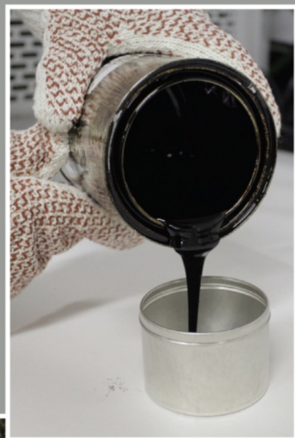
NCAT Report 14-08



RECALIBRATION PROCEDURES FOR THE STRUCTURAL ASPHALT LAYER COEFFICIENT IN THE 1993 AASHTO PAVEMENT DESIGN GUIDE

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1. INTRODUCTION

Pavement thickness design in the U.S. has been predominantly empirically-based since the 1960's. The American Association of State Highway and Transportation Officials (AASHTO) pavement design guides published from 1962 through 1993 (*1,2*) were based primarily on the AASHO Road Test (*1*) conducted in Ottawa, Illinois from 1958 until 1960. A more recent edition of the AASHTO Guide was published in 1998 but focused primarily on improving rigid pavement design and is outside the scope of this document. Though updated and improved over time, the design guides still rely heavily upon observed pavement performance during the road test. The performance resulted from the cross-sections, climate, materials, construction practices and traffic applications representing late 1950's conditions and technology at this one test location. For example, the thickest asphalt section placed at the AASHO Road Test was 6 inches. Furthermore, the advances in pavement engineering, design, materials and construction fields over the past 52 years has made the AASHTO Design Guide (*2*) more outdated with every passing year, forcing designers to extrapolate well beyond the original conditions of the road test. These advances include the development of the Superpave asphalt mix design procedures, the development of the performance graded (PG) asphalt binder specification, the use of polymers and other modifiers in asphalt, improved asphalt plant production controls, improved construction techniques and quality control procedures, to name just a few.

As documented previously (*3*), the National Cooperative Highway Research Program (NCHRP) recognized the need for an improved and updated pavement design system and began Project 1-37A in 1998 entitled, "Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures: Phase I." The project ran through 2004 and resulted in the Mechanistic Empirical Pavement Design Guide (MEPDG). In 2008, the MEPDG was transitioned to the AASHTOWare series of programs and was renamed DARWin-ME as the program developers continued to improve the program's capabilities. In 2013, the software became commercially available under the name AASHTOWare™ Pavement ME Design. The software and accompanying documentation (*4*), represents a tremendous leap forward from the 1993 Design Guide (*2*) and software, DARWin.

Though the MEPDG is recognized as a technological advance in pavement design, there are costs associated with implementing the new procedure. The costs include software licensing and training, development of numerous data sets through laboratory and field testing required to run the software and validation/calibration studies that must be conducted before fully implementing the new procedure. These activities can also take significant amounts of time to accomplish. Currently, the older empirically-based design procedure is the most popular approach in the U.S. with 78% of states using some edition (i.e., 1972, 1986 or 1993 Design Guide) of the older empirical AASHTO procedure (*3,5*). A recent survey of state agencies, as summarized in Figure 1.1, indicated that many states plan to adopt the MEPDG, but only three have currently done so and fourteen expect to implement within the next two years (*5*). The other states are at least two years from implementing the MEPDG while six do not currently plan to implement (*5*). For states that have already begun working toward implementing the MEPDG, there are many data sets (i.e., traffic, material properties, performance records) that are common between the empirical and mechanistic-empirical approaches, so it would make sense to update the old method while implementing the new approach. Finally, given the complexities of the MEPDG and design software, there may be many design scenarios (e.g., facilities such as city streets, county roads, lower volume state routes) that simply do not warrant such a detailed analysis.

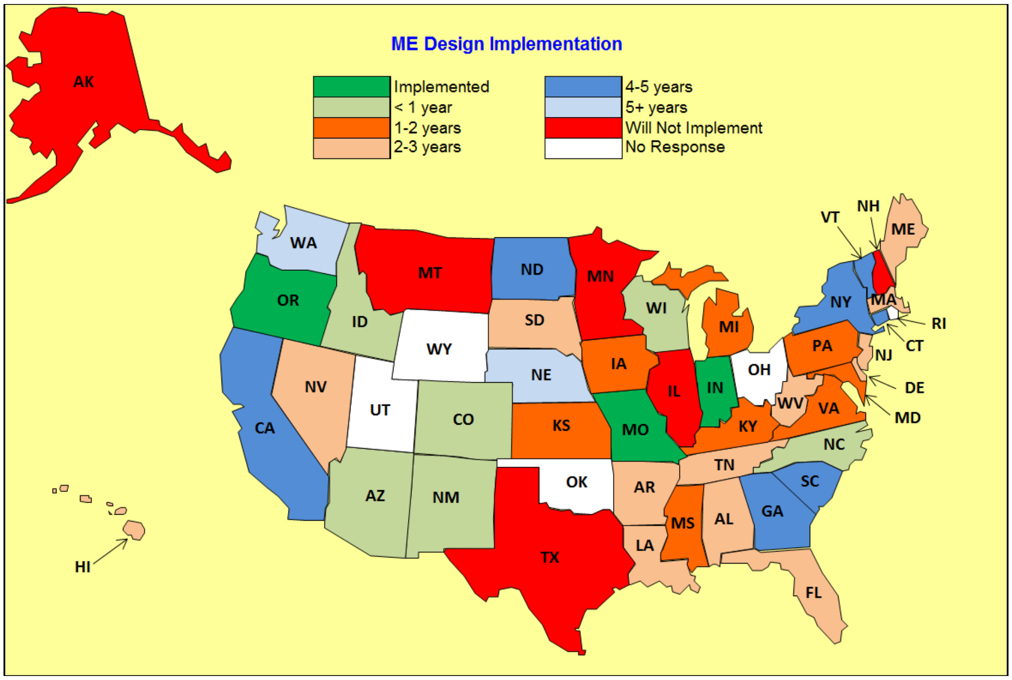


Figure 1.1 MEPDG and Design Software Implementation (*data from 5*).

Clearly, there is a gap between the outdated empirically-based procedure and the MEPDG that should be filled to achieve optimal pavement structural designs. The purpose of this document is to provide recommended procedures for updating the empirically-based design method to reflect modern pavement performance. As explained below, focus is placed on recalibrating the asphalt structural coefficient as it has the strongest correlation amongst all the design variables to pavement thickness. Further rationale for recalibrating the asphalt coefficient is that it was AASHTO's original intent that states develop agency-specific structural coefficients. As stated by George (*6*), "Because of wide variations in environment, traffic and construction practices, it is suggested that each design agency establish layer coefficients based on its own experience and applicable to its own practice."

2. OVERVIEW OF THE AASHTO EMPIRICAL DESIGN PROCEDURE

Before discussing methods for updating the AASHTO empirical design procedure, it is important to establish a firm understanding of the design process and how it was developed. Subsections 2.1 through 2.3 explain the process and its limitations and were excerpted from a previous report (*3*), while section 2.4 further explains the importance of the structural coefficient.

2.1 AASHTO Empirical Design Inputs

Observations from the AASHO Road Test established correlations between the following four main factors for flexible pavements:

- Soil condition as quantified by the subgrade resilient modulus (M_r)
- Traffic as quantified by equivalent single axle loads (ESALs)
- Change in pavement condition as quantified by the change in pavement serviceability index (ΔPSI)
- Pavement structure as quantified by a structural number (SN)

The soil resilient modulus describes the inherent ability of the soil to carry load and can be measured in the laboratory through triaxial resilient modulus testing or in the field through falling weight deflectometer (FWD) testing. Generally, lower M_r values will require more pavement thickness to carry the given traffic. The soil modulus during the AASHO road test was approximately 3,000 psi, and care should be taken when using the AASHTO empirical method to be sure M_r values obtained through modern means are adjusted to reflect test conditions (*1,2*). For example, AASHTO recommends dividing the soil modulus obtained through FWD testing by three before using in the empirical design equation (*2*). It is also important to emphasize that there was only one soil type used during the AASHO Road Test (*1*). Though there were seasonal fluctuations in the soil modulus from which empirical correlations between soil modulus and pavement condition were developed, they are strictly limited to that soil type.

