NCAT Report 09-03

RECALIBRATION OF THE ASPHALT LAYER COEFFICIENT

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August 2009

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ABSTRACT
The Alabama Department of Transportation (ALDOT) currently uses the 1993 DARWin version of the American Association of State Highway and Transportation Officials (AASHTO) Guide for the Design of Pavement Structures when designing flexible pavements. The AASHTO design methodology is based on information obtained at the AASHO Road Test, which was performed from 1958 to 1960 near Ottowa, Illinois. This road test provided an empirical correlation between pavement thickness and traffic loadings. However, the results stemming from the road test are limited to the pavement materials utilized, applied traffic and the climate of Illinois. Using the results of the AASHO Road Test, a flexible pavement design equation was developed and introduced in the 1986 AASHTO Guide for Design of Pavement Structures that includes inputs of soil resilient modulus, traffic, structural capacity (structural number), reliability, variability, and ride quality (change in serviceability). The structural number is calculated using the layer thicknesses, material drainage properties, and layer coefficients, which are used to express the relative strength contribution of each pavement layer to the overall pavement structure. In this study, these inputs were analyzed to determine the relative influence of each on the resulting hot mix asphalt (HMA) thickness. It was found that the HMA layer coefficient, resilient modulus, and traffic inputs are by far the most influential parameters. Since the soil modulus and traffic are generally given parameters for a particular design, it was decided that the layer coefficient be recalibrated to provide the greatest potential savings in HMA thickness. Furthermore, the layer coefficient has not been updated to account for advancements made in construction methods, gradation requirements, and paving materials since the AASHTO Road Test, and therefore should be reanalyzed.

The recalibration was performed using traffic and performance data collected for the structural sections of the 2003 and 2006 National Center for Asphalt Technology (NCAT) Test Track cycles. These data were used in conjunction with traffic equations developed from the AASHO Road Test as well as the AASHTO flexible pavement design equation to find both the calculated and predicted equivalent single axle loads (ESALs). Once these values were found for each section, a least squares regression was performed to determine new HMA layer coefficients. The resulting average layer coefficient was 0.54 for all sections, with a standard deviation of 0.08. Using this parameter instead of the AASHTO recommended coefficient of 0.44 results in an approximate HMA thickness savings of 18%. From these results, it is recommended that ALDOT adopt this value as their new layer coefficient for flexible pavement designs.

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CHAPTER 1 - INTRODUCTION

BACKGROUND

The Alabama Department of Transportation (ALDOT) currently uses the 1993 DARWin version of the American Association of State Highway and Transportation Officials (AASHTO) Guide for the Design of Pavement Structures when designing flexible pavements. The AASHTO design method is based on information obtained at the AASHO Road Test, which was performed from 1958 to 1960 near Ottowa, Illinois. This road test provided an empirical correlation between pavement thickness and traffic loadings. Equations were developed from the road test to determine the pavement thickness required for a particular design, and although they have been modified somewhat, are still in use today. Through the road test research and the developed equations, the structural number (initially termed the thickness index) was introduced to define the overall structural capacity of a pavement cross section. The structural number for flexible pavement design is mathematically defined by the following equation (AASHTO, 1993):

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \qquad (1.1)$$

Where a_i is the empirically based layer coefficient that represents the structural capacity of the material used in the i^{th} layer, the *m* coefficients are used to describe the drainage properties of each layer, and the *D* terms are the thicknesses of each respective layer. The structural number can also represent the capacity of each individual layer in the pavement cross section. For example, the structural number for the HMA can be expressed as:

$$SN_1 = a_1 D_1 \qquad (1.2)$$

The same concept can be used to calculate the *SN* for other pavement layers. The structural numbers for each layer in the pavement cross section can be summed to equal the total structural number for the pavement as expressed in Equation 1.1. While the respective layer thicknesses and drainage conditions are relatively easy to quantify, the layer coefficients are not so straightforward. No direct method exists for establishing new layer coefficients as new HMA mix types are created, and they are dependent upon many different parameters including material stiffness, tensile strength, compressive strength, moisture conditions, and even the layer's position within the pavement cross section.

The recommended layer coefficient for dense graded HMA mixes (a_1) is 0.44, which comes from the results of the AASHO Road Test (HRB, 1962; AASHTO, 1993). ALDOT uses this value for designing flexible pavements, as do many other transportation agencies. This coefficient is based on the limited parameters used at the road test. The trucks had only bias-ply tires with pressures of around 70 psi. No triple or quad axles were utilized, and no super singles tires were used. Additionally, a maximum of 2 million equivalent single axle loads (ESALs) were applied over the course of the road test. Only a limited number of cross sections were tested, one soil type was used as the subgrade, one type of gravel was used as the base material, and one type of HMA was used. The thickest HMA pavement was 6 inches. The entire road test only lasted two years. All the results from the road test are a product of the climate in northern Illinois. Furthermore, no Superpave mixes, open-graded friction courses, stone-matrix asphalts, or other advanced paving materials were available at the time. These new mixes provide better performance against rutting, fatigue cracking, and other distresses, and therefore it seems logical that these improved asphalt mixes would have a higher structural capacity (*SN*). Since there are only two inputs needed to calculate the structural number of the HMA as shown in Equation 1.2, this implies that the layer coefficient should be higher for these new mixes. If a new, and presumably higher, layer coefficient were established for these improved mixes, then the required HMA thicknesses would decrease. Consequently, this would result in lower material and construction costs and an overall more efficient pavement structure.

Due to this reasoning, it was concluded that there is a need to analyze the current recommended value for the HMA layer coefficient of 0.44. It should be updated to reflect the changes and improvements that have occurred in HMA materials and construction over the last 50 years. In addition, while a savings in HMA thickness is expected, the magnitude of that savings is uncertain; therefore, the sensitivity of the layer coefficient on the resulting HMA thickness should be analyzed as well.

OBJECTIVES

There were two primary objectives of this investigation. One was to determine the sensitivity of the layer coefficient (a_1) on the resulting HMA thickness using the 1993 AASHTO method for flexible pavement design. The second objective was to recalibrate the layer coefficient for newer mixes, and compare that value to the currently used layer coefficient of 0.44.

SCOPE

The sensitivity analysis was performed using the 1993 AASHTO Design Guide flexible pavement design equation to create a large database of resulting HMA thicknesses from changing individual inputs to the equation. This database allowed easy viewing of the trends and relative influences of each of the inputs, including the HMA layer coefficient.

The primary objective of this investigation was to recalibrate the layer coefficients for current asphalt mixes. Data from the structural sections of the 2003 and 2006 National Center for Asphalt Technology (NCAT) Test Track cycles were utilized to achieve this objective. The Test Track provides unique opportunities for such research: accelerated traffic is applied to create accelerated levels of pavement damage in a relatively short period of time, which was very useful for this investigation. Detailed traffic and performance data collected over the course of the test cycles were used to perform a least squares regression to arrive at new layer coefficients for each test section and recommendations were made for the use of a new HMA layer coefficient.

ORGANIZATION OF REPORT

A literature review is provided in Chapter 2 that further describes the AASHTO design method, ALDOT's pavement design procedure, and the origins of the layer coefficient.

Additionally, Chapter 2 contains information on past research efforts regarding the HMA layer coefficient. Chapter 3 provides details on the sensitivity of the flexible pavement design equation to the required inputs, and ranks their influence on the resulting HMA thickness. Chapter 4 provides a description of the test facility and the cross sections used in this study, and details the recalibration procedure and the resulting layer coefficients for each test section. Trends in layer coefficients are discussed in Chapter 4, as well as the impact of changing the layer coefficient. Chapter 5 presents conclusions and recommendations.

CHAPTER TWO - LITERATURE REVIEW

INTRODUCTION

The current flexible pavement design methodology used by the Alabama Department of Transportation (ALDOT) is derived from the results of the American Association of State Highway Officials (AASHO) Road Test conducted in the late 1950's. A basic nomograph and equation were developed from the road test results that includes inputs of soil modulus, traffic, ride quality (serviceability), reliability, variability, and the capacity of the pavement structure (structural number). Using the aforementioned flexible pavement design equation or nomograph in conjunction with these inputs, the designer arrives at a design thickness for the hot mix asphalt (HMA).

To find the structural number of the entire pavement cross section or its respective layers, the layer thicknesses, drainage properties, and layer coefficients are needed. Layer coefficients, also called structural coefficients, are used to quantify a particular layer's relative ability to function and perform within the pavement cross section. For dense graded HMA mixes, the AASHTO recommended layer coefficient is 0.44, and this value is used by ALDOT for all plant mix designs (AASHTO, 1993; ALDOT, 2004). Recommended layer coefficients for other materials are listed in Chapter 2. This value is based on results from the very limited test conditions of the AASHO Road Test, and has not been recalibrated for newer pavement types and other variables.

AASHO ROAD TEST

Overview and Limitations

The AASHO Road Test was conducted from 1958 to 1960 near Ottowa, Illinois. The primary purpose of the road test was to determine the effect of various axle loadings on pavement behavior. Both flexible and rigid pavements were tested in the study, along with several short span bridges. Six two-lane test loops were created for trafficking, including four large loops and two small loops. Hot mix asphalt (HMA) and base thicknesses were varied within each test loop to determine the effect of axle loadings on different pavement cross sections. Individual lanes were subjected to repeated loadings by a specific type and weight of vehicle. Single and tandem axle vehicles were used for trafficking. Bias-ply tires were used with pressures of approximately 70 psi. Only 2 million equivalent single axle loads (ESALs) were applied over the course of the test. The test vehicles ranged in gross weight from 2,000 lb to 48,000 lb, and are shown in Figure 2.1. The improved paving materials that are used today such as Superpave mixes, stone matrix asphalts, and open graded friction courses were not available during the road test. Within the pavement cross section, only one type of HMA, granular base material, and subgrade soil were used. The thickest HMA pavement was 6 inches. All results from the road test are a product of the climate of northern Illinois within a two-year period (HRB, 1962).

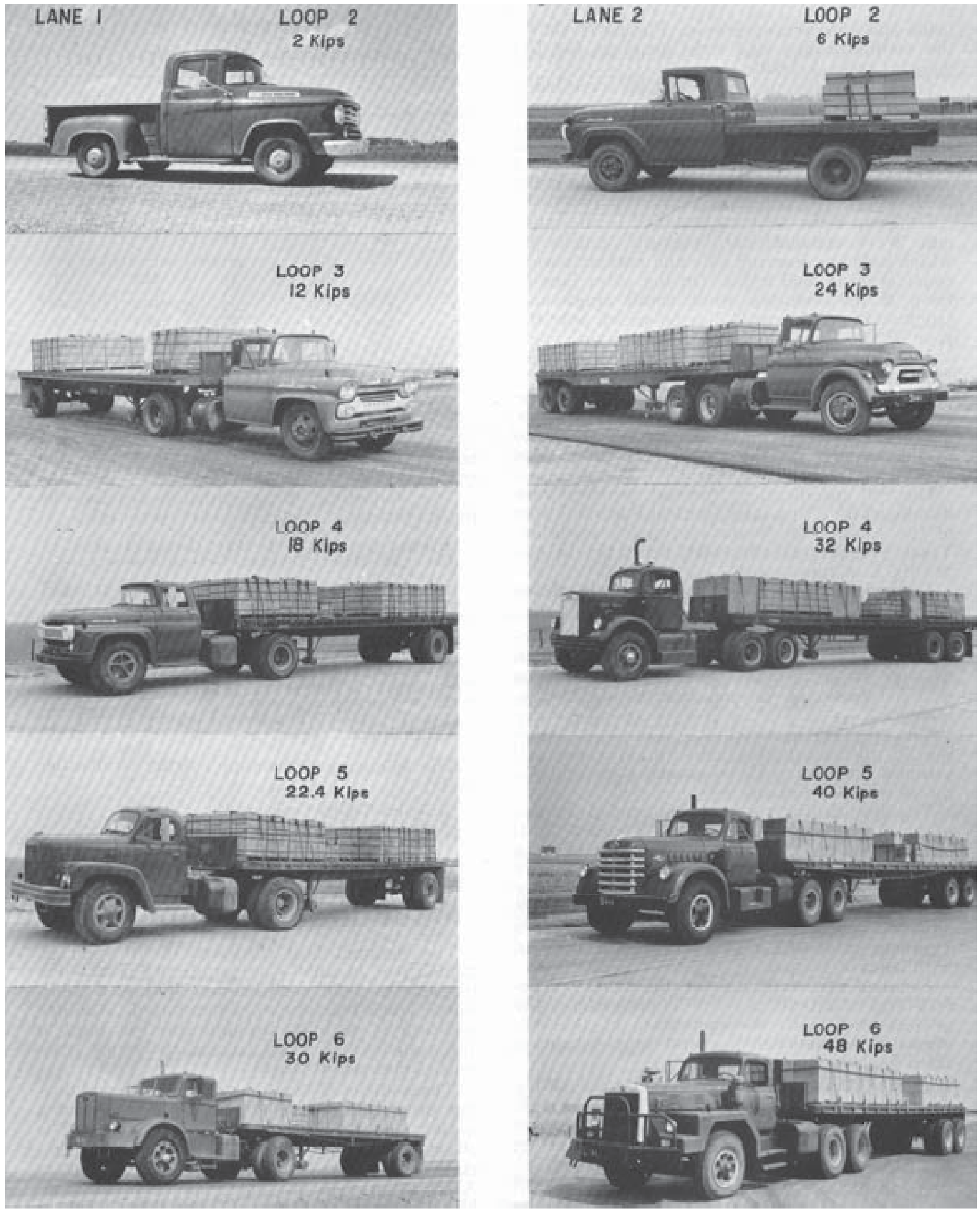

Figure 2. 1 AASHO Road Test Vehicles (Smith et al., 2004).

Results

The results of the AASHO Road Test were used to develop the first pavement design guide, known as the *AASHO Interim Guide for the Design of Rigid and Flexible Pavements*. This design guide was issued in 1961, and had major updates in 1972, 1986, and 1993. The 1993 AASHTO Design Guide is essentially the same as the 1986 Design

Guide for the design of new flexible pavements, and is still used today by many transportation agencies, including ALDOT.

The primary objective for the AASHO Road Test was to determine the relationship between pavement loading and deterioration. Using replicate cross sections in different test loops (that were loaded with different axle weights), researchers at the road test were able to view the differences in pavement distresses such as rutting, cracking, and slope variance, that were caused by increasing axle loads. The relationship found was an approximate fourth power relationship: a unit increase in axle weight causes increased damage to the fourth power. To put this relationship into context, if the axle weight is doubled, it causes approximately sixteen times more damage to the pavement. Figure 2.2 illustrates the general relationship between loading and damage found at the road test.

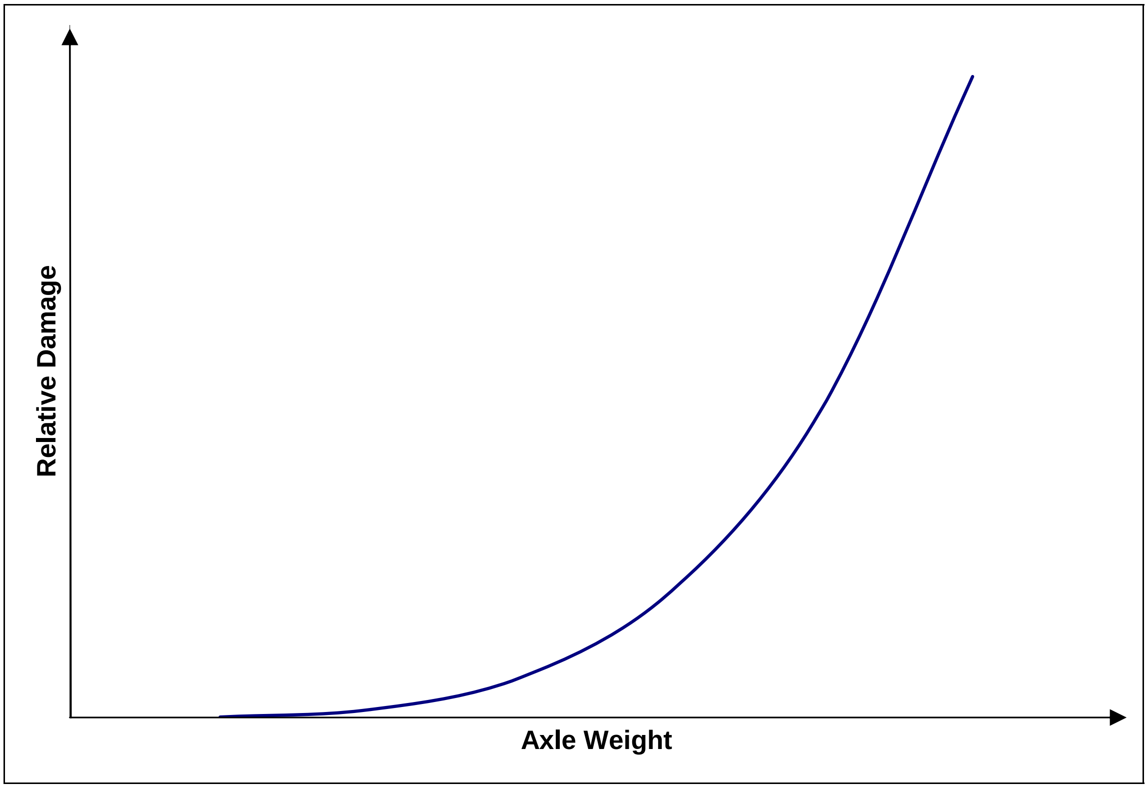

Figure 2. 2 4th Power Relationship between Axle Weight and Pavement Damage (after HRB, 1962).

Origins of the Layer Coefficient

It was from this fourth-power relationship that the concept of pavement capacity was derived. Knowing the expected traffic that will load a pavement and its associated damage, the pavement must have a certain capacity to withstand said traffic and resulting damage. The researchers at the AASHO Road Test developed an equation termed the "thickness index" (similar to the structural number), which can be mathematically expressed as (HRB, 1962):

$$D = a_1 D_1 + a_2 D_2 + a_3 D_3 \quad (2.1)$$

Where the *a* terms are the layer coefficients, the *D* terms are the thicknesses of each layer, and the subscripts 1, 2 and 3 represent the HMA, base, and subbase pavement layers, respectively. If the layer coefficients are all equal to one, then the thickness index is simply the total thickness of the pavement cross section. However, for the AASHO Road Test investigation, these parameters were allowed to vary so that each pavement layer could have a certain capacity per unit thickness (HRB, 1962). These parameters were varied because, for example, a four inch HMA layer contributes considerably more to pavement capacity than a four inch subbase layer. From this concept stems the general definition of the layer coefficient: an empirical relationship between the layer thickness and structural number that expresses a layer's relative contribution to the performance of the pavement structure (AASHTO, 1993). The layer coefficient depends upon many variables, including the resilient modulus, layer thickness, underlying support, position in the pavement structure, and stress state (AASHTO, 1993; Pologruto, 2001).

Using the thickness index equation, several analyses of variance were conducted on the data from the AASHO Road Test to determine the layer coefficients for each pavement sublayer. Table 2.1 shows the HMA layer coefficients (a_1) found from those analyses, the number of test sections in each loop used to find those coefficients, and the model R^2 values. Loop 1 is not included in the table because it was never trafficked; it was used to evaluate environmental impacts on pavements.

Table 2. 1 HMA Layer Coefficients from AASHO Road Test (after HRB, 1962)

Loop	Layer Coefficient (a_1)	Test Sections	R^2
2	0.83	44	0.80
3	0.44	60	0.83
4	0.44	60	0.90
5	0.47	60	0.92
6	0.33	60	0.81

Based upon this table and other details from the results, the value of 0.44 was recommended for use as the HMA layer coefficient (HRB, 1962). According to the authors, a weighted average was taken to arrive at this recommended value. However, it is uncertain how these values were weighted.

In 1972, a relationship was created that linked the layer coefficient to the elastic modulus (*E*) of the HMA at 70°F, and is shown in Figure 2.3. This graph can only be used if the modulus is between 110,000 and 450,000 psi. The AASHO Road Test recommended layer coefficient of 0.44 corresponds to a modulus of 450,000 psi (AASHTO, 1993). In 2006, Priest and Timm found a relationship relating temperature and stiffness for all the structural sections in the 2003 Test Track cycle. Using their relationship, the average HMA modulus was calculated as 811,115 psi. If the curve in Figure 2.3 were extrapolated out to this modulus value, the resulting layer coefficient would equal 0.54.

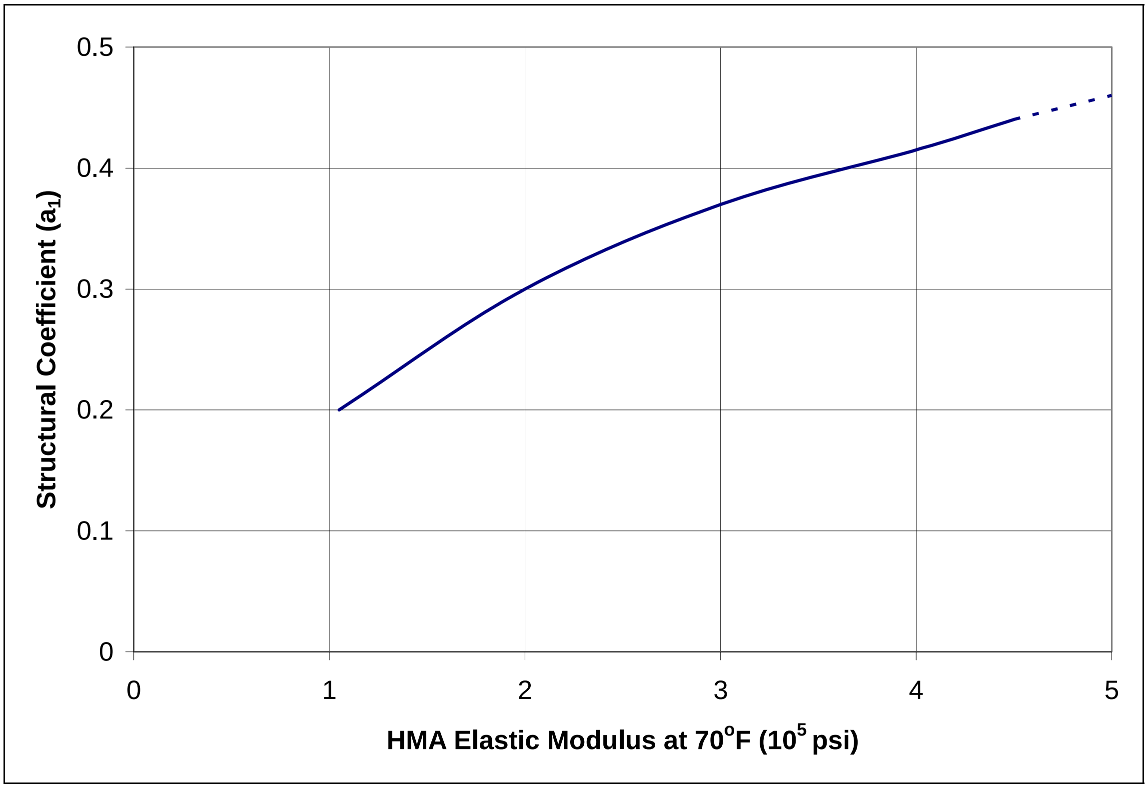

Figure 2. 3 Determining a_1 based on HMA Modulus (after AASHTO, 1993).

AASHTO TRAFFIC EQUATIONS

Using the fourth-power relationship found at the AASHO Road Test, equations were derived to relate axle loading to pavement damage. Replicate cross sections were constructed in different test loops to apply varying repeated axle loads on the same pavement structure. This allowed the researchers at the road test to view the damage caused by heavier axles, and create mathematical relationships based upon that damage.

The resulting pavement damage was quantified using Equivalent Axle Load Factors (EALFs), which are used to find the number of ESALs. An EALF is used to describe the damage done by an axle per pass relative to the damage done by a standard axle per pass. This standard axle is typically an 18-kip single axle, as defined in the road test. From the AASHO Road Test results, the EALF can be expressed in the following form according to Huang (2004):

$$EALF = \frac{W_{t18}}{W_{tx}} \tag{2.2}$$

Where W_{tx} is the number of x axle load applications at time t, and W_{t18} is the number of 18 kip axle load applications at time t.

The EALFs for each axle load group are used to find the total damage done during the design period, which is defined in terms of passes of the standard axle load, as shown in the following equation (Huang, 2004):

$$ESAL = \sum_{i=1}^{m} EALF_i n_i \tag{2.3}$$

Where m is the number of axle load groups, $EALF_i$ is the EALF for the ith axle load group, and n_i is the number of passes of the ith axle load group during the design period.

These basic traffic equations can be used in conjunction with the following equations that were also developed from the AASHO Road Test to characterize traffic for a given flexible pavement design (Huang, 2004):

$$G_t = \log\left[\frac{4.2 - p_t}{4.2 - 1.5}\right] \tag{2.4a}$$

$$\beta_x = 0.40 + \frac{0.081 \cdot (L_x + L_2)^{3.23}}{(SN + 1)^{5.19} \cdot L_2^{\ 3.23}} \tag{2.4b}$$

$$\log\left[\frac{W_{tx}}{W_{t18}}\right] = 6.1252 - 4.79\log(L_x + L_2) + 4.33\log L_2 + \frac{G_t}{\beta_x} - \frac{G_t}{\beta_{18}} \tag{2.4c}$$

Where G_t is the log of the ratio of loss of serviceability (ride quality) at some time t to the potential loss of serviceability at terminal serviceability (p_t) = 1.5. The initial serviceability is assumed to be 4.2; this value was typical for flexible pavements at the AASHO Road Test, and is used as the initial value for ALDOT pavement designs as well. β_x is a function of design and load variables, L_x is the axle group load in kips, L_2 is the axle code (1 for single, 2 for tandem and 3 for tridem), SN is the structural number, W_{tx} is the number of x axle load applications at time t, W_{t18} is the number of 18 kip axle load applications at time t, and β_{18} is the value of β_x when L_x is equal to 18 and L_2 is equal to one.

