 1.1 of the MEPDG that is available on the web.

Project Information

MEPDG is an analysis tool that gives levels of distress for terminal IRI, longitudinal cracking, alligator cracking, thermal cracking and permanent deformation (rutting). When performing an analysis the user selects failure criteria for each distress or selects default values. Reliability levels for the analysis may be selected as well. The recommended default values, at a reliability level of 50 percent, were used for the analysis and are shown in Table 36. Initial pavement smoothness is also required and the recommended default value of 63 in/mi, based on IRI, was used.

Table 36. Default Performance Criteria (4)

Distress	Limit
Terminal IRI (in/mile)	172
Longitudinal Cracking (ft/mile)	2000
Alligator Cracking (%)	25
Thermal Cracking (ft/mile)	1000
Permanent Deformation (in)	0.75

The MEPDG requires traffic, climatic and soil information as well as material properties. Table 37 shows the default input parameters used as baseline values for evaluation. Default values for a CL soil for the subgrade were selected as typical for the project location. A 30 year design life was selected.

Table 37. Baseline MEPDG Inputs

Design Parameter	Input Value
Pavement	4-Lane Rural Interstate
Design Life	30 years
Truck Traffic	15,000 vpd
Traffic Opening	Spring
Climate	Stillwater, OK
Depth to Water Table	30 feet
Layers	
1st Asphalt Layer	2 inch S-4, PG 70-28
2nd Asphalt Layer	3 inch S-3, PG 70-28
3rd Asphalt Layer	3 inch S-3, PG 64-22
4th Crushed Stone Base	*8 inch, Mr = 30,000 psi
Subgrade	*CL, Mr = 16,000 psi

* Default values of MEPDG except for thickness.

Structure

The pavement structure modeled (Figure 33) consisted of three lifts of HMA, two inches of an S-4 mix with PG 70-28 asphalt, three inches of an S-3 mix with PG 70-28 and 3 inches of an S-3 mix with PG 64-22 asphalt. Aggregate base consisted of eight inches of crushed stone with a default resilient modulus of 30,000 psi. The subgrade was a default CL clay with a resilient modulus of 16,000 psi.

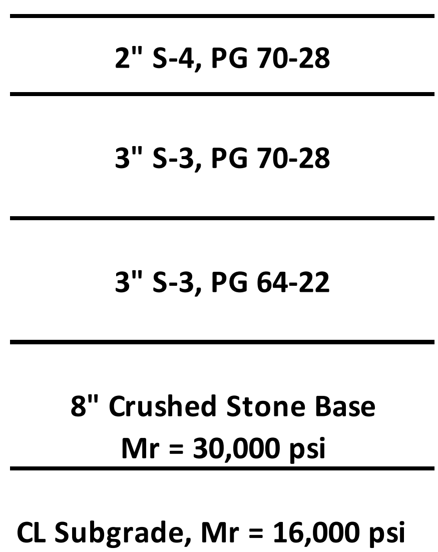

Figure 33. MEPDG trial section.

Material Properties

A sensitivity analysis of the MEPDG input parameters was performed to determine which parameters had a significant impact on predicted pavement performance. The sensitivity analysis was performed using default or level 3 values for HMA, aggregate base and subgrade material properties. Each parameter in Table 37 was evaluated individually by varying the single input parameter and holding all other input variables to the baseline values. The different input variables for each parameter are shown in Table 38.

Table 38. Parameters for Sensitivity Analysis

Design Parameter	Baseline Input Value	Lower Range	Upper Range
Traffic Opening	Spring	Fall	Summer
Truck Traffic (vpd)	15,000	8,000	20,000
Climate	Stillwater	Oklahoma City	Tulsa
Depth to Water Table	30 feet	20 feet	40 feet
Asphalt Layers			
2 inch S-4 3 inch S-3 3 inch S-3	PG 70-28 PG 70-28 PG 64-22	PG 76-28 PG 76-28 PG 64-22	PG 64-22 PG 64-22 PG 64-22
Crushed Stone Base Mr = 30,000 psi	8 inch thick	6 inch thick	12 inch thick
8” Crushed Stone Base	Mr = 30,000 psi	Mr = 20,000 psi	Mr = 40,000 psi
CL Subgrade	Mr = 16,000 psi	Mr = 13,500 psi	Mr = 18,500

The results of the sensitivity analysis are shown in Table 39. The value of each performance criteria is shown along with, in parenthesis) the percent increase or decrease in the distress from the baseline value. A negative value indicates a decrease in distress from the baseline value and a positive value an increase in distress.