The AASHO Road Test featured various test loops that were constructed of asphalt concrete thicknesses ranging from 1 to 6 inches and trafficked with different axle types and load levels (*1*). The researchers noted an approximate fourth-power relationship between the amount of pavement damage and the load level applied to the pavement section. This relationship was the central idea in the equivalent single axle load (ESAL), which was selected to be an 18,000-lb single axle with dual tires. AASHTO developed empirical equations to relate the number of applications of all other axle types (single, tandem and tridem) and load magnitudes to that of the ESAL. Figure 2.1 illustrates ESAL values for single and tandem axles over a range of axle weights. The single and tandem curves clearly show the fourth-order nature of ESALs versus axle weight. The benefit of spreading the load over more axles is evident in Figure 2.1 by the dramatic reduction in ESALs for the tandem axle group at any given axle weight, relative to the single axles. Finally, the ESAL standard is shown in the plot at 18 kip with an ESAL value of one. Within the AASHTO empirical design system, total traffic must be decomposed into vehicle types with known axle weight distributions. The axle weight distributions are then used with the ESAL equations to determine ESALs per vehicle from which a total design ESAL over the pavement life is computed. It should also be noted that the ESAL assumes a tire inflation pressure of 70 psi and a tire with a bias-ply design. Today, tire pressures in excess of 100 psi are common with a radial design. These factors are not accounted for in the ESAL equations.

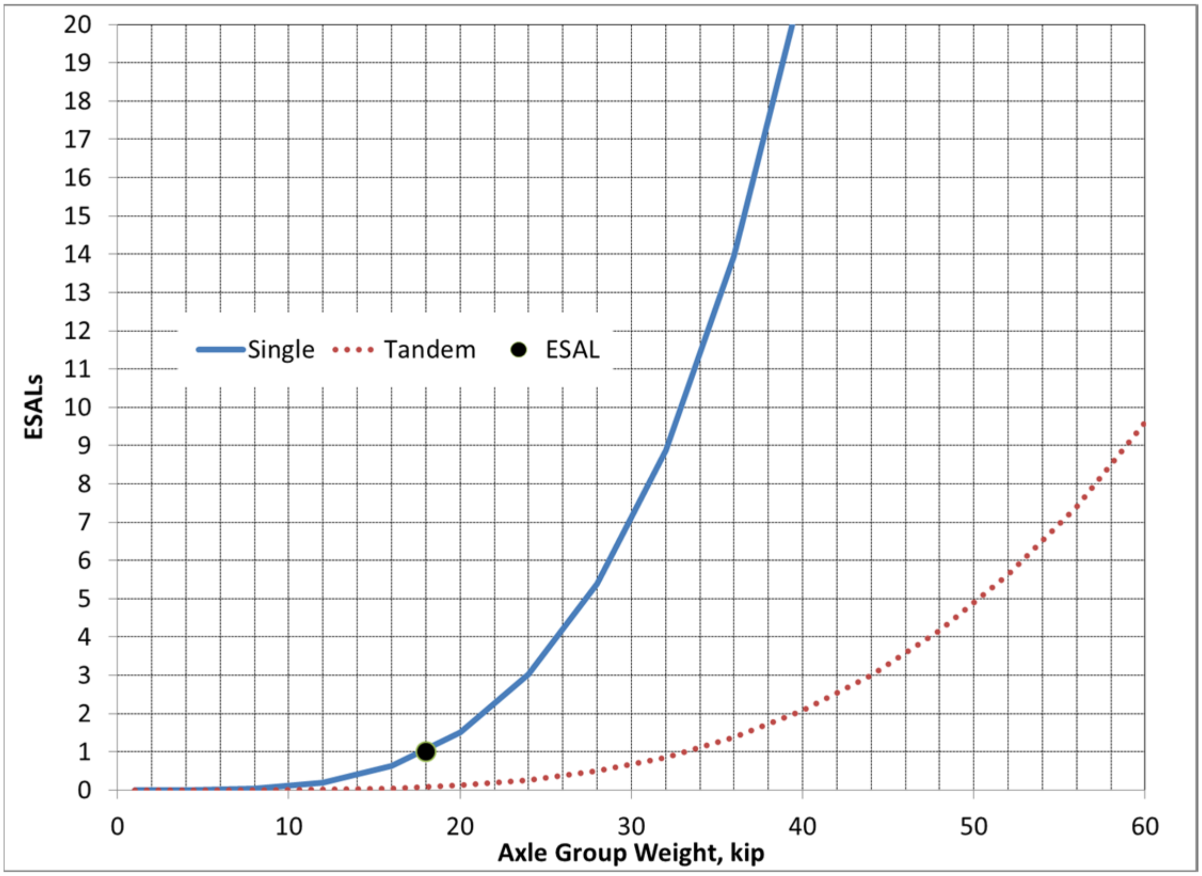


Figure 2.1 ESALs versus Axle Weight (*3*).

During the AASHO Road Test, routine inspections of each section were made by a panel of raters. Figure 2.2 shows the rating form and the zero to five scale used by the raters to quantify current pavement condition. Though actual pavement distress measurements were made during the road test, this rating scale was the only performance parameter used in the thickness design procedure. The researchers compiled the average ratings and plotted them against the amount of applied traffic in each section to develop performance history curves as shown schematically in Figure 2.3. The AASHTO design procedure relies upon characterizing the change in serviceability (ΔPSI) from the start (p_o) to the end (p_t) of the design life as a function of applied ESALs. Typical ΔPSI design values range from 2 to 3 as a function of roadway classification (*2*). For example, a high volume interstate would be designed with a smaller ΔPSI compared to a low volume county road.

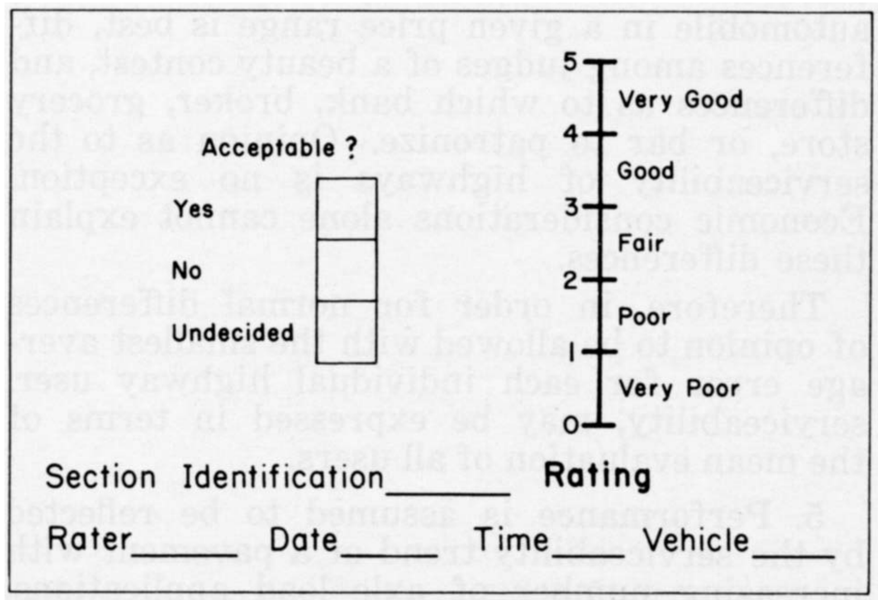


Figure 2.2 AASHO Road Test Present Serviceability Rating Form (*1*).

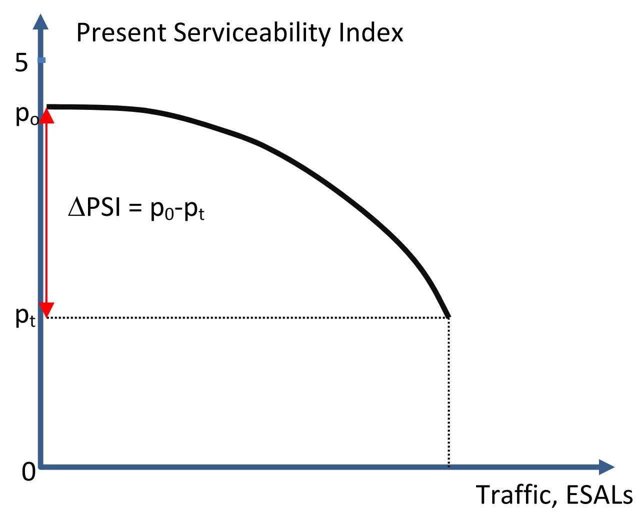


Figure 2.3 Pavement Performance History Quantified by PSI (*3*).

