DETERMINATION OF INPUT DATA FOR OKLAHOMA SMA MIXTURES FOR USE IN THE MEPDG

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USE IN THE MEPDG

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CHAPTER I

INTRODUCTION

INTRODUCTION

Since its first adoption to U.S. from European countries in 1991, the use of Stone Matrix Asphalt (SMA) for surface courses on medium and high traffic roads has been growing tremendously. The two main reasons for its growing acceptance by many states have been its improved rut resistance and durability. Today many states use SMA as one of their standard mix types (1).

SMA mixtures are prepared using higher asphalt contents and more durable aggregates than conventional superpave mixtures. This makes their initial cost higher. However, experience has showed that this higher initial cost may be more than offset by the expected increase in pavement life (2).

SMA is difficult to work with because of its high coarse aggregate content and relatively stiff asphalt binder (3). When SMA was first introduced to the U.S, there were not many contractors experienced enough to handle the construction of SMA. Over time many contractors developed the experience needed to work with this mix without difficulty (1).

The current AASHTO pavement design guide (4) uses empirical performance equations, which were developed using 1950's AASHO road test data, to design new and rehabilitated highway pavements. This guide cannot continue to be used as a primary pavement design guide because it has many limitations. To overcome these limitations, the AASHTO Joint Task Force on

Pavements (JTFP) initiated an effort to develop an improved pavement design guide which is based on mechanistic principles. The new mechanistic-empirical pavement design guide (MEPDG) (5) is the end result of this effort.

The new pavement design guide was first developed under NCHRP project 1-37 A between 1998 and 2004. This design guide was known as the *AASHTO 2002 Design Guide* but its name was later changed to the *Mechanistic Empirical Pavement Design Guide* (MEPDG). The MEPDG have been updated many times after its first introduction to the public in 2004. The design guide is still under evaluation and has not been adopted by AASHTO to date (6).

The MEPDG uses dynamic modulus (E*) and poison's ratio (μ) inputs to compute critical responses for HMA materials. The design guide requires a dynamic modulus input at a minimum of three temperatures and three frequencies to develop a master curve. It is from this master curve that modulus of HMA at all levels of temperatures and frequencies are determined in the MEPDG. The dynamic modulus of HMA mixes can be measured experimentally in accordance with NCHRP 9-29 PP 02 testing protocol (7) or can be computed using different predictive equations. Typical assumed or correlated values are used for poison's ratio. (5)

The MEPDG uses a hierarchical approach for materials characterization. There are three levels to characterize the asphalt layers in which the first level provides the highest design reliability and each succeeding level is a drop in design reliability. The MEPDG uses actual laboratory measured dynamic modulus and asphalt binder data at input level 1 to develop a master curve. At input levels 2 and 3, the master curves are constructed using E* values computed from predictive equations. (5)

PROBLEM STATEMENT

SMA has been under utilized in Oklahoma. This is because of:

a) The extra expenses required to prepare this mix.

b) A lack of data in the state which indicates that SMA performs better than conventional superpave mixtures.

c) A lack of input data which are required for use in the MEPDG.

OBJECTIVES

The objectives of this research project were to compare the performance of SMA to conventional S-4 mixes to see which one performs better and to develop input data of SMA for use in the MEPDG.

The mixture performance at the end of the design period was predicted using the MEPDG software. The software needs dynamic modulus, aggregate gradation and volumetric properties of the mixes to predict distresses.

RESEARCH METHODOLOGY

To accomplish the aforementioned objectives, six tasks were performed in this research project. These are: Literature Review, Materials, Dynamic Modulus Test, Laboratory Test Results, Analysis of Test Results, Performance Prediction using MEPDG Software, and Conclusions and Recommendations.

Task 1: Literature Review: available literatures which deal with MEPDG and dynamic modulus were reviewed. This was done to get insight on the use of dynamic modulus in the MEPDG. Literature about SMA was also reviewed to get ideas on the general and specific features of SMA mixtures.

Task 2: Materials: SMA and S-4 mixtures, which were similar to field produced mixtures, were prepared in accordance with ODOT's requirements. Five SMA and two S-4 mixes were prepared using PG 76-28 asphalt cement. A voids analysis was then made on each specimen to verify if their mix properties meet ODOT mix requirements.

Task 3: Dynamic Modulus Test: the most efficient way of preparing test specimens for dynamic modulus was identified. The equipment necessary to prepare and test the specimens for dynamic modulus were also arranged in this task.

Task 4: Test Results: A dynamic modulus test was performed on the test specimens prepared in task 3. The specimens were tested in accordance with NCHRP 9-29 PP 02 testing protocol.

Task 5: Analysis of Results: two way analysis of variance (ANOVA) was performed to determine if there was a significant difference between the mean dynamic modulus values of S-4 and SMA mixtures and to see if there were a difference in the dynamic modulus values measured at different test temperatures. ODOT's S-4 mixtures which were sampled and tested previously by Cross et al. were also included in the analysis.

After the statistical analysis, the dynamic modulus data obtained in task 4 were manipulated to develop master curves. From these master curves, E* values at the recommended temperatures and frequencies of the MEPDG were estimated. Dynamic modulus values were also predicted using the Witzack predictive equation for each mix sampled in task 3.

The mean dynamic modulus values of SMA mixtures were then compared with S-4 mixtures. The comparison was made using both the measured and predicted dynamic modulus values.

Task 6: MEPDG: A sensitivity analysis was made on MEPDG software version 1.1 to investigate the impact of input parameters on prediction of distresses. Then, using the average dynamic modulus data obtained in task 5, a comparison was made between the predicted distress values of SMA and S-4 mixtures. This was done to compare the predicted performance of SMA mixtures with that of S-4 mixtures.

Task 7: Recommendations and Conclusions: finally conclusions and recommendations were given based on the findings of this research project.

CHAPTER II

LITERATURE REVIEW

MECHANISTIC EMPERICAL PAVEMENT DESIGN GUIDE (MEPDG)

Objective

According to the design guide (5), the objective of the MEPDG is “to provide the highway community with a state-of-the-practice tool for the design of new and rehabilitated pavement structures based on mechanistic-empirical principles’’. This objective was accomplished by developing design guide and companion software which is based on the design guide procedure.

The design guide uses structural response models to compute stresses, strains and displacements at critical locations in the pavement layers. These responses are then utilized in damage models to accumulate damage over the design period. The guide then uses a field calibrated cracking model (transfer function) to predict distresses from the accumulated damages. The use of the transfer function is the main empirical part of the mechanistic empirical design procedure (5).

A designer who uses MEPDG for pavement design has the luxury to propose a trial design by first considering site (traffic, climate, subgrade, existing pavement condition for rehabilitation) and construction conditions. The trial design is then evaluated for performance criteria through the prediction of key distresses and smoothness. If the design does not meet the criteria, it is revised and the evaluation process repeated. This iteration process continues until the design meets the criteria (5).

Need For the Design Guide

One of the major concerns of the current AASHTO flexible design guide (4) is its inability to incorporate significant material properties into the design procedure. The only material property included is the layer coefficient ‘’a’’. Asphalt mixtures are assigned an ‘’a’’ coefficient based on resilient modulus. The resilient modulus test is usually performed according to ASTM D 4123. However, the test is rarely performed and ‘’a’’ coefficients are typically assigned to different mix types by departments of transportation (DOTs).

The other major concern is the performance equations. The current AASHTO guide designs pavements based on performance equations which were developed empirically using 1950’s AASHO Road Test data (4). Using these equations, the guide has been used as the primary design procedure for many decades. However it cannot continue to be used because of its incapability to address the following issues (5):

1. Heavy truck traffic design volume levels used in the AASHTO guide were based on the traffic of the highway system in the 1960’s, but the traffic on interstate pavements has increased tremendously which will forces the designer to extrapolate the data. Projects designed this way may be under designed or over designed, which leads to a significant economic loss.

2. The road test was conducted at one specific geographic location which makes it impossible to address the effects of different climatic conditions on pavement performance.

3. The AASHO Road Test did not take into consideration a design procedure for pavement rehabilitation.

4. Only one type of subgrade was used for all test sections when the fact is there exist many

types that result in different performance of highway pavements.

5. Only one HMA test mixture was used on the road test even if there are different kinds of HMA mixtures (e.g. Superpave, SMA) whose effects cannot be fully considered.
6. Even if there are many stabilized higher quality base courses that exist today, only two types were included on the original road test.
7. The truck characterization used on the road test was a representative of that time. Many of these are outmoded.
8. Sub drainage, which is common on today's highways, was not included at the road test.
9. Because of the short duration of the Road Test (2 years), the long term effects of climate and aging of materials were not considered.
10. Failure modes like rutting, thermal cracking and faulting were not considered directly on the AASHTO design guide. These might lead to more premature failures.
11. The AASHTO guide included a procedure for considering design reliability that has never been validated.

The MEPDG design guide and software addresses the aforementioned issues by using a mechanistic empirical approach.

Development of the MEPDG and its Companion Software

The design guide and companion software version 0.7 was the first of its kind to be released to the public under NCHRP Project 1-37A in June 2004. Shortly after that, an independent and compressive review was conducted on the guide under NCHRP project 1-40A. Based on the results of this review, software version 0.8 (released November 2005) and 0.9 (released July 2006) was developed by the NCHRP Project 1-40D (6).

In April 2007, version 1.0 was officially released to the public. After correcting defects which exists on this version of the software, version 1.1 was released in September 2009 (8). Version 1.1, which can only be used for evaluation purpose only, is the latest version of the software to date. The online version of the guide and its companion software is available to anyone with internet access and can be downloaded from http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm. This version of the software was used in this project.

Some of the recommendations of the project 1-40A team was accepted and incorporated on the recent versions of the software by the project 1-40D team while some were not. The recommendations which were not accepted are (6, 8):

- Elimination of calculation of IRI.
- Turning off permanent deformation model.
- Stop using of LTPP data for calibration of the distress models.
- Changing of design philosophy which means using limiting strain concept rather than predicting performances.
- Turning off HMA top down-cracking model.
- Inclusion of an option to use Hirsh model for E*.
- Focusing on local rather than national calibration.

The project team did not find it necessary to eliminate or turn off the aforementioned models but rather preferred to use more data and recalibrate the models for the newer versions.

The recommendations which were accepted and incorporated on the new versions are:

- Inclusion of endurance limit in cracking models.
- Incorporation of new E* model (G* based Witczak Equation).

- Improvement of the HMA thermal cracking model.
- Inclusion of unbound layer rutting model.
- Consideration of cold mixed asphalt-treated granular materials.
- Modification of the Enhanced Integrated Climatic Model (EICM).
- Modifying the software so that it allow runs in batch mode.
- Fixing all software bugs identified during the independent review.
- Checking consistency of guide and software.

These are some of the improvements which were made only for new flexible pavement design. Other necessary improvements were also made for rehabilitation and rigid pavement design.

Input Requirements

The input parameters for the MEPDG software are grouped into five areas: project, traffic, climate, design and layer. General information of the project, which includes the project design period and the months the pavement layers will be constructed and will be open for traffic, should be specified on the project part of the software. The type of design and analysis parameters should also be specified on this part. The type of designs incorporated in the guide are: new design of flexible and rigid pavements, restoration design of jointed plain concrete pavements (JPCP) and overlay design of asphalt concrete and PCC pavements.

The analysis parameters are performance criteria's the pavement under consideration is expected to fulfill. The parameters (distresses) listed on the guide for flexible pavements are terminal IRI, AC surface down cracking (Longitudinal cracking), AC bottom up cracking (Alligator cracking), AC 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’’. Rigid pavements have different analysis parameters.

All the input data for traffic which includes initial two-way Annual Average Daily Truck Traffic (AADTT), number of lanes in design direction, percent of truck in design direction, percent of trucks in design lane and operational speed should be given on the traffic part of the software. Other traffic inputs required, like traffic volume adjustment factors, axle load distribution factors and general traffic inputs are also incorporated in the traffic part of the guide.

The climate part of the MEPDG has a sophisticated climatic modeling tool called Enhanced Integrated Climatic Model (EICM). This tool is used to model temperature and moisture within each pavement layer including the subgrade layer. 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 (5).

The Design part of the software gives the chance for the user to select either a viscosity or a G* based HMA E* predictive model. The last input parameter which is also the main focus of this project deals about the layer section of the HMA mixtures. The input requirements for the layer section and the hierarchical approach are explained in detail below.

Layers

The MEPDG uses a hierarchical approach for materials characterization. There are three levels to characterize the asphalt layers in which the first level provides the highest design reliability and each succeeding level is a drop in design reliability. Each level has three input screens: Asphalt Mix, Asphalt Binder and Asphalt General.

Asphalt Mix screen Level 1 input for the asphalt mix screen requires a dynamic modulus (E*) input at a minimum of three temperatures and three frequencies to develop master curves and shift factors. The design guide recommends using five temperatures and four frequencies. The minimum and maximum temperatures should be between 10-20^{o}F and 125-135^{0}F respectively and at least one of the temperatures should be between 60 and 90^{o}F. AASHTO TP 62-03 (9) recommends the minimum and maximum temperature to be 14^{0}F and 130^{o}F respectively. At input levels 2 and 3, the master curves are developed directly from the dynamic modulus predictive equation. The only input required on these two levels is aggregate gradation: the cumulative percentage retained on 3/4inch, 3/8 inch and No. 4 sieves and percent passing the No. 200 sieve (5).

Asphalt Binder Screen On the asphalt binder screen, binder complex shear modulus (G*) and phase angle (δ) data are required at a loading rate of 1.59 Hz (10 rad/sec). These data should be available at least at three temperatures for input levels 1 and 2. At input level 3, it is only required to have one of the following binder related information (5).

- the performance grade (PG) of the asphalt binder based on AASHTO M 320
- viscosity grade of the asphalt binder based on AASHTO M 226
- penetration grade of the asphalt binder based on AASHTO M 20

Asphalt General Screen The required inputs for the asphalt general screen are similar for all three levels. This screen is separated into four sections: General, Poisson's ratio, as built volumetric properties and thermal properties. The general section is where the value for the reference temperature must be given. The default value is 70^{o}F but other temperatures may be entered. The user can specify the Poisson's ratio of the bituminous pavement on Poisson's ratio section. A default value of 0.35 or other realistic values can be entered on this section. The values for the volume binder effective (Vbe), air voids and compacted unit weight of the asphalt mixture should be given on the as built volumetric property section. Default values are 11.0%, 8.5% and 148 pcf respectively. The required thermal properties are thermal conductivity and heat capacity. Either default values of 0.67 BTU/hr-ft-^{o}F for thermal conductivity and 0.23 BTU/lb-^{o}F for heat capacity or user defined values may be entered for the thermal properties (5).

Dynamic Modulus

Complex dynamic modulus (E*) is defined in NCHRP Report 547 as " a complex number that relates stress to strain for linear viscoelastic materials subjected to continuously applied sinusoidal loading. The complex modulus is defined as the ratio of the amplitude of the sinusoidal stress (at any given time, t, and angular load frequency, ω), $\sigma = \sigma_o \sin(\omega t)$, and the amplitude of the sinusoidal strain $\epsilon = \epsilon_o \sin(\omega t - \emptyset)$, at the same time and frequency, that results in a steady – state response'' (10).

$$E^* = \frac{\sigma}{\epsilon} = \frac{\sigma o \, sin(\omega t)}{\epsilon o \, sin(\omega t - \emptyset)} \qquad [1]$$

Where,

E^* = complex modulus, psi;

σo = peak-to-peak stress amplitude, psi;

ϵo = peak-to-peak strain amplitude, inches/inch;

$\emptyset$ = phase angle, degrees

ω = angular velocity, rad/sec;

t = time, seconds

Mathematically, the dynamic modulus ($|E^*|$) is defined as the absolute value of the complex modulus or

$$|E^*| = \frac{\sigma o}{\epsilon o} \qquad [2]$$

The attention given for the dynamic modulus is increasing recently because this property has become the main input property of HMA in the MEPDG. The MEPDG uses the dynamic modulus to determine the temperature and rate dependent behavior of an asphalt concrete layer. The dynamic modulus test was also recommended as the primary simple performance test for predicting rutting (10).

Master Curve

In the MEPDG, the modulus of HMA at all levels of temperature and time rate of load are determined from a master curve constructed at a reference temperature. Master curves are constructed using the principle of time of loading-temperature superposition. This means the same modulus value of a material can be obtained either at low test temperatures and high loading frequencies (short loading times) or at high test temperatures but lower loading frequencies (longer loading times). The data at various temperatures are shifted with respect to time of loading until the curves merge into a single smooth function. This describes the dependency of asphalt materials to rate of loading and temperature (1, 5, and 10). Figure 1(a) shows the dynamic modulus data obtained from HMA mix and Figure 1(b) shows the master curve after shifting the data to the reference temperature (in this case 70^{o}F or 21.1^{o}C). Figure 2 shows the resulting shift factors.