Findings

Based on the sensitivity analysis performed, the following observations on overall impact on performance criteria are made.

- Climate had up to a 1% impact on performance criteria.
- Date pavement was opened to traffic had up to a 2% impact on performance criteria.
- Depth to water table had up to a 5% impact on performance criteria.
- PG Grade of the asphalt had up to a 26% impact on performance criteria.
- Subgrade resilient modulus had up to a 48% impact on performance criteria.
- Aggregate base thickness had up to a 57% impact on performance criteria.
- Traffic had up to a 59% impact on performance criteria.
- Aggregate base resilient modulus had up to a 99% impact on performance criteria.

Table 39. Results of Sensitivity Analysis on Input Parameters

		Performance criteria				
Variables	Range	Terminal IRI (in/mi)	AC surface down cracking (Longitudinal Cracking) (ft/mi)	AC Bottom up cracking (Alligator Cracking) (%)	Permanent Deformation (AC only) (in)	Permanent Deformation (Total pavement) (in)
		Change in Distresses				
Traffic opening	Fall	169.0 (-0.06%)	1080 (1.89%)	7.9 (0.00%)	0.67 (0.00%)	1.07 (0.00%)
	* Spring	169.1	1060	7.9	0.67	1.07
	Summer	168.8 (-0.18%)	1070 (0.94%)	7.9 (0.00%)	0.68 (1.49%)	1.07 (0.00%)
Traffic (AADTT)	8000	159.0 (-5.97%)	436 (-58.9%)	4.2 (-46.8%)	0.50 (-25.4%)	0.86 (-19.6%)
	* 15000	169.1	1060	7.9	0.67	1.07
	20000	175.0 (3.49%)	1560 (47.2%)	1.05 (32.9%)	0.77 (14.9%)	1.18 (10.3%)
Climate	Oklahoma city	169.1 (0.00)	1070 (0.94%)	7.9 (0.00%)	0.67 (0.00%)	1.07 (0.00%)
	* Stillwater	169.1	1060	7.9	0.67	1.07
	Tulsa	169.1 (0.00)	1060 (0.00%)	7.9 (0.00%)	0.67 (0.00%)	1.07 (0.00%)
Water table	20'	169.30 (0.12%)	1070 (0.94%)	7.9 (0.00%)	0.68 (1.49%)	1.08 (0.93%)
	* 30'	169.1	1060	7.9	0.67	1.07
	40'	168.9 (-0.12%)	1110 (4.72%)	7.9 (0.00%)	0.67 (0.00%)	1.06 (-0.93%)
Aggregate Base Thickness	6	169.3 (0.12%)	1660 (56.6%)	9.0 (13.9%)	0.66 (-1.49%)	1.06 (-0.93%)
	*8	169.1	1060	7.9	0.67	1.07
	12	168.6 (-0.30%)	643 (-39.3%)	6.5 (-17.7%)	0.74 (4.48%)	1.08 (0.93%)
PG Grades Asphalt layers	76-28,76-28,64-22	164.7 (-2.60%)	842 (-20.6%)	7.32 (-7.34%)	0.575 (-14.2%)	0.97 (-9.35%)
	* 70-28,70-28,64-22	169.1	1060	7.9	0.67	1.07
	64-22,64-22,64-22	171.4 (1.36%)	1330 (25.5%)	8.1 (2.53%)	0.73 (8.96%)	1.13 (5.61%)
Aggregate base layer (Mr)	20000	170.2 (0.65%)	2110 (99.1%)	10.1 (27.9%)	0.65 (-2.99%)	1.07 (0.00%)
	*30000	169.1	1060	7.9	0.67	1.07
	40000	168.1 (-0.59%)	446 (-57.9%)	6.1 (-22.8%)	0.69 (2.99%)	1.07 (0.00%)
Subgrade (Mr)	13500	171.1 (1.18%)	603 (-43.1%)	8.7 (10.1%)	0.66 (-1.49%)	1.11 (3.74%)
	*16000	169.1	1060	7.9	0.67	1.07
	18500	167.6 (-0.89%)	1570 (48.1%)	7.3 (-7.59%)	0.68 (1.49%)	1.04 (-2.80%)

Based on the sensitivity analysis performed, the following observations on the effect of input variables on individual performance criteria are made.