Since flexible pavements are typically comprised of diverse layers with varying engineering properties, it was necessary for AASHTO to introduce the pavement structural number (SN) concept. SN represents the cumulative pavement structure above subgrade expressed as a product of individual layer thicknesses (D_i), their respective structural coefficients (a_i) and drainage coefficients (m_i) as illustrated in Figure 2.4. The layer thicknesses are output from the AASHTO design process as will be described below. The structural coefficients are empirical values meant to relate the relative load-carrying capacity of different materials. For example, many state agencies use 0.44 for asphalt and 0.14 for granular base as originally recommended by AASHO (*1*). These particular structural coefficients mean that one inch of asphalt is roughly equivalent to 3.1 inches (0.44÷0.14) of aggregate base. The drainage coefficients are meant to empirically adjust the design according to site-specific rainfall expectations and quality of drainage provided by the material itself (*1*). Drainage coefficients range from 0.4 to 1.4 with the original AASHO Road Test condition represented as 1.0.

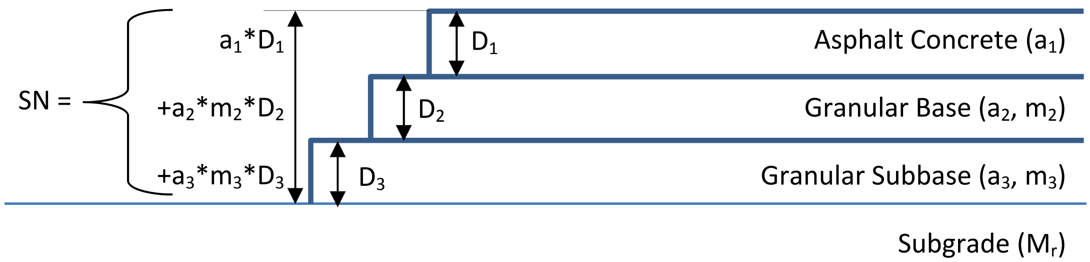


Figure 2.4 Structural Number Concept (*3*).

2.2 AASHTO Empirical Design Procedure

As described above, the AASHO Road Test (*1*) established a correlation between soil condition, traffic, change in pavement condition and pavement structure. This relationship is shown in Equation 1 (*2*). The M_r, ΔPSI and SN terms are as defined above. ESALs are represented by the W_{18} term. The Z_R and S_0 terms are reliability and variability factors not originally part of the AASHTO design procedure but added later to incorporate a safety factor into the design. They are not present in the 1972 edition of the Design Guide (*7*) which some states still use (*3*). The other quantities in the equation are regression coefficients that provided the best match between the independent variables (SN, ΔPSI, M_r) and the performance of the pavement section as quantified by ESALs.

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

While the purpose of Equation 1 is to determine the required structural number of a proposed pavement section, it is written to compute ESALs (W_{18}) and solving algebraically for SN is a daunting task. To alleviate this problem, AASHTO published a design nomograph (Figure 2.5) that solves for SN given the other inputs. Notice that W_{18} (ESALs) is treated as another input with the nomograph solving toward SN. Alternatively, the DARWin software developed for AASHTO, or solver subroutines in spreadsheets, are used to solve the equation for SN. It is important to note that Equation 1 uses Z_R to represent reliability while in the nomograph, reliability is used directly as a percentage. More precisely, Z_R represents the z-statistic corresponding to the chosen level of reliability. When using the equation, Z_R must be entered. When using the nomograph, the reliability percentage must be entered. AASHTO has recommended levels of reliability (*2*), based upon highway functional classification, and the value should be carefully selected as pavement thickness is correlated to the reliability level and choosing values outside of the recommended ranges can greatly increase pavement thickness.

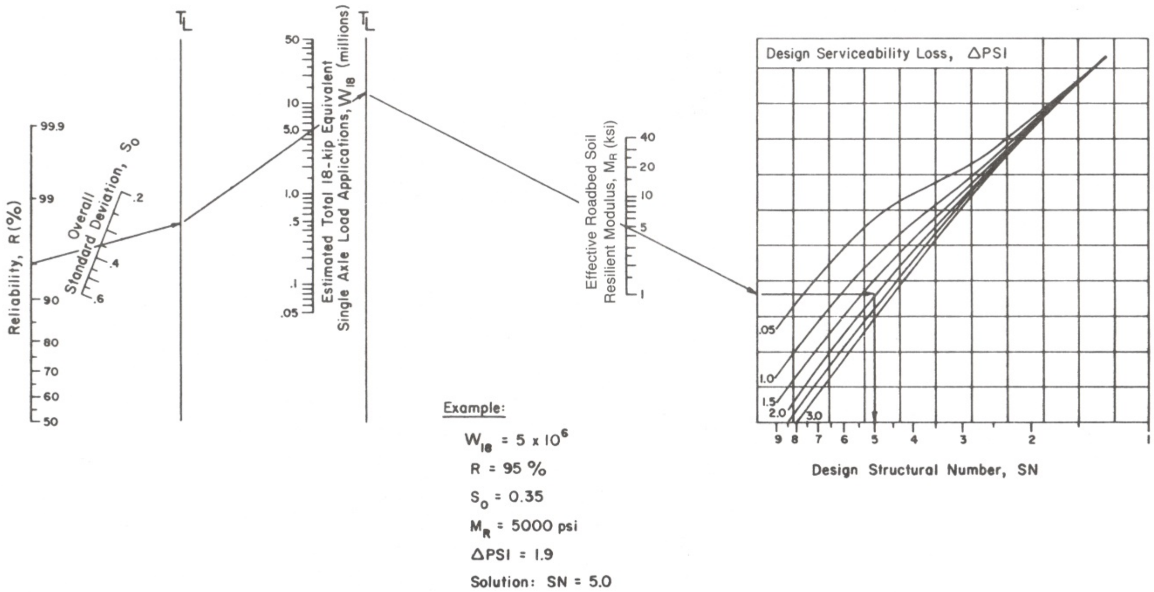
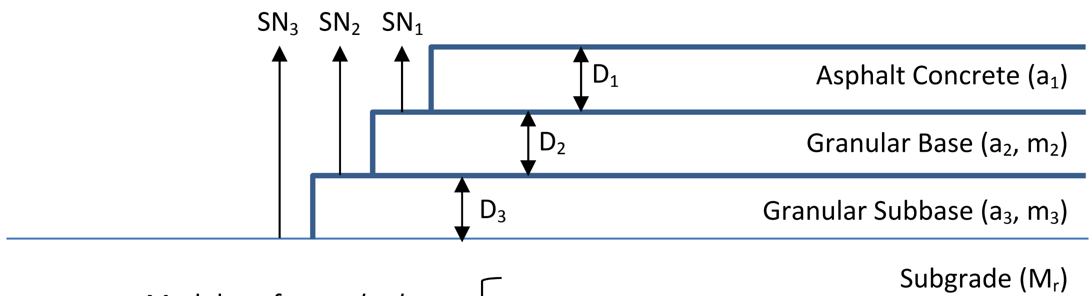


Figure 2.5 AASHTO Flexible Pavement Design Nomograph (*2*).

The AASHTO design equation (Equation 1 or Figure 2.5) is meant to be used for each layer in a multilayer pavement structure to determine the required pavement thicknesses. As described by AASHTO (*2*), this is done in a top-down fashion as depicted in Figure 2.6. The design begins by finding the required structural number above the granular base (SN_1) using the granular base modulus and other input parameters in the design equation or nomograph. By definition, this structural number is the product of the structural coefficient and thickness of layer one, so it can be used to solve for the thickness of the first layer. Next, the required structural number above the granular subbase (SN_2) is found by using the subbase modulus and other input parameters in the design equation or nomograph. As shown in Figure 2.6, SN_2 is the sum of the layer one contribution (a_1*D_1) and the layer two contribution ($a_2*m_2*D_2$). Since D_1 was already found in the previous step, the SN_2 equation can be solved for D_2. This procedure is followed again for the subgrade (or next sublayer, if present), as shown in Figure 2.6, to arrive at a unique set of pavement layer thicknesses.