AASHTO FLEXIBLE PAVEMENT DESIGN

Fundamental Equations

The 1993 AASHTO Design Guide is the current standard used for designing flexible pavements for many transportation agencies. In the AASHTO design methodology, the subgrade resilient modulus (M_R), applied ESALs (W_{18}), reliability (with its associated normal deviate, Z_R), variability (S_o), change in serviceability (ΔPSI), and structural number (SN) are used in the nomograph in Figure 2.4 and the following corresponding equation to design flexible pavements (AASHTO, 1993):

$$\log W_{18} = Z_R S_0 + 9.36\log(SN+1) - 0.20 + \frac{\log\left[\frac{\Delta PSI}{4.2 - 1.5}\right]}{0.4 + \frac{1094}{(SN+1)^{5.19}}} + 2.32\log M_R - 8.07 \tag{2.5}$$

The reliability level selected is typically based upon the predicted traffic level. A low volume road (defined as less than 500 ESALs per day traveling in both directions) requires 85% reliability, medium volume (between 500 and 1750 ESALs per day) requires and high volume (greater than 1750 ESALs per day) requires 95% (Holman, 1990). The reliability level selected corresponds to a standard normal deviate, Z_R, which is calculated using the following equation (Huang, 2004):

$$Z_R = \frac{\log W_{18} - \log W_{t18}}{S_0} \tag{2.6}$$

Where W_{t18} is the number of single-axle load applications to cause the reduction of serviceability to the terminal level (p_t). The standard normal deviates needed for design are typically tabulated and do not need to be calculated for individual designs.

The standard deviation, S_o, is typically assumed to be 0.49 for flexible pavements based upon previous research (AASHTO, 1993).

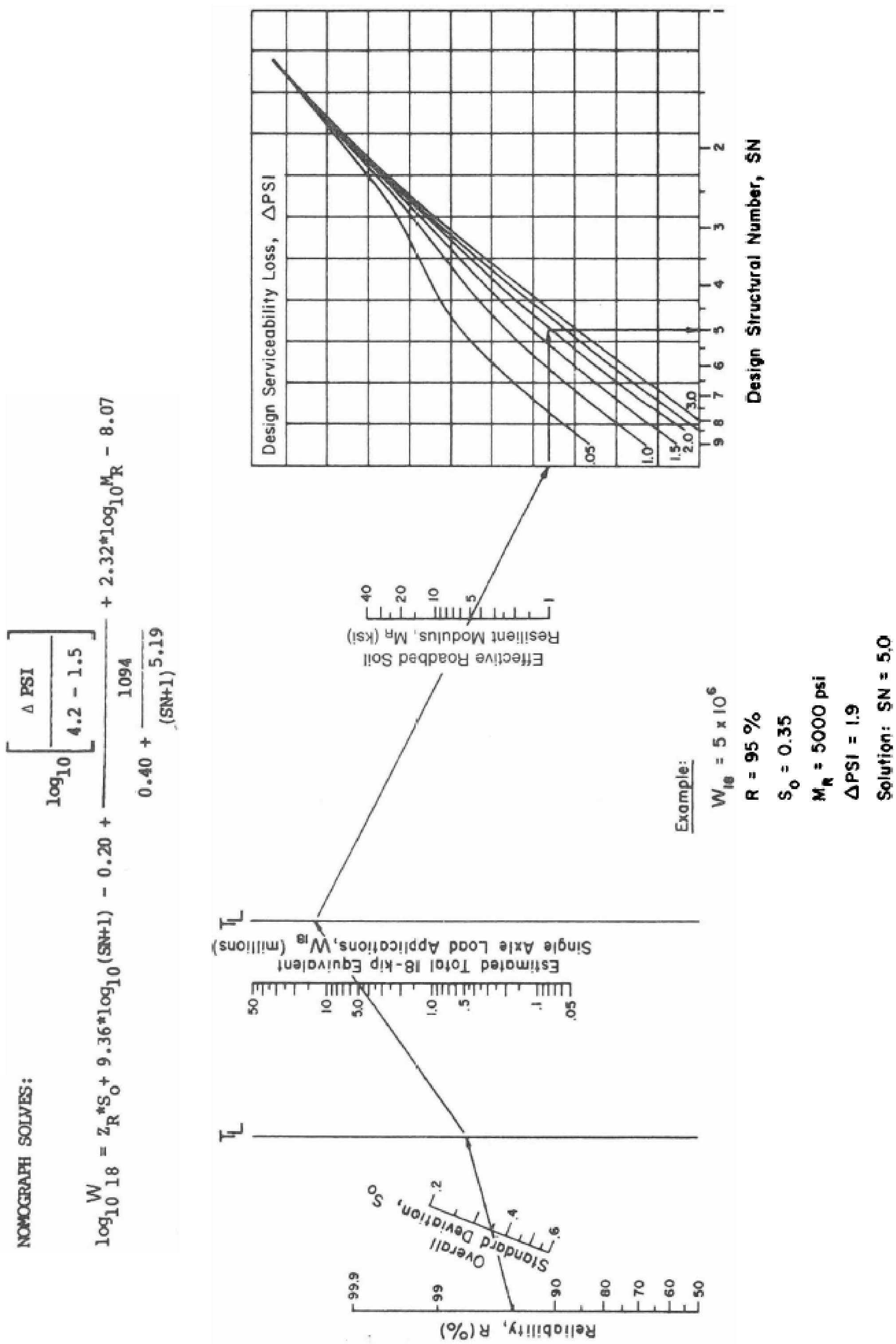

Figure 2. 4 AASHTO Flexible Design Nomograph (AASHTO, 1993).

The M_R of the subgrade soil seen in the equation has been adjusted to account for seasonal changes, and is termed the effective M_R. The AASHTO Design Guide (1993) recommends taking an annual average for backcalculated resilient modulus data and dividing it by three to obtain the effective modulus. This is done to account for differences in testing procedures from the road test and the current testing method using the falling weight deflectometer (FWD). At the road test, screw driven laboratory devices were used to determine the soil stiffness. Due to the slow response time of such devices, the apparent stiffness of the soil was very low (around 3,000 psi). With the much more rapid loading of FWD testing, the moduli are typically around three times higher, and therefore are divided by three to arrive at similar numbers to those used at the road test.

PSI and IRI

The change in serviceability (ΔPSI) seen in Equation 2.5 is the difference between the initial serviceability rating of the pavement when opened to traffic and the terminal serviceability that the pavement will reach before rehabilitation, resurfacing or reconstruction is required. The present serviceability index (PSI), also known as the present serviceability rating (PSR), is a subjective measure by the road user of the ride quality, ranging from zero (impassible) to five (perfect ride). Studies conducted at the AASHO Road Test found that for a newly constructed flexible pavement, the initial serviceability (p_o) was approximately 4.2 (AASHTO, 1993). For the selection of a terminal serviceability (p_t), the AASHTO Design Guide recommends selection based upon the same traffic levels used for reliability selection: for low traffic, 2.5, for medium traffic, 3.0, and for high traffic, 3.5. To demonstrate the subjectivity of the measurement, studies from the AASHO Road Test found that an average of 12% of road users believe that a pavement receiving a rating of 3.0 is unacceptable for driving while 55% of road users believe that 2.5 is unacceptable (AASHTO, 1993).

Due to the subjective nature of serviceability measurements, most current road roughness measurements are now standardized to the international roughness index (IRI). This index provides a measure of the longitudinal wavelengths in the pavement profile in inches per mile or meters per kilometer. These measurements are taken by inertial profilers, and can be closely replicated from machine to machine (Sayers and Karamihas, 1998). This removes the subjectivity of assessing the ride quality, and therefore is a more accurate measurement. However, since the AASHTO flexible pavement design procedure still requires serviceability levels as inputs, a conversion must be made from IRI to PSI.

In 1990, an in-house study was conducted by Holman at ALDOT to relate IRI to PSI. The derived relationship can be expressed as:

$$PSI = 5e^{(-0.0051118 \cdot IRI - 0.0016027)} \tag{2.7}$$

Al-Omari and Darter (1994) studied the relationships between serviceability, IRI, and pavement distresses. Plots were created that relate IRI to serviceability (PSR) as shown in Figure 2.5. The data in this graph comes from the states of Louisiana, Michigan, New Jersey, New Mexico, and Indiana, and include flexible and rigid pavements, as well as combinations of the two. The equation shown in the figure is for all pavement types and for units of mm/m, and is recommended by the National Highway Institute (NHI) for use if no state-specific data is available (Holman, 2003). Based upon only the flexible pavement data from their study, Al-Omari and Darter developed an equation that converts IRI (in/mile) measurements to PSI:

$$PSI = 5e^{(-0.0038 \cdot IRI)} \tag{2.8}$$

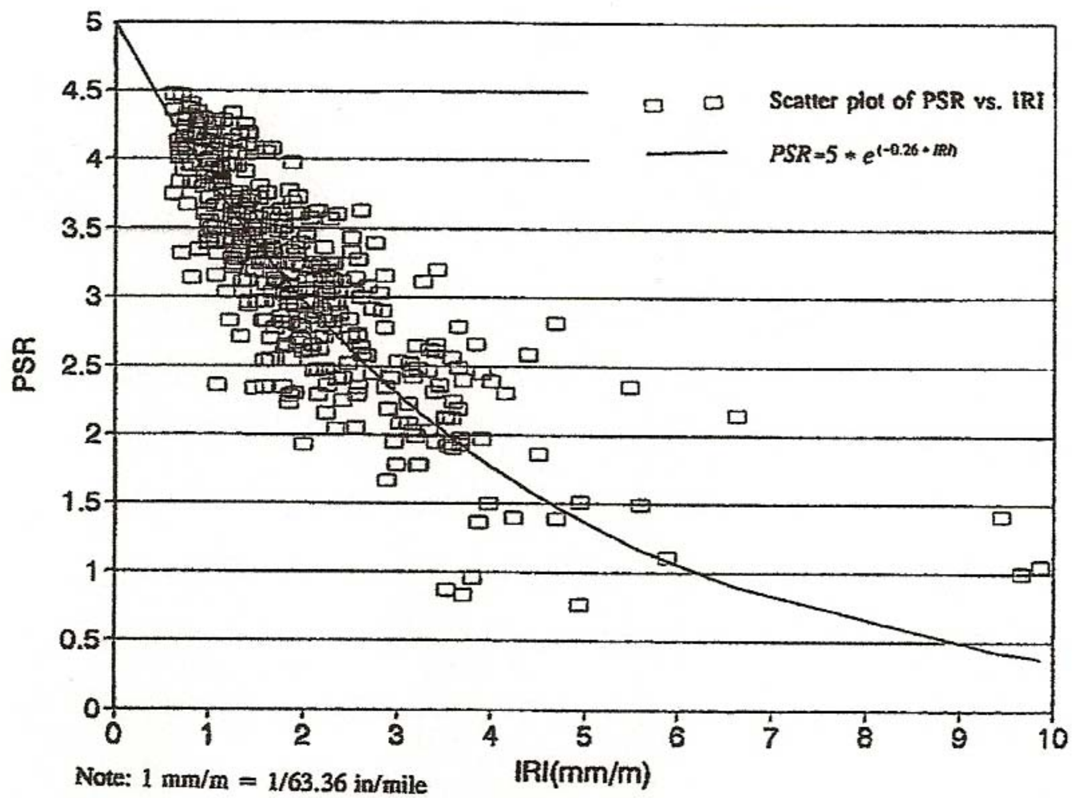

Figure 2. 5 Relationship between IRI and PSR for All Pavement Types (Al-Omari and Darter, 1994).

A study conducted in 1994 by Gulen et al. stated that the equation developed by Al-Omari and Darter was biased and not statistically correct because it was forced to pass through an IRI value of zero when PSR was equal to 5. The authors developed their own relationships relating IRI and PSI for the state of Indiana and recommended that the selection of a model be based upon the needs of the end user (Gulen et al., 1994).

In 1999, Hall and Munoz developed relationships for relating IRI and PSI for both asphalt and concrete pavements. They analyzed data from the AASHO Road Test that included parameters of slope variance (SV) and PSI, and then developed a correlation between SV and IRI for a broad range of road roughness levels. Their findings for flexible pavements can be expressed mathematically as:

$$PSI = 5 - 0.2397x^4 + 1.771x^3 - 1.4045x^2 - 1.5803x \tag{2.9a}$$

$$x = \log(1 + SV) \tag{2.9b}$$

$$SV = 2.2704(IRI)^2 \tag{2.9c}$$

Based upon the similarity of the Al-Omari/Darter and Holman equations, it was decided to focus on those relationships for this study. Since the equation developed by Al-Omari and Darter is the result of a much larger performance database and is recommended by the NHI, it was determined that their equation would be optimal for converting the IRI data to present serviceability values.

Structural Number

As seen in the nomograph in Figure 2.4, the designer arrives at a structural number for a given set of inputs. This structural number is used to find the design HMA thickness (D_1) for a given base and subbase thickness (D_2 and D_3, respectively) from the following relationship (AASHTO, 1993):

$$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3 \tag{2.10}$$

This equation is very similar to the thickness index presented in Equation 2.1. However, Equation 2.10 accounts for the drainage properties present in the base and subbase layers with the m_2 and m_3 coefficients, respectively.

While the structural number is found directly from the nomograph, it cannot be solved for directly using Equation 2.5. Therefore, an iterative procedure must be used to arrive at the appropriate structural number for a given design. A seed value for the SN must be input into the equation, and the resulting calculated ESALs must be compared to the design ESALs. From there, the SN is adjusted, and the process is repeated. However, it is not expected that the design ESALs will perfectly match the calculated ESALs due to the effect of taking the logarithm (base-10) of such large numbers. For example, the logarithm of 5 million equals 6.70, and the logarithm of 6 million equals 6.78. Due to this logarithm effect, there can be very similar answers to the $logW_{18}$ term using the iterative procedure, yet these answers can result in large differences between the calculated ESAL values (W_{18}).

Minimum Thickness

When designing flexible pavements, minimum thickness values should be used to prevent impractical or uneconomical designs. The 1993 AASHTO Design Guide recommends the minimum thicknesses shown in Table 2.2 for asphalt concrete and aggregate base layers based upon the traffic level (ESALs) of the design.

Table 2. 2 Minimum Thicknesses (after AASHTO, 1993)

Traffic (ESALs)	Asphalt Concrete (in)	Aggregate Base (in)
Less than 50,000	1.0 (or surface treatment)	4
50,001-150,000	2.0	4
150,001-500,000	2.5	4
500,001-2,000,000	3.0	6
2,000,001-7,000,000	3.5	6
Greater than 7,000,000	4.0	6

Sensitivity to Inputs
A search was conducted to find the relative influence of each of the inputs in the flexible pavement design equation on the resulting HMA thickness. Very little information was available for this topic. Therefore, it was concluded that research should be conducted to determine the sensitivity of these inputs.

Some information can be inferred from the design nomograph shown in Figure 2.4. For example, looking at the far left of the nomograph, the reliability scale changes as the value increases. This means that at lower reliability levels (50 to 70%), changing the design reliability by a single percentage has little effect on the resulting HMA thickness. However, at higher levels (90 to 99%), a unit change will have a more apparent affect on the HMA thickness. This trend of changing scales is also seen on the standard deviation, resilient modulus, and structural number, though the scales change at varying degrees. The traffic level does not demonstrate the same trend because it is on a log scale between the two turning lines. In other words, for traffic, it is the order of magnitude that is critical rather than relatively small differences within an order of magnitude.

ALDOT PRACTICE

To design flexible pavements in the state of Alabama, ALDOT currently follows the 390 Procedure (ALDOT, 2004). This procedure provides guidance for conducting pavement designs according to the 1993 AASHTO Design Guide, and includes additional information for materials testing and obtaining traffic information. The end result of the 390 procedure is a “materials report” that contains traffic data, materials test results, and printouts detailing the pavement structural design from the DARWin™ software, which is the embodiment of the 1993 AASHTO Design Guide. The traffic data and materials test results are inputs to the DARWin software, and the output obtained from the software is the pavement structural design.

ALDOT also has another pavement design guidance document, *Guidelines for Flexible Pavement Design in Alabama* that describes how to determine each of the inputs for the flexible pavement design equation to be entered into the DARWin software. Further details can be obtained from the original document (Holman, 1990).

The current ALDOT flexible pavement design procedure, employed through DARWin, utilizes Equation 2.5 and the corresponding nomograph shown in Figure 2.4. ALDOT currently uses default recommended values for the layer coefficients, change in serviceability, reliability, and variability inputs (Holman, 1990). These values are summarized in Table 2.2. The remaining inputs of drainage coefficients, traffic, and subgrade resilient modulus, are calculated on a project-by-project basis. The traffic ranges used for reliability and serviceability are the same as those discussed previously.

Table 2. 3 Recommended Design Values (after Holman, 1990).

Input	Recommended Values
*HMA layer coefficient (a_1)**	
414 and 416 mixes (plant mix)	0.44
411 mixes	0.30
*Base layer coefficient (a_2)**	
Granular base	0.14
Bituminous treated base	0.30
Cement or lime treated base	0.23
*Subbase layer coefficient (a_3)**	
Granular subbase	0.11
Terminal Serviceability (p_t)	
Low Traffic	2.5
Medium Traffic	3.0
High Traffic	3.5
Reliability (R)	
Low traffic	85%
Medium traffic	90%
High traffic	95%
Variability (S_o)	
Flexible pavements	0.49

* Note: Layer coefficients for other materials can be found in the reference cited.

The drainage coefficients (m_i) are calculated based upon the percent passing the number 200 sieve (P_{200}) of the material in question and the average annual rainfall in inches (AAR) for the project location, as expressed by the following equations:

$$S_i = 1.2 - 0.6 \cdot (AAR)/100 \tag{2.11a}$$

$$D_q = 1.2 - 0.6 \cdot (P_{200})/100 \tag{2.11b}$$

$$m_i = S_i \cdot D_q \tag{2.11c}$$

Where S_i is the level of saturation and D_q is used to describe the drainage quality.

The design traffic (ESALs) is calculated for each project from the average annual daily traffic and percent trucks for the design. To quantify the traffic, truck volumes are calculated at various locations within the project length and an average is computed. The truck volume from the location closest to, but just over, the average value is used as the design value. This design traffic value is then multiplied by several factors (365 days/year, Lane Distribution, Directional Distribution, Growth Factor, EALF) to arrive at the design ESALs (Holman, 1990).

The ALDOT procedure (Holman, 1990) has a provision for estimating subgrade soil modulus (M_R) from California Bearing Ratio (CBR) testing according to the following equation:

$$M_R = 10^{(0.851 \cdot \log CBR + 2.971)} \tag{2.12}$$

ALDOT assumes this value to remain the same throughout the year unless there are data to the contrary (Holman, 1990). The CBR value is found from the soil in the saturated condition, and therefore is expected to be the worst case scenario for the soil subgrade.

In addition to using CBR data to estimate the resilient modulus, ALDOT has been conducting extensive triaxial resilient modulus tests (AASHTO T307) of their subgrade materials for the past seven years. Procedure 390 states that the design M_R for soils classified as A-1, A-3, A-2-4, and A-2-5 shall be the average M_R values generated by AASHTO T307 at a confining pressure of 4 psi and optimum moisture content. For soil classes A-6 or A-7 (A-7-5 or A-7-6), 2 psi confining pressure is used, and samples are compacted on the wet side of optimum moisture to generate more conservative design soil moduli. For all other soil classes, the design M_R used is the average M_R value generated at 2 psi confining pressure and optimum moisture content (ALDOT, 2004).

PAST CALIBRATION EFFORTS

Many studies have been conducted to determine layer coefficients for new materials or for materials that were not used at the AASHO Road Test. However, little change has occurred in the actual coefficients used for design due to the complex nature of this parameter. As mentioned previously, the layer coefficient is dependent upon several factors, including resilient modulus, layer thickness, underlying support, position in the pavement structure, and stress state (AASHTO, 1993; Pologruto, 2001). Based upon these facts, AASHTO urges agencies to determine their own layer coefficients if data are available to do so.

Van Wyk, Yoder, and Wood (1983) performed an investigation to determine the layer coefficients of recycled asphalt pavement. Deflection basins from non-destructive testing were compared to theoretical deflection basins using a layered elastic software program called BISTRO. Once the deflections basins matched adequately based upon a pavement cross section selected for use in BISTRO, distresses such as tensile strain at the bottom of the asphalt, compressive subgrade strain, and deformations were computed for 25 test sections. These distresses were calculated so two similar pavement sections, one with a recycled layer and one without, and with the same predicted time before failure could be compared to determine the new layer coefficient for the recycled layer. This accomplished using Equation 2.10 and setting the structural number of both pavement sections equal to each other. The only parameter that could vary was the layer coefficient of the recycled asphalt layer; the layer coefficient of the comparison layer on the control pavement was held constant (0.44). The authors stated that the layer coefficients should be determined using the distress criteria that constitutes the shortest pavement life. The results of their study found that the layer coefficients increased slightly over time after construction completion. They also concluded that the elastic modulus was the single most important parameter for determining layer coefficients, and that the layer coefficient changed slightly with changes in thickness. The authors noted that their recommended procedure for determining the layer coefficients of recycled materials was time consuming and required extensive testing to determine fatigue characteristics. Based upon the wide range of layer coefficients found in their study (0.11 to 0.39), they did not recommend any particular layer coefficient for recycled asphalts, and stated that the selection of the coefficient was still the responsibility of the pavement designer (Van Wyk et al., 1983).

In 1989, Coree and White determined layer coefficients for ten bituminous mixes as single values and as distributions. These mixes were typically used by the Indiana Department of Highways (IDOH). They used Odemark's principle of equivalent stiffness, which compares different material types using a ratio of strength parameters. Applying this principle to find a new layer coefficient resulted in the following equation (Coree and White, 1989):

$$a_{IDOH} = a_{AASHO}\left[\frac{E_{IDOH}}{E_{AASHO}}\right]^{1/3} \tag{2.13}$$

Where a_{IDOH} is the desired layer coefficient for an IDOH mixture, a_{AASHO} is the AASHO Road Test layer coefficient, and E_{IDOH} and E_{AASHO} are the moduli of an IDOH asphalt mixture and the AASHO Road Test asphalt mixture, respectively.

Equation 2.13 can be solved for a single value of the layer coefficient. To determine the coefficient distributions, equations termed the Poel/Bonnaure et al. relationships were utilized. A full description of these equations and concepts is beyond the scope of this report, but can be found in the reference (Coree and White, 1989). Distributions of parameters were created for the AASHO Road Test and the IDOH mixtures. Three approaches were used to determine the layer coefficients, including a deterministic method, a probabilistic method, and a method combining both deterministic and probabilistic methods. The deterministic method used single values for each parameter in Equation 2.13, while the probabilistic method used distributions for all parameters in the same equation. The combination method used distributions for the mixture stiffness variables only. Using the deterministic approach, the average layer coefficient was 0.44. From the strictly probabilistic approach, the average coefficient was 0.53 with an average standard deviation of 0.287. The combination approach resulted in an average layer coefficient of 0.47 with an average standard deviation of 0.101. The authors cautioned against using the results of the probabilistic approach since the idea of a layer coefficient being represented by a distribution is not widely accepted. They stated that the larger values for the layer coefficients of IDOH mixtures compared to those of the AASHO Road Test could be attributed to the mixes being stiffer; the penetration grade of the AASHO Road Test mixes was 85 – 100, and the IDOH mixes were classified as 60 – 70. Finally, the authors think that the shift in resulting layer coefficients from the three different approaches was due to the distributions being asymmetrical.

To determine the layer coefficients for crumb-rubber modified (CRM) mixes for the Kansas Department of Transportation (KDOT), Hossain et al. (1997) used falling weight deflectometer (FWD) data collected from testing the in-place pavement structure. Three backcalculation methods were used to convert the deflection data from 41 different test locations into moduli. Good agreement was found between the methods, and the moduli were determined to be sound. To determine the layer coefficients from the moduli, the structural number was found using the following equation recommended by the AASHTO Design Guide (1993):

$$SN_{eff} = 0.0045 \cdot D \cdot \sqrt[3]{E_p} \tag{2.14}$$

Where D is the total thickness of the pavement cross section above the subgrade (in inches), and E_p is the effective modulus of the pavement cross section above the subgrade (in psi). Using the backcalculated moduli and the layer thicknesses, the layer coefficients were calculated for the CRM mixes.

Another method was used to calculate the layer coefficients for comparison to the results of the AASHTO method. This method was termed the equal mechanistic response (EMA) and used layer equivalency concepts (similar to the procedure of Van Wyk et al., 1983) to find another set of layer coefficients. The average layer coefficient for CRM mixes found using the AASHTO method was 0.28. The average layer coefficient from the EMA method was 0.33. From this study, a particular layer coefficient was not recommended for use in design. However, an equation was developed to estimate the layer coefficient for CRM mixes, and is defined as follows (after Hossain et al., 1997):

$$a_{CRM} = 0.315\log(E) - 1.732 \tag{2.15}$$

Where E is the modulus of the CRM mix in MPa.

From the study, the authors concluded that using the AASHTO method to estimate layer coefficients results in very high variabilities, and the equal mechanistic approach was much less variable. They found a range of layer coefficients for CRM asphalt mix overlays using the EMA ranging from 0.11 to 0.46, with most values falling around 0.30. For newly constructed CRM pavements, they found a range of layer coefficients of 0.25 to 0.50, with the average being approximately 0.35 (Hossain et al., 1997). However, these values and the equation developed are only applicable to the mixes tested.