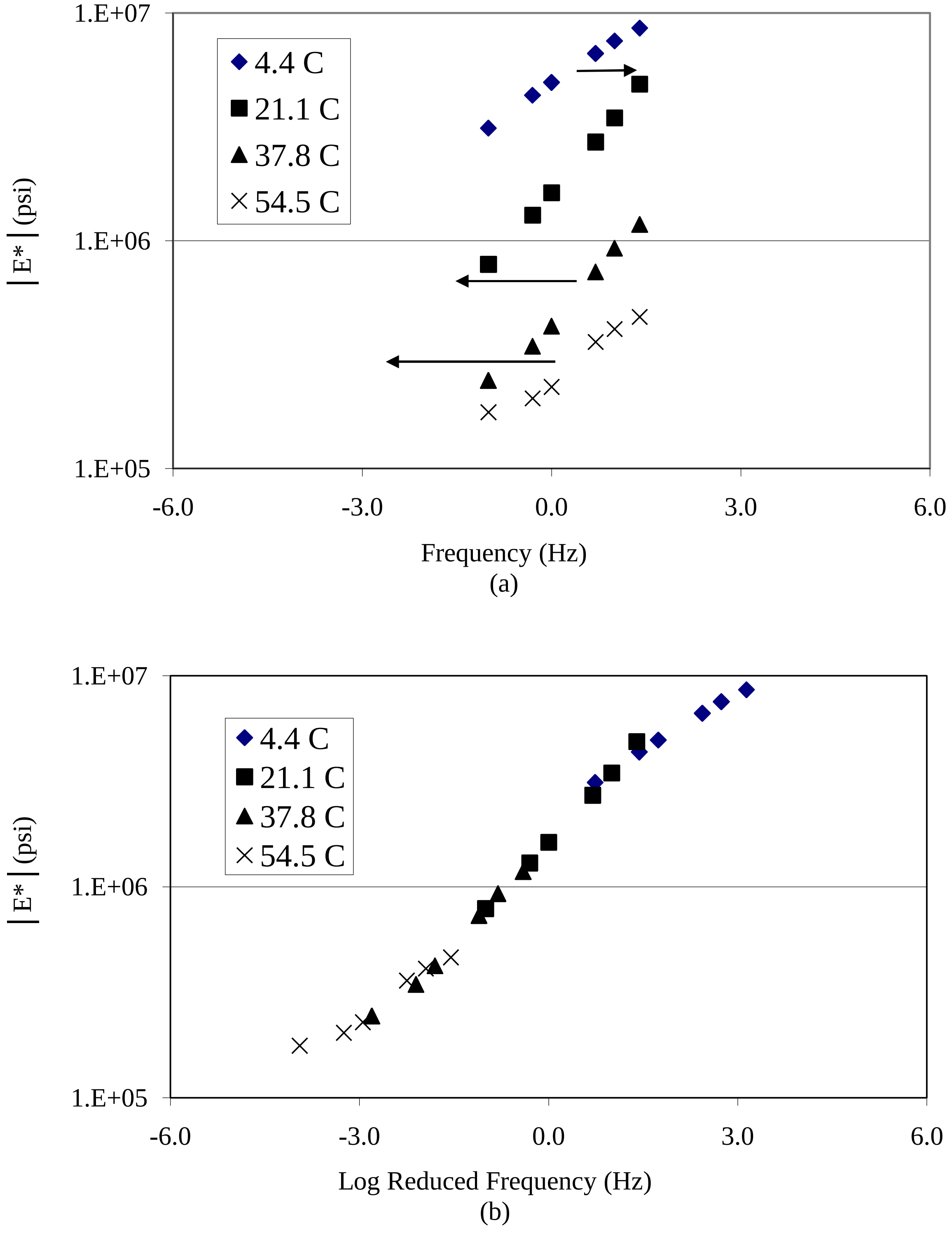

FIGURE 1 (a) Results of dynamic modulus test on HMA sample; (b) Dynamic modulus master curve after shifting the test data.

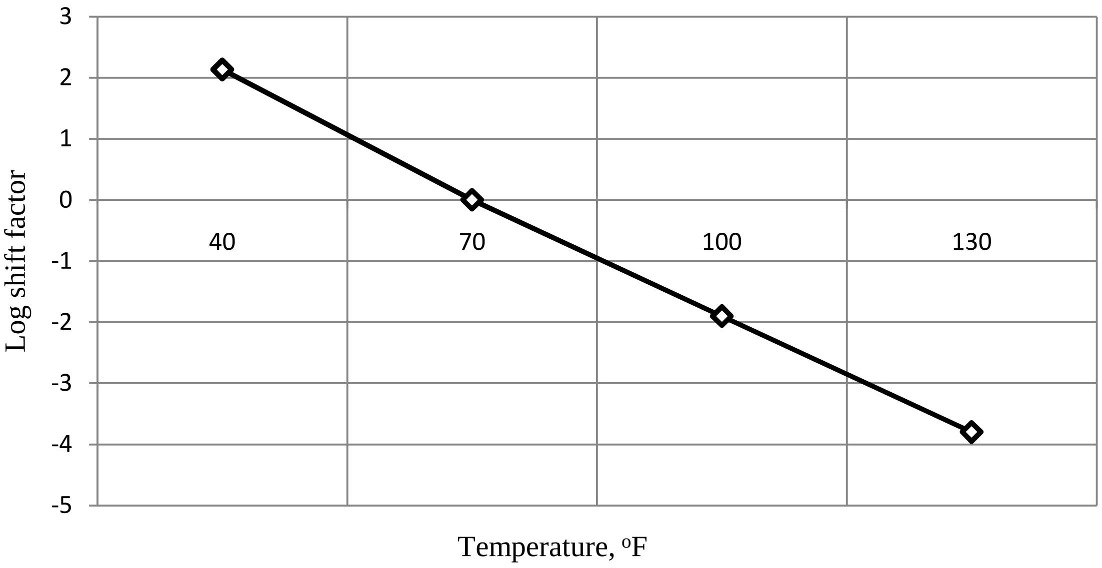

FIGURE 2 Shift factor versus temperature.

In general, the dynamic modulus master curve can be mathematically modeled by a sigmoidal function described as (5)

$$\text{Log} \left| E^* \right| = \delta + \frac{\alpha}{1+e^{\beta+\gamma(\log tr)}} \quad [3]$$

Where:

$|E^*|$ = dynamic modulus.

t_r = reduced time of loading at reference temperature.

δ, α = fitting parameters; for a given set of data, δ represents the minimum value of E* and $\delta + \alpha$ represents the maximum value E*.

β,γ = parameters describing the shape of the sigmoidal function

The sigmoidal function describes the time dependency of the modulus at the reference temperature. The shift factor, which describes the temperature dependency of the modulus, can be shown in the following form (5):

$$t_r = \frac{t}{a\ (T)} \qquad [4]$$

$$\text{Log}\ (t_r) = \log\ (t) - c\ (\log \eta - \log \eta_{TR}) \qquad [5]$$

Where,

a (T) = shift factor as a function of temperature.

T = temperature of interest.

t_r = time of loading at the reference temperature.

t = time of loading at desired temperature.

η = viscosity of the binder at the test temperature, cP

η_{TR} = viscosity of the binder at the reference temperature, cP

c = fitting coefficient.

Input Levels

The MEPDG uses actual laboratory measured E* data at input level 1 while it uses E* values predicted from the Witczak E* predictive equation (11) at input levels 2 and 3. The master curve and shift factors for input level 1 are developed by determining the fitting parameters of equation [3] and [5] using non linear optimization to shift the laboratory mixture test data into a smooth curve. Before shifting the test data, the relationship between binder viscosity and temperature must be established. This is done by first converting the binder stiffness data at each temperature to viscosity using equation [6]. The parameters of the ASTM Ai-VTSi equation are then found by linear regression of equation [7] after log-log transformation of the temperature data. These parameters can then be used to calculate the viscosity at any temperature (5).

$$\eta = \frac{G*}{10}\left(\frac{1}{\sin\delta}\right)^{4.8628} \qquad [6]$$

$$\log \log \eta = A + VTS \log T_R \qquad [7]$$

Where

η = binder viscosity, cp

G* = binder complex shear modulus, pa

δ = binder phase angle, degree

A, VTS = regression parameters.

T_R = temperature in Rankine at which the viscosity was estimated.

The master curve at input level 2 is developed from Witczak's dynamic modulus predictive equation using actual binder test data. There are two Witczak predictive equations. The first one is viscosity-based (11) while the second one is G*-based. The MEPDG uses both equations but the G*- based model is not nationally calibrated. Hence, the use of viscosity based equation (equation 8) is preferable. At input level 3, the same predictive equation as level 2 is used but no laboratory test data is required either for the asphalt mixture or the asphalt binder. The MEPDG uses default A and VTS values to calculate the viscosity of the asphalt binder (5).

$$log\ E* = 3.750063 + 0.02932\rho 200 - 0.001767(\rho 200)2 - 0.002841\ \rho 4 - 0.058097Va$$

$$-0.802208\left(\frac{Vbe}{Vbeff+Va}\right) + \frac{3.871977-0.021\rho 4+0.003958\rho 38-0.000017(\rho 38)2+0.005470\rho 34}{1+e^{(-0.603313-0.31335logf-0.393532log\eta)}}$$

[8]

Where

E* = dynamic modulus, psi

η = bitumen viscosity, 106 poise

f = loading frequency, Hz

Va = air void content, %

Vbeff = effective binder content, %

P_{34} = cumulative % retained on the ¾ in sieve

P_{38} = cumulative % retained on the 3/8 in sieve

P_4 = cumulative % retained on the No. 4 sieve

P_{200} = % passing the No.200 sieve

The Hirsh model (12) is the other dynamic modulus predictive equation other than the two Witczak equations. This model is not incorporated in the MEPDG. The material property input requirements vary among the three models. One of their differences is on the gradation of the mix. The two Witczak models incorporate the gradation of the mix while the Hirsh model does not. The other difference is whether the dynamic shear modulus value (G*) is used directly on the predictive equation or not. In the viscosity based Witczak model, the G* values have to be translated to binder viscosity values while in the other methods the G* values are used directly in the predictive equations.

STONE MATRIX ASPHALT (SMA) MIXES

Introduction

The European study tour, which took place in mid-September 1990, found some technologies which had the potential to be transferred to United States. The tour participants found SMA to be the most promising special-purpose mixture which could be used in the United States (13).

Accordingly, four states (Wisconsin, Georgia, Michigan, and Missouri) constructed the first SMA projects in 1991 (14) and its use has been growing since that time.

SMA was first developed in the 1960's in Germany as an overlay to minimize the effects of studded tire damage. Today, it is used in many European countries as an overlay or surface course to resist rutting and to improve durability (13).

Just as in Europe, the main reasons SMA has been used in the United States are its improved resistance to rutting and its increased durability. Other reported benefits of SMA include less tire noise, improved skid resistance and reduced thermal cracking (1, 2, and 15).

The initial cost of SMA is higher due to increased asphalt contents and the use of more durable aggregates. However, experience showed that this higher initial cost may be more than offset by the expected increase in pavement life (2, 16).

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 their improved rutting resistance and durability, these mixes are usually used for surface courses on high volume traffic roads. Sometimes they are used for intermediate and base layers when there are slow moving, heavy vehicles (2).

Materials

The materials used to produce SMA include crushed aggregate, mineral filler, asphalt cement and additives. SMA needs high quality aggregates, which have 100 percent of the particles with one or more fractured faces, because the mixture's rut resistance comes from the stone-on-stone aggregate skeleton. Natural sand should not be used in SMA mixtures. Where SMA is used as a surface course, aggregates which can polish easily (ex - limestone) should not be used because they create low skid resistance on pavement surfaces (1, 2, and 17).

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.

TABLE 1 Typical Gradation Requirements for SMA Mixture

Sieve Size	Percent Passing
¾ in. (19 mm)	100
½ in. (12.5 mm)	90-100
3/8 in. (9.5 mm)	65-80
No. 4 (4.75 mm)	22-30
No. 8 (2.36 mm)	16-24
No. 200 (0.075 mm)	9-12

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. To control the 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.

Mix Design

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}) (1). ODOT doesn’t consider this as a mix design requirement as recent changes in SMA gradation requirement ensure stone-on-stone contact.

SMA mixes have been 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.

Construction

SMA mixtures are difficult to work with because these mixes have 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 (1, 3).

Early compaction by keeping roller right behind the paver is needed because SMA mixtures tend to set up quickly. If they become cool this will make it very difficult to compact them. 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 (1, 17).

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 (18). The European experience shows that SMA mixes are generally expected to last up to 25% longer than conventional mixes (13).

The increase in cost for SMA mixtures is more than offset by the expected increase in pavement life. The saving in cost becomes truly significant when the savings from the expected increase in pavement life combines with the savings from fewer user delays (16).

CHAPTER III

MATERIALS

INTRODUCTION

SMA mixtures have not been utilized in some parts of Oklahoma as often as they could have been. One of the reasons mentioned for this is a lack of data in the state which shows SMA mixtures perform better than conventional mixtures. There is also a lack of input data which are required for use in MEPDG.

To overcome this, SMA mixtures were prepared and tested on this project and were compared to S-4 mixtures made with the same asphalt cement. S-4 is a designation for ODOTs superpave mixtures which has a nominal maximum aggregate size of ½ inch. The mixes used for comparison and their producers are shown in Table 2.

TABLE 2 Mix Types

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
S-4	T.J. Campbell Const. Co.	S4QC0190900600	3M+	100	TJC
S-4	Cornell Const. Co.	S4PV0110902000	30M+	125	CL-3

MIXTURES

As shown in table 2, five SMA and two S-4 mixtures were sampled on this project. These mixtures were obtained in different ways. Some were acquired directly from stockpiles by OSU personnel while others were collected by contacting contractors and ODOT personnel. S-4 mixtures, which were previously prepared and tested by Cross et al (19), were also used in this project in addition to the two S-4 mixture sampled.

The two S-4 mixes and one of the SMA mixes sampled were cold feed belt samples. The other four SMA samples were prepared by blending materials from different stockpiles to the job mix formula (JMF). Sources of SMA and S-4 mixtures are given in tables 3 and 4, respectively.

TABLE 3 Sources of S-4 Mixtures

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

TABLE 4 Sources of SMA Mixtures

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

Asphalt Cement

PG 76-28 asphalt cement was used for both SMA and S4 mixes.

MIXTURE VERIFICATION

It was not the intention of this project to reproduce the field mixtures. The main objective was to produce mixtures similar to field produced mixtures so that they can be tested and checked if they meet ODOT's requirements.

In order to make sure replicate samples of the produced mixture has the same gradation, the aggregates from each mix were sieved over a 1- inch sieve through No- 50 sieve. The sieved aggregates were then stored by size and recombined to the batch weights required.

To verify the mix, two specimens were prepared to the JMF gradation using asphalt contents on either side of the JMF asphalt content. The specimens were then compacted to N_{des} in accordance with AASHTO T 312. Different mix design number of gyration (N_{des}) values was used for S4 and SMA mixtures as shown in table 2.

A voids analysis was performed on the compacted specimens to determine the asphalt content which gives a 4% air void and to verify if the mix properties meet ODOT's requirements. Blended gradation of aggregates and mix properties of the mixtures with ODOT's mix property requirement are presented in tables 5 and 6.

TABLE 5 Blended Gradation of Aggregates and Mix Properties of SMA Mixtures

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

*Produced under old SMA specification

NR = No requirement

TABLE 6 Blended Gradation of Aggregates and Mix Properties of S-4 Mixtures

Mix Code	TJC	CL-3	ODOT Spec.
Sieve Size	Percent Passing		
3/4"	100	100	100
1/2"	97	96	90-100
3/8"	90	87	≤ 90
No. 4	52	69	
No. 8	36	47	34-58
No. 16	28	36	
No. 30	24	28	
No. 50	19	16	
No. 100	11	9	
No. 200	4.6	5.2	2-10
% AC	4.6	4.9	min. 4.6
Ndes	100	125	
% VTM	4.0	4.0	4.0
% VMA	14.4	14.7	≥ 14.0
% VFA	72.3	72.8	65-75

CHAPTER IV

DYNAMIC MODULUS TEST PROCEDURES

INTRODUCTION

One of the objectives of this project was to obtain data to determine if SMA mixtures perform substantially better than conventional Superpave mixtures. To accomplish this task, S-4 mixtures were made using similar aggregates and the same asphalt cement as SMA mixtures. The mixture samples were then tested for dynamic modulus to evaluate their performance properties.

All of the test specimens, which were made using the same PG 76-28 asphalt cement from Valero, were prepared to the target air void content (VTM) and aging condition in accordance with NCHRP 9-29 PP 01 (20).

DYNAMIC MODULUS TEST

Specimen Size Requirements

Dynamic modulus testing requires a nominal 100 mm diameter by 150 mm high cylindrical test specimen that is sawed and cored from a 150mm diameter by 170mm high cylindrical superpave gyratory compacted (SGC) specimen. Testing should only be performed on test specimen which meets the specified air void content. The gyratory specimen air void content, which is required to obtain the specified test specimen air void content, must be determined by trial and error procedure. The VTM of the final test specimen shouldn’t differ by more than 0.5 percent from the specified target air voids (7).

The air void content that is recommended for HMA mixtures is 4-7%. The target air void content of the final test specimen for this project was 5±1%. After several trials, it was determined that a gyratory specimen which was compacted to 7±1% VTM would obtain a test specimen of 5±1% air void content.

Batch Weights

The batch weights used for S-4 mixes were different from the SMA mixes. A 6500 to 6800 gm batch of aggregate, which was batched to the required gradation, was used to get a 7±1 % VTM S4 mixes while a 6000 to 6350 gm batch of aggregate was used to get the same VTM for the SMA mixes.