Modulus of *granular base*, and other inputs (W_{18}, Z_R, S_0, ΔPSI) used to find SN_1

$$SN_1 = a_1 * D_1$$

$$D1 \geq \frac{SN_1}{a_1}$$

$$SN_2 = a_1 * D_1 + a_2 * m_2 * D_2$$

$$D_2 \geq \frac{SN_2 - a_1 * D_1}{a_2 * m_2}$$

Modulus of *granular subbase*, and other inputs (W_{18}, Z_R, S_0, ΔPSI) used to find SN_2

Modulus of subgrade, and other inputs (W_{18}, Z_R, S_0, ΔPSI) used to find SN_3

$$SN_3 = a_1 * D_1 + a_2 * m_2 * D_2 + a_3 * m_3 * D_3$$

$$D_3 \geq \frac{SN_3 - a_1 * D_1 - a_2 * m_2 * D_2}{a_3 * m_3}$$

Figure 2.6 Pavement Design with Empirical AASHTO Design Equation (*3*).

2.3 AASHTO Empirical Design Limitations

Though the empirical AASHTO design procedure has been used since the 1960's, there are many factors that limit its continued use and provide motivation for developing and implementing more modern methods. Most notably among these factors is the very nature of the method itself: *empirical*. This means that the design equations described above are strictly limited to the conditions of the original road test. This includes all the coefficients in Equation 1, the structural coefficients (a_i), drainage coefficients (m_i), ESAL equations and so forth. Any deviation from these conditions results in an unknown extrapolation.

The limitations of the AASHO Road Test are numerous. The experiment had one soil type, one climate, one type of asphalt mix (pre-Marshall mix design), limited pavement cross-sections, limited load applications and tires inflated to 70 psi (*1*). Any deviation from these factors in modern design means extrapolation, which can lead to under or over-design. Most designs conducted today are extrapolations beyond the original experimental conditions. Consider, for example, the thickness design curves published in 1962 as part of the AASHO Road Test report shown in Figure 2.7. The shaded gray area above 1.1 million axle loads is entirely extrapolated. Also, the dashed portions of the curves are extrapolations. As evidenced by Figure 2.7, there was very little, even in 1962, that was *not* an extrapolation.

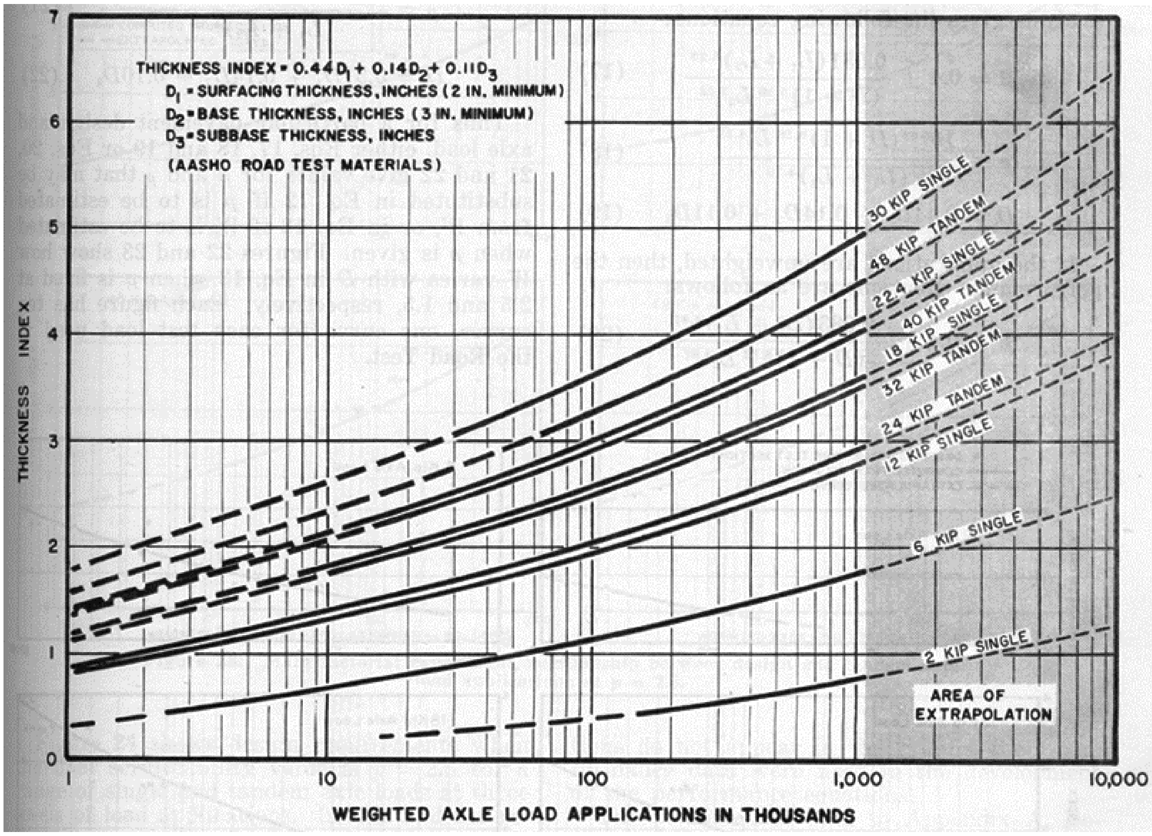


Figure 2.7 Flexible Pavement Design Curves (*1*).

2.4 Structural Coefficients

The structural coefficients are of great importance in the AASHTO procedure. These empirical terms are meant to reflect the relative structural contributions of each pavement layer and have a direct impact on the derived layer thicknesses as demonstrated in Figure 2.6. Though AASHO recommended 0.44 for the asphalt layer in 1962, a range of values were actually reported. Table 2.1 lists the reported values by test loop ranging from 0.33 to 0.83. Loop 1 is not included in the table because it was never trafficked; it was used to evaluate environmental impacts on pavements. The authors of the 1962 report (*1*) stated that a weighted average was used to determine 0.44 as the recommended value, but inspection of the data does not clearly indicate how the values were weighted to achieve 0.44.

As described by Peters-Davis and Timm (8), a relationship was created in 1972 that linked the layer coefficient to the elastic modulus (E) of the HMA at 70°F, and is shown in Figure 2.8. Strictly speaking, 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 (*2*). In 2006, Priest and Timm (*9*) found a relationship relating temperature and stiffness for all the structural sections in the 2003 research cycle of the

National Center for Asphalt Technology's Pavement (NCAT) Test Track. Using their relationship, the average HMA modulus was calculated as 811,115 psi. If the curve in Figure 2.3 was extrapolated out to this modulus value, the resulting layer coefficient would be equal to 0.54.