In 1999, Romanoschi and Metcalf conducted research to analyze pavement structural capacity on pavements at the Louisiana Transportation Research Center Pavement Research Facility (PRF). FWD testing was performed on several sections at the PRF, and backcalculation was performed to determine moduli values. The authors chose to not use the AASHTO recommended method (Equation 2.14) because it was cumbersome, difficult to solve in a database environment, and had no unique solution (Romanoschi and Metcalf, 1999). They decided to use alternate equations developed by other researchers to find the structural number. However, once the backcalculated moduli were found, it was decided that the data were far too variable to use for the determination of layer coefficients. Therefore, laboratory tests were performed to find the resilient modulus for HMA cores taken from the sections at the PRF. The 1993 AASHTO Design Guide cautions against using laboratory moduli values for determining layer coefficients, as these values are known to differ vastly from the in situ moduli. While the authors recommended assigning layer coefficients based on laboratory moduli values, no HMA layer coefficients were presented as the results of their analysis.

Pologruto conducted a study in 2001 to determine the layer coefficients for various paving materials in Vermont for the Vermont Transportation Agency (VTrans). FWD data were gathered from the Strategic Highway Research Program (SHRP) database. Seasonal data, collected over more than 5 years, were used for eight different locations throughout the state of Vermont to determine the procedure for finding the layer coefficients. The effective structural number (Equation 2.14) was calculated for the backcalculated moduli and the results were plotted against the days of the year from which they were calculated. The resulting graph illustrated that for Vermont, the SN_{eff} remained fairly constant from May to October. High variations were found during the spring-thaw period. Therefore, only the data from the stable months were used to find the layer coefficients. A pilot project was created to determine the resulting layer coefficients for one test section to verify the validity of the recommended procedure. To find the layer coefficients for each respective layer in the pavement structure, the SN_{eff} was calculated immediately above and below the layer in question. The difference between these two values was found and then divided by the layer thickness. The SN_{eff} at the top of the subgrade was defined as zero. The results of the pilot project found layer coefficients within the AASHTO range for all materials except the HMA, which was 0.639. This value, although approximately 50% higher than the recommended 0.44, was not discounted from the results because the Marshall stability and resilient modulus for the material were well beyond the AASHTO recommended upper limits for each parameter. Since the results were encouraging, FWD testing was performed at three other test sites to find layer coefficients. A total of 25 individual testing locations were used in the author's analysis. Elastic layer simulation (ELS) was used to verify the layer coefficients found from the AASHTO procedure, and the two methods were compared. From the AASHTO method, the average HMA layer coefficient was 0.60. The average found using the ELS was 0.59. From these results, the author recommended using the AASHTO method for determining new layer coefficients, and that further research should be performed to see if similarly high HMA layer coefficients are found for other HMA types and in other locations (Pologruto, 2001).

In 2005, Jess and Timm used the AASHTO procedure to find layer coefficients from backcalculated moduli at the NCAT Test Track in Opelika, Alabama. The AASHTO method was used (Equation 2.14) to find the effective structural numbers for 26 different test sections. A control test section with a HMA layer coefficient of 0.44 was used to determine the comparison layer coefficients for the other test sections. From this study, the average layer coefficient found was 0.59 with a standard deviation of 0.13. Additionally, the sections included in the study were unusually thick; on the order of 24 inches of hot mix asphalt. Because of this, the sections did not experience significant pavement distress and consequently the structural coefficients were merely calibrated to surface deflection and not changes in serviceability. Therefore, it was decided to recommend a conservative value based on the average minus one standard deviation which resulted in 0.46 (Jess and Timm, 2005). Obviously, switching from 0.44 to 0.46 does not significantly alter pavement cross sections in design.

Harold Von Quintus conducted a study on layer coefficients for the Kansas Department of Transportation in 2007. He concluded that the HMA layer coefficient for the wearing surface and base mixtures should be increased. He stated that the magnitude of the increase should be dependent upon a detailed analysis of material properties, constructions records, and pavement performance, and not solely on the HMA modulus (Von Quintus, 2007).

SUMMARY

This literature review briefly discussed the AASHO Road Test and its limitations, along with the findings from the road test. The origins of the layer coefficient were explained, and the recommended value for the HMA layer coefficient was discussed. The AASHTO traffic and flexible pavement design equations and their respective inputs were presented and described. An explanation was provided for ALDOT's flexible pavement design procedure, and default values were given that are commonly used by ALDOT for the AASHTO design equation. Finally, the procedures and results from past layer coefficient calibration efforts were provided. As noted in this chapter, there was a lack of information available on sensitivity of the AASHTO equations to their inputs. Therefore, the following chapter presents a sensitivity analysis as part of this investigation.

CHAPTER 3 - SENSITIVITY ANALYSIS

INTRODUCTION

The 1993 AASHTO Design Guide flexible pavement design equation requires several inputs to find a resulting HMA thickness. These inputs obviously all affect the HMA thickness since they are included in the calculation to find it; however, obtaining the relative influence of each would be a valuable tool for optimizing pavement designs. A sensitivity analysis of all the inputs was performed to achieve that goal. The results can provide a pavement designer with the knowledge of which inputs have the greatest influence on the HMA thickness, and which have the least. Such information could be used to determine further research needs for the more influential inputs, or the need to better characterize those inputs for the greatest benefit to cost ratio.

METHODOLOGY

Fundamental Equations

A sensitivity analysis was conducted to determine which parameters in the AASHTO Design Guide (1993) flexible pavement design equation are the most influential on the thickness of the hot mix asphalt (HMA). Equation 3.1 shows the parameters needed to arrive at the HMA thickness (D_1), which comes from the structural number (*SN*) shown in Equation 3.2. The other parameters include the number of ESALs over the design period (W_{18}), the design reliability (*R*), the amount of variability (S_o) in the design, the soil modulus (M_R), and the expected loss of serviceability over the lifetime of the pavement (ΔPSI).

$$\log W_{18} = Z_R S_o + 9.36\log(SN+1) - 0.20 + \frac{\log\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.4 + \frac{1094}{(SN+1)^{5.19}}} + 2.32\log M_R - 8.07 \tag{3.1}$$

The structural number (*SN*) is used to define the structural capacity of a flexible pavement structure, and is calculated using the layer coefficients of the asphalt, base, and subbase layers (a_1, a_2, and a_3, respectively), the layer thicknesses (D_1, D_2, and D_3), and the drainage coefficients (m_1 and m_2) for the base and subbase layers as shown in Equation 3.2.

$$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3 \tag{3.2}$$

Inputs

Before conducting the sensitivity analysis, a baseline pavement condition was created to determine the relative influence of altering the inputs. A three-layer pavement cross-section of HMA, granular base, and subgrade soil was used. For the base layer of granular material, a thickness (D_2) of 6 inches was used with a layer coefficient (a_2) of 0.14. The layer coefficient is typical for crushed stone aggregate bases. A value of 1.0 was used for the drainage coefficient (m_2). This value was chosen for simplicity, but is also frequently used for ALDOT pavement designs. It was assumed that no subbase layer was used, so the final term in Equation 3.2 was zero. All other parameters in the design equation were varied to determine their relative influence on HMA thickness. Table 3.1 shows the values used for each input. These values were chosen based upon typical values used for flexible pavement design, and extremes were added to get a wide range of thicknesses and to characterize trends.

Table 3. 1 Values Used for Flexible Design Equation Inputs

Parameter	Values
Layer coefficient (a_1)	0.20, 0.30, 0.44, 0.60
Traffic level (W_{18})	1e6, 1e7, 1e8, 1e9 ESALs
Resilient modulus (M_R)	3,000, 10,000, 20,000, 30,000 psi
Reliability (R)	50%, 80%, 90%, 99%
Change in serviceability (ΔPSI)	1, 1.5, 2.0, 2.5
Variability (S_o)	0.20, 0.30, 0.40, 0.50, 0.60

To obtain the HMA layer thickness using the flexible design equation, first the structural number was found using the bisection method. This method was needed since equation 3.1 is difficult to solve explicitly. Once a structural number was computed, Equation 3.2 was rearranged to solve for D_1, which is the HMA layer thickness. Using this concept and the varied parameters as shown in Table 3.1, the HMA thickness was recalculated each time an input was changed. There were five inputs used for the variability, and four inputs used for the other inputs (traffic, resilient modulus, reliability, change in serviceability, and layer coefficient). Therefore, a total of $5 \times 4^5 = 5{,}120$ HMA thicknesses were calculated for this investigation.

RESULTS AND DISCUSSION

A Pearson's correlation was performed on the entire data set (all of the 5,120 thickness calculations) to determine which of the parameters were the most influential on the resulting HMA thickness. Table 3.2 shows the resulting sample Pearson correlation coefficients (R), where the values range from -1 to 1. Values closer to those range extremes are considered to be strongly correlated, and values close to zero are not as closely correlated. They are shown in the table in order from the most influential (a_1) to the least (S_o).

Clearly, the layer coefficient has the greatest influence on the HMA thickness. The next two parameters in Table 3.2, traffic level and resilient modulus, though influential cannot be changed from a design perspective; they are simply the conditions of the design. Therefore, the layer coefficient is the most influential parameter as measured by correlation coefficient with the other design parameters having much less influence. To better illustrate the significance of the correlation results, the trends and interdependency of each of these inputs is discussed below.

Table 3. 2 Correlation between HMA Thickness and Other Inputs

Parameter	Correlation Coefficient
Layer coefficient (a_1)	-0.518
Traffic level (W_{18})	0.483
Resilient modulus (M_R)	-0.425
Reliability (R)	0.157
Change in serviceability (ΔPSI)	-0.141
Variability (S_o)	0.083

Input Trends

To determine the general trend of each input on the resulting HMA thickness, a few points were selected of the 5,120 available for each input and were plotted to view the relationship. These plots were not meant to exactly quantify the relationship of each input with the HMA thickness, but rather to view the general trend and further illustrate the relative influence of each input on the HMA thickness.

The thicknesses found from altering the layer coefficient for the HMA (a_1) generally follow a negative trend as shown in Figure 3.1. The graph shown is for a set traffic level (1e8 ESALs), resilient modulus (20,000 psi), variability (0.40), reliability (80%), and change in serviceability (2.0). To generate graphs for the other inputs, the layer coefficient was set to 0.44. Figures 3.1 through 3.6 are ordered from the most influential input to the least (as in Table 3.2), and observation of the graphs provides a better understanding of the correlation results (i.e., that a_1 has the most influence and S_o has the least). For example, the overall magnitude of difference in HMA thickness from changing the layer coefficient is approximately 12 inches (Figure 3.1), while changing the variability only creates an overall difference of 1 inch in HMA thickness (Figure 3.6).

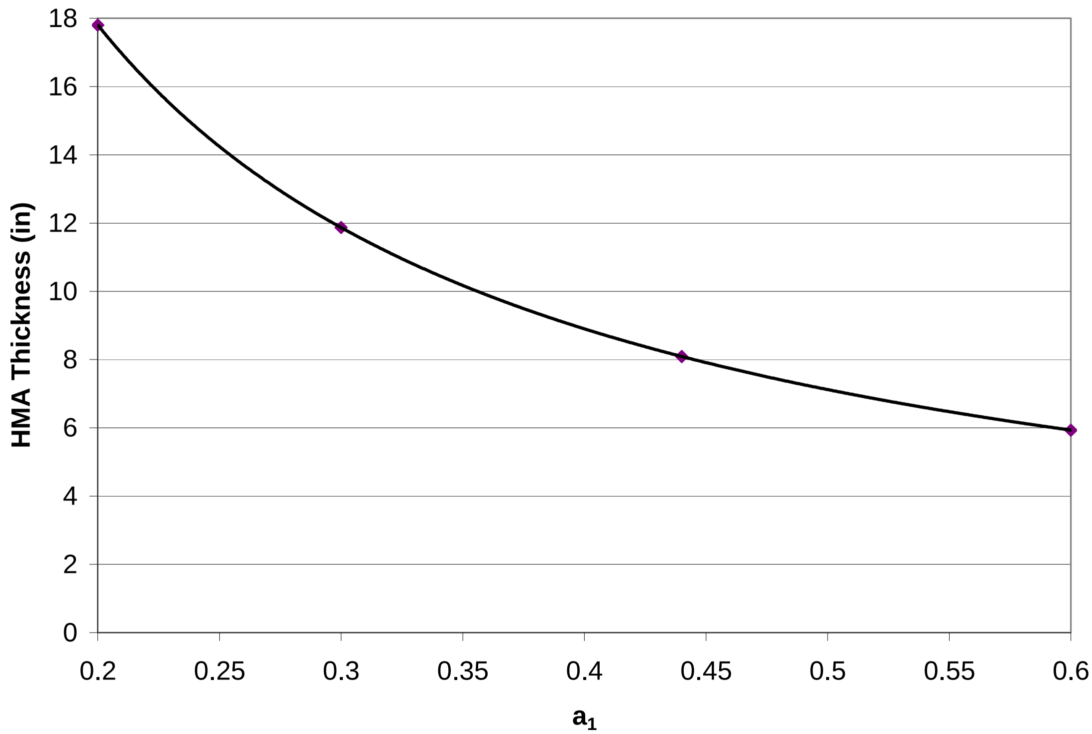

Figure 3. 1 General Trend of HMA Thickness with Layer Coefficient (a_1).

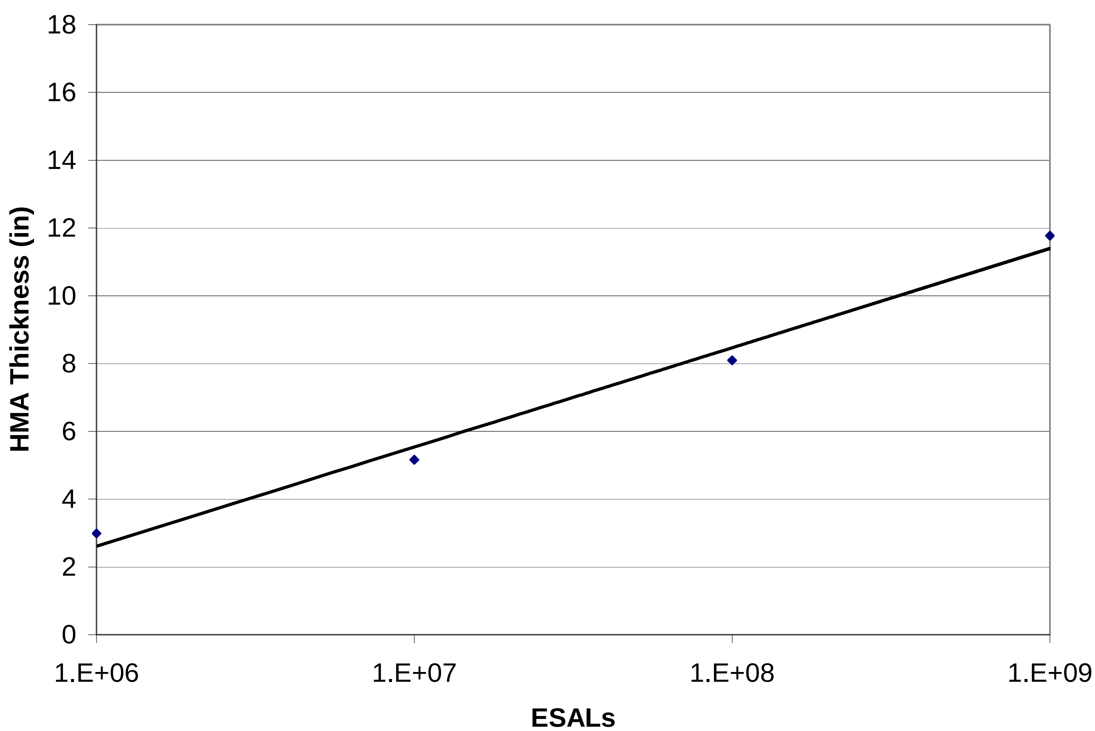

Figure 3. 2 General Trend of HMA Thickness with Traffic (W_{18}).

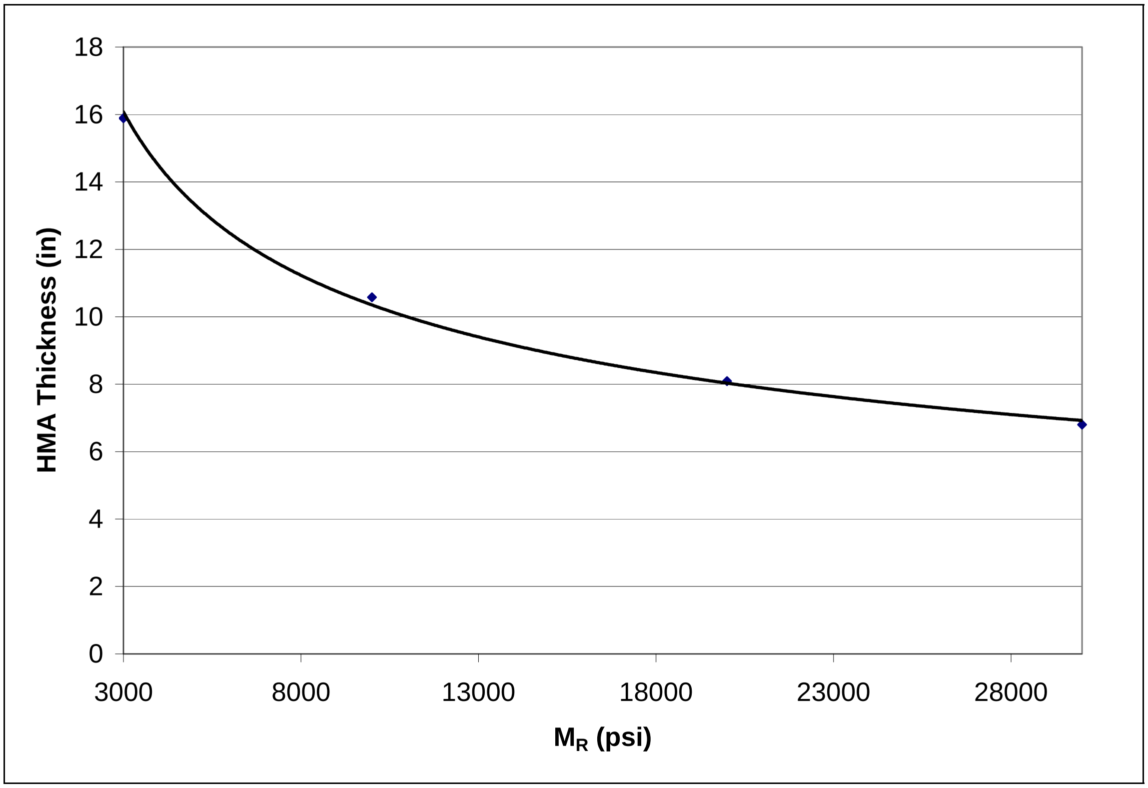

Figure 3. 3 General Trend of HMA Thickness with Resilient Modulus (M_R).

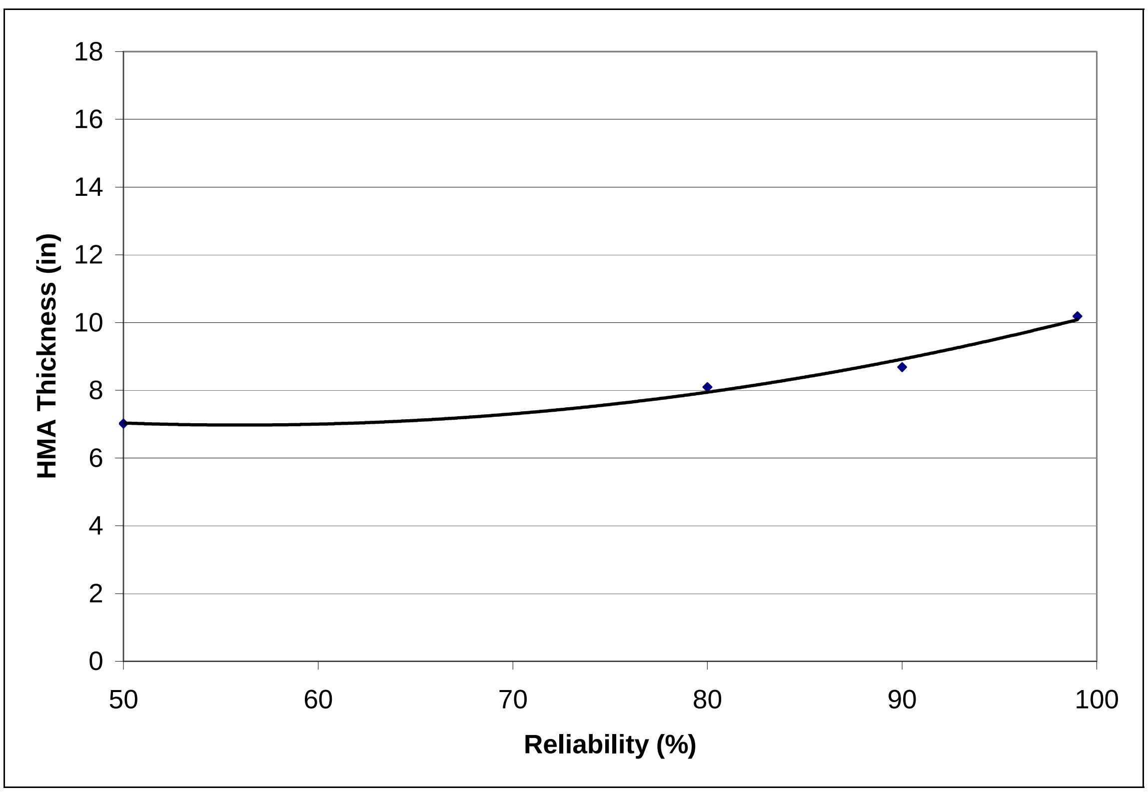

Figure 3. 4 General Trend of HMA Thickness with Reliability (R).

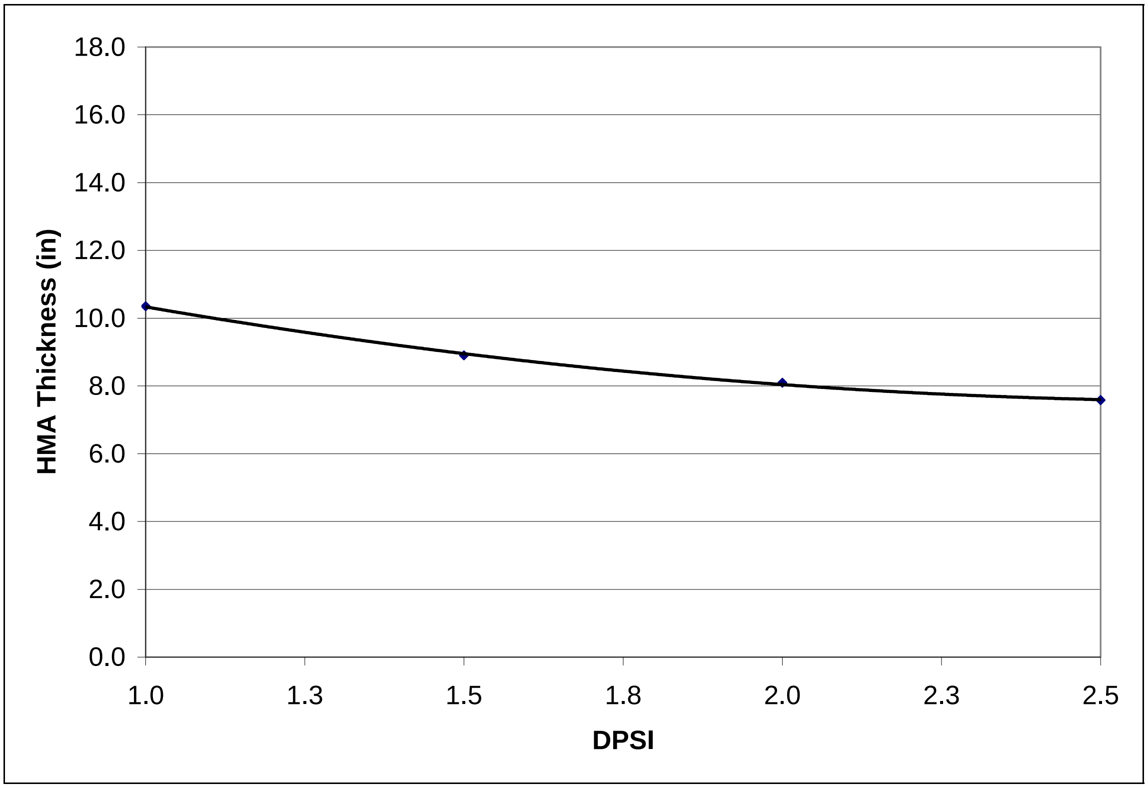

Figure 3. 5 General Trend of HMA Thickness with Serviceability (ΔPSI).