- With the exception of traffic and PG grade of the asphalt, none of the variables evaluated had more than a 1.2% impact on pavement roughness. Traffic and PG grade of the asphalt had up to a 6% impact on roughness.
- Date of traffic opening, climate and depth to water table had less than a 5% impact of top down longitudinal cracking.
- Aggregate base resilient modulus had the largest impact on top down cracking (up to 99%), followed by traffic (up to 59%), aggregate base thickness (up to 57%), subgrade resilient modulus (up to 48%) and PG grade of the asphalt (up to 26%).
- It is interesting to note that for HMA and aggregate base, the softer the binder the more top down cracking, but for subgrade, the stiffer the subgrade the more top down cracking.
- Date of traffic opening, climate and depth to water table had no effect on bottom up (alligator) cracking.
- Traffic had the largest impact on bottom up (alligator) cracking (up to 47%), followed by aggregate base resilient modulus (up to 28%), aggregate base thickness (up to 18%), subgrade resilient modulus (up to 10%) and PG grade of the asphalt (up to 7%).
- Date of traffic opening, climate, depth to water table, aggregate base thickness, aggregate base resilient modulus and subgrade resilient modulus had little effect on permanent deformation of the asphalt layer (less than 5%).
- Traffic had the largest impact on permanent deformation of the asphalt layer (up to 25%), followed by PG grade of the asphalt (up to 14%).
- Date of traffic opening, climate, depth to water table, aggregate base thickness, aggregate base resilient modulus and subgrade resilient modulus had little effect on total permanent deformation (less than 4%).
- Traffic had the largest impact on total permanent deformation (up to 20%), followed by PG grade of the asphalt (up to 9%).
- Although the effect is minor on asphalt and total permanent deformation, there appears to be minor inconsistencies with date of traffic opening, aggregate base thickness, aggregate base resilient modulus and subgrade resilient modulus.

Based on the sensitivity analysis performed, it appears that default values for date of traffic opening, climate and depth to water table could be used as they had little impact on performance criteria. Subgrade resilient modulus had its biggest impact on top down cracking (up to 48%) followed by bottom up alligator cracking (up to 10%). With the exception of top down fatigue cracking, and there are issues associated with the model for top down fatigue cracking that will be discussed later, subgrade resilient modulus is not a major performance factor for the thicker pavements evaluated. Traffic, aggregate base thickness, aggregate base resilient modulus, and PG grade of the asphalt impacted all performance criteria. Reasonable values for Oklahoma materials should be used and their effect on performance evaluated when using the MEPDG for design of Oklahoma asphalt pavements.

DESIGN TRIALS AND RESULTS

To evaluate the effect of SMA on performance, the trial section shown in Figure 33 was considered as the baseline for trial runs using the MEPDG. Two inches of SMA, and the corresponding dynamic modulus, was substituted for the top lift of S-4 HMA using PG 76-28 asphalt. Both default E* and measured E* values were used for SMA and the top lift of S-4. In addition, ODOT typically uses a high water table and lower subgrade resilient modulus values than recommended by the MEPDG. The evaluation was performed using the baseline depth to water table and baseline subgrade resilient modulus as well as a five foot depth to the water table with a subgrade resilient modulus of 5,000 psi. The results are shown in Table 40.

Findings

Based on the comparisons made between SMA and S-4, the following observations on the effect of input variables on individual performance criteria are made.

- SMA, with its lower dynamic modulus, had more roughness. The percent increase was not impacted by subgrade resilient modulus and depth to water table.
- SMA, with its lower dynamic modulus, had more top down cracking. The lower the subgrade resilient modulus and less depth to water table the less top down cracking. There is a tremendous increase in top down cracking with a stiffer subgrade. However, all values were considerably below the threshold value.
- SMA, with its lower dynamic modulus, had more bottom up (alligator) cracking. The lower the subgrade resilient modulus and less depth to water table the more alligator cracking. All values were considerably below the threshold value.
- SMA, with its lower dynamic modulus, had more permanent deformation in the asphalt layers. There is a slight increase in permanent deformation in the asphalt layers with a stiffer subgrade. All values exceeded the threshold value. MEPDG results contradict Hamburg rut test data and published literature.
- SMA, with its lower dynamic modulus, had more total permanent deformation. The lower the subgrade resilient modulus and less depth to water table, the more total permanent deformation, the opposite effect seen with permanent deformation in the asphalt layers. All values exceeded the threshold value.