Table 2.1 HMA Layer Coefficients from AASHO Road Test (*data from 1*)

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

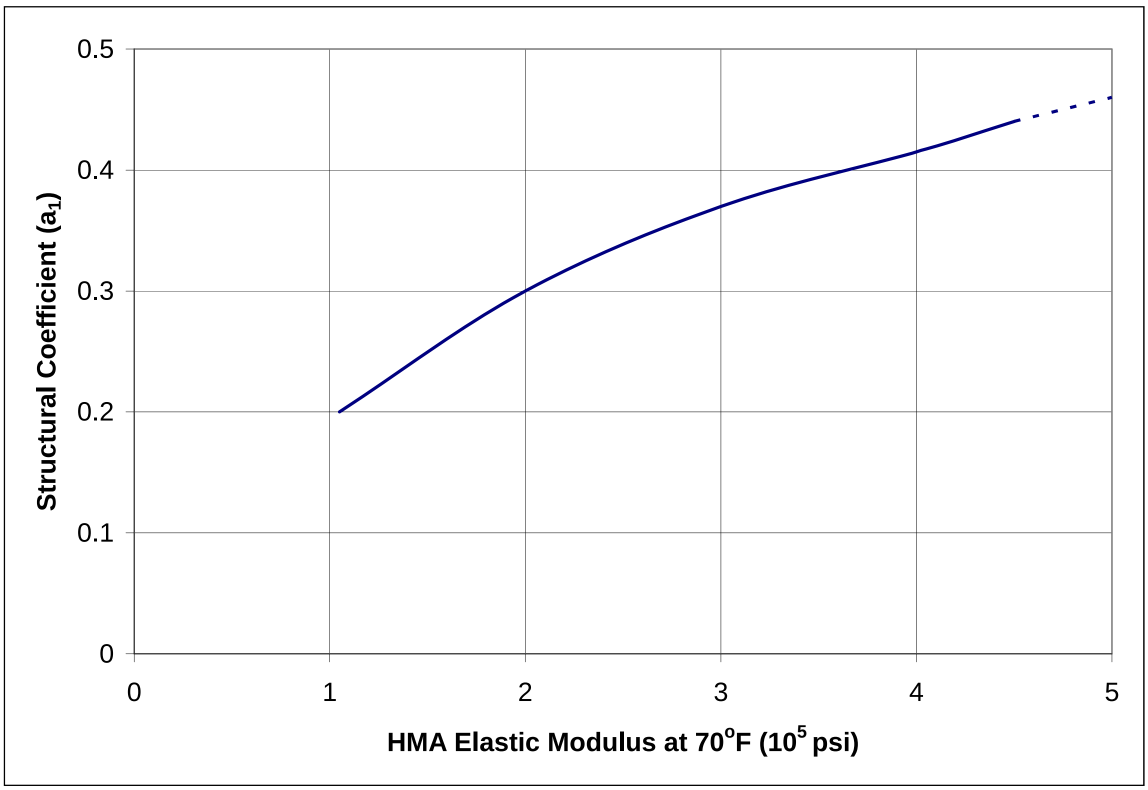


Figure 2.8 Determining a_1 based on HMA Modulus (*data from 2*).

The structural coefficients not only appear in the structural design equations (Equation 1, Figures 2.5 and 2.6) but they are also present in the ESAL computations. The 4th order relationship between axle weight and pavement damage was mentioned in Section 2.1. More specifically, at the AASHO Road Test, replicate cross sections were constructed in different test loops to apply repeated axle loads at various load levels on the same pavement structure. This allowed the researchers to measure the damage caused by axles at various weights and create mathematical relationships based upon that damage, which included a factor accounting for the pavement structure. This factor was the structural number, as is used in the design equations shown above (Equation 1, Figures 2.5 and 2.6), and is a product of the layer thicknesses, drainage coefficients and structural coefficients. Since ESALs are needed in the structural design equation to determine the required SN from which thicknesses are computed, and an SN is required to determine ESALs, the design process follows circular reasoning. To overcome this problem, many designers simply assume an SN equal to 5 to compute ESALs as the starting point, from which the actual design SN may be determined from the structural design equation.

When considering updating the empirically-based procedure, one may consider adjusting values other than the asphalt layer coefficient. A previous investigation (*8*) conducted a sensitivity analysis to determine which parameters had the greatest impact on asphalt concrete (AC) thickness. The analysis considered a wide range of layer coefficients (a_1), traffic levels (ESALs), soil moduli (M_r), reliability (R), change in serviceability (ΔPSI) and design variability (S_0). Table 2.2 summarizes the Pearson correlation coefficients for the 5,120 design thicknesses determined in the sensitivity analysis with the parameters ranked from most to least influential (*8*). Clearly, the asphalt layer coefficient is the most influential. The next two parameters, though also strongly correlated, may be considered simply part of the design scenario or site-specific conditions. The remaining parameters are much less correlated and do not affect pavement thickness as significantly as the first three. Therefore, it makes sense to focus recalibration efforts on the asphalt layer coefficient to better align observed performance with performance predicted by the design procedure.

Table 2.2 Correlation between HMA Thickness and Input Parameters (*8*)

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

The asphalt structural coefficient plays a vital role in pavement design and should reflect performance characteristics of modern materials. However, a recent survey of state agencies (*10*), summarized in Figure 2.9, shows the distribution of asphalt structural coefficients across the U.S., where 45% of states currently use 0.44 for at least one paving layer, though some states specify according to the lift or mix design using a number of design gyrations (N_{des}). Many states (28%) use less than the originally recommended AASHO value of 0.44 (*3*). Two states, Alabama (*8*) and Washington (*11*), recently revised their structural coefficients to 0.54 and 0.50, respectively. These increases reflect modern advances in the materials and construction practices and are more consistent with field performance of flexible pavements in these states. The changes result in optimum asphalt pavement thickness design that can potentially provide significant savings to the state agencies. A change from 0.33 to 0.44 would result in 25% thinner sections. An increase from 0.44 to 0.54, as done in Alabama, reduces the pavement thickness by 18.5%. As stated by Larry Lockett (*12*), the ALDOT State Materials and Tests Engineer at the time the change was implemented, "This means that our resurfacing budget will go 18% farther than it has in the past. We will be able to pave more roads, more lanes, more miles, because of this 18% savings." Any change considered by a state agency should be carefully evaluated and supported by actual pavement performance data.

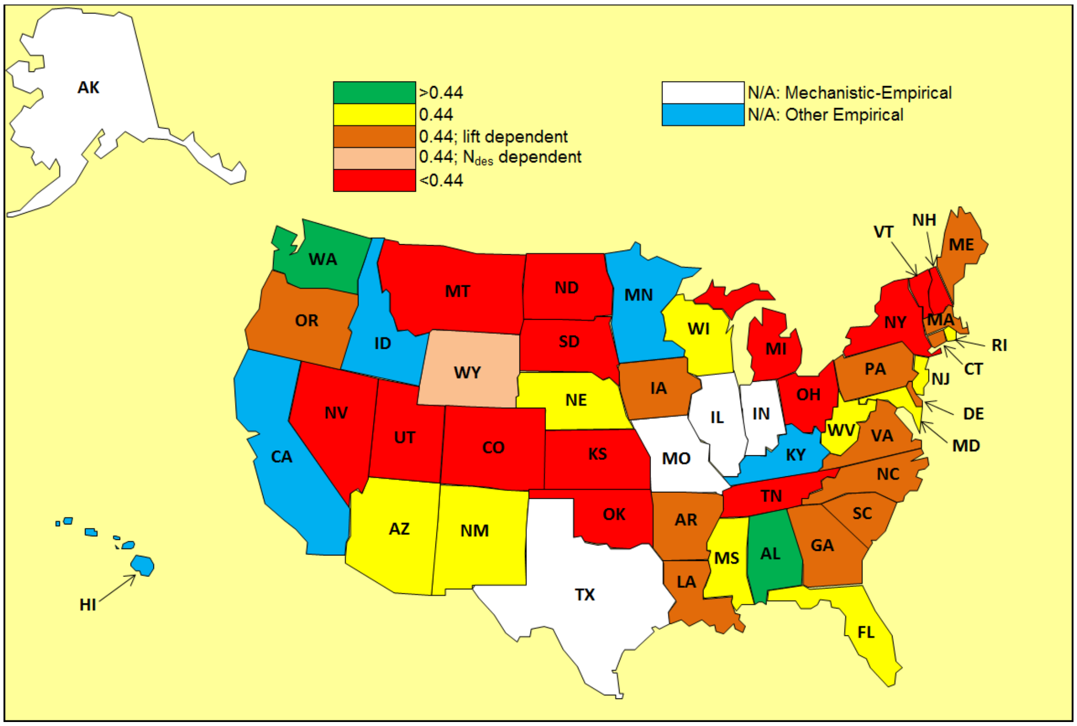


Figure 2.9 Asphalt Structural Coefficients (*data from 10*).

3. RECALIBRATION PROCEDURES

The outcome of any pavement design procedure is a set of layer thicknesses that will be sufficient to carry the expected traffic, in the given environment, with a specified level of performance over a fixed period of time. The success of the design procedure hinges on the ability of the procedure to make accurate predictions of pavement performance given a set of input parameters. Safety factors may be added to the predictions to account for uncertainty in the process as is done in the AASHTO method (*2*) through the reliability (R or Z_R) and variability (S_0) terms. Three general classes of recalibration methods are discussed in the following subsections, each of which should be judged against the ability to make accurate predictions of pavement performance over time.