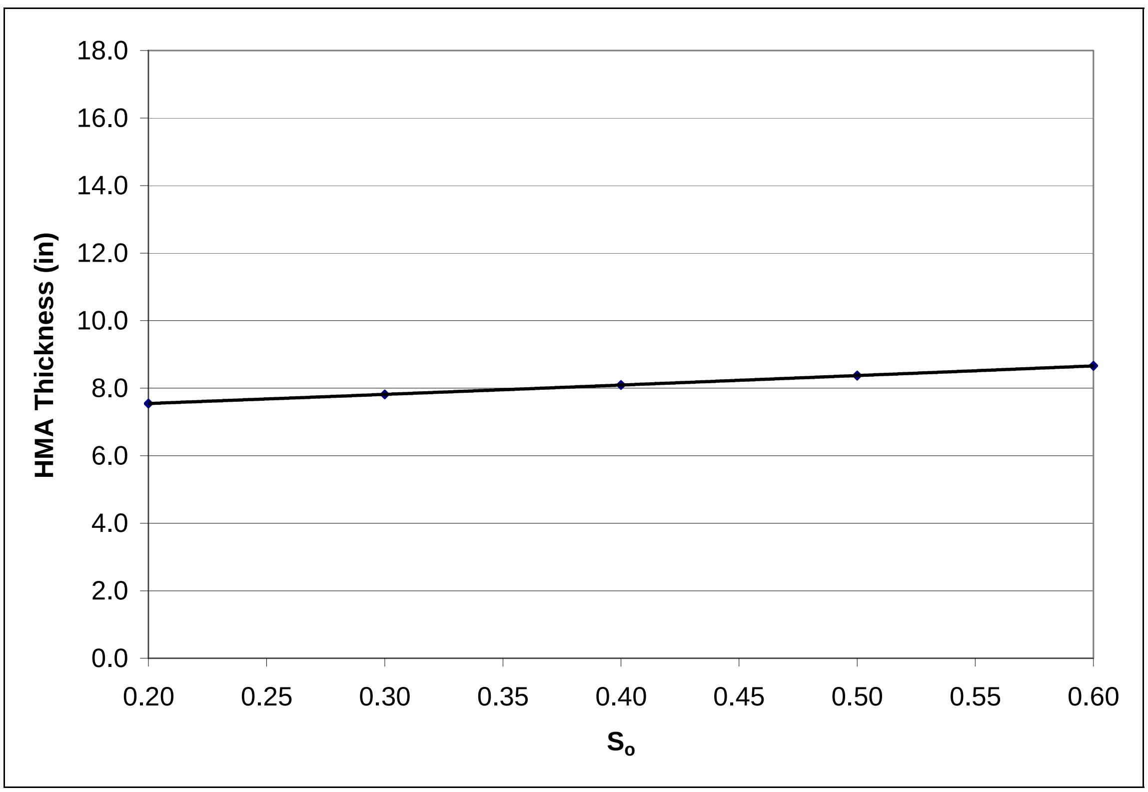

Figure 3. 6 General Trend of HMA Thickness with Variability (S_o).

Input Dependency

During the analysis, it became apparent that all of the inputs are dependent upon the traffic level. For example, the layer coefficient value has more of an influence on the resulting HMA thickness as the traffic level increases, as illustrated in Figure 3.7. This chart shows how the HMA thicknesses change on average for the different traffic levels specified when the layer coefficient is varied. This process was repeated for all the inputs (Figures 3.8 through 3.11) except the traffic level because the relative influence of each other input changed only as the traffic level changed.

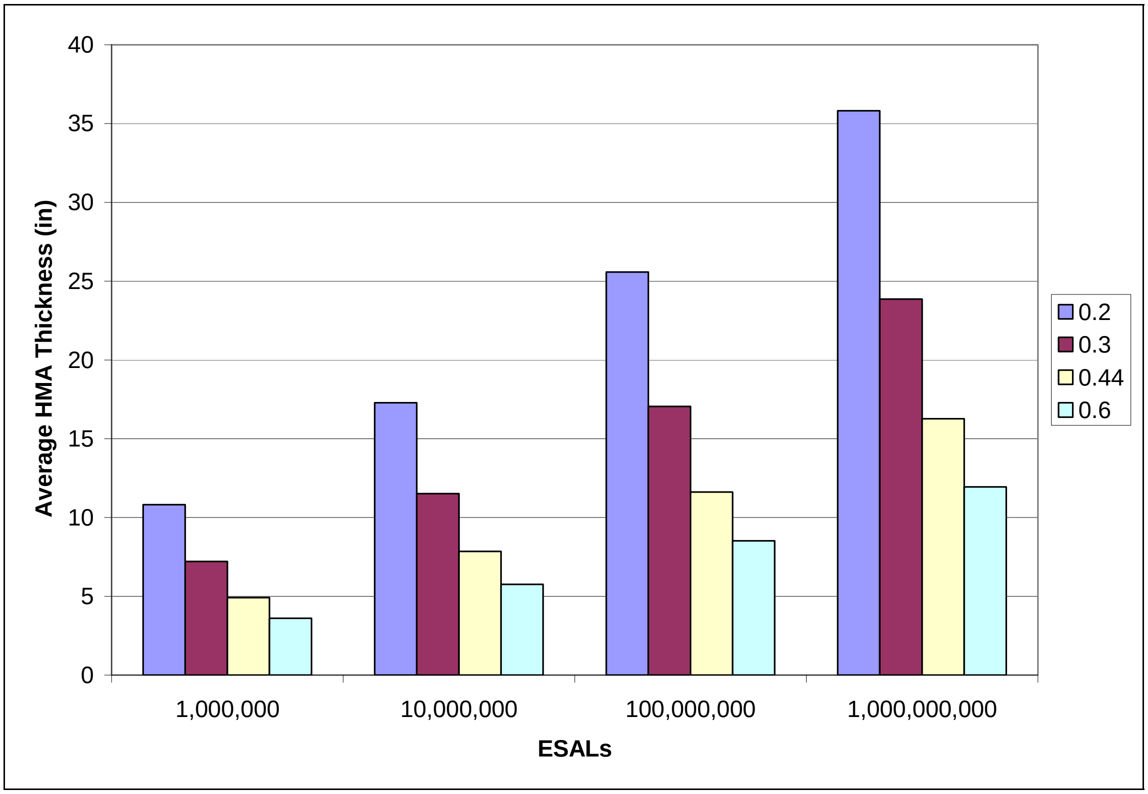

Figure 3. 7 Resulting HMA Thickness from Changing Layer Coefficient (a_1).

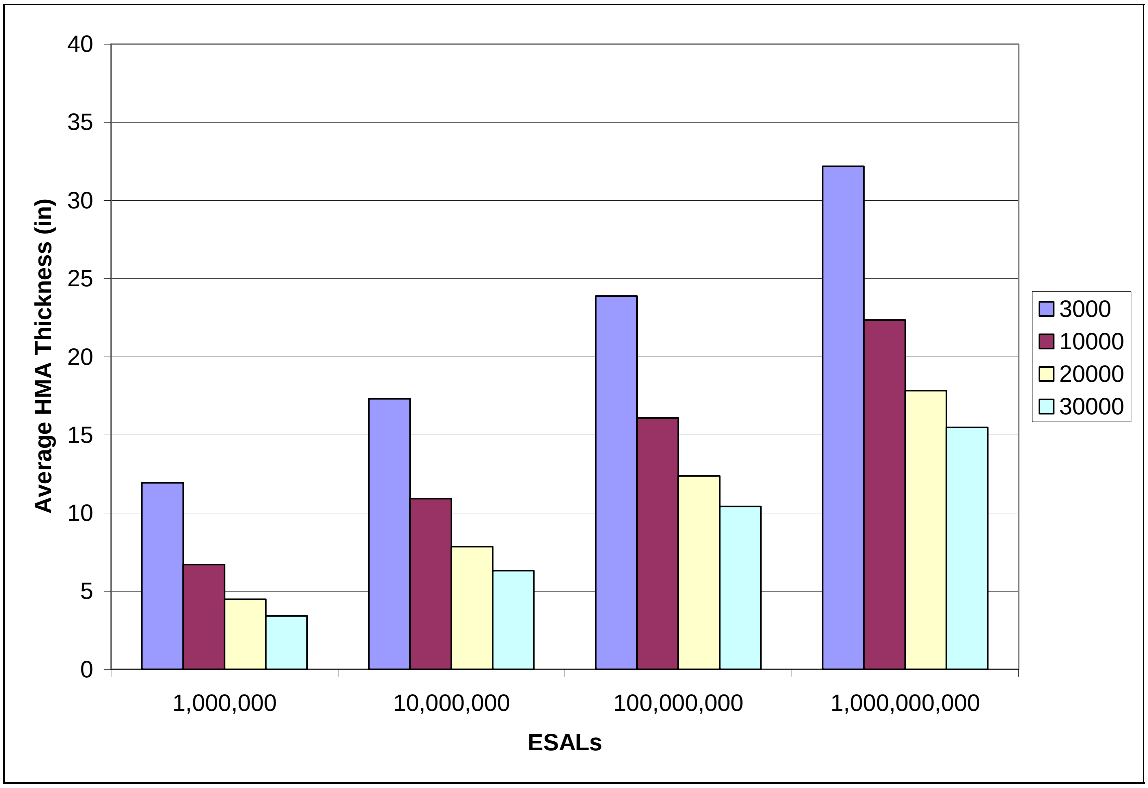

Figure 3. 8 Resulting HMA Thickness from Changing Resilient Modulus (M_R).

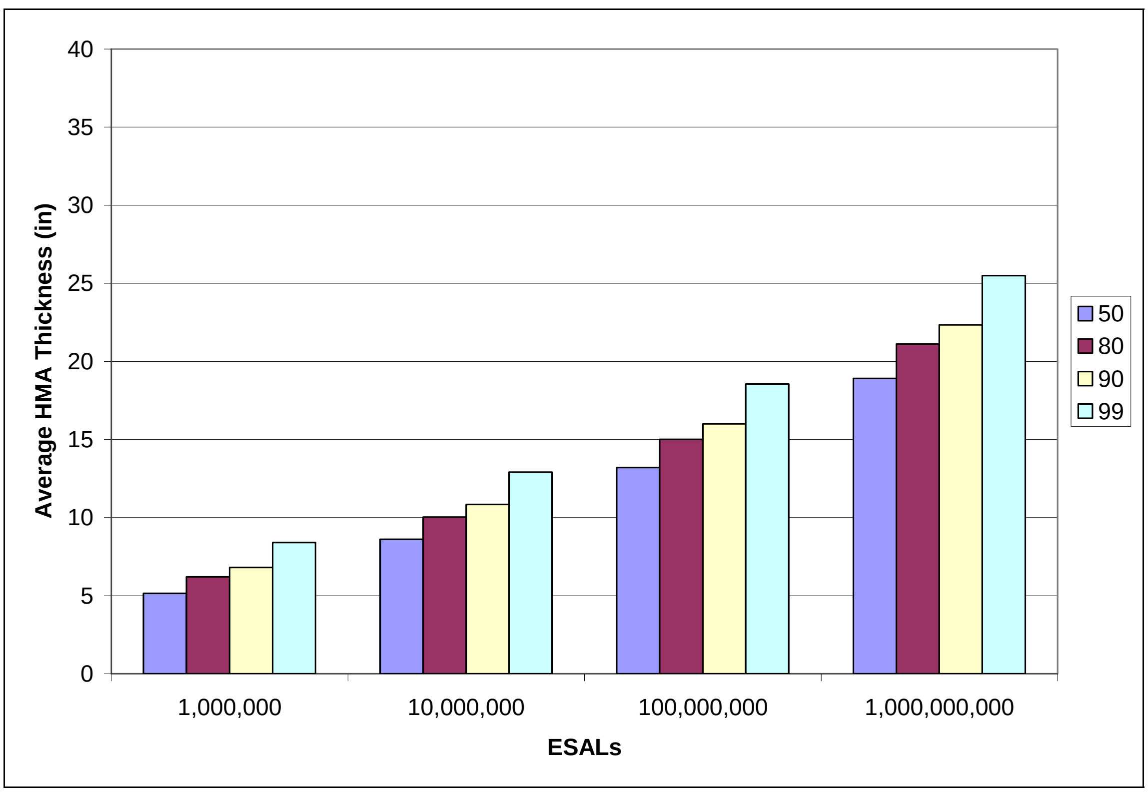

Figure 3. 9 Resulting HMA Thickness from Changing Reliability (R).

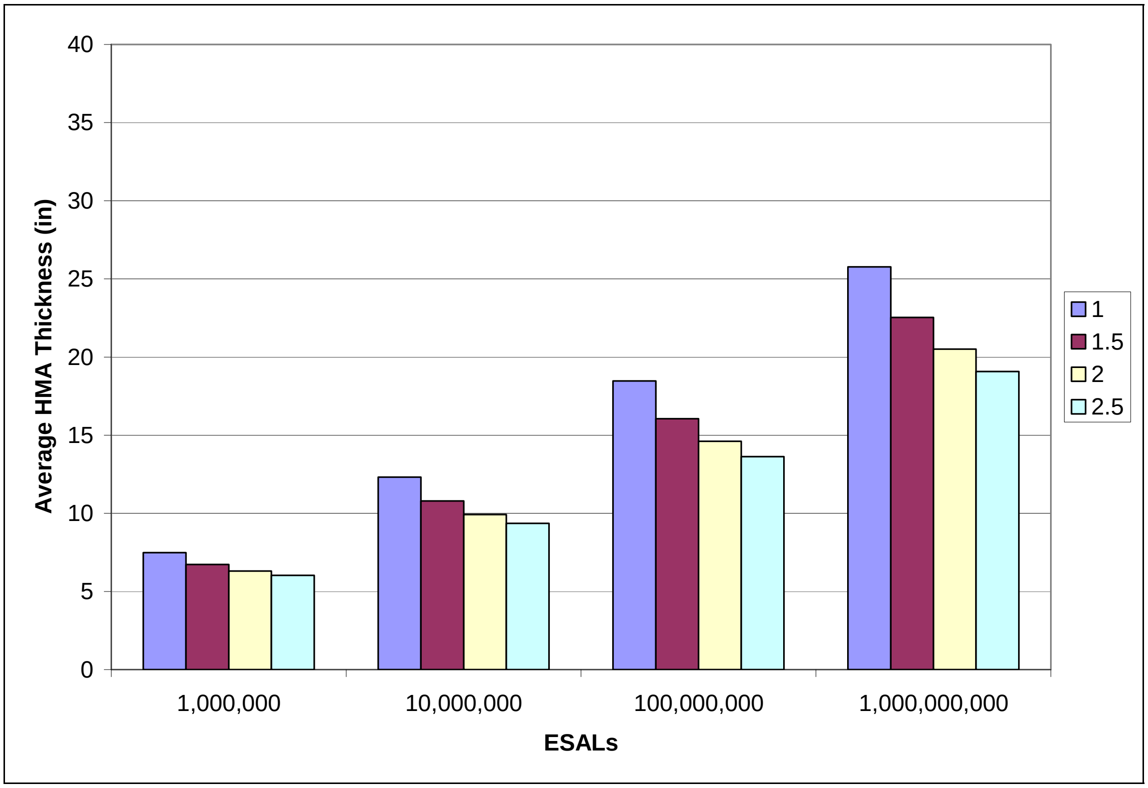

Figure 3. 10 Resulting HMA Thickness from Changing Serviceability (ΔPSI).

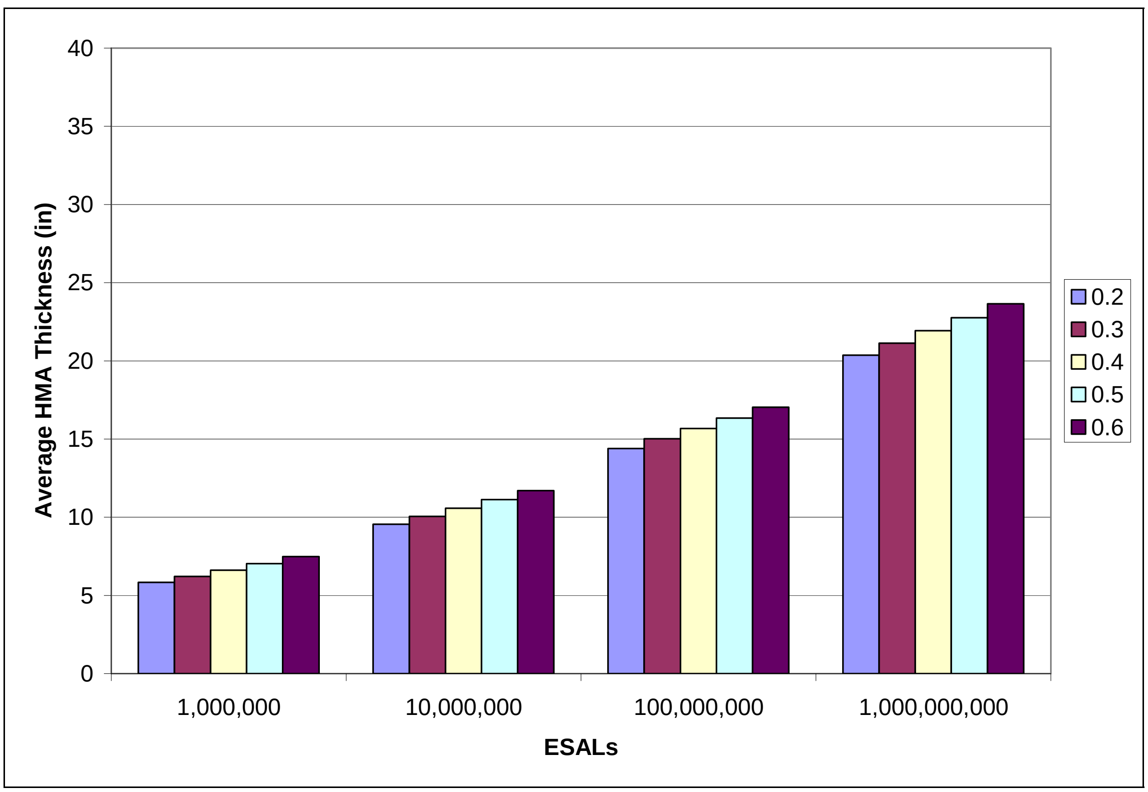

Figure 3. 11 Resulting HMA Thickness from Changing Variability (S_o).

It is important to note several trends from these data. First, the layer coefficient (Figure 3.7) and resilient modulus (Figure 3.8) seem to follow similar trends; namely, they have a greater effect on the HMA thickness as the traffic level increases. For example, at the 1 million ESAL traffic level, going from a layer coefficient of 0.2 to 0.3 causes a thickness decrease of approximately 4 inches, whereas that same shift at the 100 million ESAL traffic level causes a decrease of approximately 9 inches (Figure 3.7). This trend can also be seen in the reliability (Figure 3.9) and change in serviceability (Figure 3.10) charts, though not to the same extent. It is also slightly apparent in the variability chart (Figure 3.11), but it is even less obvious than the other charts. By looking at these graphs, it is evident that the traffic level does have a significant impact on the calculated HMA thickness, and that the other inputs are dependent upon it to varying degrees. The layer coefficient and resilient modulus (along with the traffic level) have the greatest influence on the resulting HMA thickness; the reliability, variability, and change in serviceability are not as significant. This is apparent in the graphs, and further illustrates the correlation results found previously (Table 3.2), as well as the general trends seen in Figures 3.1 through 3.6.

SUMMARY

A sensitivity analysis was performed on the variables of the 1993 AASHTO Design Guide flexible pavement design equation to see which inputs had the greatest influence on the resulting HMA thickness. A correlation between the inputs was performed to analyze the significance of the Pearson sample *R* values. The correlation results showed the layer coefficient was the most influential on the HMA thickness, and the variability was the least influential. Between those inputs (in order from more influential to least) were the traffic, resilient modulus, reliability, and change in serviceability. Plots were created for each input versus the resulting HMA thickness to observe general trends. Input dependencies were found in the sensitivity analysis. The relative influence of all inputs was dependent upon the traffic level.

The sensitivity analysis results proved the HMA layer coefficient to be the most influential parameter on the resulting design HMA thickness. The two parameters with similar magnitudes of influence, the traffic level and resilient modulus, are both generally set parameters for a given pavement design. Therefore, this increases the importance of being able to accurately characterize the HMA layer coefficient. This input is the only one of the three most influential that can be changed; consequently, better characterization of it would provide the greatest potential savings in HMA thickness.

CHAPTER 4 - RECALIBRATION OF LAYER COEFFICIENTS

INTRODUCTION

This investigation used pavement performance and detailed traffic data collected from the National Center for Asphalt Technology (NCAT) Pavement Test Track from the 2003 and 2006 test cycles to recalibrate the HMA layer coefficient (a_1) used in the 1993 AASHTO Design Guide flexible pavement design equation as illustrated in the flowchart in Figure 4.1.

As shown in Figure 4.1, there were two data sets needed for calibration: the traffic and surface performance data. Both data sets were needed to calculate the ESALs applied at the Test Track (an estimation of the actual ESALs), and only the performance data set was needed to calculate the predicted ESALs. Before the performance data could be used, it first had to be converted from IRI (in./mile) to serviceability (PSI). This was achieved using the Al-Omari/Darter equation. Once converted, these data were used to create plots for each test section that illustrated the change in PSI over time. Points were selected from these charts to obtain terminal serviceability levels (p_t) for recalibration purposes. These p_t values were used to find the calculated ESALs as illustrated in the Figure 4.1.

The PSI versus time charts were also used to find the change in serviceability (ΔPSI) needed to calculate the predicted ESALs. This was achieved by taking the terminal serviceability points (p_t) just described, and subtracting them from the initial serviceability level (p_o) for each test section.

The other primary input, the structural number (*SN*) was necessary for the calculation of both the calculated and predicted ESALs. This number was originally calculated using thickness data from each test section as well as an assumed HMA layer coefficient (a_1) of 0.44. This layer coefficient comes from the AASHO Road Test and is currently used by ALDOT for flexible pavement designs.

To find the calculated ESALs, the traffic data set, terminal serviceability, and structural number were used as inputs for the traffic equations derived from the AASHO Road Test as illustrated in Figure 4.1. The predicted ESALs were calculated using the change in serviceability and structural number as inputs to the AASHTO flexible pavement design equation. Once found, the calculated and predicted ESALs were compared and a simple linear least squares regression was performed. The error was minimized between the two data sets by changing only the HMA layer coefficient. This process was repeated for each structural section of the 2003 and 2006 Test Track cycles, resulting in a new regressed layer coefficient for each section. A more detailed discussion of this process is provided later in this chapter.

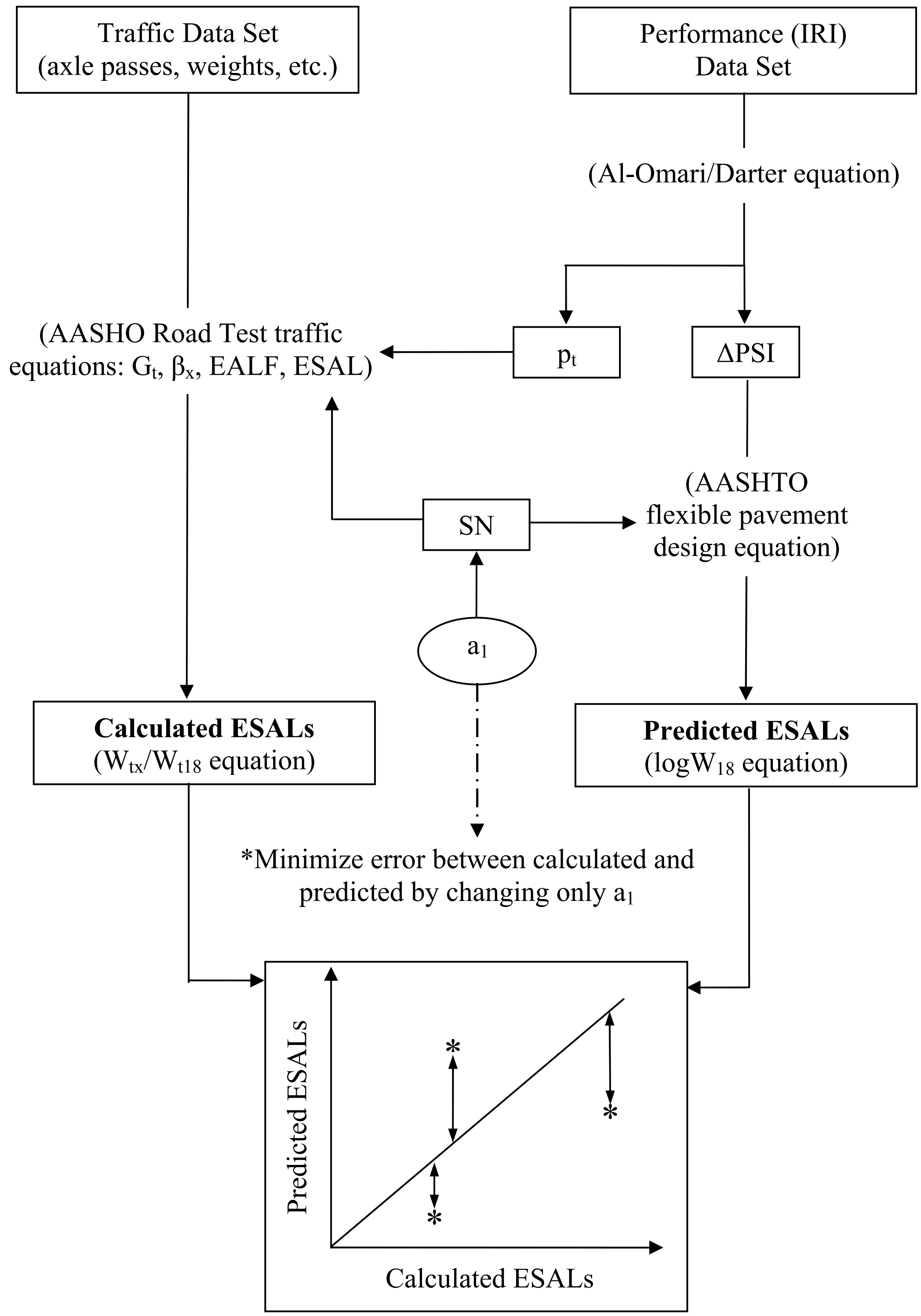

Figure 4. 1 Recalibration Procedure Illustration.

TEST FACILITY

Overview

The recalibration of the layer coefficients was performed using data from Auburn University's NCAT Pavement Test Track located in Opelika, Alabama. The Test Track is a 1.7 mile oval that is divided into 46 sponsored sections that are 200 ft long as shown in Figure 4.2. The Test Track provides sponsors with a facility that supports hot mix asphalt (HMA) research by applying live traffic in an accelerated testing environment. The test sections consist of varying pavement cross sections and materials based upon each sponsors' needs. Live traffic is applied 16 hours a day for 5 days a week, which adds up to approximately 10 million ESALs over a 3-year test cycle. The data used in this investigation were from the 2003-2006 and 2006-2009 test cycles. A full description of the 2003 and 2006 Test Track cycles are beyond the scope of this report, but have been documented elsewhere (Timm et al., 2004; Timm, 2009).