Mixing

The aggregates were heated for a minimum of four hours at ODOTs mixing temperature of 325^{o}F. The asphalt cement was also heated until it reached 325^{o}F. The asphalt cement was stirred occasionally during the heating process to prevent overheating. While the aggregates and the asphalt cement were heating, all the mixing implements such as bucket mixer, spatulas and other tools were also heated for about an hour before mixing.

To mix the samples, the aggregates were placed in a pre-heated bucket mixer and the desired amount of asphalt cement was added. The samples were then mixed until the aggregates were thoroughly coated, which took approximately two minutes. The mixtures were then placed in a flat pan and placed in an oven, which was set at ODOT's compaction temperature of 300 ^{0}F, for two hours oven aging in accordance with AASHTO R30.

Gyratory Specimen Compaction

To compact the samples, the mixtures were transferred from the flat pan in the oven to a heated compaction mold. The compaction molds, top plates and other necessary tools were heated at compaction temperature (300 ^{0}F) for an hour before compaction. The samples were compacted in a 150 mm diameter mold to a height of 170 mm using a Superpave Gyratory Compactor (SGC) in accordance with AASHTO T 312.

After compaction, the samples were extruded from the compaction molds, labeled and set aside to cool to room temperature. The bulk specific gravity of each compacted sample and the theoretical maximum specific gravity (Gmm) of the loose mix samples were then determined in accordance with AASHTO T 166 and AASHTO T 209, respectively. From the bulk and theoretical maximum specific gravity values, the air void content (VTM) of the gyratory specimens were determined and the results came out to be 7±1 % for the aggregate batch weights mentioned above.

Final Test Specimen Preparation

From the compacted gyratory specimens, a nominal 100 mm diameter by 150 mm tall cylindrical test specimens were cored and sawed. A nominal 100 mm diameter test specimens were cored from the center of the gyratory specimens using a diamond studded core barrel. The ends of the cored samples were sawed to obtain a nominal 150 mm tall test specimens. The final test specimens were checked to see if they met the dimensional tolerance requirements set by NCHRP 9-29 PP 01 (20). Specimens which did not meet the tolerance requirements shown in table 7 were rejected.

The bulk specific gravity (AASHTO T 166) was then determined on those specimens which met the criteria. Using the bulk specific gravity and the theoretical maximum specific gravity (Gmm) values, the air void content of the final test specimens was calculated. Specimens with air voids that were outside the target range of 5±1% VTM were rejected as recommended by NCHRP 9-29 PP 02 (7).

TABLE 7 Test Specimen Dimensional Tolerances (20)

Item	Specification
Average Diameter	100 mm to 104 mm
Standard Deviation of Diameter	0.5 mm
Height	147.5 mm to 152.5 mm
End Flatness	0.5 mm
End Perpendicularity	1.0 mm

Test Specimen Instrumentation

Six steel studs, which are used to hold three axial linear variable displacement transducers (LVDTs), were attached to the sides of the final test specimens with epoxy cement. Because the LVDTs have a gauge length of 4 inches, the steel studs were also positioned 4 inches apart between their centers. Once the epoxy was dry and the studs were attached to the specimen, they were ready for testing.

Testing

The test specimens were tested for dynamic modulus according to NCHRP 9-29 PP 02 test protocol (7). This protocol requires a Simple Performance Test System (21) to be used to test the specimens and analyze the results for developing a dynamic modulus master curve. OSU has a dynamic modulus testing machine which meets the requirements of the Simple Performance Test System equipment specification (21). This machine was used to test the specimens on this project.

Figure 3 shows the set up of OSU dynamic modulus testing machine. The machine has a control and operating unit which are connected to a power supply. The control unit compromises the computer and temperature control unit. The computer is used to give commands to the operating unit by using software which was provided by Interlaken Inc., the manufacturer of the machine. The temperature control unit is used to regulate different test temperatures in the testing chamber (which is located in the operating unit) according to the specifications in the test procedures.

The operating unit consists of the test chamber, actuator, which is connected to the hydraulic pump, and a load cell which is attached to the actuator. The test chamber has the capacity to maintain a temperature of -10 ^{0}C to 125 ^{0}C with an accuracy of ± 1 ^{0}F. Two load cells of 10 and 2 kip capacity are used for testing. The 10 kip load cell is used for testing at 4 ^{0}C and the 2 Kip load cell is used for testing at 20 ^{0}C and 45 ^{0}C. The deformation of the test sample is recorded in a data file using three LVDTs.

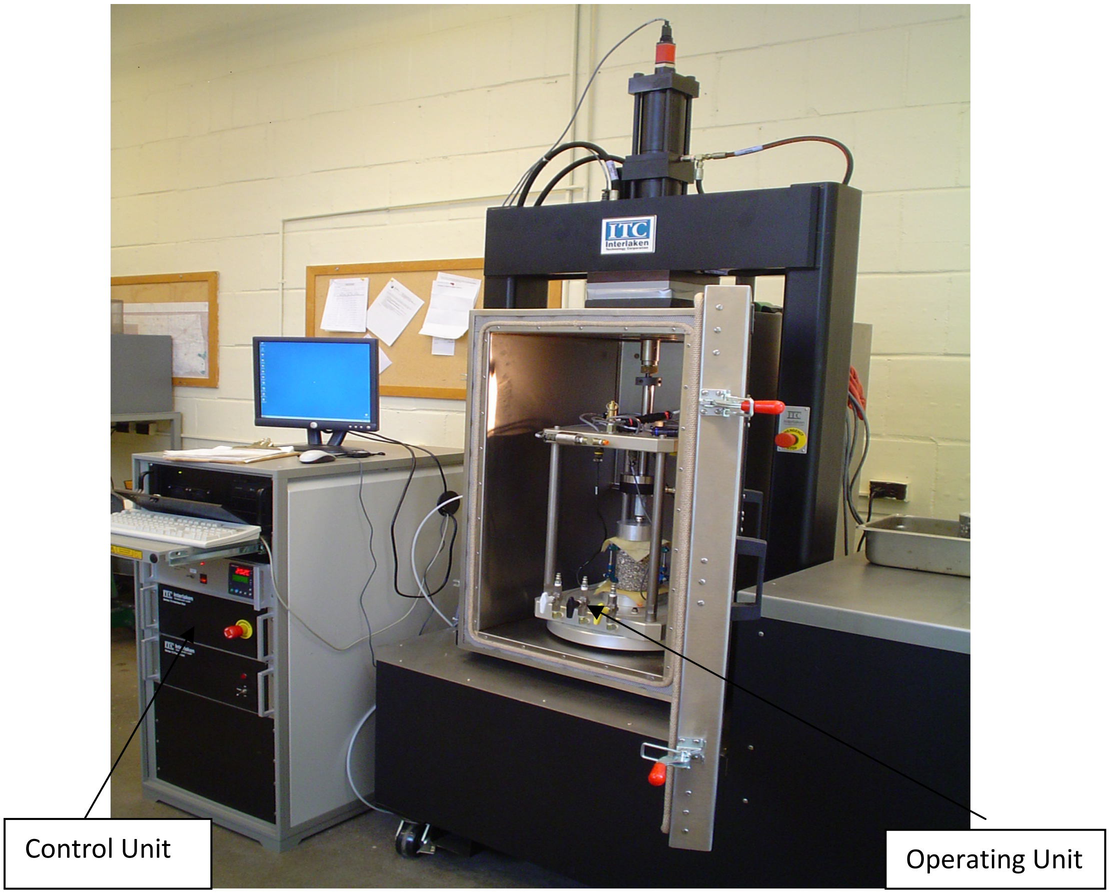

FIGURE 3 OSU Dynamic Modulus Machine.

The testing protocols, which were used to define the dynamic modulus test, are shown in Table 3. The user has to define basic specimen and operator information for the test. The user also has to define the test temperature in ^{0}C and confining pressure in psi if the test is going to be performed with confinement. For this project, 0.0 psi value for confinement was used because an unconfined dynamic modulus master curve is typically used in mechanistic-empirical pavement analysis methods (7). The number of test frequencies, initial dynamic load as a function of temperature and number of cycles as a function of frequency are the other inputs which have to be defined by the user. AASHTO TP 62-03 (9) has typical initial dynamic load and number of cycle's values as shown in Table 8. These typical values were used on this project.

TABLE 8 Testing Protocols for Defining a Dynamic Modulus Test (7, 9)

Description	Values
Temperatures (^{o}C)	4, 20 , (35 or 40 or 45) *
	Use 35 for PG 58-XX and softer
	40 for PG 64-XX and 70-XX
	45 for PG 76-XX and stiffer
Frequencies (Hz)	10,5,1,0.5,0.1
Load at Test Temperature	At 4 ^{0}C : 175 psi
	At 20 ^{0}C : 75psi
	At 45 ^{0}C : 40 psi
Number of 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

* The highest temperature depends on the binder grade of the asphalt cement.

CHAPTER V

LABORATORY TEST RESULTS

DYNAMIC MODULUS TEST RESULTS

One of the main objectives of this project was to develop input data, which are required for use in MEPDG, for Oklahoma SMA Mixtures. To accomplish this, SMA mixtures were prepared and tested for dynamic modulus. S-4 mixtures were also prepared and tested for comparison. Void analysis was performed on the compacted specimens of these mixtures to determine their volumetric properties.

NCHRP 9-29 PP 02 testing protocol (7) was followed on this project to test the specimens for dynamic modulus. This testing protocol recommends specimens, which are made using PG 76-28 asphalt cement, to be tested at 4, 20 and 45 ^{0}C temperatures. Accordingly, all of the specimens in this project were tested at these temperatures.

Two types of load cells were used to test the specimens for dynamic modulus. A 2 kip load cell was used to test specimens at 20 and 45 ^{o}F temperatures while a 10 kip load cell was used at 4 ^{o}F. Each specimen was tested at five frequencies (Hz): 10, 5, 1, 0.5 and 0.1 at each temperature.

The dynamic modulus results of both SMA and S4 mixes are shown below in tables 9 - 15.

TABLE 9 Dynamic Modulus Test Results, Haskell Lemon 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 10 Dynamic Modulus Test Results, Haskell Lemon 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 11 Dynamic Modulus Test Results, Cornell 1 (SMA)

Temperature (˚C)	Frequency (Hz)	Dynamic Modulus (psi)		
		Sample 1	Sample 2	Average
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
20	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
45	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

TABLE 12 Dynamic Modulus Test Results, Cornell 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 13 Dynamic Modulus Test Results, Silver Star (SMA)

Temperature (°C)	Frequency (Hz)	Dynamic Modulus (psi)		
		Sample 1	Sample 2	Average
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 14 Dynamic Modulus Test Results, T.J. Campbell (S4)

Temperature (°C)	Frequency (Hz)	Dynamic Modulus (psi) Sample 1	Sample 2	Sample 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 15 Dynamic Modulus Test Results, Cornell 3 (S4)

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

INPUTS FOR PREDICTIVE EQUATION

Dynamic modulus values of HMA mixes can be determined from predictive equations. The predictive equations estimate modulus values using material properties and volumetrics. The Viscosity-Based Witczak equation (11) was used in this project to predict E* values of SMA and S-4 mixtures. This equation, which is presented as equation 8 in chapter two of this report, requires aggregate gradation and mixture volumetric values to predict dynamic modulus. Summary of the required mix properties for this equation is presented in table 16.

TABLE 16 Aggregate Gradations and Mixture Volumetric Properties

Mixes	% Retained			% Pass.	Va (%)	Vbeff(%)
	3/4 "	3/8 "	No. 4	No. 200		
			SMA Mixtures			
Haskel Lemon 1	0	31	71	10	5.2	13.31
Haskel Lemon 2	0	35	71	10	5.4	13.87
Clinton 1	0	22	68	10	6.2	13.17
Clinton 2	0	32	70	10	5.5	13.94
Silver Star	0	25	70	11	5	13.29
Average	0.0	29.0	70.0	10.2	5.5	13.5
Std. Dev.	0.0	5.3	1.2	0.4	0.5	0.4
			S-4 Mixtures			
T.J. Campbell	0	10	47.7	4.6	5.2	10.28
Clinton 3	0	13.2	31	5.2	5.3	10.58
Average	0	11.6	39.35	4.9	5.25	10.43
Std. Dev.	0	2.3	11.8	0.4	0.1	0.2
			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 by Cross et al. (19)

CHAPTER VI

ANALYSIS OF RESULTS

STATISTICAL ANALYSIS

Two types of mixes (S-4 and SMA) were prepared and tested for dynamic modulus in this project. The samples were tested at three temperatures (4 °C, 20 °C and 45 °C) in accordance with NCHRP 9-29 PP 02 testing protocol (7). A two-way analysis of variance (ANOVA) was performed on the data to determine if there is a statistical difference between measured dynamic modulus values of S-4 and SMA mixes and to see if the mean dynamic modulus values are significantly different at different test temperatures. A study by Cross et al. (19) showed that frequency has a consistent effect on dynamic modulus. For this reason, the ANOVA was only performed on the middle frequency (1 Hz). The result of the ANOVA is shown in table 17.

TABLE 17 Results of ANOVA on Mix Types and Test Temperatures

Source	Degrees Freedom	Sum Squares	Mean Square	F value	Prob. > Fcr
Type	1	6.4804796E+11	5.4080017E+12	34.26	< 0.0001
Temp.	2	1.0816003E+13	6.4804796E+11	285.88	< 0.0001
Type*Temp	2	2.5491131E+11	1.2745566E+11	6.74	0.0031
Error	38	7.1884364E+11	1.8916938E+10		
Total	43	1.24378E+13			

According to the result of the ANOVA, both test temperature and type of mix showed a significant effect on measured E* values. To determine which means were statistically different, Duncan's Multiple Range Test was performed. The result also showed a significant interaction between type of mixes and test temperature. Because of this interaction, Duncan's multiple range tests was performed on type of mixes and test temperature as shown in table 18-19. The result of this test indicates which means were significantly different at a confidence limit of 95% ($\alpha = 0.05$).

TABLE 18 Duncan's Multiple Range Test for Dynamic Modulus on Mix Type

Grouping*	Mean Dynamic modulus (psi)	N	Type
A	806835	15	S4
B	568027	29	SMA

* Means with the same letter are not significantly different.

TABLE 19 Duncan's Multiple Range Test for Dynamic Modulus on Test Temperatures

Grouping *	Mean Dynamic modulus (psi)	N	Temp
A	1315129	15	4
B	458466	15	20
C	140814	14	45

* Means with the same letter are not significantly different.

As shown in table 18, the mean dynamic modulus values of S-4 and SMA mixtures are significantly different. The mean E* value of S-4 mixtures is greater than SMA mixtures. This indicates that S-4 mixtures are stiffer than SMA mixtures.

It can be observed from table 19 that the average measured dynamic modulus value of all the specimens tested in this project is different for different temperatures. Referring to the same table it can also be observed that modulus values decreases with an increase in temperature.

To confirm if the mean dynamic modulus values of both S-4 and SMA mixes are different at different temperatures, Duncan’s multiple range tests was performed on the two types of mixes separately by test temperature. The results are shown in tables 20-22.

TABLE 20 Duncan's Multiple Range Test for Dynamic Modulus at 4 °C

Grouping*	Mean Dynamic modulus (psi)	N	Type
A	1611195	5	S4
B	1167096	10	SMA

* Means with the same letter are not significantly different.

TABLE 21 Duncan's Multiple Range Test for Dynamic Modulus at 20 °C

Grouping*	Mean Dynamic modulus (psi)	N	Type
A	636436	5	S4
B	369480	29	SMA

* Means with the same letter are not significantly different.

TABLE   22 Duncan’s Multiple Range Test for Dynamic Modulus at 45 °C

Grouping	Mean Dynamic modulus (psi)	N	Type
A	172873	5	S4
B	123003	9	SMA

* Means with the same letter are not significantly different.

Tables 20-22 show that there is a significant statistical difference between the mean dynamic modulus values of S-4 and SMA mixtures. In all three tables, the mean dynamic modulus values of the S-4 mixtures are greater than the SMA mixtures. This indicates that S-4 mixtures are stiffer than SMA mixtures at all test temperatures.

A second two-way analysis of variance (ANOVA) was performed at 1 Hz frequency to determine if there is a statistical difference in dynamic modulus values of individual SMA mixes and test temperature. The results of the ANOVA are shown in table 23.

TABLE 23 Results of ANOVA on SMA Mixes and Test Temperatures

Source	Degrees Freedom	Sum Squares	Mean Square	F value	Prob. > Fcr
Mix	4	7.4815333E+10	1.870383E+10	34.26	< 0.0001
Temp.	2	5.8007833E+12	2.900392E+12	285.88	< 0.0001
Mix*Temp	8	1.5862684E+11	1.982836E+10	6.74	0.0031
Error	14	6.8968807E+10	4.926343E+09		
Total	28	6.1031943E+12			

Table 23 shows that SMA mixes and test temperature had a significant effect on measured dynamic modulus values. The result of the ANOVA also indicated that there was a significant interaction between SMA mixes and test temperatures. Hence, Duncan's multiple range tests was performed on SMA mixes, by test temperature at 1 Hz frequency, as shown in table 24 and 25.

TABLE 24 Duncan's Multiple Range Test for SMA Dynamic Modulus on Test Temperatures

Grouping*	Mean Dynamic modulus (psi)	N	Temp
A	1167096	10	4
B	369480	10	20
C	123003	9	45

* Means with the same letter are not significantly different.