3.1 Deflection-Based Procedures

Deflection-based procedures rely on field testing of existing pavements to determine in-place modulus. The in-place modulus is then correlated to a structural coefficient through existing empirical equations. The main advantage of this approach is that it requires relatively little data generation through deflection testing. The main disadvantage is that the approach relies upon existing empirical equations that are based on past performance and may not accurately reflect the performance of the pavement materials under evaluation. The general procedure includes the following steps discussed in the subsections below:

1. Identify and characterize pavement sections to be evaluated.
2. Perform deflection testing on pavement sections.
3. Backcalculate pavement layer properties.
4. Compute new structural coefficients.

3.1.1 Identify and Characterize Pavement Sections to be Evaluated

The first step is to identify and select candidate pavement sections to include in the analysis. Recently constructed, undamaged pavement sections should be characterized since the structural coefficient is meant to represent "new" conditions. Information regarding the pavement cross section that includes material type and as-built layer thicknesses at the test location is critical.

3.1.2 Perform Deflection Testing on Pavement Sections

Deflection testing, using a falling weight deflectometer (FWD), should be conducted on the selected pavement sections. For details regarding FWD best practices, consult the FHWA manual on field guidelines for FWD testing (*13*).

Depending on the backcalculation scheme to be used, as discussed in the next subsection, the FWD should be configured to measure deflections at critical offsets. At a minimum, the center and outer (60 or 72 inches from load center) deflections should be measured. Typically, deflections may be measured with 6 to 9 sensors, which include the center and outermost deflection measurements.

Many FWD's are configured to test at multiple load levels. To determine the structural coefficient, it is important to have test results at 9,000 lb, which is the AASHTO standard load level. This may be achieved either by setting the drop height to achieve 9,000 lb or by interpolating results from multiple load levels. For example, Figure 3.1 shows the interpolation process from data collected at the NCAT Test Track (*14*). In the figure, the center (D1) and outermost (D9) deflections were plotted against load level. Regression equations were determined for each set of deflections from which deflection at the target load level was determined.

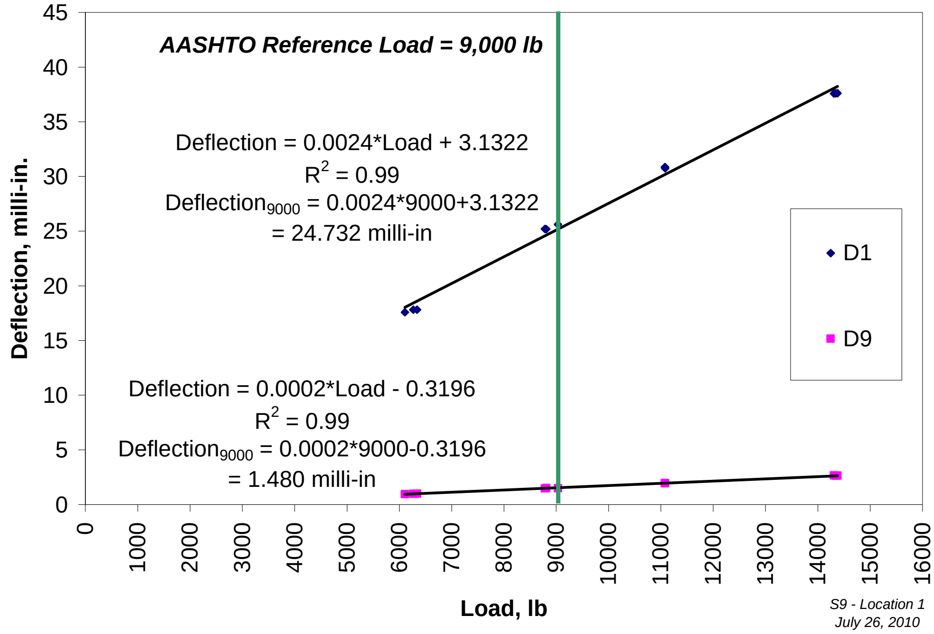


Figure 3.1 Deflection versus Load Example (*14*).

It is also important to measure and account for the temperature at the time of testing. The AASHTO system is currently based on a 68F (20C) pavement reference temperature. Therefore, any deflection data must be adjusted to this reference temperature. AASHTO (*2*) has published temperature correction charts that may be used to correct the center deflection for backcalculation. Alternatively, tests could be conducted over a range of temperatures and deflections interpolated at the reference temperature as shown by the example in Figure 3.2. The center deflection (D_1) shows a strong dependence on temperature characterized by the corresponding regression equation. The equation was used to establish the best estimate of deflection at 68F, and the deflections at other temperatures were corrected to 68F represented by the D1 at 68F data series in Figure 3.2. The outermost deflection (D_9) shows very little correlation with temperature, as expected, since it represents the behavior of the subgrade soil, and no temperature correction is needed. This approach, though effective, would require accessing the pavement at multiple times to gather the required data and may not be practical in all situations. In any case, it is important to have deflections representing the AASHTO reference temperature.

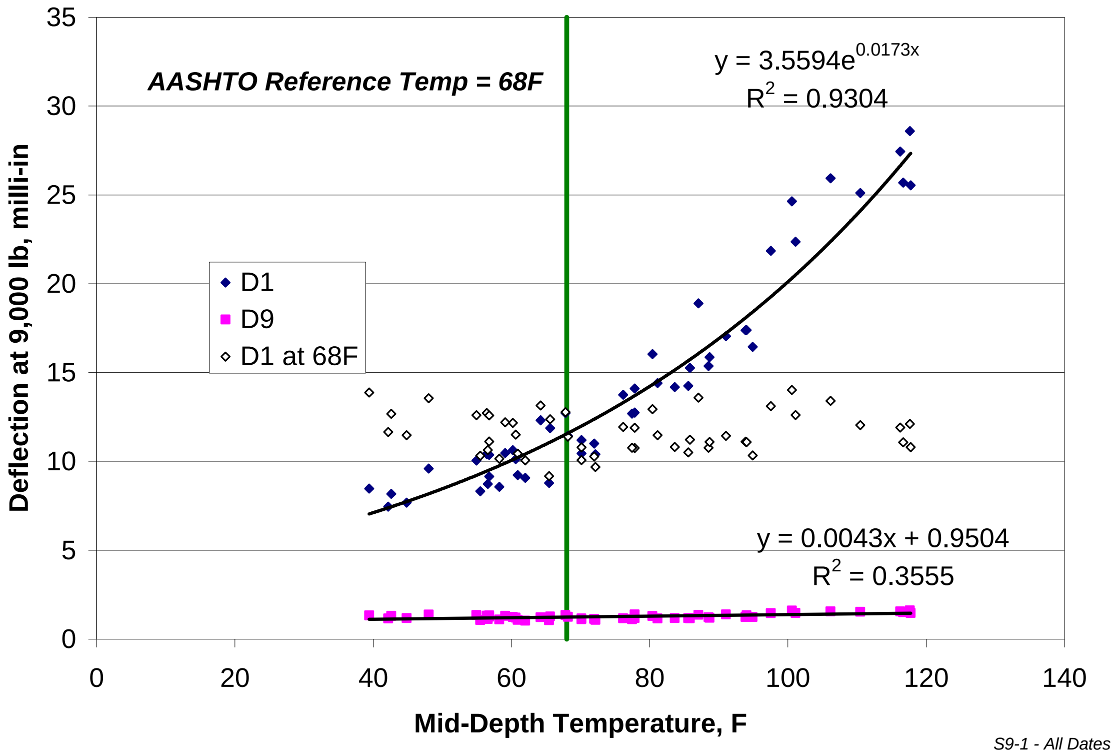


Figure 3.2 Deflection vs. Temperature Example (*14*).