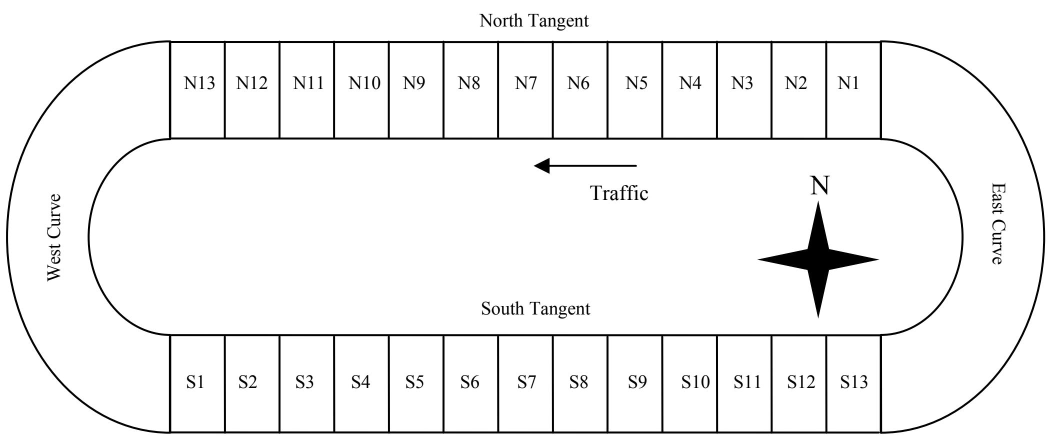

Figure 4. 2 Layout of the NCAT Test Track.

Structural Experiment

In the 2003 Test Track cycle, there were eight sections that comprised the structural experiment. The structural experiment included sections with embedded instrumentation to more accurately characterize pavement response under traffic loadings. These sections were designed with varying thicknesses and materials as shown in Figure 4.3. The thicknesses were varied to provide a wide array of distresses to analyze at the end of the test cycle. As seen in Figure 4.3, each section shared the same subgrade material that was already present at the Test Track. This soil, commonly termed the "Track soil", can be classified as an AASHTO A-4(0) soil type (Timm, 2009). The test sections also shared the same 6 inch crushed aggregate base course. Sections N1, N4, and N5 used modified PG 76-22 HMA layers of 5, 9, and 7 inches, respectively. Sections N2, N3, and N6 were also 5, 9, and 7 inches thick; however, they were unmodified PG 67-22 HMA layers. Section N7 consisted of 6 inches of unmodified PG 67-22 HMA placed under a 1 inch thick layer of PG 76-22 stone matrix asphalt (SMA). Finally, section N8 had a 2 inch thick rich bottom PG 67-22 layer with an additional 0.5% binder, which was placed under 4 inches of unmodified PG 67-22, and then topped with 1 inch of SMA (Timm et. al, 2004).

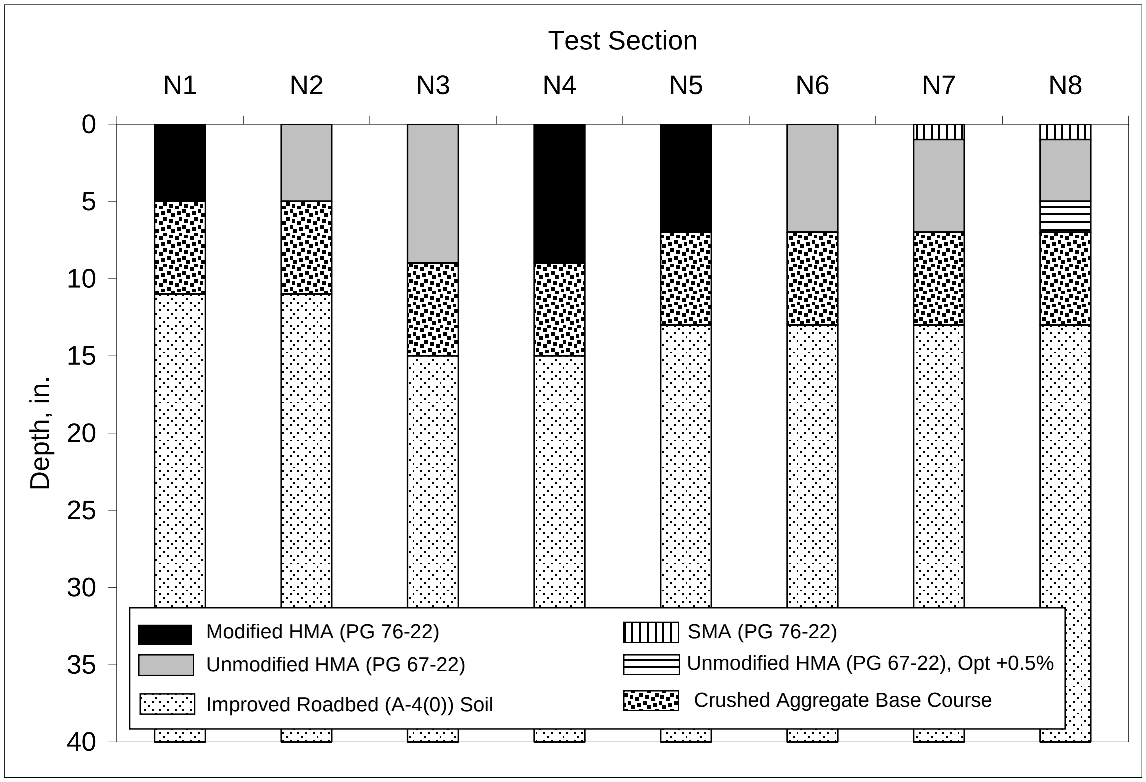

Figure 4. 3 2003 Test Track Structural Sections (Timm et. al, 2004).

For the 2006 Test Track, five of the original eight structural sections (N3 through N7) were left in place from the 2003 cycle; however N5 was milled and inlaid with a 2 inch asphalt layer to control top-down cracking that was present throughout the section. Three of the sections were reconstructed (N1, N2, and N8), and three new sections were added to the structural experiment (N9, N10, and S11). The cross sections of the 2006 Test Track structural sections are shown in Figure 4.4 along with their respective sponsors. As seen in the figure, all sections other than N8 and N9 still utilized the Track soil as subgrade material. The subgrade material used in sections N8 and N9 can be classified as an AASHTO A-7-6 soil, and is known as Seale subgrade material since it was imported from a borrow pit in Seale, Alabama (Taylor, 2008).

In sections N1 and N2, 10 inches of Florida limerock material was used as the base layer. Both sections were topped with 7 inches of HMA: unmodified PG 67-22 for all lifts of section N1, and a 3 inch lift of the same unmodified binder followed by two 2 inch lifts with modified PG 76-22 binder for section N2. For sections N3, N4, N5, N6, N7, and S11, 6 inches of granite aggregate base material supplied from Vulcan materials was used as the base layer. Sections N3 and N4 both consisted of 9 inches of HMA. N3 used unmodified PG 67-22 binder in all HMA lifts, and N4 used modified PG 76-22 in all lifts. N5, N6, and N7 contained 7 inches of HMA in each section: section N5 had a HMA layer created with unmodified PG 67-22 binder placed over modified PG 76-22 layers, section N6 was comprised of unmodified PG 67-22, and section N7 contained 6 inches of unmodified binder topped with a 1 inch lift of PG 76-22 SMA. Section N8 consisted of approximately 6 inches of Track soil as the base layer, followed by one layer of HMA with PG 64-22 binder designed with 2% air voids, one layer of HMA with PG

64-22 binder, one layer of HMA with PG 76-28 binder, and finally topped with PG 76-28 binder SMA for a total HMA thickness of 10 inches. Section N9 used approximately 9 inches of the Track soil as base material, followed by one layer with PG 64-22 compacted to 2% air voids, two layers with PG 64-22, a layer with PG 76-28, and finally topped with PG 76-28 SMA for a total HMA thickness of 14 inches. Section N10 used 4 inches of a dolomitic limestone base material, termed Missouri Type 5 base. Above the base was one HMA layer with PG 64-22 topped with two HMA layers with PG 70-22 binder for a total of approximately 8 inches of HMA. Section S11 consisted 8 inches of HMA: two upper layers with modified PG 76-22 binder over two layers with unmodified PG 64-22. These layers were placed atop the 6 inches of granite aggregate base material utilized in sections N3 through N7.

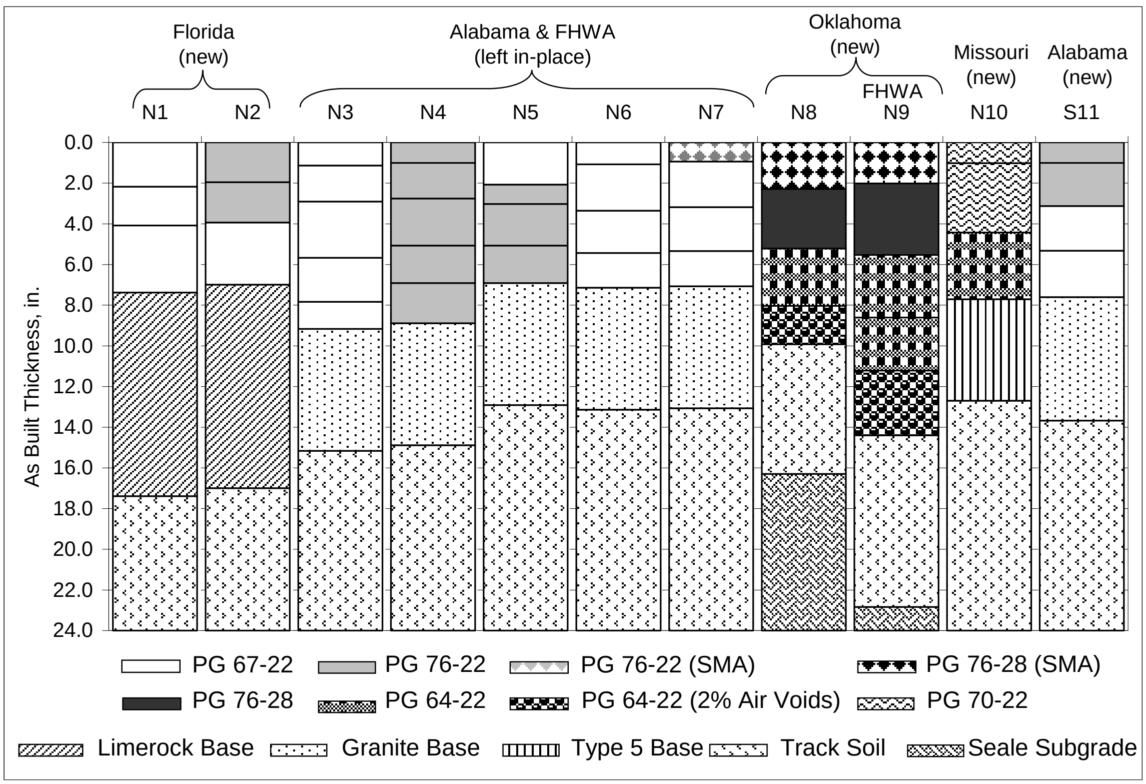

Figure 4. 4 2006 Test Track Structural Sections (Timm, 2009).

Performance Monitoring

During each Test Track cycle, the condition of each section is monitored to determine the accumulation of distresses such as fatigue cracking and rutting over time. These distresses contribute to an increase in roughness on the pavement surface, which adversely affects ride quality. NCAT uses an Automatic Road Analyzer (ARAN) Inertial Profiler (shown in Figure 4.5) at the Test Track to measure the small wavelengths in the longitudinal profile in the pavement surface at high speeds for each wheel path. It achieves this by using an accelerometer, lasers, a speedometer, and a computer. The accelerometer is used to measure the acceleration of the vehicle, which is processed through data algorithms to establish an inertial reference. This reference is used to determine the instantaneous height of the accelerometer in the vehicle. High frequency lasers are used to determine the distance between the accelerometer and the ground, and the speedometer is used to determine the distance between the measurements taken by the laser (Sayers and Karamihas, 1998). These distance measurements are continually stored in an on-board computer, and are later equated to a standard measurement known as the international roughness index (IRI).

Figure 4. 5 ARAN Inertial Profiler at NCAT Test Track.

METHODOLOGY

IRI Data

The IRI data collected with the ARAN van over the course of each Test Track cycle were used for the recalibration process. The IRI data (recorded in inches per mile) were converted to units of present serviceability index (PSI), a term that is a direct input into the AASHTO flexible pavement design equation. This conversion was made using the following relationship (Al-Omari and Darter, 1994):

$$PSI = 5e^{(-0.0038 \cdot IRI)} \tag{4.1}$$

Once the data were converted, plots were created for each section that showed the decrease in PSI over time for the right (RPSI) and left wheel paths (LPSI), as well as the average of the two (AvgPSI). From each of these plots, various points were selected for calibration of the flexible pavement design equation. If considerable deterioration was present over time, multiple points were used per section to provide a better fit. Figure 4.6 shows a plot of PSI over time for section N1 during the 2003 Test Track cycle. As seen in the figure, there is more deterioration in the right wheel path than in the left wheel path. This was not the case for all the sections; therefore the wheel path with the most deterioration (the lower PSI values) was used in the recalibration procedure to be conservative. Five points are shown in Figure 4.6 as circles on the right wheel path PSI line. These five points were chosen to represent the trend of decreasing PSI over time. It is important to note that there is a jump in the PSI level around January 2005, which is when the section was milled and inlaid. When cases such as this occurred, points were not selected beyond the jump in PSI.

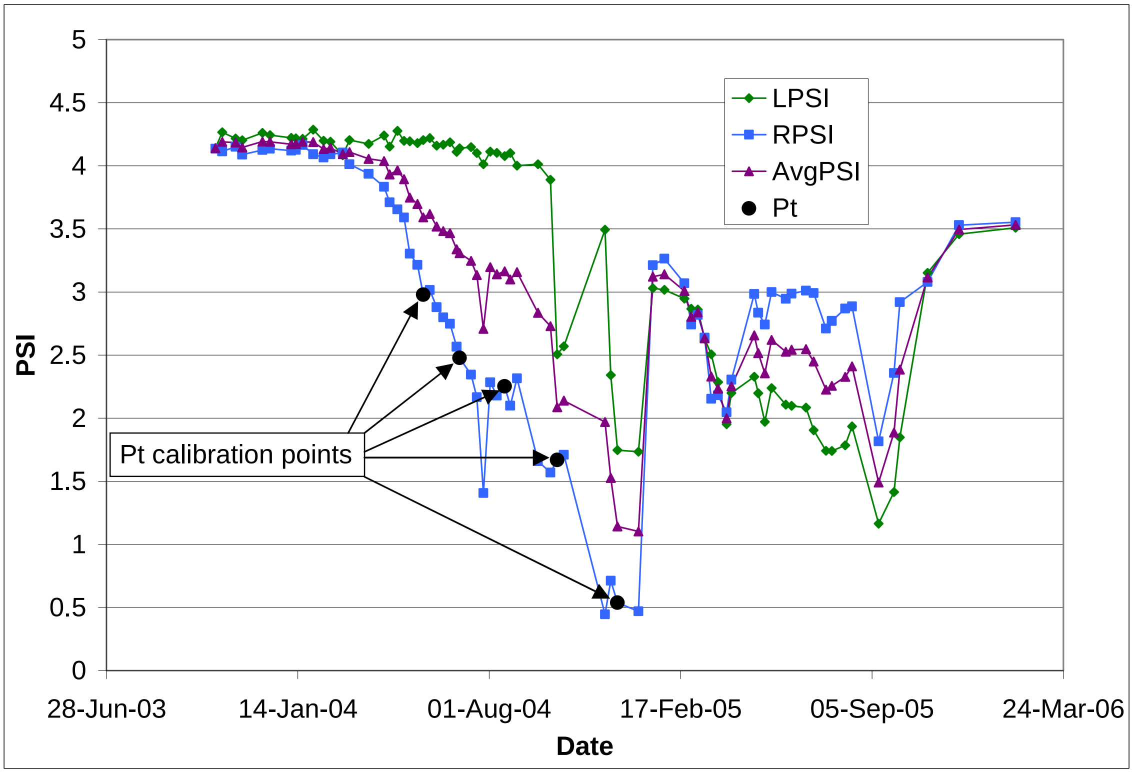

Figure 4. 6 PSI Data Derived from IRI Data from Section N1 (2003 Test Track).

While the deterioration in Figure 4.6 is quite apparent, other sections did not show similar trends. For example, Figure 4.7 shows section N3 during both the 2003 and 2006 Test Track cycles (as mentioned previously, this section was left in place after the 2003 cycle). From the figure, it is seen that there is no major deterioration in this section over the course of 6 years and 20 million ESALs of traffic; therefore, point selection in this section was not possible. In sections such as this, an artificial terminal PSI was assigned to the section to perform the recalibration. Each value assigned was lower than the actual PSI of the section to be conservative. For this particular case, a value of 3.5 was used, and the actual final PSI measurement was 4.3. This value was chosen to be conservative, and because the regression procedure could not be performed unless there was a substantial difference between the initial and final PSI values. The other PSI versus time graphs for each section can be found in Appendix A. If point selection was possible, the points were denoted on the figures in the Appendix as they are in Figure 4.6.

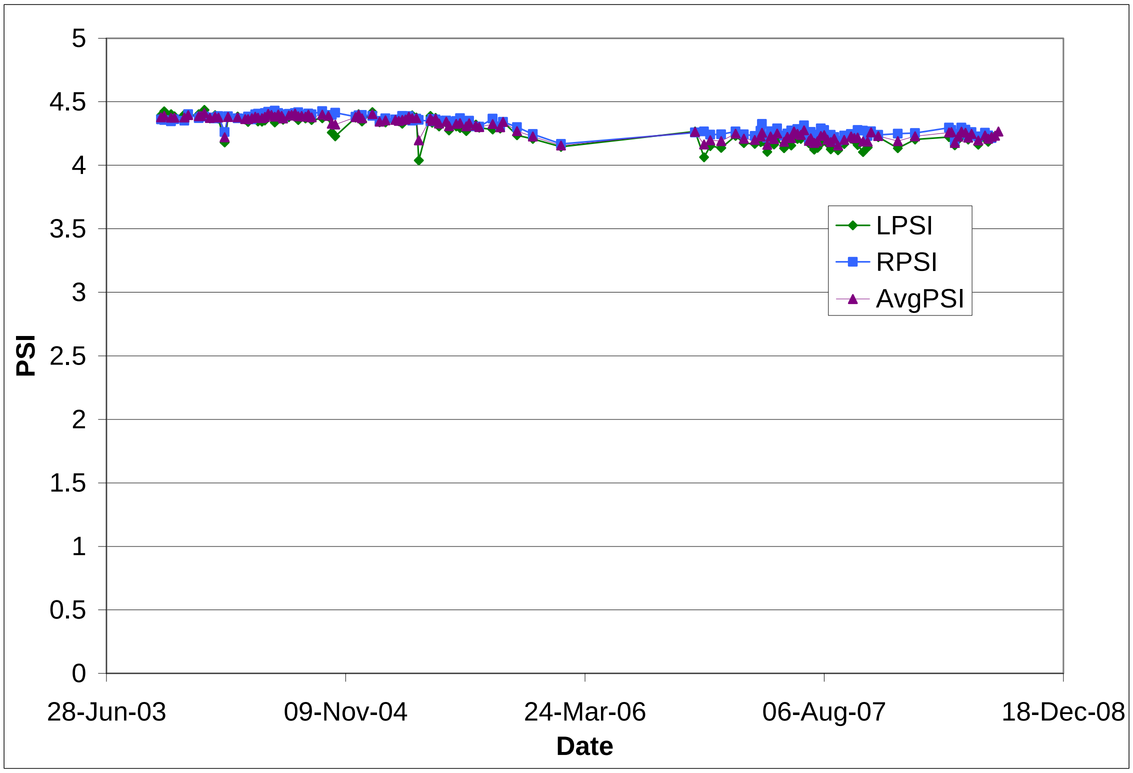

Figure 4. 7 PSI Data from Section N3 (2003 and 2006 Test Track Cycles).

Predicted ESALs

The AASHTO Design Guide flexible pavement design equation was used to predict the amount of applied ESALs, and is expressed as:

$$\log W_{18} = Z_R S_o + 9.36\log(SN+1) - 0.20 + \frac{\log\left[\dfrac{\Delta PSI}{4.2-1.5}\right]}{0.4+\dfrac{1094}{(SN+1)^{5.19}}} \tag{4.2}$$

$$+ 2.32\log M_R - 8.07$$

Where Z_R is the normal deviate for a given reliability, S_o is the standard deviation, ΔPSI is the expected loss of serviceability over the lifetime of the pavement, M_R is the resilient modulus of the subgrade, and SN is the structural number, as defined by the following equation:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \tag{4.3}$$

Where a_1, a_2, and a_3 are the layer coefficients for the hot mix asphalt, base, and subbase layers, respectively; D is the thickness of each respective layer in inches, and m is the drainage coefficient for the base and subbase layers. For this analysis, there were no subbase layers for the sections at the Track, so the final term was dropped from the equation.

For Equation 4.2, the reliability was set at 50%. This value was chosen because higher reliabilities are used to artificially increase the predicted traffic to account for uncertainty in the design process. Since all the inputs necessary for use in the equation were known, it was not necessary to provide this artificial traffic increase, and therefore the reliability was set at 50%. This reliability resulted in a normal deviate (Z_R) of zero; therefore, the first term in the design equation was zero.

For the calculation of the structural number (*SN*), the thicknesses were obtained from construction records for each section, the drainage coefficient for the base layer (m_2) was assumed to be equal to one, and the layer coefficient for the base layer (a_2) was assumed to be 0.14. This layer coefficient is a recommended value for a crushed stone base course (AASHTO, 1993). The layer coefficient for the HMA (a_1) was set at a seed value of 0.44, which is the current value recommended in the 1993 AASHTO Design Guide and also commonly used by ALDOT. The thicknesses used for the analysis are shown in Table 4.1. These thicknesses represent section-wide averages based on surveyed depths.

Table 4. 1 Thickness Data for the Test Track Sections

Section	Test Track Year	D_1 (in)	D_2 (in)
N1	2003	5.3	6.0
N1	2006	7.4	10.0
N2	2003	4.8	6.0
N2	2006	7.1	10.0
N3	2003-2006	9.2	6.0
N4	2003-2006	8.9	6.0
N5	2003-2006	6.9	6.0
N6	2003-2006	7.1	6.0
N7	2003-2006	7.1	6.0
N8	2003	7.0	6.0
N8	2006	9.9	6.4
N9	2006	14.4	8.4
N10	2006	7.7	5.0
S11	2006	7.6	6.1

To obtain the change in serviceability (ΔPSI) for each test section, the serviceability data was utilized. The initial PSI (p_o) value for each section was set to the first PSI reading taken after the section's construction. The final PSI value (p_t) was selected based upon the point selection procedure described previously. The difference between these two values was the ΔPSI used in the equation.

The resilient modulus (M_R) was calculated for each section based on backcalculated falling weight deflectometer (FWD) data collected over the course of each Test Track cycle. An average was taken of the backcalculated data for each section and then divided by three as recommended in the AASHTO Design Guide (1993). As mentioned previously, this value is divided by three to account for differences in testing procedures used to find the subgrade moduli. Table 4.2 shows the resulting M_R values used in Equation 4.2.

Table 4. 2 Resilient Modulus Data (after Taylor, 2008)

Section	Test Track Year	M_R (psi)
N1	2003	8093
N1	2006	12279
N2	2003	8497
N2	2006	11749
N3	2003	10838
N4	2003	11296
N5	2006	10053
N6	2003	11427
N7	2003	11154
N8	2003	9800
N8	2006	10038
N9	2006	15630
N10	2006	14731
S11	2006	9593

After identifying all the inputs, the predicted traffic ($logW_{18}$) was found for each test section. The estimated ESALs were found by solving $10^{\log W_{18}}$. For example, for section N1 of the 2003 Test Track, Table 4.3 shows the serviceability data that were used in the predicted ESALs calculation (also shown graphically in Figure 4.6).

Table 4. 3 Serviceability Data for Section N1 (2003 Test Track)

Date	Initial Serviceability (p_o)
10/20/2003	4.14
Date	**Terminal Serviceability (p_t)**
5/24/2004	2.98
7/01/2004	2.48
8/17/2004	2.25
10/11/2004	1.67
12/13/2004	0.54

Using these data in conjunction with the resilient modulus and thickness data mentioned previously, the predicted ESALs based upon this damage were calculated and are shown in Table 4.4.

Table 4. 4 Predicted ESALs Applied for Section N1 (2003 Test Track)

Date	Predicted ESALs
5/24/2004	802,367
7/01/2004	1,126,574
8/17/2004	1,270,712
10/11/2004	1,638,661
12/13/2004	2,340,290

Calculated ESALs

The AASHTO Design Guide quantifies pavement damage using Equivalent Axle Load Factors (EALFs), which are used to find the number of ESALs. An EALF is used to describe the damage done by an axle per pass relative to the damage done by a standard axle (typically an 18-kip single axle) per pass. This equation comes from the results of the AASHO Road Test, and is expressed as follows according to Huang (2004):

$$EALF = \frac{W_{t18}}{W_{tx}} \tag{4.4}$$

Where W_{tx} is the number of x axle load applications at time t, and W_{t18} is the number of 18 kip axle load applications at time t.

The EALFs for each axle load group are used to find the total damage done during the design period, which is defined in terms of passes of the standard axle load, as shown in the following equation (Huang, 2004):

$$ESAL = \sum_{i=1}^{m} EALF_i n_i \tag{4.5}$$

Where m is the number of axle load groups, $EALF_i$ is the EALF for the ith axle load group, and n_i is the number of passes of the ith axle load group during the design period.