Based on the trial runs, it appears that when it comes to asphalt layers, stiffer is better. The MEPDG results seem to contradict themselves with permanent deformation and go against Hamburg rut test results and published literature (22,23,24,31).

Table 40. Results of Sensitivity Analysis on SMA and S-4 Parameters

		Default E*						Expermental E*					
Depth to Water Table		5 ft			30 ft			5 ft			30 ft		
Subgrade Resilient Modulus		Mr = 5,000 psi			Mr=16,000 psi			Mr = 5,000 psi			Mr=16,000 psi		
Performance criteria	***Distress Target***	**S4**	**SMA**	**% Increase**	**S4**	**SMA**	**% Increase**	**S4**	**SMA**	**% Increase**	**S4**	**SMA**	**% Increase**
Terminal IRI (in/mi)	172	185.5	192.7	3.9	164.7	172.2	4.6	182.9	191.3	4.6	162.6	171.4	5.4
AC Surface Down Cracking, Long. Cracking (ft/mile)	2000	1.3	2.2	69.2	842	1110	31.8	0.5	1.4	180	295	612	108
AC Bottom Up Cracking, Alligator Cracking (%)	25	14.5	17.1	17.9	7.3	8.9	21.9	12.3	15.1	22.8	6	7.6	26.7
Permanent Deformation, AC only (in.)	0.25	0.52	0.62	19.2	0.58	0.73	25.9	0.51	0.64	25.5	0.55	0.73	32.7
Permanent Deformation, Total Pavement (in.)	0.75	1.39	1.53	10.1	0.97	1.13	16.5	1.36	1.53	12.5	0.93	1.13	21.5

ASPHALT INSTITUE FATIGUE ANALYSIS

There are numerous other mechanistic-empirical thickness design programs available. Many use the Asphalt Institute's (32) fatigue equation:

$$Nf = 0.079488C\varepsilon_t^{-3.291}E^{*-0.854} \quad [7]$$

where: Nf = log load applications to failure
C = Mix correction factor
ε_t = tensile strain at bottom of bound layer
E* = dynamic modulus
M = mix correction factor

The above equation is adjusted for mix properties through a mix correction factor C as shown in equation [8].

$$C = 10^M \quad [8]$$

where: M = 4.84 [Vb / (Va + Vb) – 0.69]

with: Vb = volume of binder
Va = air voids of mix

The Asphalt Institute (AI) recommends a standard mix if void properties are not known, assuming a default binder volume of 11% and air voids of 5%. These assumptions result in an M of zero and a C of 1(32).

The expected fatigue life of S-4 and SMA mixes were evaluated by equation [7] with average measured E* values. Default Vb and Va values were used along with average values determined from the mixes evaluated. The results are shown in Table 41. Tensile strain values were determined by using the typical pavement section shown in Figure 33 and using Everstress (33) to calculate the strain. From the calculated strain, additional strain values were selected to calculate the fatigue life at various strain levels.

The results of the AI fatigue analysis using for the default C values and the calculated C values are presented graphically in Figures 34 and 35, respectively. The SMA mix has a significantly longer fatigue life than the S-4 mix regardless whether the mix is adjusted for mix properties using the C values. It should be noted that adjusting the fatigue equation for SMA mix properties significantly increases the fatigue life compared to using default mix properties. The same is true for the S-4 mix but the increase in fatigue life is not near as large.

Table 41. Asphalt Institute Fatigue Input Parameters and Results

Mix	E* (1 HZ, 70 F)	Va	Vb	M	C	et	Nf
SMA	404,000				1.000	0.00015	4,983,644
					1.000	0.00020	1,933,631
					1.000	0.00025	927,775
					1.000	0.00030	509,164
SMA	404,000	5.26	14.17	0.1901	1.5493	0.00015	7,721,196
		5.26	14.17	0.1901	1.5493	0.00020	2,995,789
		5.26	14.17	0.1901	1.5493	0.00025	1,437,409
		5.26	14.17	0.1901	1.5493	0.00030	788,851
S4	690,000				1.000	0.00015	3,155,147
					1.000	0.00020	1,224,183
					1.000	0.00025	587,375
					1.000	0.00030	322,352
S4	690,000	4.33	10.15	0.05308	1.1300	0.00015	3,565,325
		4.33	10.15	0.05308	1.1300	0.0002	1,383,330
		4.33	10.15	0.05308	1.1300	0.00025	663,735
		4.33	10.15	0.05308	1.1300	0.0003	364,258