3.1.3 Backcalculate Pavement Layer Properties

There are several approaches to backcalculating in-place pavement layer properties from measured deflections. They range from relatively simple equations solved by hand or in a spreadsheet to very complex computational algorithms executed in self-contained computer programs. Regardless of the approach, the objective of any backcalculation scheme is to determine the layer properties under the given applied load and environmental conditions that produced the measured deflections. While only the relatively simple AASHTO two-layer backcalculation procedure (*2*) is discussed here, there are many more sophisticated multi-layer backcalculation programs available that include EVERCALC, MODCOMP and MICHBACK, to name a few.

The AASHTO two-layer backcalculation approach (*2*) is based on fundamental pavement mechanics and determines the in-place subgrade soil modulus (M_r) and the composite modulus of all pavement layers (E_p) above the subgrade soil. The approach was originally intended to provide estimates of in-place effective structural number (SN_{eff}) as part of the AASHTO overlay design procedure (*2*). However, it can also be used to provide the information necessary to find structural coefficients as will be demonstrated in the next subsection.

The first step in the AASHTO backcalculation approach is to determine the soil modulus according to (*2*):

$$M_R = \frac{0.24 * P}{\delta_r * r} \qquad \text{(Equation 2)}$$

where:

M_R = subgrade modulus, psi
P = load magnitude, lb (9,000 lb recommended by AASHTO)
δ_r = measured deflection at offset, r, in.
r = radial offset, in.

AASHTO (*2*) recommends a radial offset that exceeds 70% of the effective radius (a_e) of the stress bulb at the subgrade/pavement interface. This is to insure that the sensor chosen provides only a measure of subgrade deflection while providing sufficiently high deflections to minimize the impact of measurement error. The effective radius may be calculated by (*2*):

$$a_e = \sqrt{a^2 + \left(D \sqrt[3]{\frac{E_p}{M_R}} \right)^2} \qquad \text{(Equation 3)}$$

where:

a_e = effective radius of stress bulb at subgrade/pavement interface, in.
a = FWD load plate radius, in.
D = total pavement depth above subgrade, in.
M_r = subgrade modulus computed from Equation 2, psi
E_p = composite pavement modulus computed from Equation 4, psi

Note that a specific sensor offset must be chosen to compute the subgrade modulus according to Equation 2, but whether it satisfies the 70% of a_e criteria cannot be checked until further computations are made since Equation 3 also requires the composite pavement modulus (E_p). After the subgrade modulus has been determined, assuming a sensor offset, E_p is backcalculated from the center deflection using the following equation (*2*):

$$\delta_1 = 1.5 * p * a \left(\frac{1}{M_R \sqrt{1 + \left(\frac{D}{a} \sqrt[3]{\frac{E_p}{M_R}} \right)^2}} + \frac{\left(1 - \frac{1}{\sqrt{1 + \left(\frac{D}{a} \right)^2}} \right)}{E_p} \right) \qquad \text{(Equation 4)}$$

where:

δ_1 = center deflection, in. (called D1 above)

p = contact pressure, psi (computed from load, P, and circular contact radius, a)

a = FWD load plate radius, in.

D = total pavement depth above subgrade, in.

M_r = subgrade modulus computed from Equation 2, psi

E_p = composite pavement modulus, psi

Equation 4 is easily solved for E_p in a spreadsheet using some kind of iterative solution like the built-in Solver function in Excel® or using a bisection method to determine the correct E_p that will produce the measured center deflection. After computing E_p, it should be used in Equation 3 with the other variables to check that the selected sensor met the radial offset requirement.

3.1.4 Compute New Structural Coefficients

There are two approaches to finding the AC structural coefficient both of which use the composite pavement modulus (E_p). The first may be used with individual pavement sections if the underlying (non-AC) structural and drainage coefficients are known or assumed. The second may be used if there are paired sections where the only difference between sections is one particular AC layer. Both approaches rely on computing the effective structural number from the composite pavement modulus. The effective structural number represents the structural integrity of the pavement as an empirical function of the thickness of the pavement and the composite pavement modulus. The equation was based on performance at the AASHO Road Test (*1,2*) and is expressed as:

$$SN_{eff} = 0.0045 * D * \sqrt[3]{E_p} \qquad \text{(Equation 5)}$$

where:

SN_{eff} = effective structural number of in-place pavement

D = total pavement depth above subgrade, in.

E_p = composite pavement modulus, psi

The first approach, depicted in Figure 3.3, assumes that the structural and drainage coefficients of the layers beneath the AC are known. If that is the case, then SN_{eff} may be computed according to Equation 5 and equated to the other parameters by:

$$SN_{eff} = a_1*D_1 + a_2*m_2*D_2+a_3*m_3*D_3 \qquad \text{(Equation 6)}$$

Since every parameter in Equation 6 is known except for a_1, the AC structural coefficient may be simply calculated as:

$$a_1 = [SN_{eff} - a_2*m_2*D_2 - a_3*m_3*D_3] / D1 \qquad \text{(Equation 7)}$$

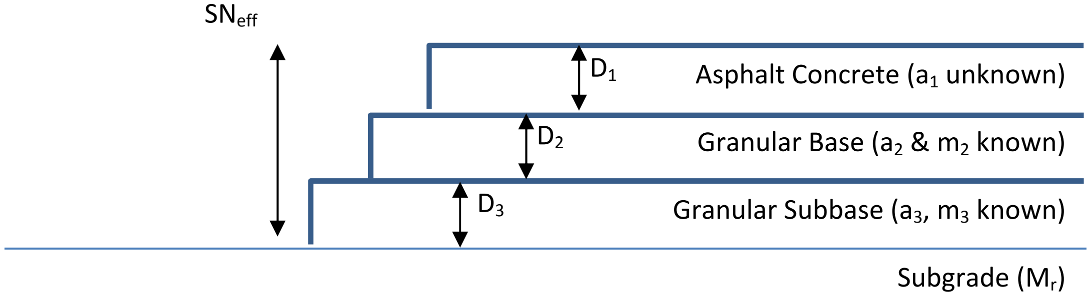


Figure 3.3 SN$_{eff}$ Schematic.

This approach was used in a Kansas study (*15*) to determine the structural coefficient of crumb rubber modified asphalt mixtures. In that study, the underlying base and subgrade layer moduli were determined through backcalculation and correlated to structural coefficients through existing equations published by Ullidtz (*16*).

The second approach that uses the SN$_{eff}$ computation relies on having two nearly identical pavements where only one layer differs between the two sections and the structural coefficient of one of the two different materials is known or assumed. Figure 3.4 shows an example from the NCAT Test Track where two sections differed only in their surfacing layers while the underlying materials were nearly identical with only slight differences due to inevitable construction variation (*14*). In this particular case, the objective was to establish a structural coefficient of the open graded friction course in Section S8 (*14*).

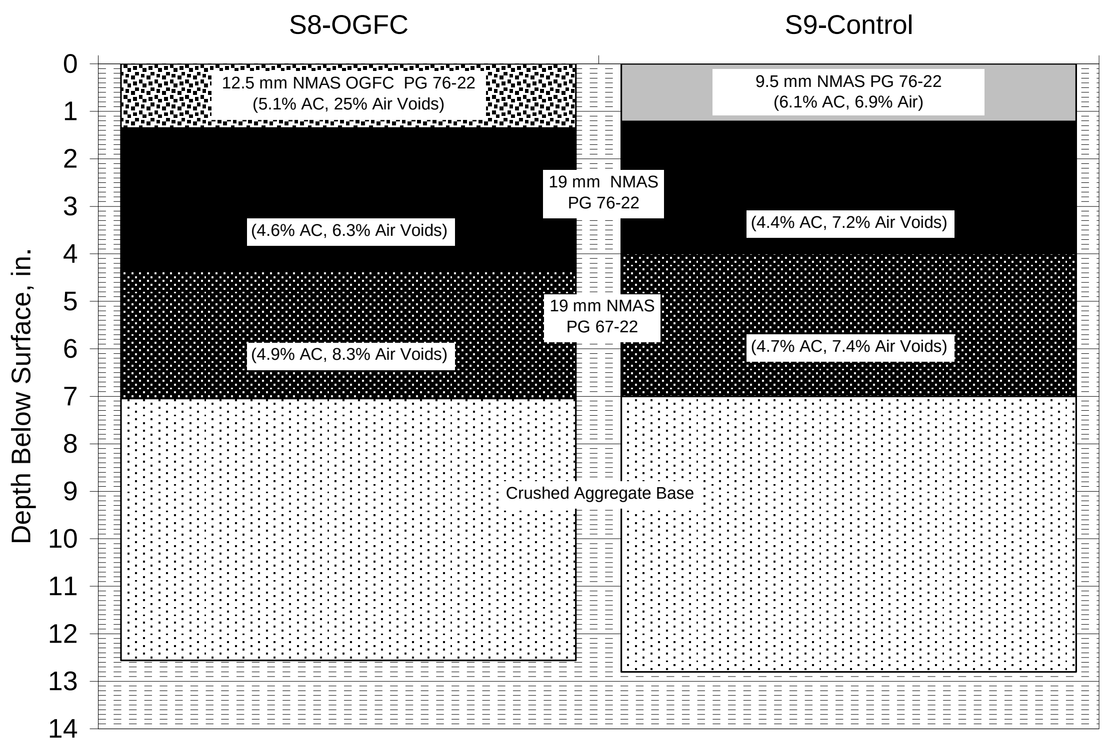


Figure 3.4 Paired Test Sections (*14*).