TABLE 25 Duncan's Multiple Range Test for Dynamic Modulus on SMA Mixes

Grouping*	Mean Dynamic modulus (psi)	N	Mixes
A	610811	6	SS
A	602324	6	HL1
A	589787	6	HL2
A	563061	5	CL2
B	473325	6	CL1

* Means with the same letter are not significantly different.

Table 24 shows the same result as table 19 but only for SMA mixes. From the result of this analysis it was observed that the average measured dynamic modulus values of SMA mixes tested in this project are different at different test temperatures. A decrease in measured modulus values were also observed with an increase in temperature.

From the five SMA mixes tested in this project only one of them was found to have a significantly different mean dynamic modulus value. Cornell 1was the only SMA mix which had a different mean dynamic modulus value. This mix was produced under the old ODOT SMA specification.

Duncan's multiple test range was performed by test temperature to cross check if the mean dynamic modulus values of the SMA mixes are similar at each test temperature and also to conform if the mixes have different dynamic modulus at different test temperatures. The results of the test are presented in tables 26-28.

TABLE 26 Duncan's Multiple Range Test for SMA mixes Dynamic Modulus at 4 °C Test Temperature.

Grouping	Mean Dynamic modulus (psi)	N	Mixes
A	1391738	2	SS
A & B	1236421	2	HL1
A & B	1220653	2	HL2
B	1010174	2	CL1
B	976494	2	CL2

* Means with the same letter are not significantly different.

TABLE 27 Duncan's Multiple Range Test for SMA mixes Dynamic Modulus at 20 °C Test Temperature.

Grouping	Mean Dynamic modulus (psi)	N	Mixes
A	430111	2	HL1
A	420887	2	HL2
A	377947	2	CL2
A	331442	2	SS
A	287016	2	CL1

* Means with the same letter are not significantly different.

TABLE 28 Duncan’s Multiple Range Test for SMA mixes Dynamic Modulus at 45 °C Test Temperature.

Grouping	Mean Dynamic modulus (psi)	N	Mixes
A	140441	2	HL1
A	127823	2	HL2
A	122786	2	CL1
A	109253	2	SS
A	106424	2	CL2

* Means with the same letter are not significantly different.

Table 27 and 28 shows no significant difference between mean dynamic modulus values of SMA mixes. The only significant difference was observed at 4°C test temperature between Silver Star (SS) and Cornell 1 and 2 mixes (CL1 and CL2).

Referring to table 26-28, it can be said that the SMA mixes tested in this project had mean dynamic modulus which are not significantly different. This means the dynamic modulus values of the different SMA mixes tested in this project can be represented by one mean dynamic modulus value at each test temperature.

DYNAMIC MODULUS MASTER CURVE

At input level 1, MEPDG software requires laboratory measured dynamic modulus (E*) data to develop master curve and shift factors. The guide recommends to use dynamic modulus values at five temperatures (-10, 4.4, 21.1, 37.8, and 54.4 ^{0} C) and four frequencies (0.1, 1, 10, and 25 Hz) (5). On this project, NCHRP 9-29 PP 02 testing protocol (7) was followed to determine the dynamic modulus values at the recommended temperatures and frequencies.

As per the recommendation of the NCHRP 9-29 PP 02 testing protocol (7), dynamic modulus test data were first collected at three temperatures (4, 20, and 45 ^{0}C) and four frequencies (10, 1, 0.1, and 0.01). Then by manipulating these test data, a master curve was constructed from which E* values at the recommended temperatures and frequencies were estimated.

Master Curve Equation

The general form of the dynamic modulus master curve equation used on the NCHRP 9-29 02 report is the modified version of the master curve equation included in the MEPDG (7).

$$\log |E^*| = \delta + \frac{(\mathrm{Max} - \delta)}{1 + e^{\beta + \gamma(\log fr)}} \qquad [9]$$

Where:

$|E^*|$ = dynamic modulus, psi

f_r = reduced frequency, Hz

Max = limiting maximum modulus, psi

δ, β, and γ = fitting parameters

The reduced frequency (f_r) is computed using the following equations (7):

$$\log f_r = \log f + \log [a(T)] \; ; \qquad [10]$$

$$\log [a(T)] = \frac{\Delta Ea}{19.14714} \left(\frac{1}{T} - \frac{1}{Tr}\right) \qquad [11]$$

Where:

f_r = reduced frequency at the reference temperature, Hz

f = loading frequency at the test temperature, Hz

[a(T)] = shift factor at temperature T

Tr = reference temperature, ^{0} K

T = test temperature, ^{0} K

ΔEa = activation energy (treated as a fitting parameter)

The maximum limiting modulus value is estimated from mixture volumetric properties using equation 12 (7).

$$|E|max = Pc\left[4{,}200{,}000\left(1 - \frac{VMA}{100}\right) + 435{,}000\left(\frac{VFA*VMA}{10{,}000}\right)\right] + \frac{1-Pc}{\frac{\left(1-\frac{VMA}{100}\right)}{4{,}200{,}000} + \frac{VMA}{435{,}000\,(VFA)}} \qquad [12]$$

Where:

$$Pc = \frac{\left(20+\frac{435{,}000(VFA)}{VMA}\right)^{0.58}}{650+\left(\frac{435{,}000(VFA)}{VMA}\right)^{0.58}} \qquad [13]$$

$|E^*|_{max}$ = limiting maximum mixture dynamic modulus, psi

VMA = voids in mineral aggregates, %

VFA = voids filled with asphalt, %

Fitting the Dynamic Modulus Master Curve

To fit the master curves, the limiting maximum modulus values (E^*_{max}) were first estimated using the average VMA and VFA of the specimens tested. The logarithm of the E^*_{max} values, which were calculated using equation 12 and 13, were computed and designated as Max. These values, with that of the reference temperature T_r (20 ^{0}C was used in this project) value, was then substituted in equation 9. The fitting parameters (δ, β, γ and ΔEa) were then determined using numerical optimization techniques (7).

A spreadsheet, which is capable of performing numerical optimization, was prepared for this project using the solver function in Microsoft EXCEL. This was done by preparing a spreadsheet which computes the sum of squared errors between the logarithm of the average measured dynamic moduli at each temperature/frequency combination and the values predicted using equation 9. The use of the solver function was to minimize the sum of squared errors by varying the fitting parameters in equation 9. The following initial estimates, which were recommended by the NCHRP 9-29 02 report (20), were used for the fitting parameters: $\delta=0.5$, $\beta=-1.0$, $\gamma= -0.5$, and $\Delta Ea= 200{,}000$ (7).

Finally, by using the logarithm of the limiting modulus (Max) value computed and the fitting parameters determined using the numerical optimization, the dynamic modulus at the recommended temperatures and frequencies were computed using equation 9. These values are given in tables 29 – 30. Figure 4-10 shows the fitted dynamic modulus master curves.

TABLE 29 Fitted Dynamic Modulus Values at MEPDG's Recommended Temperatures and Frequencies- SMA Mixtures

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

Table 30 Fitted Dynamic Modulus Values at MEPDG's Recommended Temperatures and Frequencies-S4 Mixtures

Temp. (°C)	Freq. (Hz)	Dynamic Modulus		
		TJ Campbell	CL-3	*
-10	25	3,112,369	3,095,250	3,153,904
	10	3,051,478	3,035,752	3,089,316
	5	2,993,845	2,979,341	3,030,241
	1	2,811,257	2,800,145	2,851,853
	0.5	2,707,443	2,697,990	2,754,430
	0.1	2,398,352	2,392,914	2,473,893
4.4	25	2,557,881	2,550,530	2,617,325
	10	2,364,715	2,359,640	2,443,901
	5	2,198,006	2,194,540	2,296,035
	1	1,756,416	1,755,855	1,904,617
	0.5	1,553,312	1,553,507	1,721,328
	0.1	1,096,385	1,097,185	1,289,845
21.1	25	1,264,609	1,265,345	1,452,786
	10	1,017,406	1,018,181	1,211,151
	5	850,014	850,641	1,038,528
	1	543,476	543,623	693,635
	0.5	447,223	447,219	574,288
	0.1	292,299	292,157	365,686
37.8	25	392,918	392,845	503,765
	10	308,954	308,816	389,281
	5	261,314	261,175	320,928
	1	188,076	188,020	210,345
	0.5	167,762	167,756	178,433
	0.1	136,482	136,591	128,253
54.4	25	167,202	167,197	177,545
	10	147,356	147,418	145,839
	5	136,170	136,280	127,746
	1	118,725	118,931	99,155
	0.5	113,745	113,986	90,873
	0.1	105,845	106,151	77,517

* From previous study by cross et al. (19)

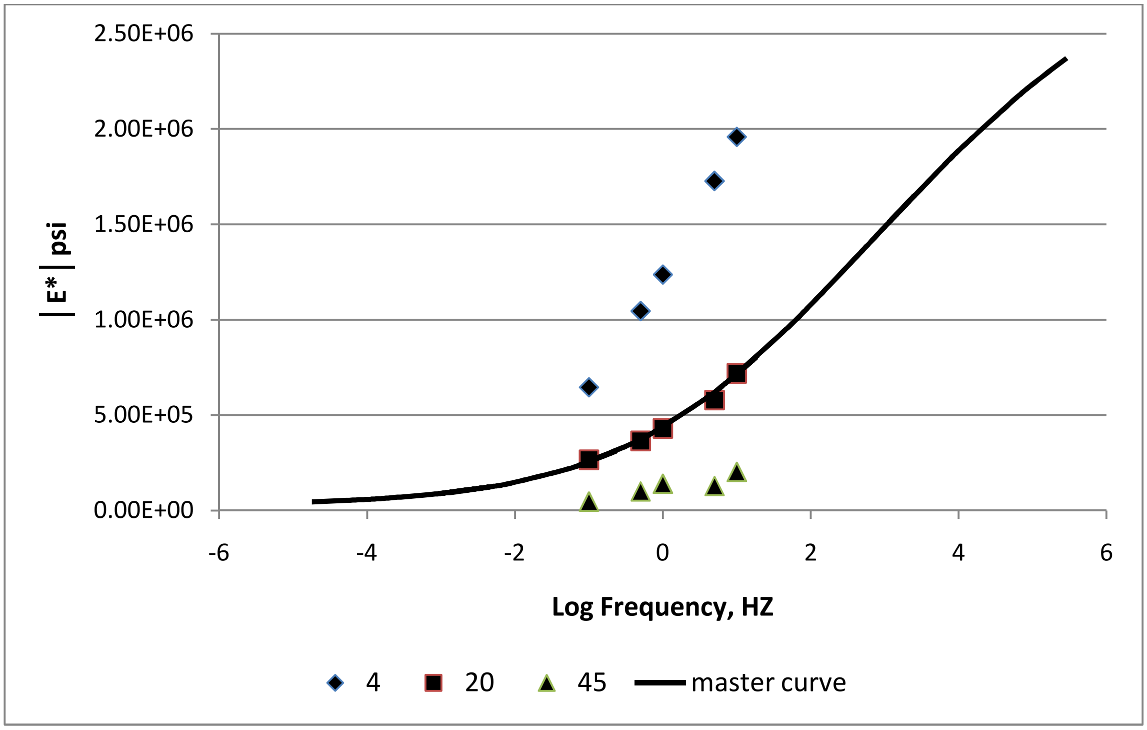

FIGURE 4 Master curve for Haskell Lemon 1 - SMA.

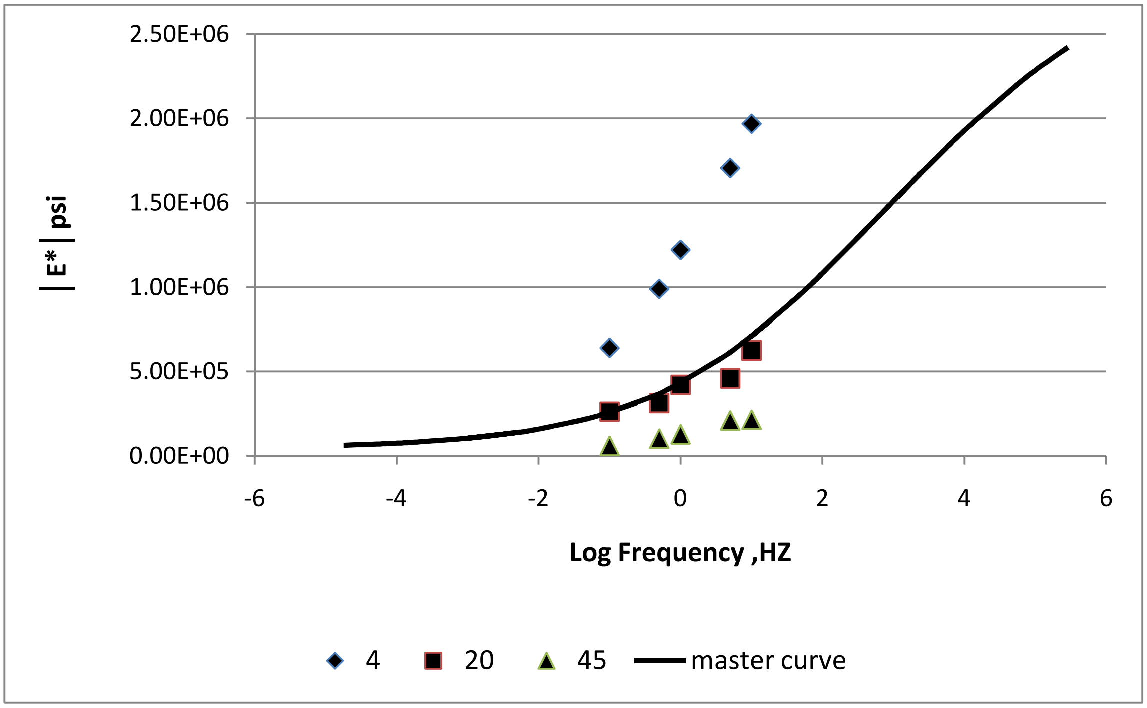

FIGURE 5 Master curve for Haskell Lemon 2 – SMA.

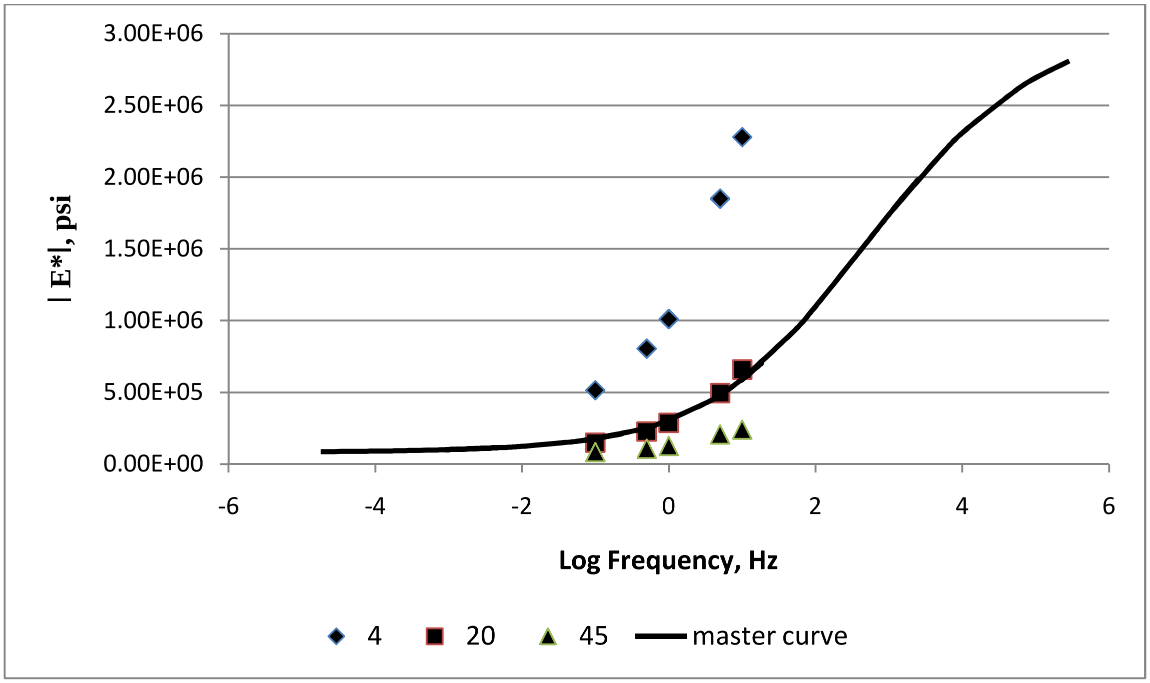

FIGURE 6 Master curve for Cornell 1 - SMA.