These basic equations were used in conjunction with the following equations to that were developed from the AASHO Road Test to characterize the traffic for the recalibration process (Huang, 2004).

$$G_t = \log\left[\frac{4.2 - p_t}{4.2 - 1.5}\right] \tag{4.6a}$$

$$\beta_x = 0.40 + \frac{0.081 \cdot \left(L_x + L_2\right)^{3.23}}{\left(SN + 1\right)^{5.19} \cdot L_2^{\ 3.23}} \tag{4.6b}$$

$$\log\left[\frac{W_{tx}}{W_{t18}}\right] = 6.1252 - 4.79\log\left(L_x + L_2\right) + 4.33\log L_2 + \frac{G_t}{\beta_x} - \frac{G_t}{\beta_{18}} \tag{4.6c}$$

Where G_t is the logarithm of the ratio of loss of serviceability at some time t to the potential loss of serviceability at terminal serviceability (p_t) = 1.5, and the initial serviceability is assumed to be 4.2; this value was typical for flexible pavements at the AASHO Road Test, and is used as the initial value for ALDOT pavement designs as well. β_x is a function of design and load variables, L_x is the axle group load in kips, L_2 is the axle code (1 for single, 2 for tandem and 3 for tridem), SN is the structural number, W_{tx} is the number of x axle load applications at time t, W_{t18} is the number of 18 kip axle load applications at time t, and β_{18} is the value of β_x when L_x is equal to 18 and L_2 is equal to one.

Most of the inputs needed for these equations were obtained from traffic data collected during the 2003 and 2006 Test Track cycles. In the 2003 cycle, six different trucks were used to apply traffic to the test sections. Five of the trucks (termed "triple trailers") were comprised of one steer axle, a drive tandem axle, and five trailing single axles as shown in Figure 4.8. The sixth truck (termed the "box trailer") consisted of a steer axle and two tandem axles as shown in Figure 4.9. The weights of these axles were recorded for each truck, and are shown in Table 4.5.

Figure 4. 8 Triple Trailer Truck at NCAT Test Track.

Figure 4. 9 Box Trailer Truck at NCAT Test Track.

Table 4. 5 Axle Weights for 2003 Test Track (after Priest and Timm, 2006)

	Steer, lb	Drive Tandem, lb		Single Axle, lb				
Truck ID	**1**	**1**	**2**	**1**	**2**	**3**	**4**	**5**
1-Triple	10150	19200	18550	21650	20300	21850	20100	19966
2-Triple	11000	20950	20400	20950	21200	21000	20900	20900
3-Triple	10550	20550	21050	21000	21150	21150	21350	20850
4-Triple	10500	21050	20700	21100	21050	21050	20900	21050
6-Triple	11200	19850	20750	20350	20100	21500	19500	20300
	Steer, lb	**Drive Tandem, lb**		**Rear Tandem, lb**				
5-Box	11550	16850	17000	16800	16100			

For the 2006 Test Track cycle, only five triple trailer trucks were used to apply traffic, and their axle weights are shown in Table 4.6.

Table 4. 6 Axle Weights for 2006 Test Track (Taylor, 2008)

	Steer, lb	Drive Tandem, lb		Single Axle, lb				
Truck ID	**1**	**1**	**2**	**1**	**2**	**3**	**4**	**5**
1-Triple	9400	20850	20200	20500	20850	20950	21000	20200
2-Triple	11200	20100	19700	20650	20800	20650	20750	21250
3-Triple	11300	20500	19900	20500	20500	21000	20650	21100
4-Triple	11550	21200	19300	21000	21050	21000	20750	20800
6-Triple	11450	20900	19400	20100	20450	21000	20050	20650

Using these axle weights, averages were computed for each axle type (steer, single, and tandem), for each truck type (triple and box), and for each Test Track cycle. For the tandem axles, averages were computed for each tire set (1 and 2), and then summed to obtain the total average tandem axle weight. For example, the 2003 weights resulted in the averages shown in Table 4.7. The combined values are the result of the average of steer axles, the sum of the average tandem axle weights, and the average of the single axle average weights. Table 4.8 shows the average axle weights for the 2006 Test Track cycle.

Table 4. 7 Average Axle Weights for 2003 Test Track

	Steer, lb	Drive Tandem, lb		Single Axle, lb				
	1	**1**	**2**	**1**	**2**	**3**	**4**	**5**
Averages	10680	20320	20290	21010	20760	21310	20550	20613
Combined	*10680*		*40610*					*20849*

Table 4. 8 Average Axle Weights for 2006 Test Track

	Steer, lb	Drive Tandem, lb		Single Axle, lb				
	1	**1**	**2**	**1**	**2**	**3**	**4**	**5**
Averages	10980	20710	19700	20550	20730	20920	20640	20800
Combined	*10980*		*40410*					*20728*

These averages were used in Equation 4.6 (as the L_x term) to compute the β_x and W_{tx}/W_{t18} terms, which were then used to compute the amount of ESALs applied per axle. The values chosen for p_t and the calculations for SN were discussed in the previous section. To provide an example of the calculations performed for the calculated ESALs, the values shown in Table 4.9 were computed for section N1 of the 2003 Test Track using the average axle weights shown in Table 4.7.

Table 4. 9 Section N1 (2003 Test Track) Traffic Calculation Results

β_x			
Truck	Steer	Tandem	Single
Triple	0.493	1.051	1.106
Box	0.518	0.757	--
$\log(W_{tx}/W_{t18})$			
Truck	Steer	Tandem	Single
Triple	0.719	-0.299	-0.197
Box	0.603	-0.039	--
ESALs/Axle			
Truck	Steer	Tandem	Single
Triple	0.19	1.99	1.57
Box	0.25	1.09	--

Detailed data were collected for the amount of traffic applied at the test track. For each day that trucks were driving during the test cycle, the total amount of steer, single, and tandem axle passes were recorded for both the triple and box trailers, and a sample of those data are shown in Figure 4.10. To calculate the ESALs applied by each axle, these axle passes were multiplied by their respective ESALs/Axle factors shown in Table 4.9 to obtain the total ESALs per day for each axle type. The ESALs were summed across all axle types to obtain the total amount of ESALs applied per day, and these were summed cumulatively to get the total ESALs applied up to a certain day in the test cycle, as shown the "Cumulative Sum" column in Figure 4.10.

Date	Truck Type	Sum of Steer Axle Passes	Sum of Single Axle Passes	Tandem Axle Passes	Steer ESALs	Single ESALs	Tandem ESALs	Cumulative Sum
1/8/2004	Triple	1617	8084	1617	309	12712	3219	546593
1/9/2004	Triple	1568	7839	1568	299	12327	3121	562340
1/10/2004	Triple	1564	7822	1564	299	12300	3114	578053
1/11/2004	Triple	592	2962	592	113	4658	1179	584004
1/12/2004	Box	3	0	6	1	0	6	584011
1/12/2004	Triple	259	1295	259	49	2036	515	586611
1/13/2004	Box	364	0	728	91	0	797	587499
1/13/2004	Triple	1513	7563	1513	289	11893	3011	602692
1/14/2004	Box	388	0	775	97	0	849	603637
1/14/2004	Triple	1576	7879	1576	301	12390	3137	619464
1/15/2004	Box	390	0	779	97	0	853	620415
1/15/2004	Triple	1584	7920	1584	302	12454	3153	636324
1/16/2004	Box	393	0	785	98	0	860	637282
1/16/2004	Triple	1573	7864	1573	300	12366	3131	653079
1/17/2004	Box	383	0	766	96	0	838	654013
1/17/2004	Triple	1549	7744	1549	296	12177	3083	669569
1/19/2004	Triple	181	903	181	34	1420	360	671383
1/20/2004	Box	391	0	782	98	0	856	672337
1/20/2004	Triple	1572	7862	1572	300	12362	3130	688129
1/21/2004	Box	396	0	791	99	0	866	689094

Figure 4. 10 Sample of Detailed Axle Data.

Using these axle data, the total amount of applied ESALs could easily be obtained for various dates. Using section N1 from the 2003 Test Track as an example, the calculated ESALs shown in Table 4.10 were recorded for the same dates that the ESALs were predicted for in Table 4.4. It is important to note that the calculated ESALs shown in Table 4.10 are the result of not only the number of axle passes applied up to that date in the cycle, but also from a unique set of ESAL/axle factors for each date. This is because each date represents a different p_t value and, consequently, a different set of ESAL factors for each axle type.

Table 4. 10 Calculated ESALs for Section N1 (2003 Test Track)

Date	Calculated ESALs
5/24/2004	2,267,922
7/01/2004	2,837,091
8/17/2004	2,963,064
10/11/2004	3,212,141
12/13/2004	4,321,771

Regression

When comparing the predicted and calculated ESALs for section N1 of the 2003 Test Track, it is apparent that there are some substantial differences when using 0.44 for the HMA layer coefficient. Table 4.11 shows the calculated and predicted ESALs for section N1 of the 2003 Test Track, as well as the difference between the two. In general, the current structural coefficient (0.44) results in gross underpredictions of the ESAL-capacity of the pavement structure. This is likely due to the newer and more advanced HMA materials that are used at the track, which implies that the layer coefficient should be higher. The large error percentages show a need to bring these values closer together for a more realistic ESAL prediction, and a least squares regression was performed to accomplish that task.

Table 4. 11 ESAL Differences Assuming $a_1 = 0.44$ for Section N1 (2003 Test Track)

Predicted ESALs	Calculated ESALs	Difference	% Error
802,367	2,267,922	1,465,555	-65%
1,126,574	2,837,091	1,710,517	-60%
1,270,712	2,963,064	1,692,352	-57%
1,638,661	3,212,141	1,573,480	-49%
2,340,290	4,321,771	1,981,481	-46%

To perform the regression, first the differences between calculated and predicted were squared. These values were summed to obtain the error sum of squares (*SSE*), which is defined by the following equation:

$$SSE = \sum_i (y_i - \hat{y}_i)^2 \tag{4.7}$$

The mean was obtained for the calculated ESALs, and the difference between that mean and each predicted ESAL level was taken and then squared. These values were summed to obtain the total sum of squares (*SST*), as seen in the following equation:

$$SST = \sum_i (y_i - \bar{y})^2 \tag{4.8}$$

The Pearson's coefficient of determination (R^2) was calculated from these values to determine the goodness of fit using the following formula:

$$R^2 = 1 - \frac{SSE}{SST} \tag{4.9}$$

To perform the regression, Microsoft Excel Solver was utilized. The solver was set to minimize the *SSE* term while only changing the HMA layer coefficient (a_1). This process is inherently iterative in nature: every time the layer coefficient changes (i.e. from 0.44 to a new regressed value), both the calculated and predicted ESALs change. This is because both of these values are calculated using the structural number (*SN*), and that is calculated using the layer coefficient (a_1).

RESULTS AND DISCUSSION

Regressed Layer Coefficients

The regression procedure was performed for each test section to determine the new HMA layer coefficients. Table 4.12 shows the regression statistics for section N1 of the 2003 Test Track. This regression resulted in a HMA layer coefficient of 0.50. There is a noticeable improvement in the calculated and predicted ESAL differences after the regression procedure. Regression statistics for all other sections (other than those that did not display a change in serviceability) can be found in Appendix B.

Although relatively large percent errors are observed between the calculated and predicted ESALs, especially at the lowest ESAL level, this is not a significant cause of concern. In fact, it is expected that there could be considerable differences due to the log effect of ESALs within the AASHTO design system. As discussed in Chapter 2, taking the log of 5 million and 6 million results in very similar values (6.70 and 6.78, respectively). Therefore, it is not surprising that some relatively large percent errors were found in the regression procedure. The main purpose in the regression was to minimize the errors, but it was not expected to eliminate them. Additionally, comparing the results from Table 4.12 (recalibrated) to Table 4.11 (assumed $a_1 = 0.44$) shows considerable improvement in the ESAL predictions.

Table 4. 12 Regression Statistics for Section N1 (2003 Test Track)

Predicted ESALs	Calculated ESALs	Difference	% Error	$Diff^2$	$STDiff^2$
1,314,680	2,224,691	910,012	41%	8.28E+11	3.21E+12
2,007,491	2,806,554	799,065	28%	6.39E+11	1.21E+12
2,332,763	2,939,906	607,145	21%	3.69E+11	5.98E+11
3,203,489	3,207,147	3,661	0%	1.34E+07	9.44E+09
4,996,650	4,353,456	643,194	15%	4.14E+11	3.57E+12
Average	3,106,351		*Sum*	2.25E+12	8.60E+12
				R^2	0.738

For sections that did not exhibit considerable damage at the end of the Test Track cycle, an artificial terminal serviceability was selected for use in the regression procedure. These values were always equal to or lower than the actual serviceability level of the test section, and therefore provide conservative estimates of the layer coefficient. For example, in section N3 of the 2003 and 2006 Test Track cycles (refer to Figure 4.7), the final serviceability value measured was 4.27. The initial reading was 4.36, resulting in a *ΔPSI* value of only 0.09. In cases such as this, the solver function was used to create a difference in calculated and predicted ESALs of zero by only changing the layer coefficient. For each section where artificial terminal serviceabilities were used, the first p_t used was the actual terminal level found in the section. If the solver could not converge on a solution for the layer coefficient, then the p_t was decreased by one tenth and then the process was repeated until the solver converged on an answer. In the case of N3, an artificial terminal level of 3.9 was assigned to create a large enough *ΔPSI* for convergence to a solution for the layer coefficient, which resulted in a layer coefficient of 0.62. Figure 4.11 shows the calculated versus predicted ESALs for each test section after the regression was performed. The data in Figure 4.11 illustrate section-specific regression results.

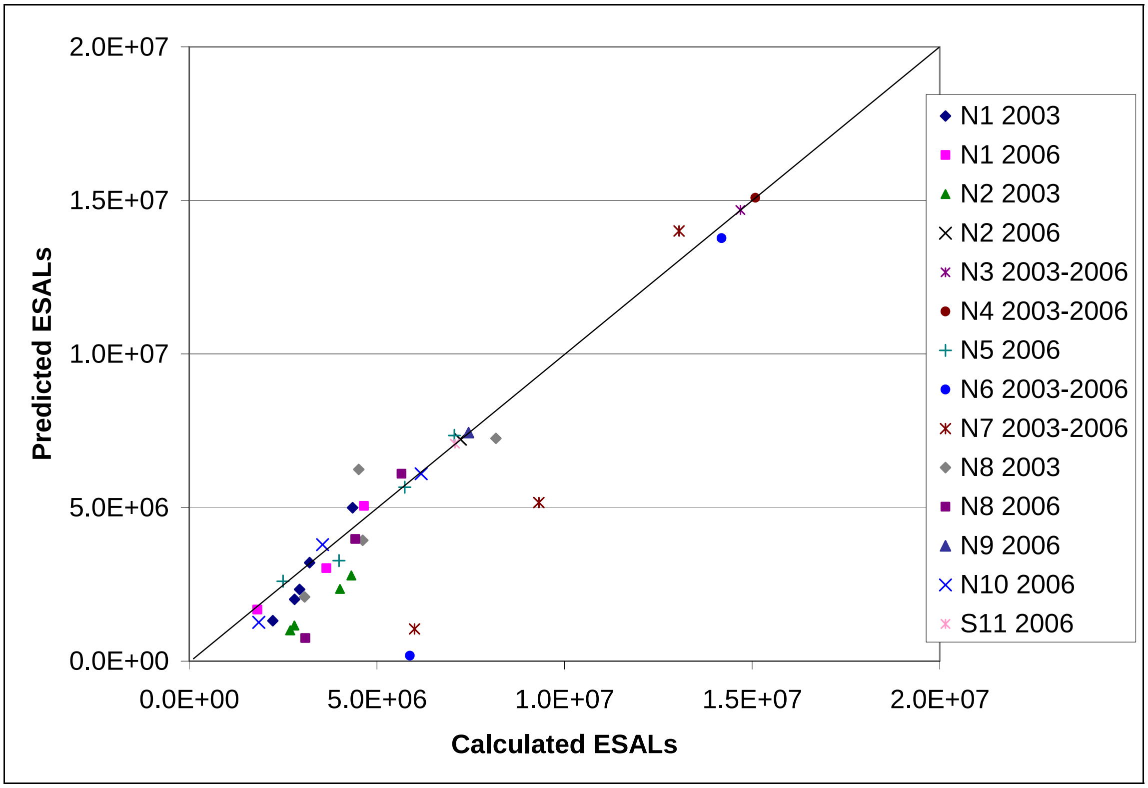

Figure 4. 11 Calculated vs. Predicted ESALs.

Table 4.13 provides a summary of the structural coefficients found for each section, as well as the R^2 values. If there is no associated R^2, then those sections were assigned artificial terminal serviceability levels as discussed previously.

Although it is apparent that the models do not describe 100% of the data used in this study, the coefficient of determination (R^2) values are still reasonably high considering the highly variable nature of research involving pavements and live traffic. Even the AASHO Road Test results had R^2 values that were not optimum; the overall average R^2 was approximately 0.85. Additionally, for those analyses, many more test sections (284) were used to arrive at a layer coefficient.

Table 4. 13 Regressed HMA Layer Coefficients

Section	Cycle Year	a_1	R^2
N1	2003	0.50	0.73
N1	2006	0.59	0.90
N2	2003	0.56	0.70
N2	2006	0.63	NA
N3	2003-2006	0.62	NA
N4	2003-2006	0.58	NA
N5	2006	0.48	0.95
N6	2003-2006	0.59	0.71
N7	2003-2006	0.58	0.61
N8	2003	0.43	0.68
N8	2006	0.48	0.64
N9	2006	0.44	NA
N10	2006	0.41	0.96
S11	2006	0.68	NA
	Average	0.54	0.76
	Std Dev	0.08	0.14

Figure 4.12 shows the individual section layer coefficients and the average layer coefficient graphically. As seen in Table 4.13 and Figure 4.12, most of the values found were higher than the current recommended value of 0.44. It is important to note that although the sections assigned artificial terminal serviceability levels do provide conservative estimates of the HMA layer coefficient, the pavements did not actually reach that level of serviceability in most cases. There were five sections that were assigned a terminal level of serviceability, and if these sections are not included in the analysis, the resulting average HMA layer coefficient is 0.51; slightly lower than the overall average value shown in Table 4.13.

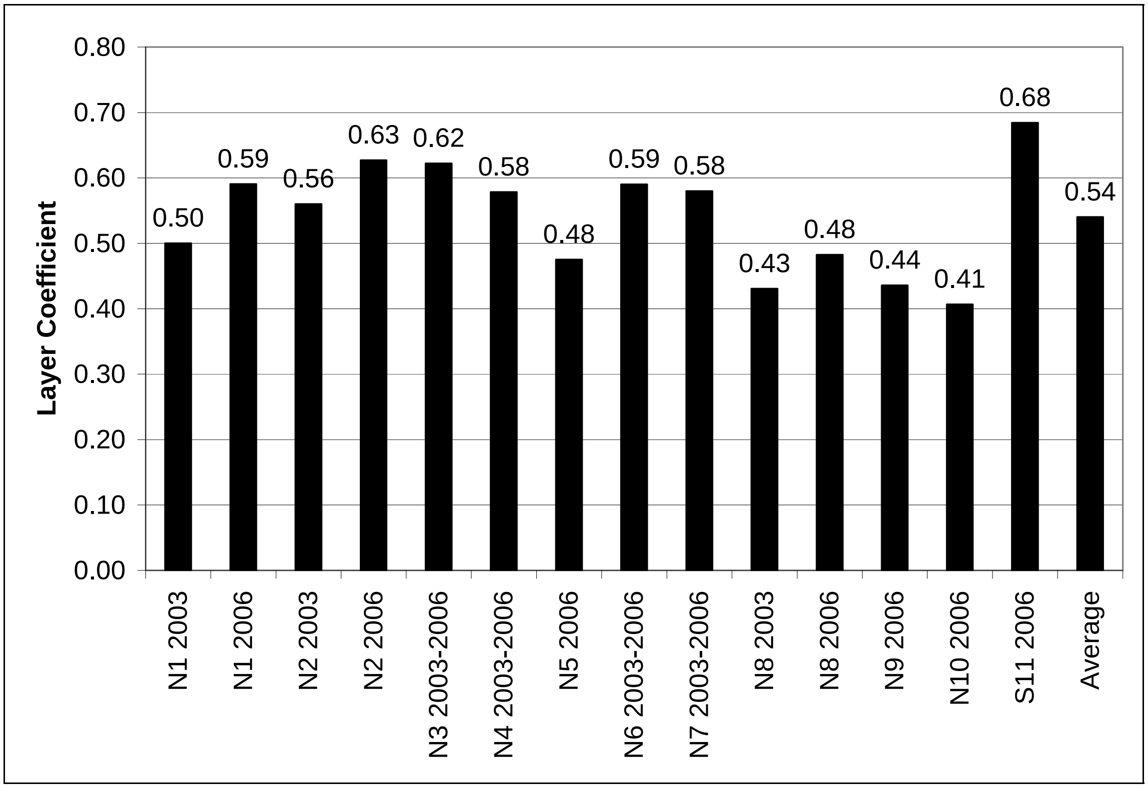

Figure 4. 12 Regressed Layer Coefficients.

There were two sections that resulted in coefficients lower than 0.44: N8 of the 2003 Track, and N10 of the 2006 Track. After the end of the 2003 Test Track cycle, section N8 showed considerable signs of fatigue cracking damage which was not expected given the pavement cross section and materials used. A forensic investigation of the section indicated that debonding had occurred between the HMA lifts, making the pavement much more prone to damage. A full report of the investigation of debonding in section N8 has been documented elsewhere (Willis and Timm, 2006). Section N10 of the 2006 Test Track also displayed more damage than expected at the end of the test cycle. Considerable surface distortion was present throughout the section, which warranted another forensic investigation. Trenches were cut in the section to view the damage throughout the cross section, and the individual HMA lifts were easily delaminated from one another using a backhoe. This phenomenon was not observed in any other sections that had trenches cut on the same day (January 29, 2009). While no reports have been officially published on this topic yet, the following pictures (Figures 4.13 and 4.14) illustrate the likelihood that debonding was occurring. Based on the published report for section N8 and the forensic photos of section N10, it is possible that the HMA layer coefficients found for these sections were lower than the recommended value of 0.44 due to their probable debonding issues.

Figure 4. 13 Trench Showing Signs of Debonding in Section N10.

Figure 4. 14 Trench Showing Delamination of HMA Lifts in Section N10.

After the regression was performed, the average value for the layer coefficient (0.54) was used to create a graph of calculated versus predicted ESALs with all test sections, and is shown in Figure 4.15. As seen in the figure, the data are fairly evenly distributed around the line of equality. A similar graph was also created using the recommended value of 0.44, and is shown in Figure 4.16. From Figure 4.16 it is apparent that using the recommended layer coefficient results in an under-prediction of calculated ESALs using the flexible pavement design equation for 89% of the sections at the Test Track, which would result in overly conservative designs.

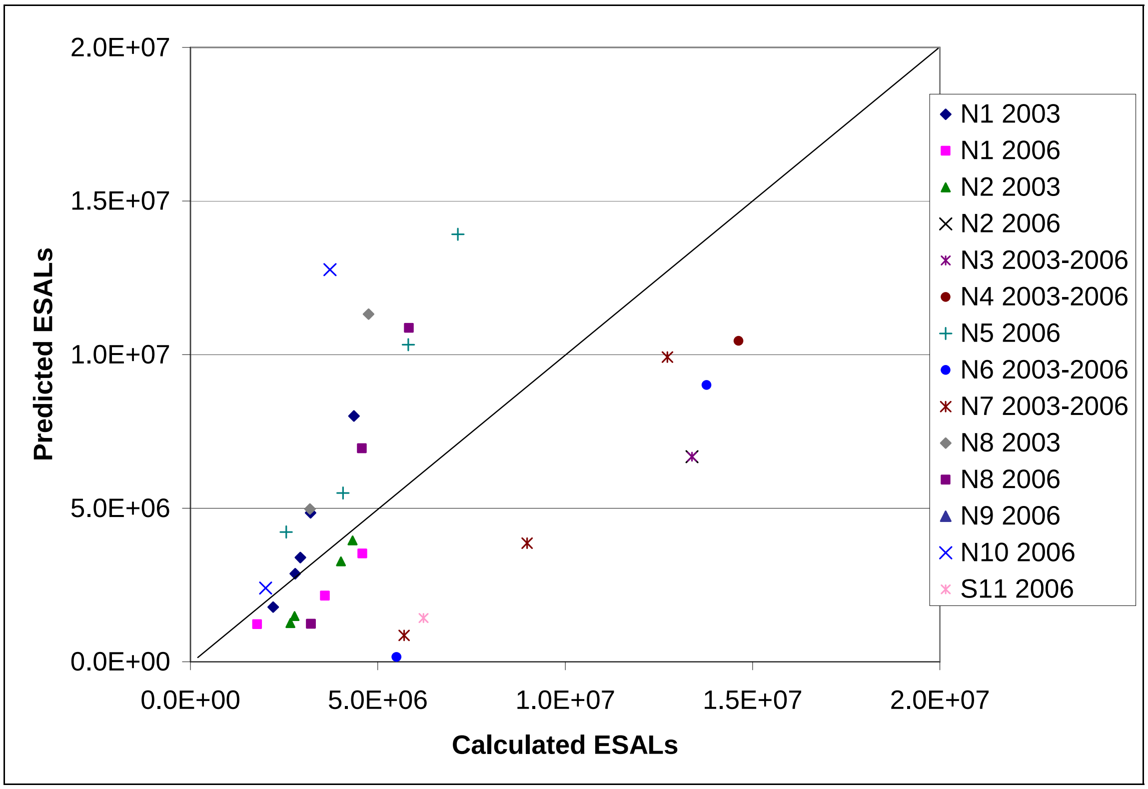

Figure 4. 15 Calculated vs. Predicted ESALs Using a_1 = 0.54.