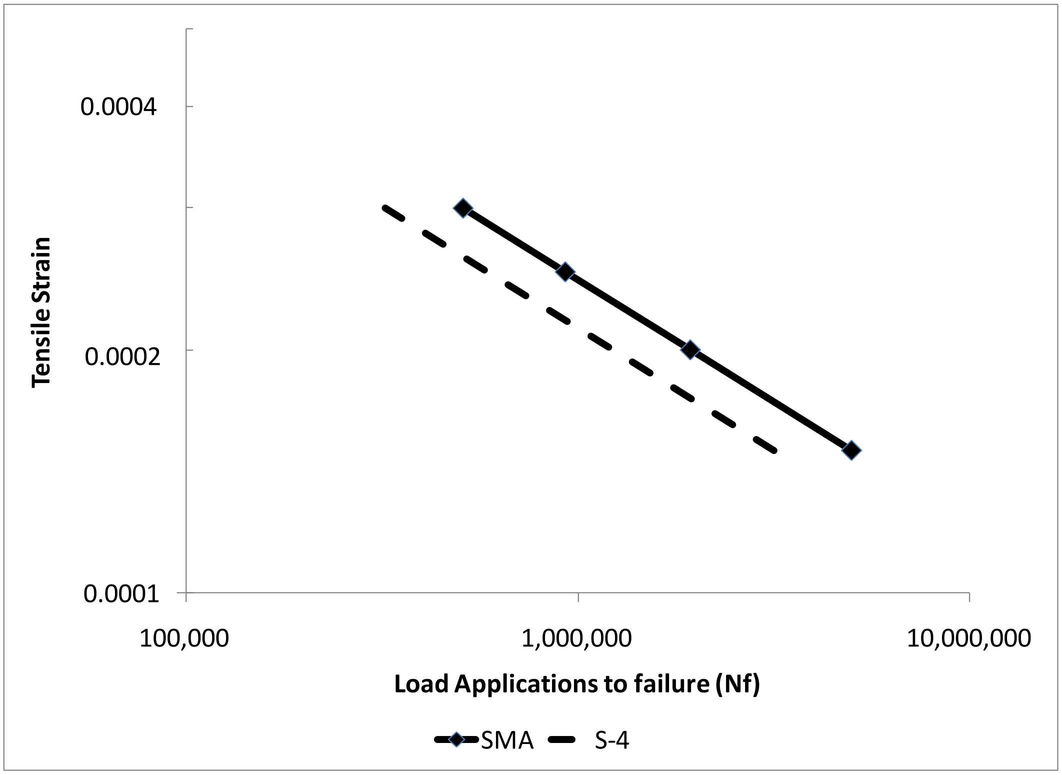

Figure 34. AI fatigue equation results, default C values.