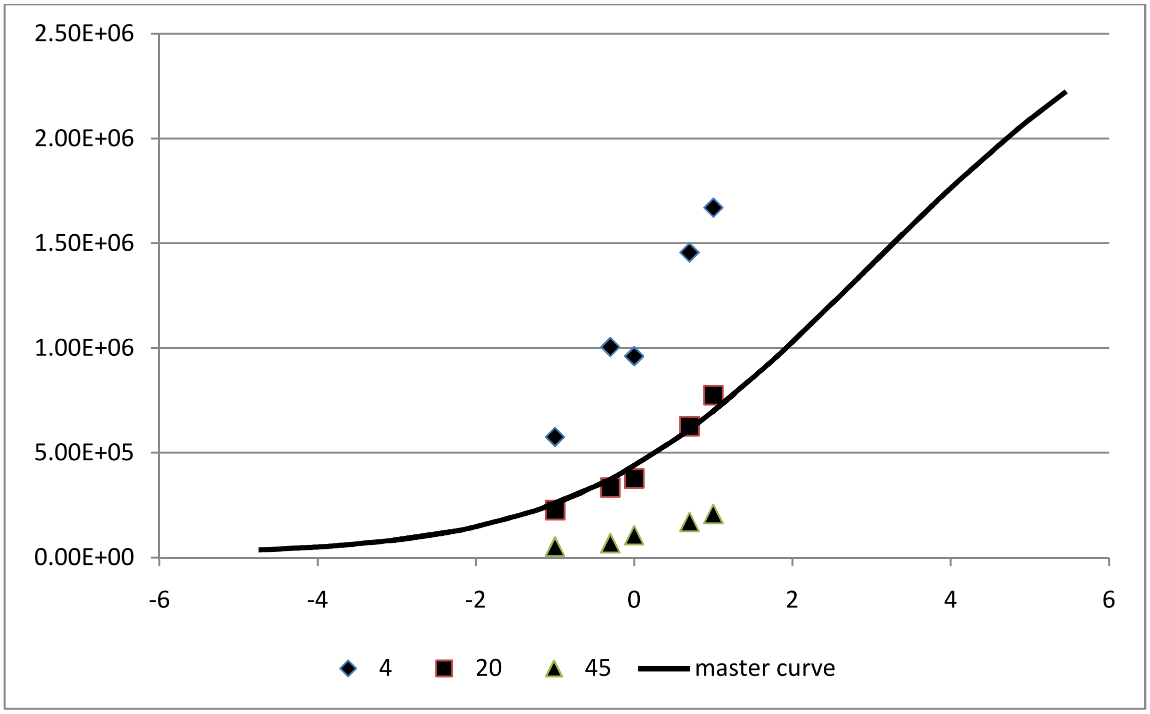

FIGURE 7 Master curve for Cornell 2 - SMA.

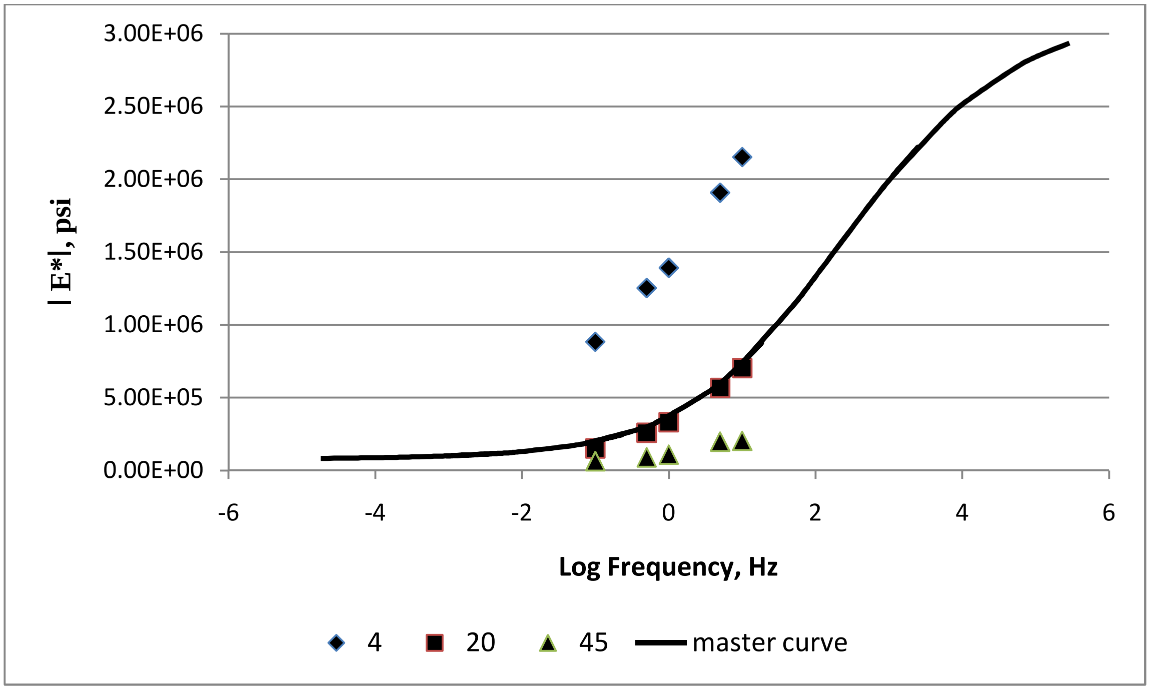

FIGURE 8 Master curve for Silver Star - SMA.

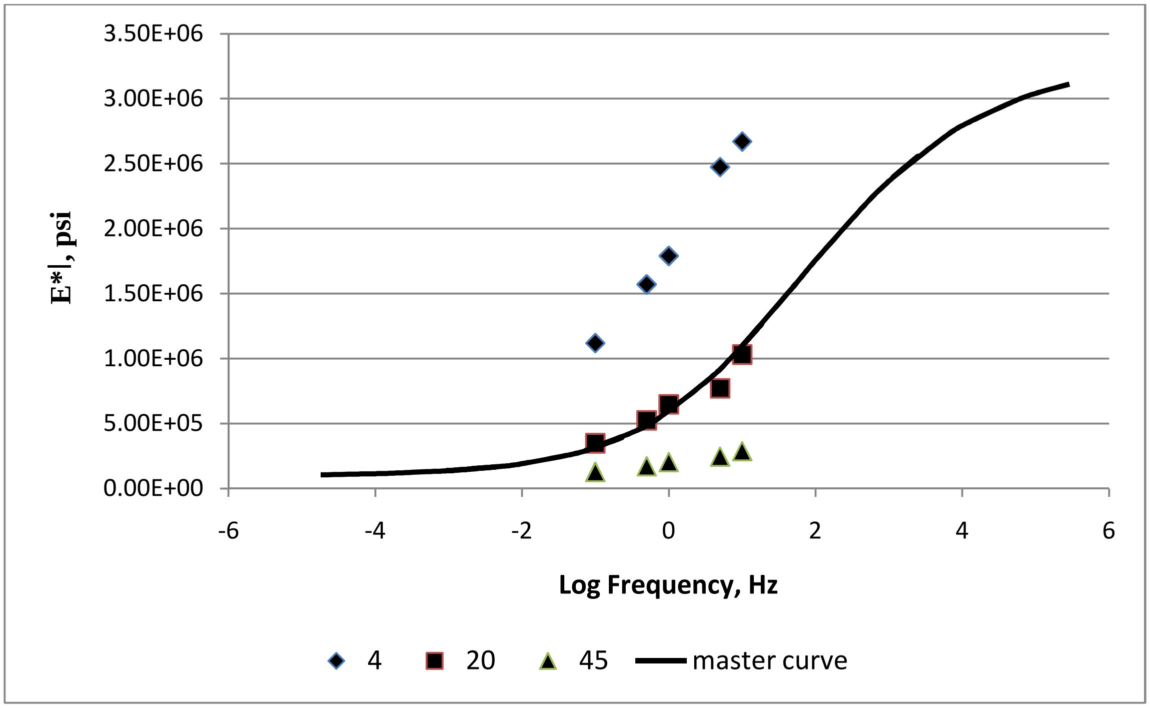

FIGURE 9 Master curve for T.J. Campbell – S-4.

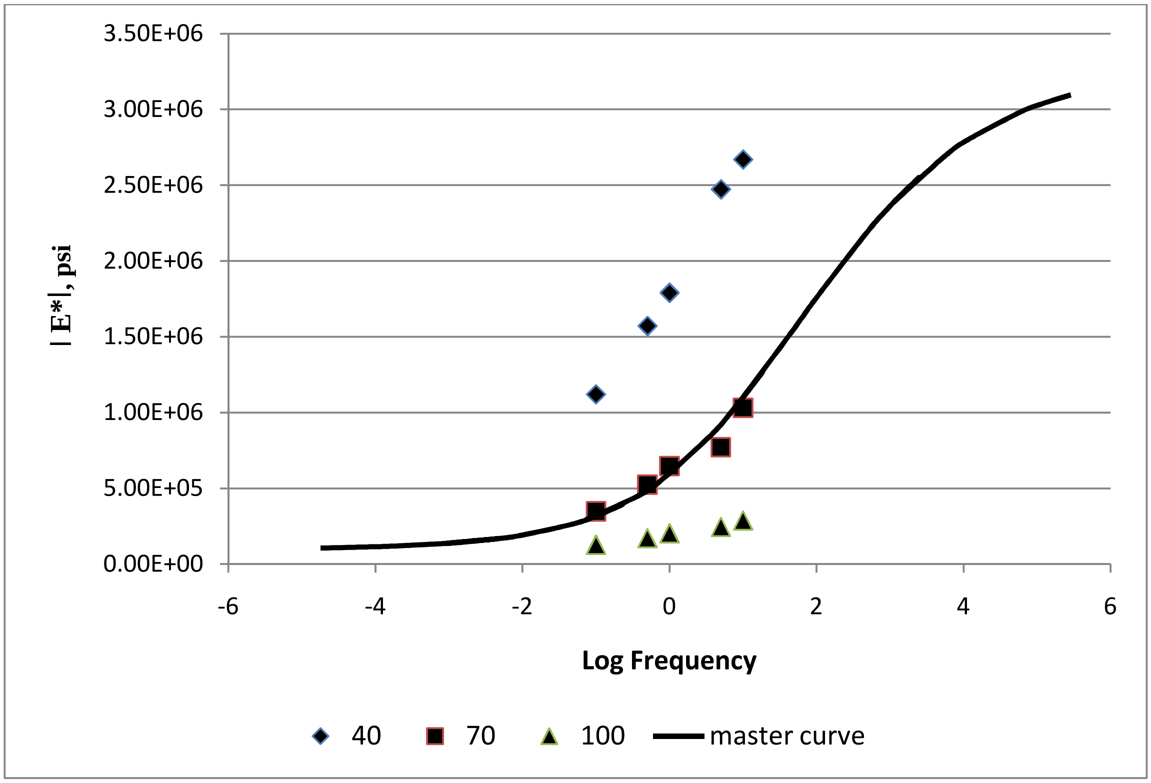

FIGURE 10 Master curve for Cornell 3 - S-4.

E* PREDICTIVE EQUATION

One of the objectives of this project was to compare the predicted dynamic modulus values of SMA mixtures with S-4 mixtures. There are different dynamic modulus predictive equations. From these equations, the MEPDG (5) uses the two equations to predict dynamic modulus values at inputs level 2 and 3.

The viscosity based Witczak equation (11), given as equation [8] in chapter 2, was developed based on data from 205 mixtures with 2,750 data points. This equation requires mixture volumetrics and aggregate gradation values as an input. It also requires the asphalt viscosity values at the temperature of interest. The viscosity is calculated from equation [7] by using default or experimentally derived A and VTS values as shown in detail on chapter 2 of this report. Default A and VTS values based on asphalt PG grade are available in the MEPDG. For PG 76-28 asphalt cement, the guide recommends to use A and VTS value of 9.2 and -3.024 respectively (5).

The viscosity based Witczak equation (11), using default A and VTS values, was used to predict the dynamic modulus values of the specimens tested in this project. The predicted values for all of the samples tested are shown in tables 31-32. The aggregate gradation and mixture volumetrics of the samples tested are given in table 16 of chapter 5.

TABLE 31 Dynamic Modulus Values - SMA Mixtures

Temp.	Freq.	Predicted Dynamic Modulus (psi)				
(°C)	(Hz)	CL - 1	CL-2	HL-1	HL-2	SS
-10	25	2,439,615	2,614,568	2,664,707	2,656,136	2,576,998
	10	2,310,412	2,475,173	2,522,818	2,514,296	2,440,335
	5	2,207,943	2,364,658	2,410,319	2,401,853	2,331,957
	1	1,956,465	2,093,589	2,134,354	2,126,091	2,066,009
	0.5	1,843,488	1,971,886	2,010,437	2,002,300	1,946,545
	0.1	1,574,895	1,682,761	1,716,014	1,708,267	1,662,574
4.4	25	1,572,588	1,680,279	1,713,486	1,705,742	1,660,135
	10	1,418,626	1,514,700	1,544,843	1,537,389	1,497,388
	5	1,303,196	1,390,640	1,418,472	1,411,271	1,375,387
	1	1,043,984	1,112,337	1,134,928	1,128,421	1,101,478
	0.5	938,254	998,951	1,019,382	1,013,215	989,779
	0.1	712,231	756,869	772,628	767,324	751,057
21.1	25	764,639	812,959	829,808	824,285	806,401
	10	645,661	685,664	700,030	695,021	680,765
	5	563,207	597,539	610,168	605,556	593,716
	1	398,459	421,743	430,852	427,154	419,842
	0.5	339,145	358,566	366,387	363,068	357,266
	0.1	227,185	239,533	244,887	242,379	239,191
37.8	25	325,839	344,402	351,933	348,704	343,229
	10	259,950	274,333	280,415	277,655	273,738
	5	217,471	229,222	234,358	231,928	228,949
	1	140,564	147,716	151,112	149,351	147,894
	0.5	115,544	121,259	124,079	122,561	121,536
	0.1	72,321	75,654	77,461	76,407	76,023
54.4	25	138,646	145,686	149,039	147,296	145,873
	10	106,867	112,092	114,710	113,281	112,396
	5	87,396	91,543	93,706	92,483	91,893
	1	54,298	56,688	58,064	57,225	57,054
	0.5	44,158	46,036	47,166	46,456	46,386
	0.1	27,396	28,468	29,185	29,152	28,759

TABLE 32 Predicted Dynamic Modulus Values – S4 Mixtures

Temp (°C)	Freq (Hz)	Predicted Dynamic Modulus (psi)		
		TJC	CL-3	*
-10	25	3,150,325	3,887,139	4,120,061
	10	2,983,411	3,678,743	3,899,834
	5	2,851,038	3,513,572	3,725,259
	1	2,526,181	3,108,632	3,297,156
	0.5	2,380,243	2,926,921	3,104,996
	0.1	2,033,306	2,495,501	2,648,618
4.4	25	2,030,325	2,491,799	2,644,701
	10	1,831,467	2,244,917	2,383,430
	5	1,682,383	2,060,042	2,187,723
	1	1,347,618	1,645,670	1,748,868
	0.5	1,211,082	1,477,010	1,570,150
	0.1	919,225	1,117,303	1,188,773
21.1	25	986,894	1,200,594	1,277,111
	10	833,272	1,011,616	1,076,652
	5	726,816	880,905	937,935
	1	514,132	620,505	661,384
	0.5	437,568	527,063	562,066
	0.1	293,065	351,283	375,077
37.8	25	420,393	506,127	539,807
	10	335,351	402,631	429,724
	5	280,528	336,076	358,889
	1	181,288	216,031	231,007
	0.5	149,006	177,139	189,533
	0.1	93,247	110,220	118,103
54.4	25	178,813	213,046	227,825
	10	137,811	163,674	175,168
	5	112,692	133,513	142,979
	1	70,000	82,453	88,428
	0.5	56,922	66,879	71,772
	0.1	35,308	41,243	44,326

* From previous study by cross et al. (19)

COMPARISON OF E* DATA

A comparison was made between dynamic modulus values of SMA and S-4 mixtures. The comparison was made on both experimentally computed and predicted E* values. A study by Cross et al (19) showed that frequency has a consistent effect on dynamic modulus. Hence, the comparison was made only on one frequency which simplified the analysis. The average measured and predicted dynamic modulus values at 1 Hz frequency are shown in table 33-34. The percent increase for S4 mixtures E* values as compared to SMA mixtures values are shown in table 35. The comparison between the modulus values of S4 and SMA mixtures at 1 Hz frequency are shown graphically in figures 11 and 12.