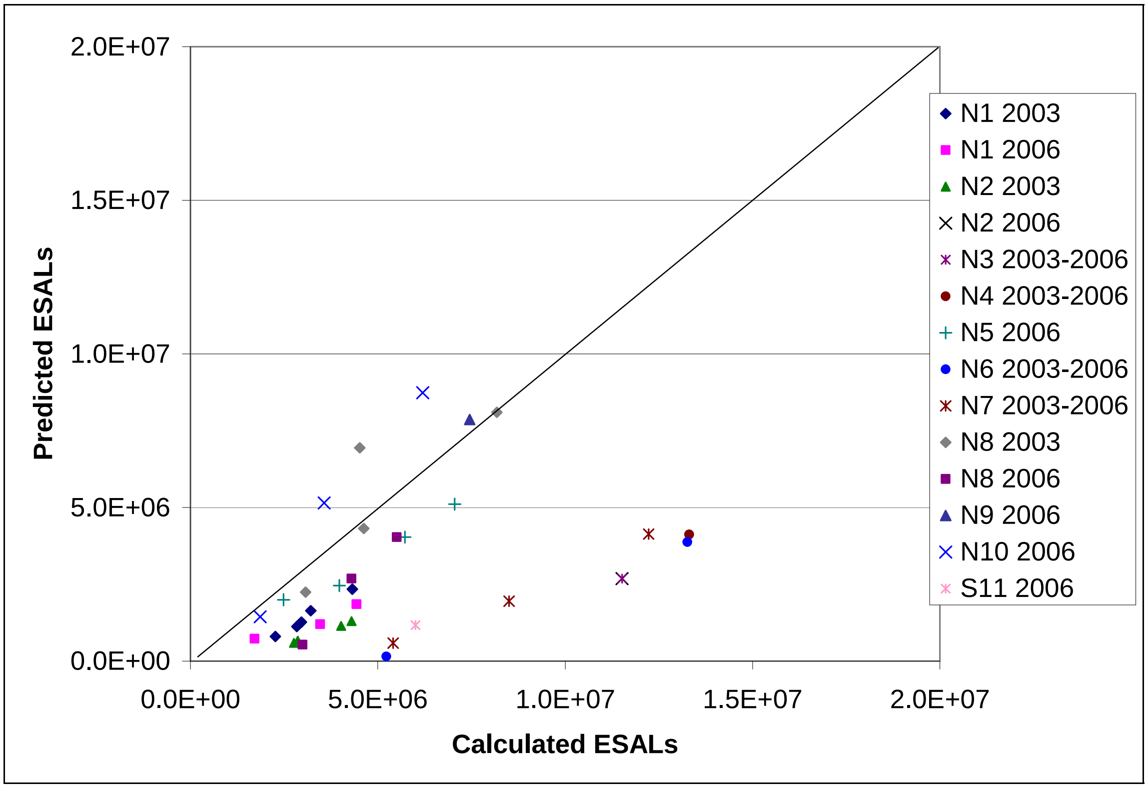

Figure 4. 16 Calculated vs. Predicted ESALs Using $a_1 = 0.44$.

Trends

No trends were apparent in the regression analysis. Pavement cross section, HMA thickness, and binder type all had no obvious effect on the resulting layer coefficient. For example, sections N1 and N2 from the 2003 Test Track both had very similar cross sections of 5 inches HMA over a 6 inch granular base. The resulting layer coefficients were 0.50 and 0.56, respectively. In the 2006 Test Track, the same sections were rebuilt with similar cross sections of 7 inches HMA over a 10 inch granular base, and the layer coefficients were 0.59 and 0.63, respectively. Other sections with similar cross sections and HMA layer thicknesses were compared, and no trend was found. Sections N3 and N4 of the 2006 Test Track both consisted of 9 inches of HMA, N3 being unmodified PG 67-22, and N4 being modified PG 76-22. The resulting layer coefficients were opposite of what was expected: 0.62 and 0.58, respectively. The highest layer coefficient was found for section S11, which consisted 8 inches of HMA: two upper layers of modified PG 76-22 binder over two layers of unmodified PG 64-22. There are several sections with thicker cross sections than S11, as well as higher PG grades. From these data and other similar comparisons, it was concluded that there were no trends found relative to overall cross section, HMA thickness, or binder type. This could be due to other factors that can affect the layer coefficient within the pavement structure. Debonding between HMA lifts likely caused lower layer coefficient values in sections N8 and N10 as discussed previously. Other factors such as compaction, binder content, air voids or pavement age might also have significant (and perhaps confounding) impacts on the differences in layer coefficients, which may be why no trends are apparent in the data. Conversely, it could be that a_1 is generally insensitive to these factors. This is evidenced by many states, ALDOT included, by using a single a_1 value in structural design for a wide variety of asphalt mixtures.

Impact of Changing Layer Coefficient

Changing the HMA layer coefficient from 0.44 to the average value of 0.54 found at the NCAT Test Track for flexible pavement designs would have a significant impact on the resulting HMA layer thickness. Figure 4.17 shows how this thickness would change over varying traffic levels for a given design (R of 50%, S_o of 0.40, M_R of 10,000 psi, ΔPSI of 2, D_2 of 6 inches, and a_2 of 0.14). At 1 million ESALs, the savings in thickness is approximately 0.7 inches; at 100 million ESALs, it is 1.7 inches. These values are only applicable to the design given. For example, a lower M_R value would result in a greater difference between using the two layer coefficients, and vice versa. Regardless of the other input values, changing to a layer coefficient of 0.54 from 0.44 would always result in a thinner pavement, and in an approximate savings of 18% in HMA thickness.

Minimum Thickness

It is important to note that the regressed layer coefficient of 0.54 was calibrated to sections with HMA thicknesses no less than 5 inches. Therefore, this coefficient should not be used for designs that result in pavements with a HMA thickness of less than 5 inches. It is recommended that for designs resulting in thicknesses of less than 5 inches, the AASHTO recommended coefficient of 0.44 be used, or the minimum thickness be set to 5 inches.

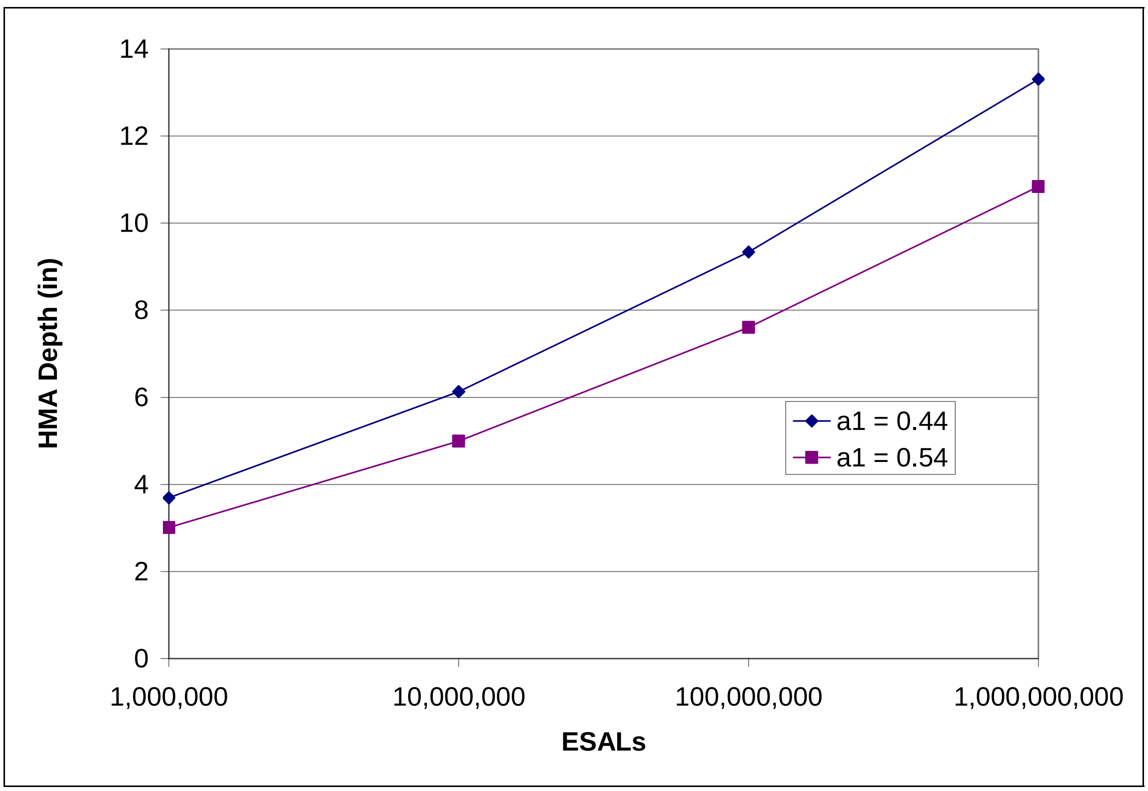

Figure 4. 17 Change in Resulting HMA Thickness from a_1 = 0.44 to a_1 = 0.54.

SUMMARY

Pavement performance and traffic data were collected over the course of the 2003 and 2006 Test Track cycles. These data were used to perform a recalibration of the HMA layer coefficient (a_1) that is used in the current ALDOT flexible pavement design methodology. The recalibration was achieved by calculating the predicted ESALs using the 1993 AASHTO Design Guide flexible pavement design equation, and also calculating the ESALs applied at the Test Track during the two test cycles. The pavement performance data (IRI measurements) were used to find a change in serviceability (ΔPSI) or a terminal serviceability level (p_t) for use in the design equations. Some sections did not exhibit an increase in roughness over the course of the Test Track cycle(s), and therefore were assigned artificial terminal serviceability levels to provide a ΔPSI term large enough for convergence to a layer coefficient. The values chosen were always less than or equal to the actual final serviceability level at the end of the cycle, and therefore are conservative. The predicted and calculated ESALs were compared, and a least squares regression was performed to minimize the difference between the two values while only changing the HMA layer coefficient. This resulted in an average layer coefficient of 0.54 with a standard deviation of 0.08 for all the test sections analyzed. If the sections assigned a terminal serviceability level were not included in the analysis, the average layer coefficient was 0.51. These values can be compared to the current value used for the HMA layer coefficient of 0.44, which comes from the AASHO Road Test. There were no trends in the data regarding the pavement cross section, HMA thickness, or binder type. The impact of changing the layer coefficient to the average value found in this analysis would result in a reduced HMA thickness, regardless of design. The amount of reduced thickness is dependent upon other design inputs such as resilient modulus, traffic, etc.

CHAPTER 5 - CONCLUSIONS AND RECOMMENDATIONS

SUMMARY

This investigation was performed to determine the relative sensitivity of HMA thickness to the inputs of the AASHTO Design Guide flexible pavement design equation, and to recalibrate the HMA layer coefficient used for flexible pavement designs. For the recalibration procedure, traffic and performance data from the 2003 and 2006 NCAT Test Track cycles were used in conjunction with flexible pavement design and traffic equations developed from the AASHO Road Test. Calculated and predicted ESALs were computed and compared, and a least squares regression was performed to determine new layer coefficients for each test section.

CONCLUSIONS

From the literature review, it was apparent that research was needed to determine the relative sensitivity of the inputs to the AASHTO flexible pavement design equation since no literature could be found on this topic. It was also concluded that based upon the limited parameters of the AASHO Road Test, the recommended default HMA layer coefficient of 0.44 should be reanalyzed to ensure accuracy for current materials. While several studies have been conducted to determine the layer coefficients of new materials, there is no recommended tried-and-true procedure that all researchers can agree upon. In addition, the results found from many layer coefficient studies tended to be highly variable, and a specific layer coefficient was typically not recommended for a particular material.

The input sensitivity analysis showed that the layer coefficient, resilient modulus, and traffic are by far the most influential parameters on the resulting HMA thickness. Since the resilient modulus and traffic are typically given parameters for a particular pavement design, it was concluded that an accurate characterization of the layer coefficient is extremely important. Input dependencies were found in the sensitivity analysis; all inputs had an increasing influence on the resulting HMA thickness as the traffic level increased.

The recalibration procedure resulted in an average HMA layer coefficient of 0.54 with a standard deviation of 0.08 from the 14 pavement sections studied. Five sections that did not exhibit considerable deterioration were assigned artificially low, yet conservative, terminal serviceability levels to obtain a ΔPSI term large enough for convergence to a layer coefficient. If these sections are not included in the calculations, the result is an average layer coefficient of 0.51. The only two test sections that resulted in regressed layer coefficients lower than the AASHTO recommended value of 0.44 had probable slippage failures, as found in forensic investigations. No trends were observed in the resulting layer coefficients when comparing binder type, HMA layer thickness, or overall pavement cross section. The impact of changing the layer coefficient to the average value found in this analysis would result in a reduced design HMA thickness, regardless of the values of other inputs to the design.

RECOMMENDATIONS

Based upon the results of this investigation, it is recommended that ALDOT use a layer coefficient of 0.54 for flexible pavement designs using the AASHTO design methodology. This coefficient is larger than the recommended value of 0.44; however, it was expected to increase due to the numerous advancements in HMA materials and construction since the AASHO Road Test was completed in 1961. Using a layer coefficient of 0.54 would result in a HMA thickness savings of approximately 18%. This coefficient should not be used for designs that result in pavements with a HMA thickness of less than 5 inches. It is recommended that for designs resulting in thicknesses of less than 5 inches, the AASHTO recommended coefficient of 0.44 be used, or the minimum thickness be set to 5 inches. Care should be taken when applying the new regressed coefficient to other states. The layer coefficient of 0.54 is the result of the environmental conditions and materials used in this study.

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APPENDIX A

PSI VERSUS TIME GRAPHS FOR EACH TEST SECTION

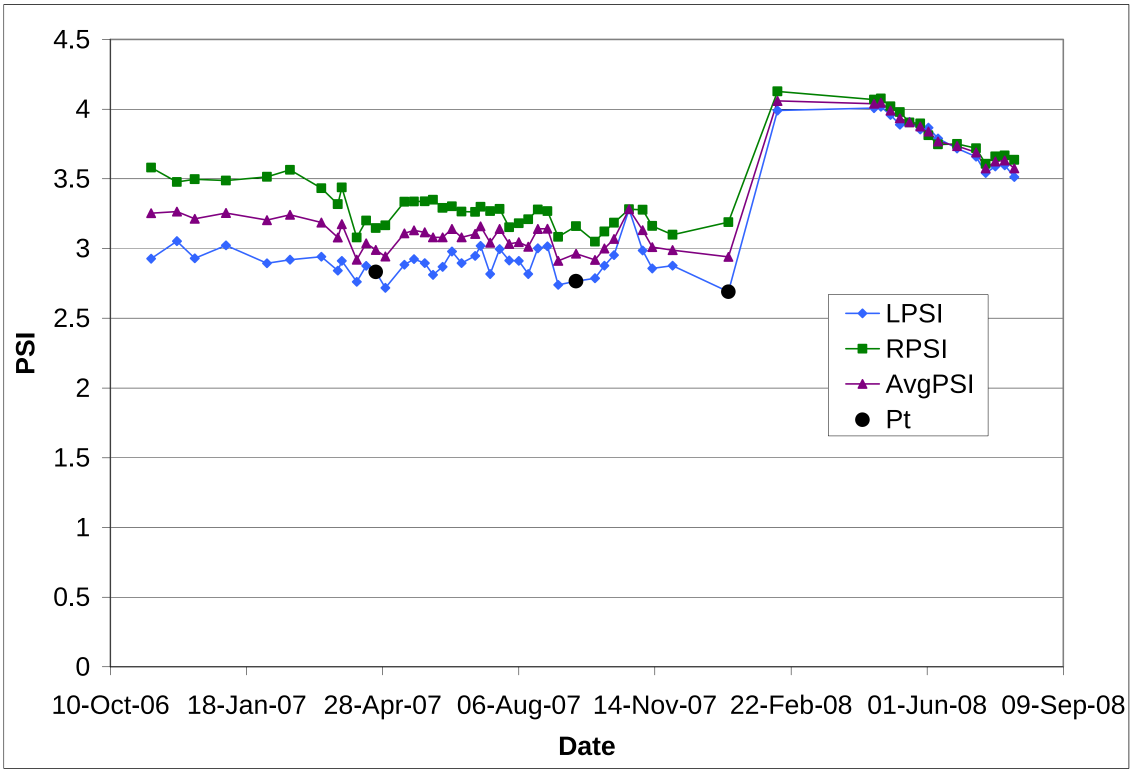

Figure A1: PSI Data from Section N1 of the 2006 Test Track.

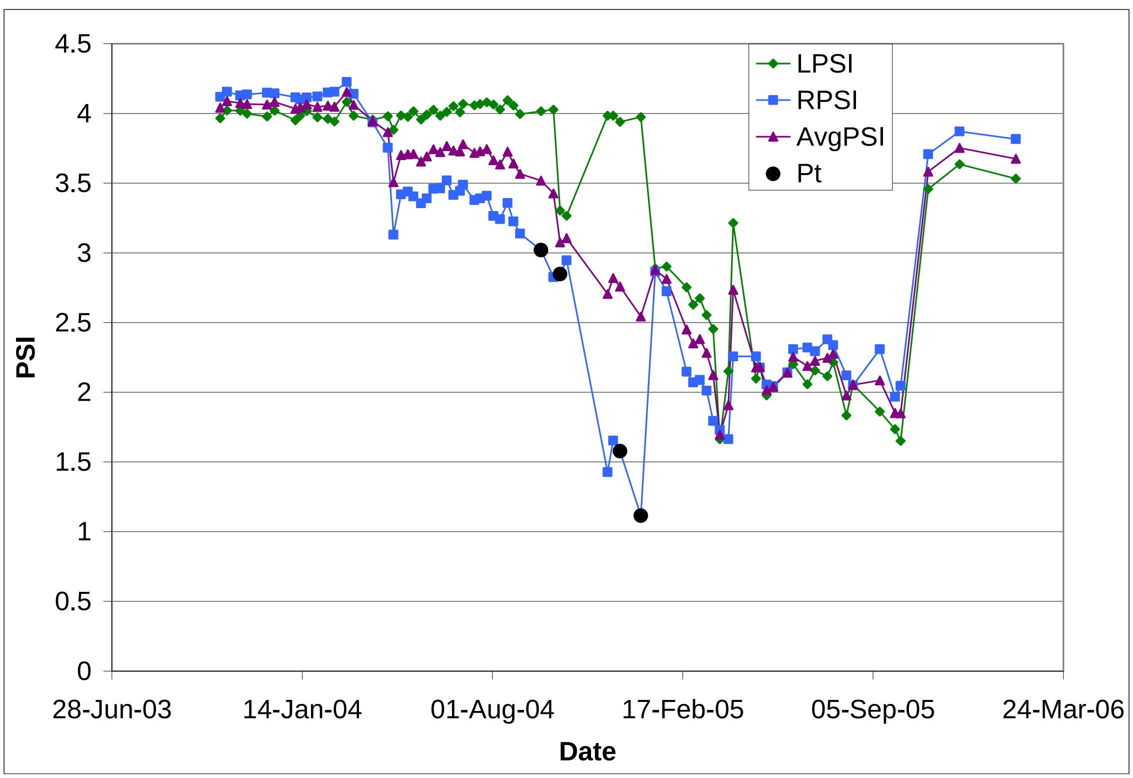

Figure A2: PSI Data from Section N2 of the 2003 Test Track.

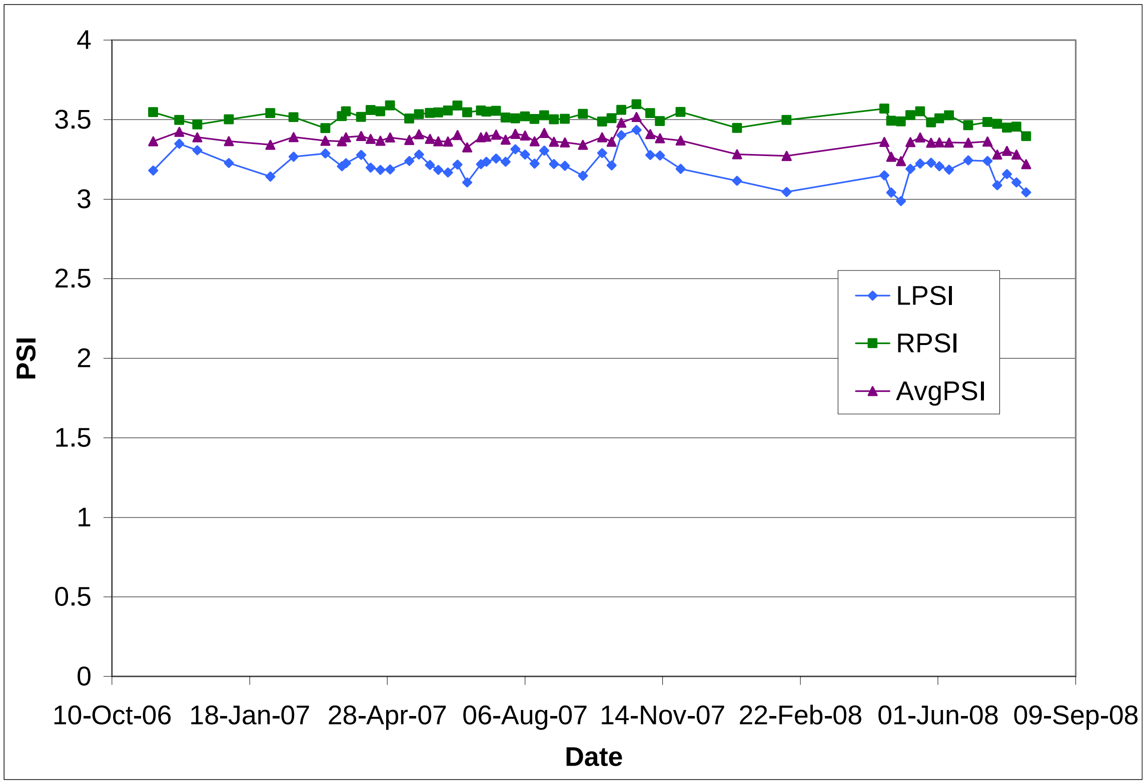

Figure A3: PSI Data from Section N2 of the 2006 Test Track.†

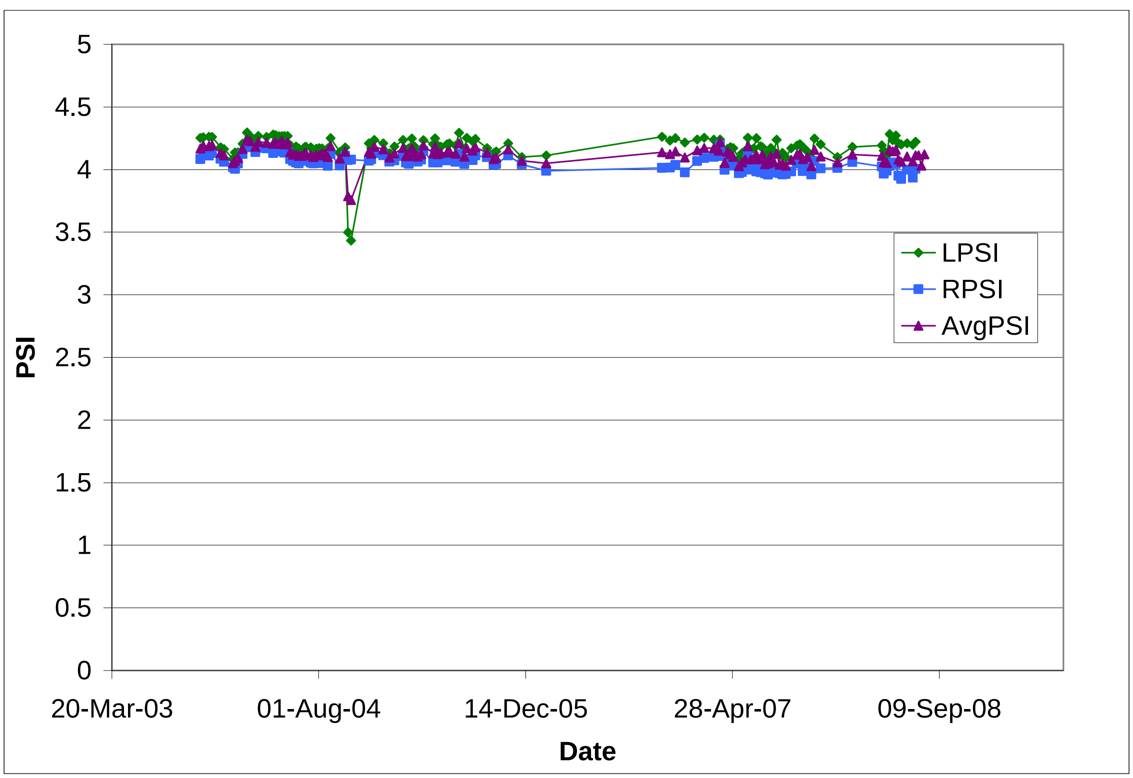

Figure A4: PSI Data from Section N4 of the 2003 and 2006 Test Track Cycles. †

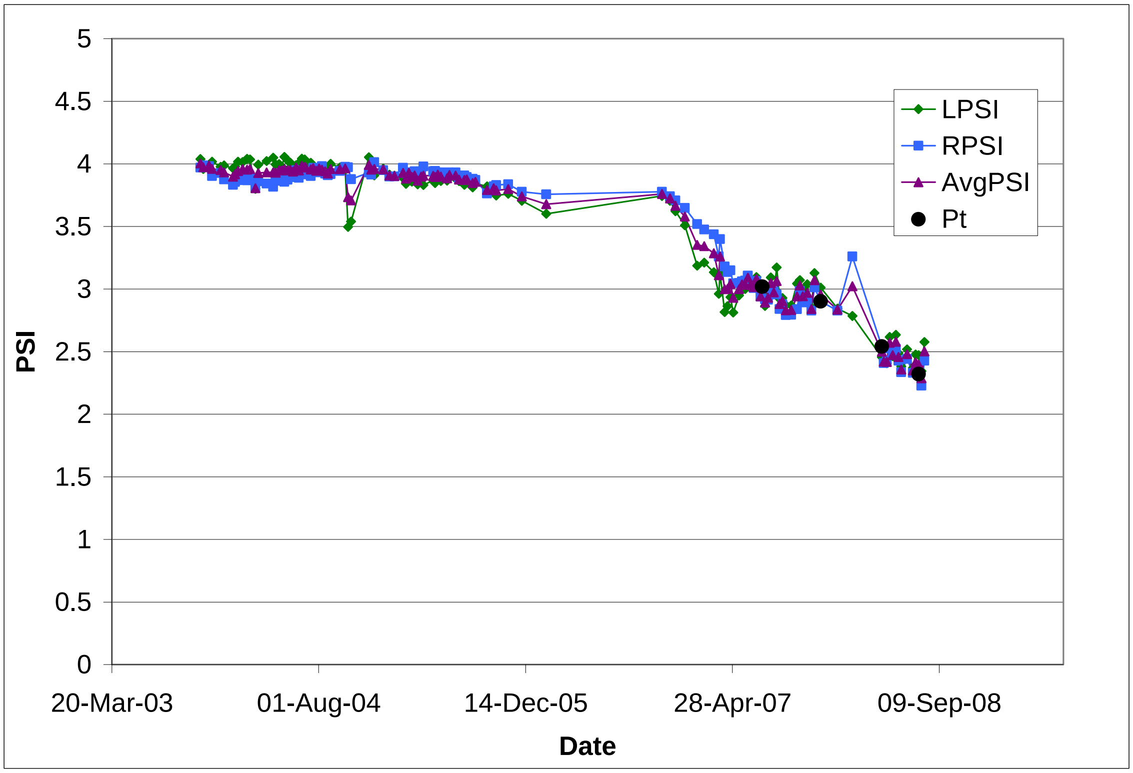

Figure A5: PSI Data from Section N5 of the 2006 Test Track.