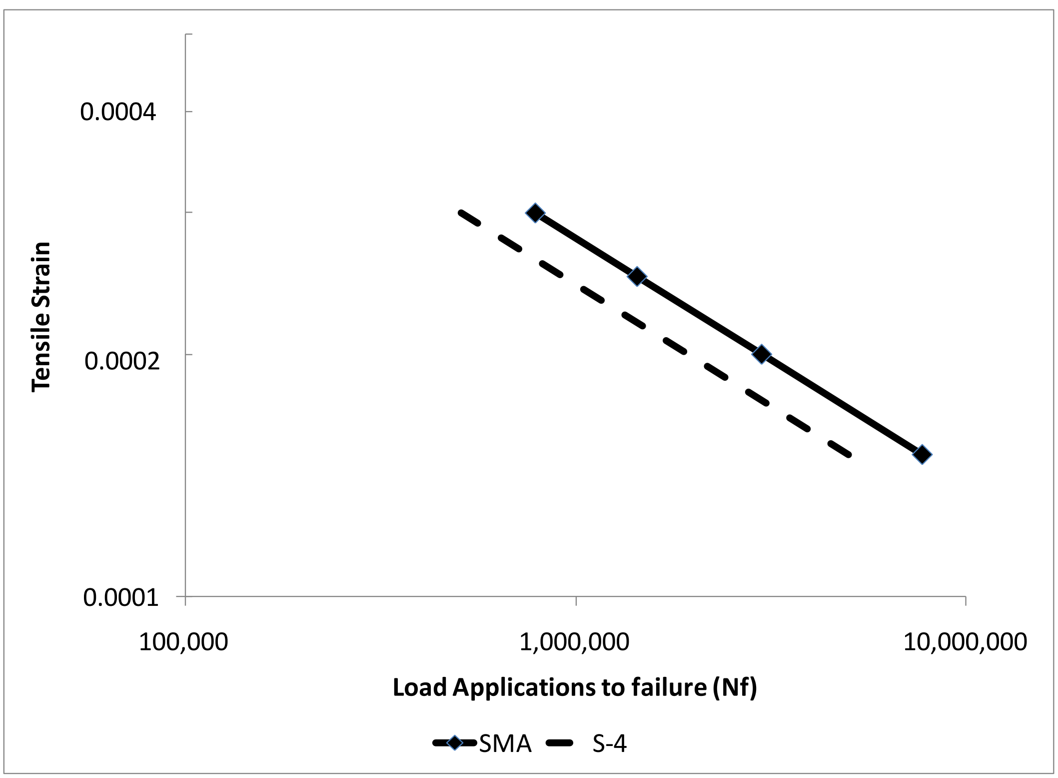

Figure 35. AI fatigue equation results, calculated C values.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

Based on the results of this study and for the materials, test methods and equipment evaluated, the following conclusions are warranted.

Hamburg Rut Testing

1. All SMA mixtures easily passed the maximum Hamburg rut depth specification limit of less than 12.5 mm rut depth at 20,000 passes.
2. Two of the four S-4 mixtures evaluated easily passed the maximum Hamburg rut depth specification limit of less than 12.5 mm rut depth at 20,000 passes.
3. Two of the four S-4 mixtures approached the maximum Hamburg rut depth specification limit of less than 12.5 mm rut depth at 20,000 passes with rut depths exceeding 10 mm.
4. The best performing mix was an S-4 mix. However, the next four mixes with the lowest rut depths were SMA mixes and the two worst performing mixes were S-4 mixes.
5. It is possible to make an S-4 mix that will resist rutting as well as an SMA mix.
6. SMA mixes had statistically significant lower Hamburg rut depths than S-4 mixes.

SMA Dynamic Modulus

1. Test temperature had a significant effect on measured dynamic modulus.
2. Test frequency had a significant effect on measured dynamic modulus.
3. For the SMA mixtures tested, there was no statistically significant difference in dynamic modulus except at the lowest test temperature (4^oC).
4. For measured dynamic modulus, SMA mixtures were not as stiff as S-4 mixtures made with the same source of PG 76-28 asphalt. The S-4 mixtures were 30-70% stiffer than SMA mixes over the range of temperatures and frequencies tested. The differences were more pronounced at lower test temperatures.
5. For predicted dynamic modulus, SMA mixtures were not as stiff as S-4 mixtures.
6. Average predicted and measured dynamic modulus of SMA mixtures were similar. At high test temperatures and low test frequencies, measured dynamic modulus was larger than predicted dynamic modulus.

MEPDG Sensitivity Analysis

1. Climate, date of traffic opening and depth to water table had little impact on pavement performance criteria.
2. Traffic, aggregate base stiffness, aggregate base layer thickness, asphalt binder grade and subgrade resilient modulus all had significant impacts on pavement performance criteria.
3. Pavement roughness was not significantly impacted (< 6% change) by any of the factors evaluated.

4. Aggregate base resilient modulus had the largest impact on top down cracking (up to 99%), followed by traffic (up to 59%), aggregate base thickness (up to 57%), subgrade resilient modulus (up to 48%) and PG grade of the asphalt (up to 26%).
5. It is interesting to note that for HMA and aggregate base, the softer the binder the more top down cracking, but for subgrade, the stiffer the subgrade the more top down cracking.
6. Traffic had the largest impact on top down cracking (up to 47%), followed by aggregate base resilient modulus (up to 28%), aggregate base thickness (up to 18%), subgrade resilient modulus (up to 10%) and PG grade of the asphalt (up to 7%).
7. Traffic had the largest impact on permanent deformation of the asphalt layer (up to 25%), followed by PG grade of the asphalt (up to 14%).
8. Traffic had the largest impact on total permanent deformation (up to 20%), followed by PG grade of the asphalt (up to 9%).