Table 33 Average Measured E* Values at 1 Hz Frequency

Temperature (C)	Dynamic Modulus (psi)		Frequency (Hz)
	SMA	S-4	
-10	2,050,896	2,851,853	1
4.4	1,136,742	1,904,617	1
21.1	404,216	693,635	1
37.8	145,790	210,345	1
54.4	73,587	99,155	1

TABLE 34 Average Predicted E* Values at 1 Hz Frequency

Temperature (C)	Dynamic Modulus (psi)		Frequency (Hz)
	SMA	S-4	
-10	2,075,302	2,977,323	1
4.4	1,104,230	1,580,719	1
21.1	419,610	598,674	1
37.8	147,327	209,442	1
54.4	56,666	80,294	1

TABLE 35 Percent Increase in S4 E* Compared to SMA E*

Temperature (C)	Percent increase in E* (psi) Measured	Predicted	Frequency (Hz)
-10	28.1	30.3	1
4.4	40.3	30.1	1
21.1	41.7	29.9	1
37.8	30.7	29.7	1
54.4	25.8	29.4	1

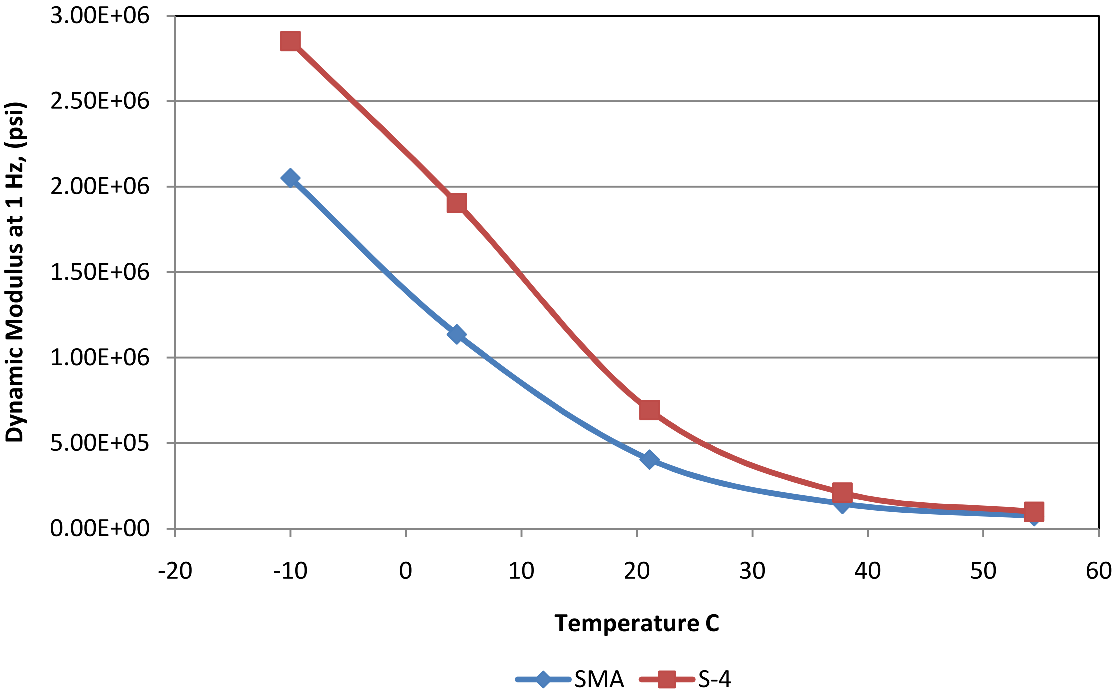

FIGURE 11 Average measured SMA and S4 E* values at 1 Hz frequency.

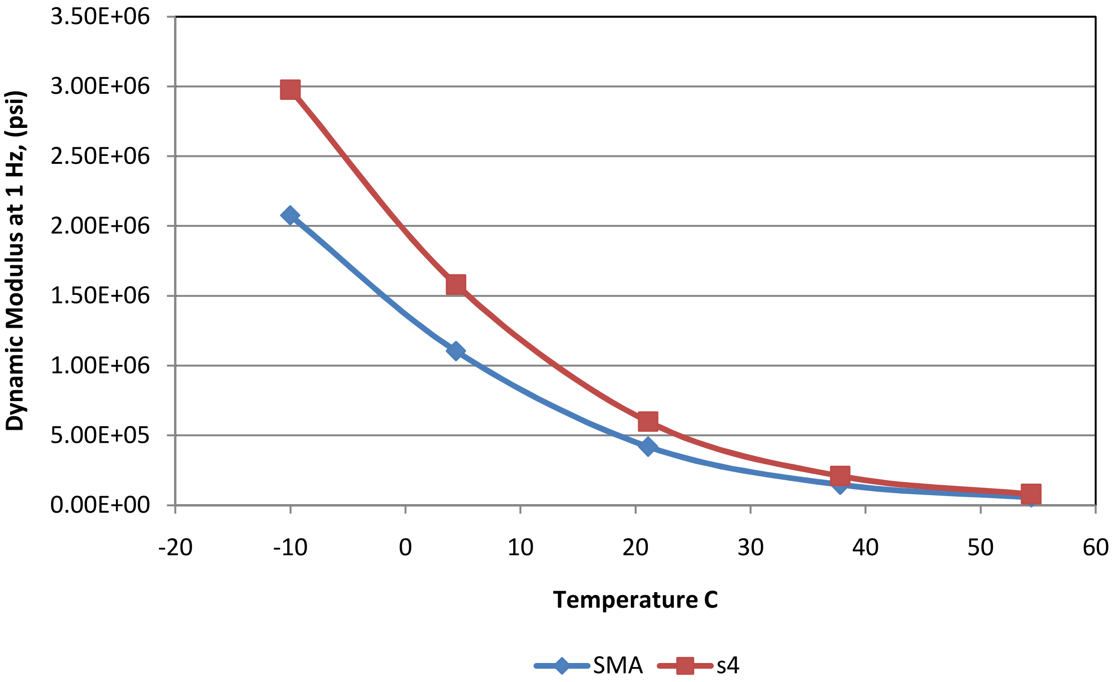

FIGURE 12 Average predicted SMA and S4 E* values at 1 Hz frequency.

As shown in figure 11 and 12, both the measured and predicted E* values of S-4 mixtures are greater than SMA mixtures. The percent increase in the predicted E* values of the S-4 mixtures compared to the SMA mixtures are around 30% at all temperatures. On the other hand, the percent increase in the experimentally measured E* values are different for different temperatures as shown in table 35. The S-4 mixtures have showed a 28, 40, 42, 31, and 26 % increase in measured E* values as compared to the SMA mixtures at -10, 4.4, 21.1, 37.8, and 54.4 ^{0}C temperatures respectively.

Findings

The following observations were made from the compared E* data:

- The measured dynamic modulus values of S-4 mixtures are greater than SMA mixtures.
- The predicted dynamic modulus values of S-4 mixtures are greater than SMA mixtures.
- S-4 mixtures are stiffer than SMA mixtures.

CHAPTER VII

MEPDG

MEPDG SOFTWARE VERSION 1.1 SENSITIVTY ANALYSIS

Input Parameters

A sensitivity analysis was performed to investigate the impact of input parameters on predicted distress. The analysis was made on MEPDG software version 1.1 at input level 3. The input parameters which were changed in the sensitivity analysis are:

- Date of traffic opening
- Traffic (AADTT)
- Climate
- Water table
- Aggregate base thickness
- Asphalt layer combination
- Aggregate base layer resilient modulus (M_R)
- Subgrade layer resilient modulus (M_R)

The impact of changing the aforementioned input parameters were studied by investigating the following distresses: Terminal IRI, AC surface down (longitudinal) cracking, AC bottom up (alligator) cracking, permanents deformation (AC only) and permanent deformation (total pavement). The baseline values used in the analysis are shown in table 36 and figure 13.

TABLE 36 Summary of Baseline Values

Parameter	Description
Design life	30 years
Traffic opening	Spring
Climate	Stillwater
Water table	30’
<u>**Traffic**</u>	
Initial two-way AADTT:	15000
Number of lanes in design direction:	2
Percent of trucks in design direction (%):	50
Percent of trucks in design lane (%):	85
Operational speed (mph):	60
Compound growth rate:	2
Layers	**See figure 13**

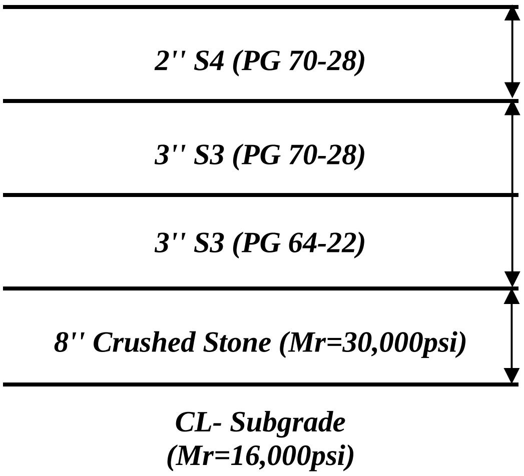

FIGURE 13 Baseline Layer Section.

As discussed in chapter 2 of this report, to define the HMA layers at input level 3 of the MEPDG, the following input parameters are needed: reference temperature, aggregate gradation, mixture volumetric properties, thermal properties, and poisons ratio. The sensitivity analysis was made using default values of the software for all of these input parameters except for aggregate gradation and mixture volumetric properties. The values for these two input parameters were taken from a previous study by Cross in 2007 (19). These values are presented in table 37.

TABLE 37 Aggregate Gradation and Volumetric Properties of S-3 and S-4 Mixtures

Mixes	% Reatined			% Pass.	Va (%)			Vbeff (%)		
	3/4 "	3/8 "	No. 4	No. 200	64-22	70-28	76-28	64-22	70-28	76-28
S4	0	12.3	35.1	5.28	4.5	4.35	4.33	9.16	9.16	9.1
S3	0	35	71	10	4.4	4.29	4.31	8.39	8.37	13.87

Using the aforementioned baseline values, distresses were predicted using the MEPDG software. The baseline parameters were then varied over a certain range and distresses were predicted for each modification. Summary of the ranges of the parameters used in the sensitivity analysis are shown in table 38.

TABLE 38 Summary of the Ranges of Parameters Used in the Sensitivity Analysis

parameters	Range
Traffic opening	Spring, Summer and Fall
Traffic	5000, 8000, 15000 and 20000 AADTT
Climate	Stillwater, Oklahoma city and Tulsa
Water table	20’, 30’ and 40’
Layers	
Asphalt layer combination *	70-70-64 76-76-64 64-64-64
Aggregate Base layer thickness	6’’, 8’’, 10’’ and12 ‘’
Crushed stone Aggregate base layer resilient modulus (M_R) **	20000, 30000 and 40000 psi
CL Sub grade layer resilient modulus (M_R)**	13,500, 16000 and 18,500 psi

* 70-70-64 indicates a 2 " PG 70-28 S4 mix, 3" PG 70-28 S3 mix, and 3" PG 64-22 S3 mix
76-76-64 indicates a 2 " PG 76-28 S4 mix, 3" PG 76-28 S3 mix, and 3" PG 64-22 S3 mix
64-64-64 indicates a 2 " PG 64-22 S4 mix, 3" PG 64-22 S3 mix, and 3" PG 64-22 S3 mix

** The ranges are the typical minimum and maximum values given in the MEPDG software (5).

Sensitivity Analysis Results

The sensitivity analysis was then made by comparing the baseline and modified distress values. This helped to see how much impact modifying input parameters have on the predicted distress values and also identify which input parameters have the larger impacts on predicted performances. Summary of the results of the sensitivity analysis is shown in table 39. Based on this result, the following observations were made about the impact each input parameters on the predicted distress.

Date of Traffic Opening

The sensitivity analysis shows that a pavement will have the same amount of distress whether it is opened to traffic in the summer, fall or spring season. This shows that changing the traffic opening season does not have any impact on the prediction of distresses.

AADTT

As it can be seen from the sensitivity analysis, changing the initial two-way AADTT alters the predicted distress values in a significant way. The analysis shows a decrease in the predicted distress values when the initial AADTT is decreased and an increase when it increases.

Climate

As in traffic opening, changing the climate from Stillwater to Oklahoma City to Tulsa did not show a significant impact on predicted distress. This shows that pavements, which are constructed in cities with similar climatic condition, will exhibit equal amount of distresses at the end of the design period as expected.

Thickness of Aggregate Base

According to the result of the sensitivity analysis, changing the thickness of the aggregate base layer changes the predicted cracking and rutting distress values in opposite ways. As shown in table 39, decreasing the thickness of the base layer increases the longitudinal and alligator cracking while it decreases the AC and total pavement rutting. Decreasing the thickness decreases the predicted cracking distress values while the rutting values increases. The change observed in terminal IRI due to change in aggregate thickness is negligible.

Binder Stiffness

The impact of the binder stiffness was also evaluated. When the PG grade of the asphalt binder was changed from PG 70-28 to PG 76-28, the values predicted for all types of distresses decreases a considerable amount. In the contrary, when it was changed from PG 70-28 to PG 64-22, the distress values increase. This means the stiffer the asphalt binder, the less the predicted distress.

Aggregate Base M_R

Aggregate base resilient modulus (M_R) was observed to have an impact on the prediction of longitudinal and alligator cracking. Decreasing the M_R of the aggregate base layer by 10,000 psi increases the alligator cracking by 30% and almost doubles the longitudinal crack .Increasing this value by the same amount decreases the longitudinal and alligator cracking by 58% and 23%, respectively. On the other hand, changing this value did not show any impact on the prediction of total pavement rutting and only showed a small impact on the prediction of AC rutting and terminal IRI.

Subgrade M_R

According to the sensitivity analysis, subgrade M_R impacts the prediction of all distresses. Decreasing the subgrade baseline M_R value to the typical minimum value decreases the longitudinal cracking and AC rutting values while increasing terminal IRI, alligator cracking and total pavement rutting values. Increasing this same baseline value to the typical maximum value increases the longitudinal cracking and AC rutting values while it decreases the terminal IRI, alligator cracking and total pavement rutting values.

TABLE 39 Sensitivity Analysis

		Performance criteria				
		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)
Parameters	Range	Change in Distresses				
Traffic opening	Fall	169.0 (0.0%)	1080 (0.0%)	7.9 (0.0%)	0.67 (0.0%)	1.07 (0.0%)
	* Spring	169.1	1060	7.9	0.67	1.07
	Summer	168.8 (0.0%)	1070 (0.0%)	7.9 (0.0%)	0.68 (0.0%)	1.07 (0.0%)
Traffic (AADTT)	5000	153.3 (-9.3%)	218 (-79.4%)	2.60 (-67.1%)	0.40(-40.3%)	0.74 (-30.8%)
	8000	159.0 (-6.0%)	436 (-58.9%)	4.20 (-46.8%)	0.50 (-25.4%)	0.86 (-19.6%)
	* 15000	169.1	1060	7.9	0.67	1.07
	20000	175 (+ 3.4%)	1560 (+47.2%)	10.5 (+32.9%)	0.77 (+14.9%)	1.18 (+10.3%)
Climate	Oklahoma city	169.1 (0.0%)	1070 (+0.9%)	7.9 (0.0%)	0.67 (0.0%)	1.07 (0.0%)
	* Stillwater	169.1	1060	7.9	0.67	1.07
	Tulsa	169.1 (0.0%)	1060 (0.0%)	7.9 (0.0%)	0.67 (0.0%)	1.07 (0.0%)
Water table	20'	169.3 (+0.1%)	1070 (+0.9%)	7.9 (0.0%)	0.68 (+1.5%)	1.08 (+0.9%)
	* 30'	169.1	1060	7.9	0.67	1.07
	40'	168.9 (-0.1%)	1110 (+4.7%)	7.9 (0.0%)	0.67 (0.0%)	1.06 (-0.9%)

Aggregate base thickness	6	169.3 (+0.1%)	1660 (+56.6%)	9 (+13.9%)	0.66 (-1.5%)	1.06 (-0.9%)
	*8	169.1	1060	7.9	0.67	1.07
	10	168.7 (-0.2%)	799 (-24.6%)	7.1 (-10.1%)	0.69 (+3.0%)	1.07 (0.0%)
	12	168.6 (-0.3%)	643 (-39.3%)	6.5 (-8.50%)	0.7 (+4.5%)	1.08 (+0.9%)
Asphalt layer Comb.	*70-28,70-28,64-22	169.1	1060	7.9	0.67	1.07
	76-28,76-28,64-22	164.7 (-2.6%)	842 (-20.6%)	7.32 (-7.9%)	0.58(-13.4%)	0.97 (-9.3%)
	64-22,64-22,64-22	171.4 (+1.4%)	1330 (+25.5%)	8.10(+2.5%)	0.73 (+9.0%)	1.13 (+5.6%)
Aggregate base layer (Mr)	20000	170.2 (+0.7%)	2110 (+99.1%)	10.1 (+27.8%)	0.65 (-3%)	1.07 (0.0%)
	*30000	169.1	1060	7.9	0.67	1.07
	40000	168.1 (-0.6%)	446 (-57.9%)	6.1 (-22.8%)	0.69 (+3%)	1.07 (0.0%)
Subgrade base layer (Mr)	13500	171.1 (+1.2%)	603 (-43.1%)	8.7 (+10.1%)	0.66 (-1.5%)	1.11 (+3.7%)
	*16000	169.1	1060	7.9	0.67	1.07
	18500	167.6 (-0.9%)	1570 (+48.1%)	7.3 (-7.6%)	0.68 (+1.5%)	1.04 (-2.8%)

* indicates baseline values.

(+ %) indicates an increase in distress from the baseline values in percentile.

(- %) indicates a decrease in distress from the baseline values in percentile.

(0.0%) indicates no change in distress.

Findings

The following observations were made from the results of the sensitivity analysis:

- Changing season of traffic opening has no impact on prediction of distresses.
- Traffic impacts prediction of distresses significantly.
- Depth of water table only impacts the prediction of alligator cracking.
- Aggregate base thickness has a considerable impact on the prediction of all types of distresses.
- Changing the stiffness of asphalt binder changes the predicted distress values in a significant way.
- Changing the aggregate base M_R value impacts the prediction of all types of distresses except total pavement rutting.
- Subgrade M_R Value impacts all types of distresses.

COMPARISON OF SMA TO S-4

One of the objectives of this project was to compare distress values of SMA and S-4 mixes at the end of the design period using the MEPDG software. The comparison was made at input levels 1 and 3 of the MEPDG. The comparison was made separately at each input level. This was done to verify the results from one of the input level with the other.

To investigate the difference in predicted distress using experimental (input level 1) and default (input level 3) inputs, comparisons were made between the two input levels. The distresses predicted at input level 1 were compared with the distresses predicted at input level 3. This comparison was done for S-4 and SMA mixes separately.

Input Parameters

Four simulations were run at each input levels. The parameters used on both input levels are presented in table 40.