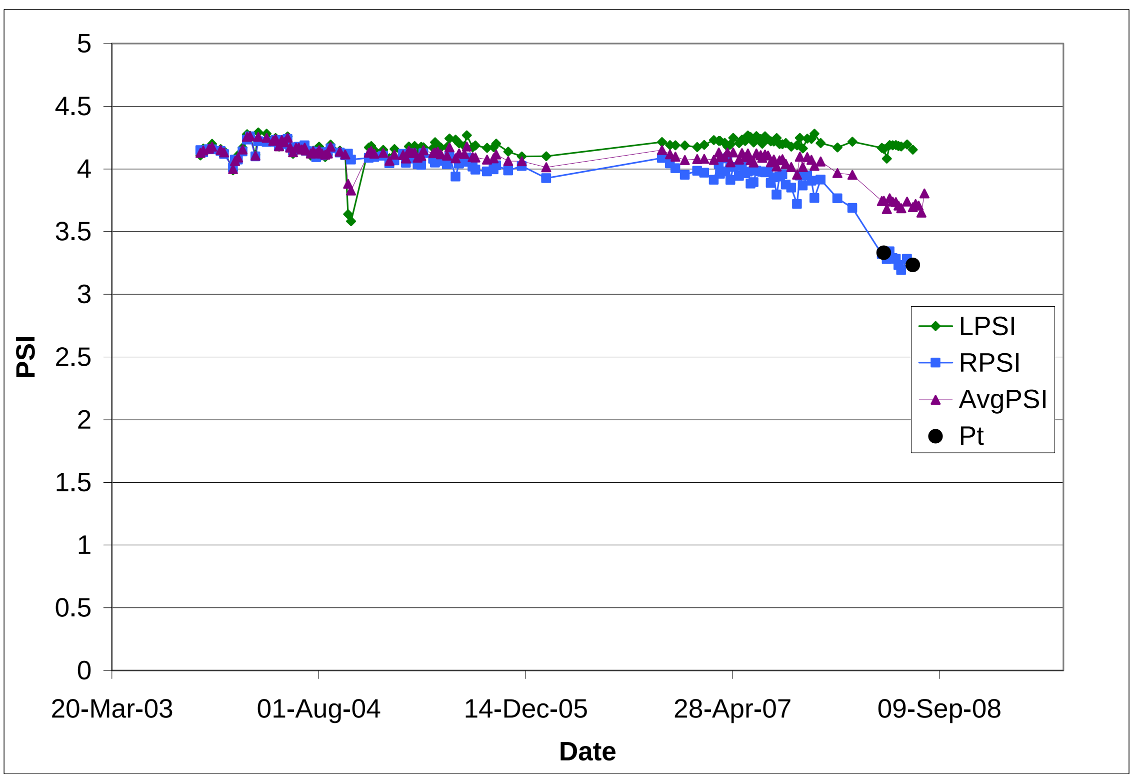

Figure A6: PSI Data from Section N6 of the 2003 and 2006 Test Track Cycles.

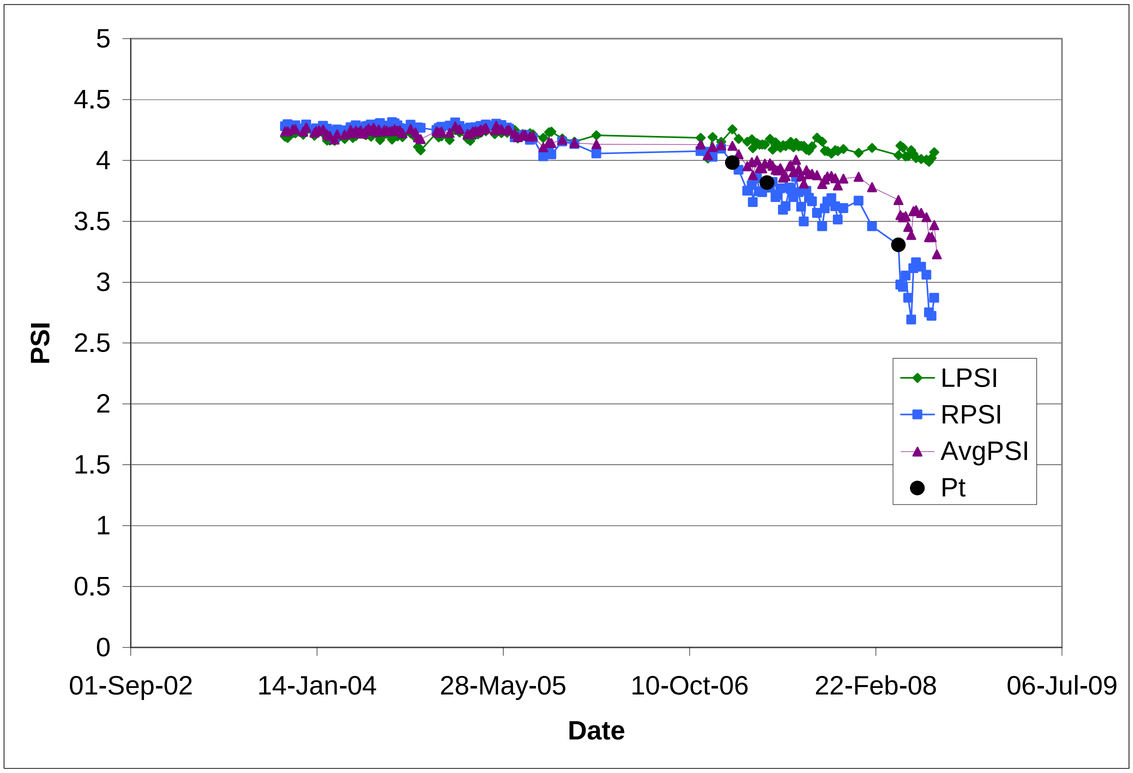

Figure A7: PSI Data from Section N7 of the 2003 and 2006 Test Track Cycles.

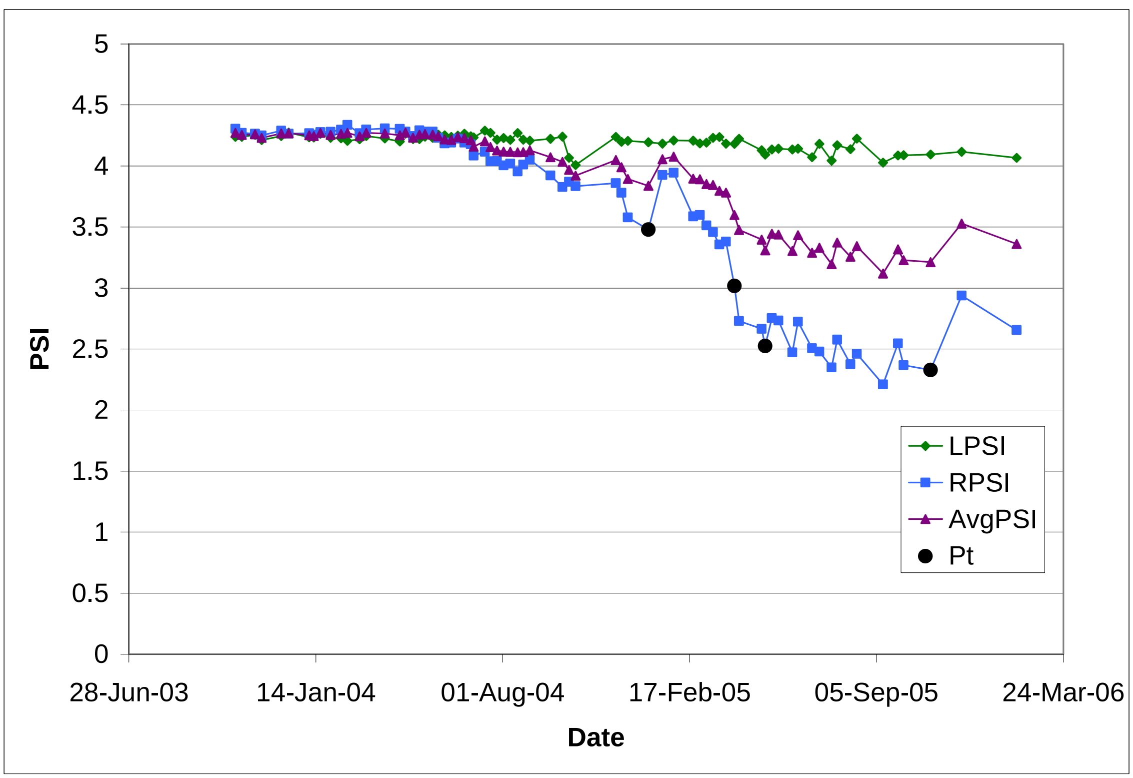

Figure A8: PSI Data from Section N8 of the 2003 Test Track.

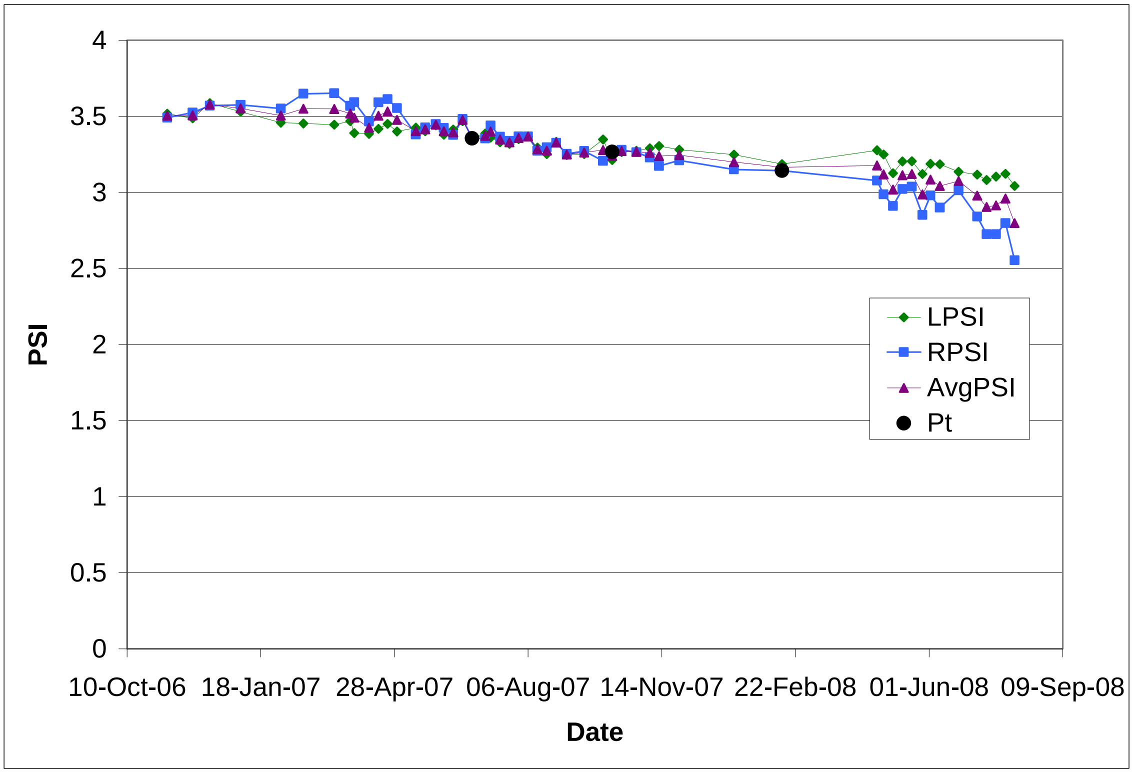

Figure A9: PSI Data from Section N8 of the 2006 Test Track.

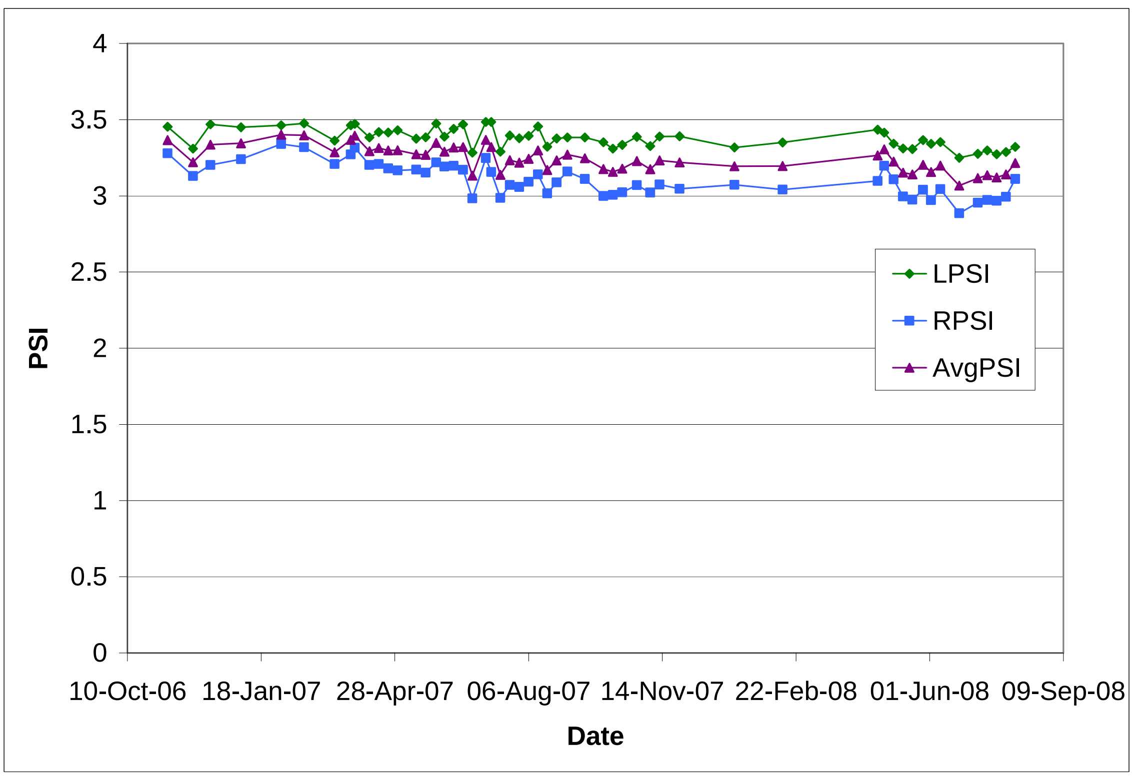

Figure A10: PSI Data from Section N9 of the 2006 Test Track. †

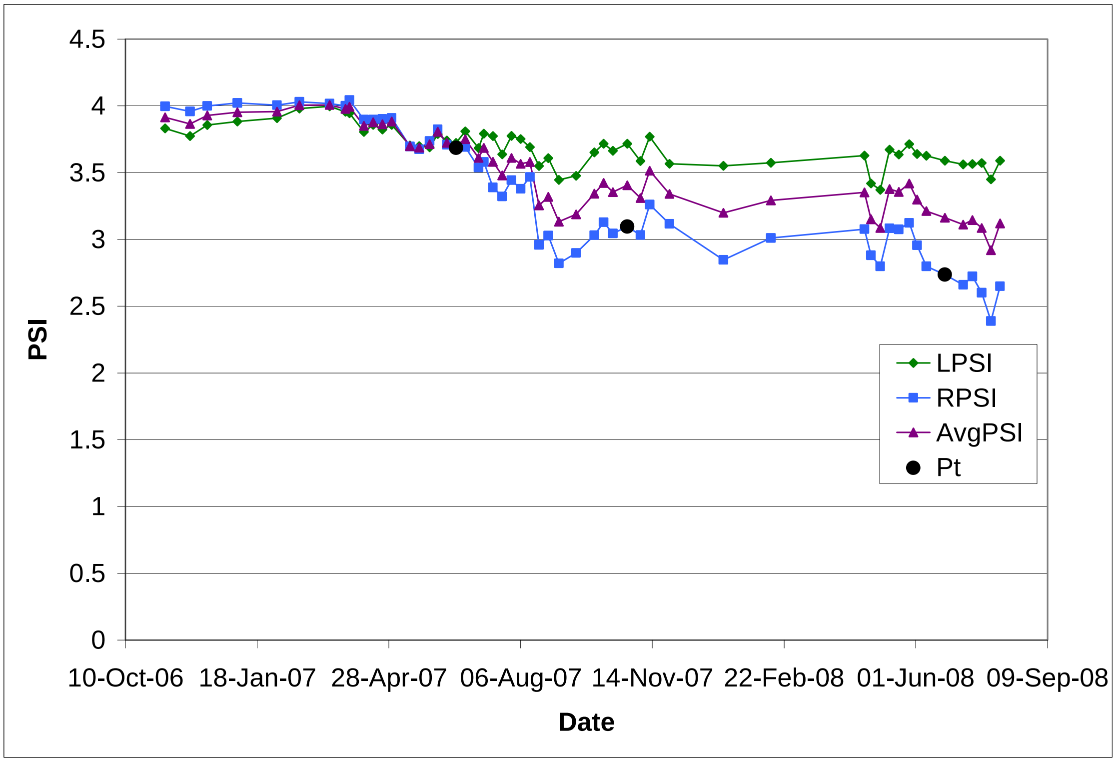

Figure A11: PSI Data from Section N10 of the 2006 Test Track.

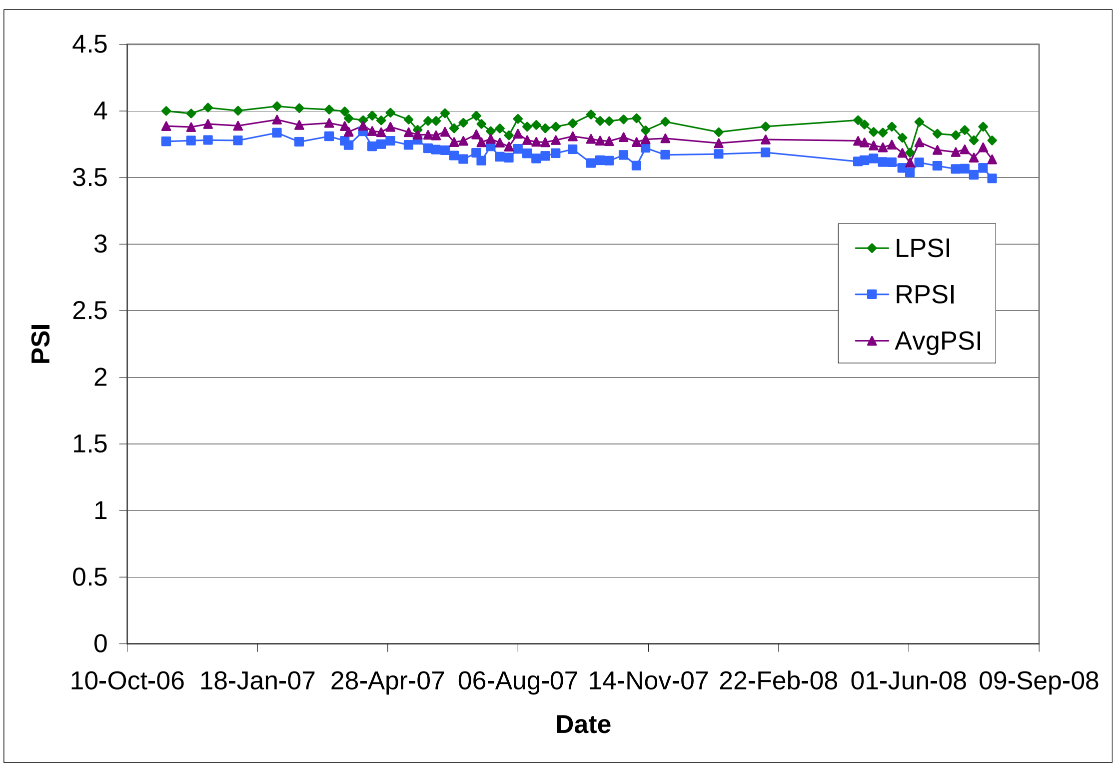

Figure A12: PSI Data from Section S11 of the 2006 Test Track. †

† Point selection not possible. No visible decrease in serviceability.

APPENDIX B

REGRESSION STATISTICS FOR EACH TEST SECTION

Table B1 Regression Statistics for Section N1 of the 2006 Test Track

Predicted ESALs	Calculated ESALs	Difference	% Error	$Diff^2$	$STDiff^2$
1,814,085	1,676,044	138042	8%	1.91E+10	2.89E+12
3,653,019	3,021,544	631475	17%	3.99E+11	1.25E+11
4,658,149	5,051,502	393354	8%	1.55E+11	2.81E+12
Average	3,375,084		*Sum*	5.73E+11	5.82E+12
				R^2	0.902

Table B2 Regression Statistics for Section N2 of the 2003 Test Track

Predicted ESALs	Calculated ESALs	Difference	% Error	$Diff^2$	$STDiff^2$
2,653,268	1,452,211	1201058	45%	1.44E+12	3.96E+12
2,767,858	1,730,741	1037117	37%	1.08E+12	2.93E+12
4,014,443	3,985,550	28893	1%	8.35E+08	2.96E+11
4,329,912	4,874,551	544639	13%	2.97E+11	2.05E+12
Average	3,441,370		*Sum*	2.82E+12	9.23E+12
				R^2	0.700

Table B3 Regression Statistics for Section N5 of the 2006 Test Track

Predicted ESALs	Calculated ESALs	Difference	% Error	$Diff^2$	$STDiff^2$
2,498,192	2,598,918	100726	4%	1.01E+10	4.95E+12
3,992,174	3,267,565	724610	18%	5.25E+11	2.42E+12
5,742,145	5,662,943	79202	1%	6.27E+09	7.03E+11
7,066,598	7,348,984	282386	4%	7.97E+10	6.37E+12
Average	4,824,777		*Sum*	6.21E+11	1.45E+13
				R^2	0.957

Table B4 Regression Statistics for Section N6 of the 2003 and 2006 Test Tracks

Predicted ESALs	Calculated ESALs	Difference	% Error	$Diff^2$	$STDiff^2$
5,880,826	176,433	5704393	97%	3.25E+13	9.72E+13
14,187,307	13,768,047	419260	3%	1.76E+11	1.39E+13
Average	10,034,067		*Sum*	3.27E+13	1.11E+14
				R^2	0.706

Table B5 Regression Statistics for Section N7 of the 2003 and 2006 Test Tracks

Predicted ESALs	Calculated ESALs	Difference	% Error	$Diff^2$	$STDiff^2$
6,004,943	1,044,458	4960485	83%	2.46E+13	7.08E+13
9,317,670	5,160,057	4157613	45%	1.73E+13	1.85E+13
13,051,495	14,008,131	956635	7%	9.15E+11	2.07E+13
Average	9,458,036		*Sum*	4.28E+13	1.10E+14
				R^2	0.611

Table B6 Regression Statistics for Section N8 of the 2003 Test Track

Predicted ESALs	Calculated ESALs	Difference	% Error	$Diff^2$	$STDiff^2$
3,072,062	2,084,428	987634	32%	9.75E+11	9.08E+12
4,625,663	3,934,268	691395	15%	4.78E+11	1.35E+12
4,519,510	6,243,857	1724347	38%	2.97E+12	1.31E+12
8,174,480	7,249,997	924482	11%	8.55E+11	4.63E+12
Average	5,097,929		*Sum*	5.28E+12	1.64E+13
				R^2	0.678

Table B7 Regression Statistics for Section N8 of the 2006 Test Track

Predicted ESALs	Calculated ESALs	Difference	% Error	$Diff^2$	$STDiff^2$
3,097,060	747,606	2349454	76%	5.52E+12	1.33E+13
4,427,676	3,972,332	455344	10%	2.07E+11	1.78E+11
5,657,763	6,096,567	438804	8%	1.93E+11	2.90E+12
Average	4,394,166		*Sum*	5.92E+12	1.64E+13
				R^2	0.638

Table B8 Regression Statistics for Section N10 of the 2006 Test Track

Predicted ESALs	Calculated ESALs	Difference	% Error	$Diff^2$	$STDiff^2$
1,853,463	1,261,542	591921	32%	3.50E+11	6.77E+12
3,553,293	3,793,560	240266	7%	5.77E+10	4.75E+09
6,180,789	6,099,393	81396	1%	6.63E+09	5.00E+12
Average	3,862,515		*Sum*	4.15E+11	1.18E+13
				R^2	0.965