MEPDG SMA Design Trials

1. SMA, with its lower dynamic modulus, had more roughness, more top down cracking, more bottom up (alligator) cracking, more permanent deformation in the asphalt layers, and more total permanent deformation than S-4 mixes.
2. MEPDG results contradict Hamburg rut test results, published literature and Asphalt Institute fatigue results.

Asphalt Institute Fatigue Life

1. SMA has a significantly longer fatigue life than S-4 regardless of whether the mix is adjusted using the mix adjustment (C) values.
2. Adjusting the fatigue equation for SMA mix properties significantly increases the fatigue life compared to using default mix properties.
3. Adjusting the fatigue equation for S-4 mix properties significantly increases the fatigue life compared to using default mix properties, but not as large as for SMA.

RECOMMENDATIONS

The literature indicated that SMA is a highly rut resistant, durable mix. Hamburg rut test data and Asphalt Institute fatigue equations confirm this. MEPDG prediction models contradict these findings. Field test results indicate that rutting that requires corrective action occurs in the top 3-4 inches of a pavement (31). The MEPDG attributes a significant portion of the permanent deformation to base and subgrade layers.

The MEPDG indicated that stiffer S-4 mixes would have less top down and bottom up fatigue cracking. The MEPDG models are calibrated from LTPP data and at least initially, no effort was made to differentiate in the field between bottom up and top down longitudinal cracking. Top down cracking models are relatively new and may need additional calibration. The MEPDG also indicated that aggregate base resilient modulus and thickness, as well as subgrade resilient modulus had significant effects on top down cracking, with softer subgrades performing better. If the MEPDG models are correct, evaluation of pavement design practices could be warranted.

Based on the results of this study and for the materials, test methods and equipment evaluated, the following conclusions are warranted.

1. The use of SMA as a wearing surface on high trafficked pavements should be encouraged due to its superior rut resistance and fatigue life.
2. Care should be exercised if using the MEPDG for design of mixtures using SMA as the MEPDG provided results that contradict field performance and published literature (22,23,24,31).
3. If using the MEPDG, the SMA mix properties shown in Table 42 should be used with the MEPDG predictive equation.
4. If using the MEPDG, the SMA dynamic modulus values shown in Table 43 could be used.

Table 42. Recommended SMA Mix Properties for E* Predictive Equations

Mix Property	SMA
% Retained 3/4" Sieve	0
% Retained 3/8" Sieve	30
% Retained No. 4 Sieve	70
% Retained No. 200 Sieve	9.8
V_a (%)	5.2
V_{beff} (%)	13.6

Table 43 Recommended E* Data for SMA

Temperature (C)	Frequency (Hz)	Dynamic Modulus (psi) Measured	Predicted	Recommended
-10	25	2,551,925	2,615,023	2,550,000
	10	2,445,606	2,475,865	2,450,000
	5	2,354,351	2,365,528	2,350,000
	1	2,103,758	2,094,852	2,100,000
	0.5	1,979,010	1,973,304	1,900,000
	0.1	1,655,495	1,684,488	1,650,000
4.4	25	1,814,876	1,682,009	1,750,000
	10	1,623,613	1,516,563	1,600,000
	5	1,473,162	1,392,582	1,400,000
	1	1,123,206	1,114,372	1,100,000
	0.5	980,564	1,000,988	990,000
	0.1	689,847	758,822	725,000
21.1	25	792,786	814,943	800,000
	10	642,811	687,565	650,000
	5	545,166	599,357	575,000
	1	368,936	423,314	400,000
	0.5	312,196	360,015	325,000
	0.1	214,930	240,692	225,000
37.8	25	279,235	345,822	325,000
	10	225,969	275,587	250,000
	5	193,819	230,350	200,000
	1	139,676	148,571	150,000
	0.5	123,070	122,008	125,000
	0.1	95,252	76,193	85,000
54.4	25	122,598	146,534	135,000
	10	105,305	112,802	110,000
	5	94,956	92,159	95,000
	1	77,576	57,124	75,000
	0.5	72,272	46,409	70,000
	0.1	63,124	28,726	60,000

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