TABLE 40 Summary of Input Parameters

Parameters	Cases	Description
HMA and aggregate base course layers	2	***2’’ S4 or SMA (PG 76-28)*** ***3'' S3 (PG 76-28)*** ***3'' S3 (PG 64-22)*** ***8'' Crushed Stone (Mr=30,000psi)***
CL- Subgrade layer	2	Good: Water table = 30 feet Good: Subgrade M_R = 16,000 psi Poor: Water table = 5 feet Poor: Subgrade M_R = 5000 psi

The values for the rest of the input parameters, which are required to run the software, are given in tables 18, 36 and 37. In addition to these data, actual laboratory measured dynamic modulus and binder test (G* and δ) data are required at input level 1. These values are presented below in tables 41 and 42. No dynamic shear rheometer (DSR) tests were performed in this project. The G* and δ values shown in table 41 are from an Oklahoma University study on the binder used in this research project (22).

The SMA E* values shown in table 42 are the average of the E* values given in table 19. The S-4 and S-3 E* values shown in this same table are taken from a previous study by Cross et. al (19).

TABLE 41 G* and δ values

Binder Type	Testing Temp (c)	G*	δ
PG 64-22	4.4	23778.84	47
	12.7	4574	48.8
	21.1	4869.11	45.5
	29.4	402.11	63.7
	43.3	56.52	71
	46.1	34.2	73.6
	54.4	10.32	78.7
PG 76-28	4.4	13726.5	46.5
	12.7	3287.2	47.5
	21.1	548.47	58.1
	29.4	181.4	56.6
	43.3	40.47	52.4
	46.1	30.03	51.9
	54.4	14.09	50.3

Table 42 Dynamic modulus (E*) values

Temp. (°C)	Freq. (Hz)	Dynamic Modulus (psi)			
		SMA	S4 *	S3*	
		PG 76-28	PG 76-28	PG 76-28	PG 64-22
-10	25	2,516,078	3,153,904	3,109,313	3,196,765
	10	2,401,956	3,089,316	3,030,898	3,134,236
	5	2,305,872	3,030,241	2,960,410	3,076,804
	1	2,050,896	2,851,853	2,753,362	2,902,125
	0.5	1,928,469	2,754,430	2,643,366	2,805,998
	0.1	1,622,148	2,473,893	2,336,431	2,526,549
4.4	25	1,771,289	2,617,325	2,491,674	2,669,902
	10	1,592,649	2,443,901	2,304,378	2,496,451
	5	1,454,594	2,296,035	2,148,272	2,347,455
	1	1,136,742	1,904,617	1,748,834	1,948,350
	0.5	1,006,235	1,721,328	1,567,881	1,759,219
	0.1	732,225	1,289,845	1,155,166	1,308,822
21.1	25	830,979	1,452,786	1,308,961	1,479,704
	10	686,253	1,211,151	1,081,724	1,225,983
	5	588,766	1,038,528	922,412	1,043,674
	1	404,216	693,635	610,731	678,126
	0.5	342,083	574,288	504,608	551,921
	0.1	232,533	365,686	320,559	333,727
37.8	25	305,368	503,765	442,223	477,677
	10	245,139	389,281	341,319	358,139
	5	208,325	320,928	281,189	287,717
	1	145,790	210,345	183,770	176,507
	0.5	126,531	178,433	155,529	145,409
	0.1	94,215	128,253	110,857	97,989
54.4	25	125,984	177,545	154,742	144,553
	10	105,904	145,839	126,563	114,360
	5	93,870	127,746	110,403	97,522
	1	73,587	99,155	84,687	71,681
	0.5	67,295	90,873	77,177	64,414
	0.1	56,526	77,517	64,968	52,956

* from previous study by cross et al.(19)

Using all of the input data mentioned above, a total of eight simulations were run using the MEPDG software. The distresses predicted by the software at the end of the design period are summarized and presented in table 43. Graphical presentations of the predicted distresses are also shown in figures 14-18.

TABLE 43 Summary of Predicted Distresses

		Level 3				Level 1			
		Poor Subgrade		Good Subgrade		Poor Subgrade		Good Subgarde	
Distresses	***Distress Target***	S4	SMA	S4	SMA	S4	SMA	S4	SMA
Terminal IRI (in/mi)	172	185.5	192.7	164.7	172.2	182.9	191.3	162.6	171.4
AC Surface Down Cracking (Long. Cracking) (ft/mile)	2000	1.3	2.2	842	1110	0.5	1.4	295	612
AC Bottom Up Cracking (Alligator Cracking) (%)	25	14.5	17.1	7.3	8.9	12.3	15.1	6	7.6
Permanent Deformation (AC only) (in)	0.25	0.52	0.62	0.58	0.73	0.51	0.64	0.55	0.73
Permanent Deformation (Total Pavement) (in)	0.75	1.39	1.53	0.97	1.13	1.36	1.53	0.93	1.13

The distress target values shown in the table are default values from the MEPDG. The distress values given in the table are at 50% reliability.

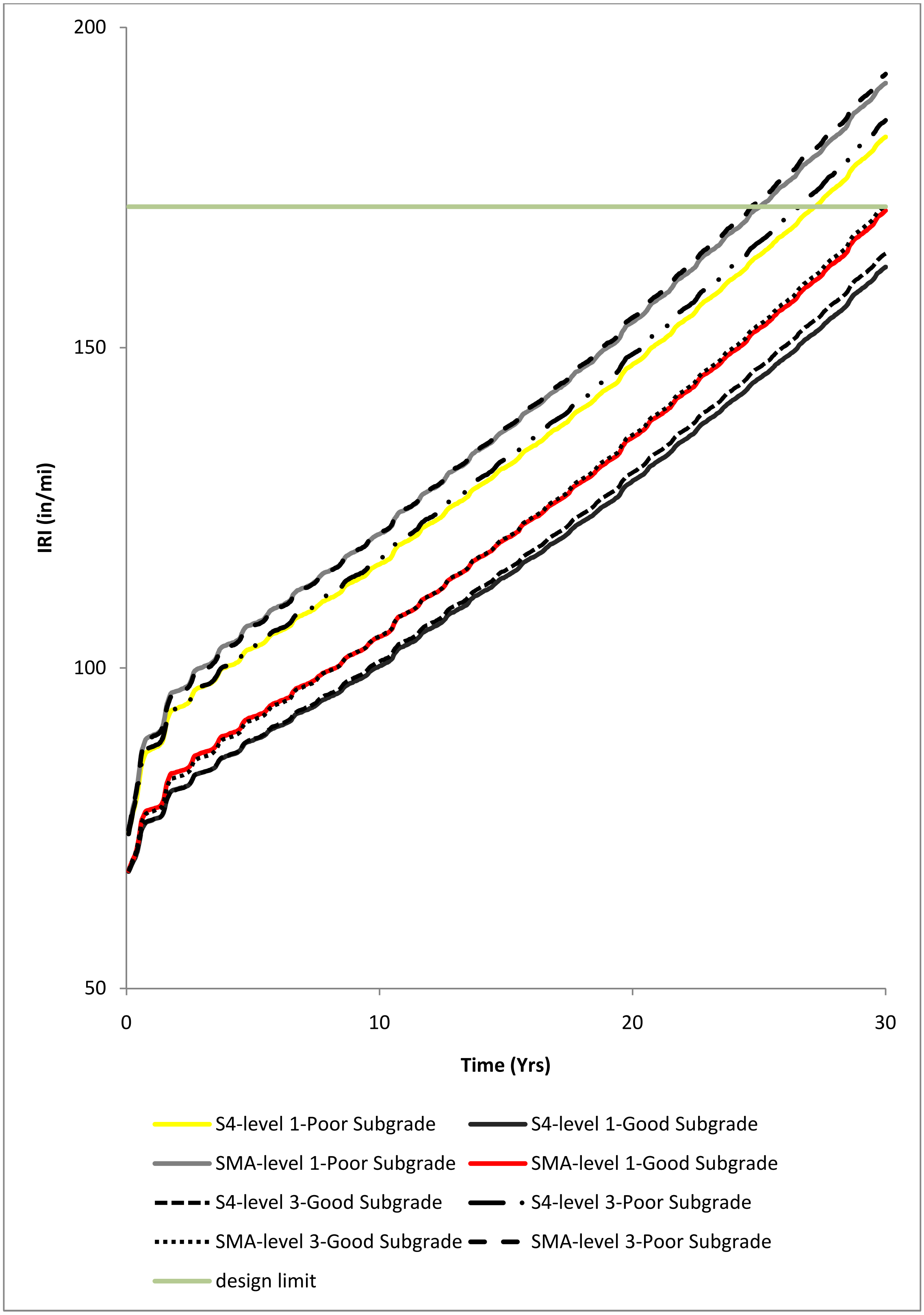

FIGURE 14 Terminal IRI Distresses.

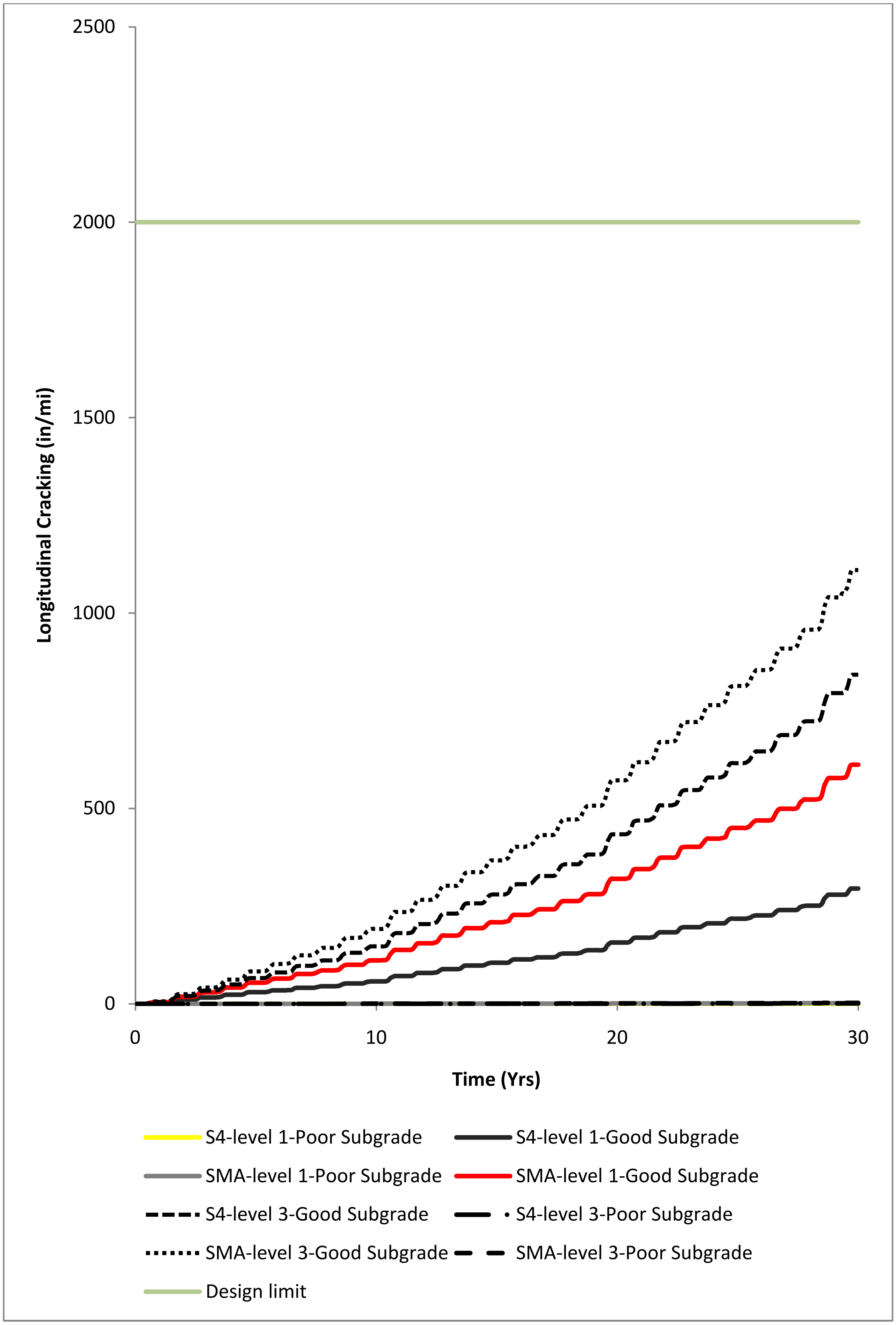

FIGURE 15 Longitudinal Cracking Distresses.

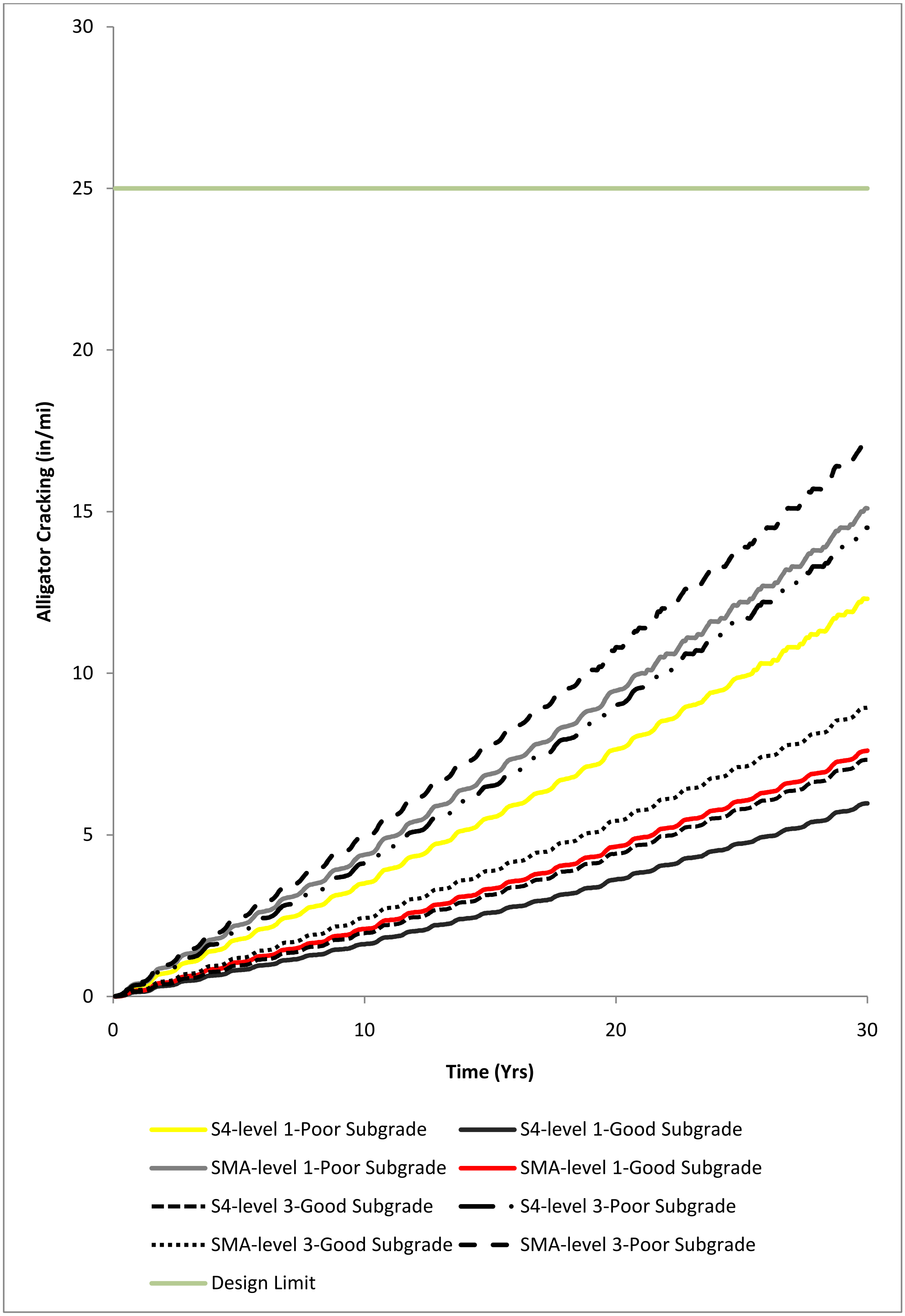

FIGURE 16 Alligator Cracking Distresses.

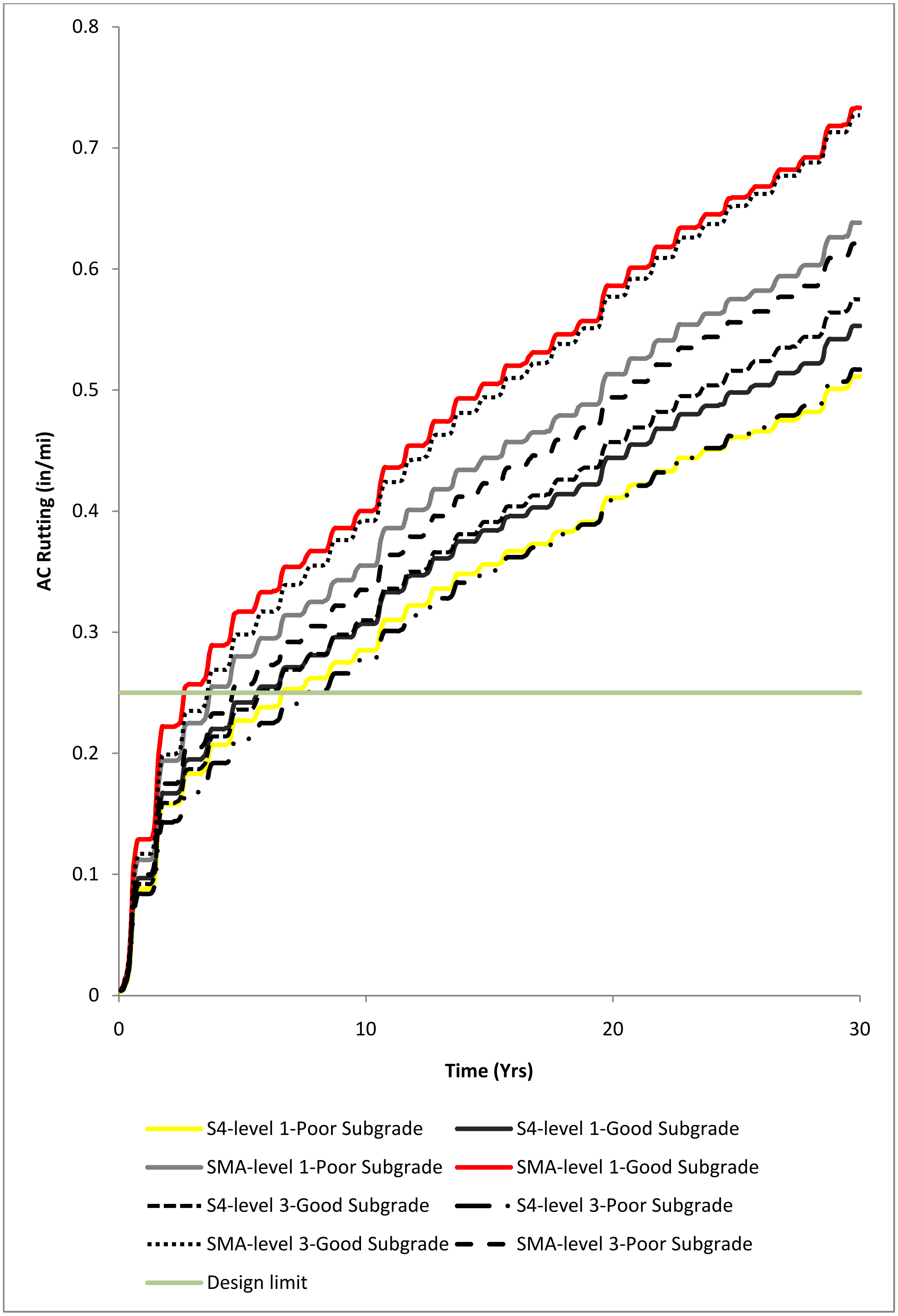

FIGURE 17 AC Rutting Distresses.

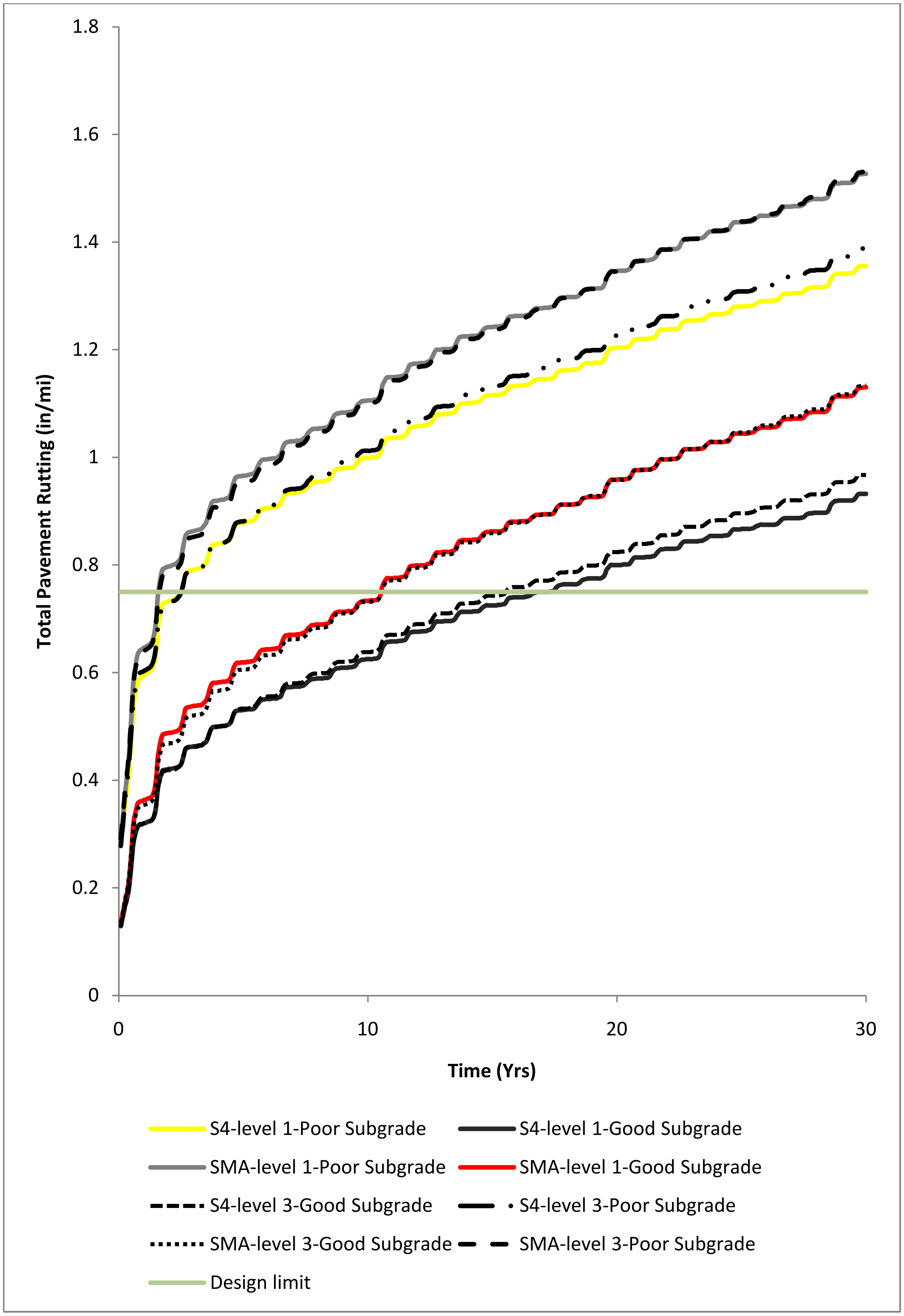

FIGURE 18 Total Pavement Rutting Distresses.

Analysis of MEPDG Software Results of SMA and S-4 Mixes

As it can be seen from table 43 and figure 14, the predicted terminal IRI distresses for pavements constructed on a “poor subgrade” are higher than pavements constructed on a “good subgrade” on both input levels 1 and 3. The sensitivity analysis, which was done on only input level 3, showed the exact same thing. The terminal IRI values predicted for pavements with S-4 surface course mixes are lower than the SMA mixes in both types of subgrades on the two input levels. As shown in figure 17, the terminal IRI values predicted at input level 1 are almost equal to the values predicted at input level 3 at the beginning of the design period. This trend continues until the 20 year design period. After this period, the values from input level 3 looked to be a little bit higher than the values from input level 1.

According to the sensitivity analysis, subgrade resilient modulus value is directly and inversely proportional to longitudinal and alligator cracking, respectively. The same thing is observed here as shown in table 43 and figure 15-16. The longitudinal cracking values predicted using “poor subgrade’’ inputs are less than those values predicted using “good subgrade” at both input levels 1 and 3 for both S-4 and SMA mixes. As shown in figure15, almost no longitudinal cracking was predicted for pavements constructed on “poor subgrade”. On the other hand, alligator cracking values are higher on pavements with “poor subgrade” layers than “good subgrade” layers. As with terminal IRI, pavements with S-4 surface course mixes exhibit less longitudinal and alligator cracking values as compared to pavements with SMA mixes throughout the design period at the two input levels. Pavements constructed using either S-4 or SMA mixes have less cracking at input level 1 than they do at level 3.

Referring to table 43 and figure 17-18, it can be said that pavements with “poor subgrade” have less AC rutting and more total pavement rutting values than pavements with “good subgrade” layer. Referring to the same table and figures, it can also be said that pavements with S-4 surface course mixes will not rut as much as pavements with SMA surface mixes. The AC rutting values of S-4 mixtures and total pavement rutting values of both S4 and SMA mixtures tends to be higher at input level 1 than input level 3 at the beginning of the design period. The trend for these values then starts to change as shown in figure 17-18 and higher values were predicted at input level 3 at the end of the design period. The AC rutting predicted at input level 1for SMA mixes were higher than the values predicted at input level 3 for the entire design period. This is the only case that distresses predicted at input level 1 were found to be higher than distresses predicted at input level 3 for the entire design period.

Findings

The following observations were made from the comparison of the distresses:

- Pavements with S-4 surface course mixes exhibit less distress than pavements with SMA surface course mixes at both input level 1 and level 3.
- Pavements with lower subgrade modulus values have less terminal IRI, alligator cracking and total pavement rutting values and more longitudinal cracking and AC rutting values than pavements with larger subgrade values.
- Distresses predicted at input level 3 are higher than distresses predicted at input level 1 except for AC rutting on SMA mixtures.

CHAPTER VII

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

SMA and S-4 mixtures were prepared and tested for dynamic modulus in this research project. The following conclusions are drawn based on the results of the testing and analysis performed.

1. Test temperature had a significant effect on measured dynamic modulus values. The laboratory measured dynamic modulus values of both S-4 and SMA mixtures tested in this research project were different at different test temperatures.
2. Stiffness of both S-4 and SMA mixes decrease as temperature increases.
3. Mix type had a significant effect on measured dynamic modulus values. The average measured dynamic modulus value of S-4 and SMA mixes were significantly different.
4. The measured dynamic modulus values of S-4 mixtures were greater than SMA mixtures. The same result was also found using the predictive equation. This indicates that the S-4 mixtures tested in this project were stiffer than SMA mixtures.
5. All of the SMA mixes prepared using the recent ODOT SMA specification had similar mean dynamic modulus values. Hence, one average dynamic modulus value at each temperature was used to represent the dynamic modulus values of all SMA mixes tested in this project.

A sensitivity analysis was done on MEPDG software version 1.1. Based on the result of the analysis, the following conclusions are made

- Changing season of traffic opening has no impact on prediction of distresses.
- Traffic impacts prediction of distresses significantly.
- Depth of water table only impacted the prediction of alligator cracking.
- Aggregate base thickness has a considerable impact on the prediction of all types of distresses.
- Changing the stiffness of asphalt binder significantly changes the predicted distress values.
- Changing the aggregate base M_R value impacts the prediction of all types of distresses except total pavement rutting.
- Subgrade M_R Value impacts all types of distresses.

The predicted distresses of S-4 and SMA mixtures were compared to investigate predicted performances. The following conclusions were drawn from the comparison:

- Pavements with S-4 surface course mixes exhibit less distresses than pavements with SMA surface course mixes at both level 1 and level 3 input levels.
- Pavements with lower subgrade modulus values have less terminal IRI, alligator cracking and total pavement rutting values and higher longitudinal cracking and AC rutting values than pavements with higher subgrade values.
- Distresses predicted at input level 3 are higher than distresses predicted at input level 1, except for AC rutting on SMA mixtures. This indicates that designing pavements using input level 3 of the MEPDG over predicts distresses for Oklahoma mixes.

RECOMMENDATIONS

1. Default A and VTS values given in the MEPDG were used to predict dynamic modulus in this project. Trial was made to use the experimentally measured G* and δ values (shown in table 31) to compute A and VTS values using linear regression. However, the modulus values predicted using these values were unreasonable. Further studies should be conducted to estimate A and VTS values which can be used to predict reasonable dynamic modulus values for all types of asphalt cements used in Oklahoma.
2. Viscocity and G* based Witzack equations are the two predictive equations included in the MEPDG. Only the viscocity based equation was used in this project. It is recommended to use the G* based equation and compare the result with the viscocity based equation and see which one is more conservative.
3. The result of this research project showed that S-4 mixtures perform better than SMA mixtures. This conclusion was drawn solely based on the output of the MEPDG software. The result of the MEPDG contradicts all literatures reviewed in this project. Literature showed that SMA mixtures perform better than conventional mixes in every aspect. It is recommended that the MEPDG be calibrated to predict distresses of SMA.
4. It is recommended to perform other tests (APA rutting, Flow number, creep tests …) on SMA and S-4 mixes to further investigate mix performance.
5. The MEPDG results with SMA contradict past performance as indicated in the literature. Therefore, it is not recommended to use the MEPDG until the program can be recalibrated or the results verified.
6. Based on the results of this study, the values shown in tables 34 and 35 are recommended for use in MEPDG for Oklahoma SMA mixtures.

TABLE 44 Recommended Mix Properties of SMA for use in MEPDG

Mix Property	Recommended Values
% of aggregate retained 3/4 " sieve	0
% of aggregate retained 3/8 "sieve	29
% of aggregate retained No. 4 sieve	70
% of aggregate passing No. 200 sieve	10.2
Va (%)	5.5
Vbeff (%)	13.5

TABLE 45 Recommended E* Values of SMA for use in MEPDG

Temperature	Frequency	Dynamic Modulus (psi)		
(°C)	(Hz)	Measured	Predicted	Recommended
-10	25	2,516,078	2,590,405	2,550,000
	10	2,401,956	2,452,607	2,427,000
	5	2,305,872	2,343,346	2,324,500
	1	2,050,896	2,075,302	2,063,000
	0.5	1,928,469	1,954,931	1,941,500
	0.1	1,622,148	1,668,902	1,645,500
4.4	25	1,771,289	1,666,446	1,718,500
	10	1,592,649	1,502,589	1,547,500
	5	1,454,594	1,379,793	1,417,000
	1	1,136,742	1,104,230	1,120,000
	0.5	1,006,235	991,916	999,000
	0.1	732,225	752,022	742,000
21.1	25	830,979	807,618	819,000
	10	686,253	681,428	683,500
	5	588,766	594,037	591,000
	1	404,216	419,610	411,500
	0.5	342,083	356,886	349,000
	0.1	232,533	238,635	235,500
37.8	25	305,368	342,821	324,000
	10	245,139	273,218	259,000
	5	208,325	228,386	218,000
	1	145,790	147,327	146,500
	0.5	126,531	120,996	123,500
	0.1	94,215	75,573	84,500
54.4	25	125,984	145,308	135,500
	10	105,904	111,869	108,500
	5	93,870	91,404	92,500
	1	73,587	56,666	65,000
	0.5	67,295	46,040	56,500
	0.1	56,526	28,592	42,500

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VITA

Robel Alemayheu Gibbe

Candidate for the Degree of

Master of Science

Thesis: DETRMINATION OF INPUT DATA FOR OKLAHOMA SMA MIXTURES FOR USE IN THE MEPDG

Major Field: Civil Engineering

Biographical:

Education:

Completed the requirements for the Master of Science in Civil Engineering at Oklahoma State University, Stillwater, Oklahoma in July, 2011.

Completed the requirements for the Bachelor of Science in Civil Engineering at Bahirdar University, Bahirdar, Ethiopia in 2007.

Experience:

Graduate Research & Teaching Assistant
Oklahoma State University, October 2009 – July 2011

- Determination of input data for Oklahoma SMA mixtures for use in the MEPDG.
- Assisting the under graduate students on laboratory testing of aggregate, concrete and asphalt materials.

Contract and Highway Engineer, Full Time
Core Consulting Engineers, October 2007 – July 2009

- Designing and supervision of highway roads
- Analyzing claims of contractors

Professional Memberships:

- Student member, American Association of Civil Engineers
- Graduate member, Ethiopian Construction Professionals
- THE HONOR Society of PHI KAPPA PHI

Name: Robel A. Gibbe Date of Degree: July, 2011

Institution: Oklahoma State University Location: Stillwater, Oklahoma

Title of Study: DETERMINATION OF INPUT DATA FOR OKLAHOMA SMA MIXTURES FOR USE IN THE MEPDG

Pages in Study: 91 Candidate for the Degree of Master of Science

Major Field: Civil Engineering

Scope and Method of Study:

The objective of this research project was to compare the performance of Oklahoma SMA mixes to conventional superpave (S-4) mixes and to develop input data of SMA for use in the new mechanistic-empirical pavement design guide (MEPDG).

To accomplish this objective, SMA and S-4 mixtures were collected from different parts of Oklahoma and were tested for dynamic modulus. These dynamic modulus values were then used in the MEPDG software to compare the performance of Oklahoma S-4 and SMA mixes.

Findings and Conclusions:

- Test temperature had a significant effect on measured dynamic modulus values.
- Stiffness of both S-4 and SMA mixtures decrease as temperature increases.
- Mix type had a significant effect on measured dynamic modulus values.
- Both the measured and predicted dynamic modulus values of S-4 mixtures were greater than SMA mixtures. This indicates that the S-4 mixtures tested in this project were stiffer than SMA mixtures.
- The output of the MEPDG software showed lower distress values for the S-4 mixtures than SMA mixtures. This result contradicts all the literatures reviewed in this research project. Literature showed that SMA mixtures perform better than conventional mixes in every aspect.
- The MEPDG should not be used to design pavements constructed with SMA until the program can be recalibrated or the results verified.

ADVISER’S APPROVAL: Dr Stephen A. Cross