The procedure involves computing E_p for each section from which SN_{eff} is determined. Since the sections are nearly identical except for one lift of AC, any difference in SN_{eff} may be attributed to the difference in that one lift. From a general perspective, the SN_{eff} of two pavements (A and B) may be computed as:

$$SN_{effA} = a_{1A}*D_{1A} + a_2*m_2*D_2+a_3*m_3*D_3 \quad \text{(Equation 8)}$$

$$SN_{effB} = a_{1B}*D_{1B} + a_2*m_2*D_2+a_3*m_3*D_3 \quad \text{(Equation 9)}$$

Taking the difference between Equations 8 and 9, assuming everything below the first layer is equivalent, yields:

$$SN_{effA} - SN_{effB} = \Delta SN = a_{1A}*D_{1A} - a_{1B}*D_{1B} \quad \text{(Equation 10)}$$

Assuming that pavement A has a known structural coefficient (a_{1A}), and having measured the ΔSN and thickness of both pavement layers, then the structural coefficient of the unknown layer may be computed by solving Equation 10 for a_{1B}:

$$a_{1B} = [a_{1A}*D_{1A} - \Delta SN] / D_{1B} \quad \text{(Equation 11)}$$

This procedure was followed for the sections in Figure 3.4, and the data are summarized in Figure 3.5. The computed difference (ΔSN) between the two sections, which was shown to be statistically significant (*14*), was 0.45. The equations shown in Figure 3.5 follow the form of Equation 11, which produced an OGFC structural coefficient equal to 0.15 (*14*).

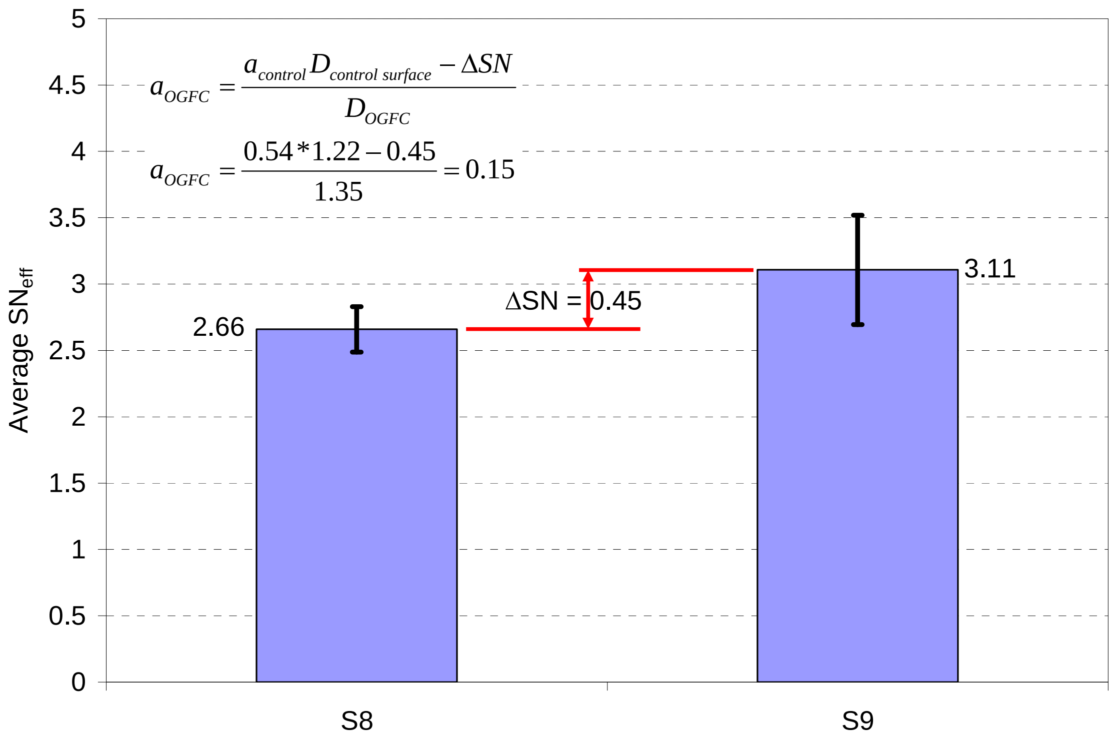


Figure 3.5 Computed SN_{eff} and Computed OGFC Structural Coefficient (*14*).

Aside from the two methods described above, there are a variety of existing equations to estimate structural coefficient from the backcalculated in-place AC modulus, which is different than composite pavement modulus (E_p) determined from the AASHTO two-layer backcalculation. A backcalculated AC modulus is determined through a multilayer backcalculation program and should be used in conjunction with the structural coefficient equations listed in Table 3.1. Since the equations are empirical, the original references should be consulted to determine if the test conditions are applicable to the pavements currently under evaluation.

Table 3.1 Asphalt Concrete Structural Coefficient Equations

Material Type	Equation	Reference
Asphalt Concrete	$a_1 = 0.171*\ln(E_{AC}) - 1.784$ where E_{AC} = AC modulus, psi	*2*
Asphalt Concrete	$a_1 = 0.4*\log(E_{AC}/3000) + 0.44$ where E_{AC} = AC modulus, MPa	*16*
Crumb Rubber Asphalt Concrete	$a_1 = 0.315*\log(E_{AC}) - 1.732$ where E_{AC} = AC modulus, MPa	*15*

As noted above, it must be re-emphasized that these deflection-based methodologies rely on past-performance characterization and may not accurately reflect performance of new or site-specific materials. They should only be used if performance data are not available, or more preferably, in conjunction with performance data as explained in the subsequent sections.

3.2 Performance-Based Procedure

Recalibration of the asphalt layer coefficient based on observed pavement performance most closely matches how the coefficients were originally calibrated (*1*). This approach should be considered an improvement over deflection-based procedures because it considers the actual performance of the material under investigation rather than relying on previously developed correlations. The main disadvantage of this approach is that detailed traffic and pavement performance records over time are needed, thus pavements selected for evaluation must be done so carefully. Also, this approach is not generally capable of discerning individual lifts of asphalt.

Figure 3.6 summarizes the performance-based recalibration procedure as previously documented by Peters-Davis and Timm (*8*). The procedure relies on two primary data sets. The first is historical traffic data in terms of axle weights, axle configuration and volume, which are needed to compute ESALs over time. The second is performance data expressed as International Roughness Index (IRI), which can be converted to pavement serviceability (PSI) over time. As shown in Figure 3.6 and explained further below, these two primary data sets are used in several equations to generate the actual ESALs applied to the pavement and the predicted ESALs that the pavement is expected to withstand. Since the asphalt structural coefficient (a_1) is used to determine the structural number (SN), which appears in both the actual and predicted ESAL equations, $\hat{a}_1$ may be iteratively adjusted to minimize the error between actual and predicted ESALs. The $\hat{a}_1$ symbol is used to indicate that it is a value that will be determined through a best-fit iterative procedure. This is the essence of how the original calibration was done for the AASHO Road Test results (*1,8*). The following subsections detail the procedural elements of Figure 3.6.

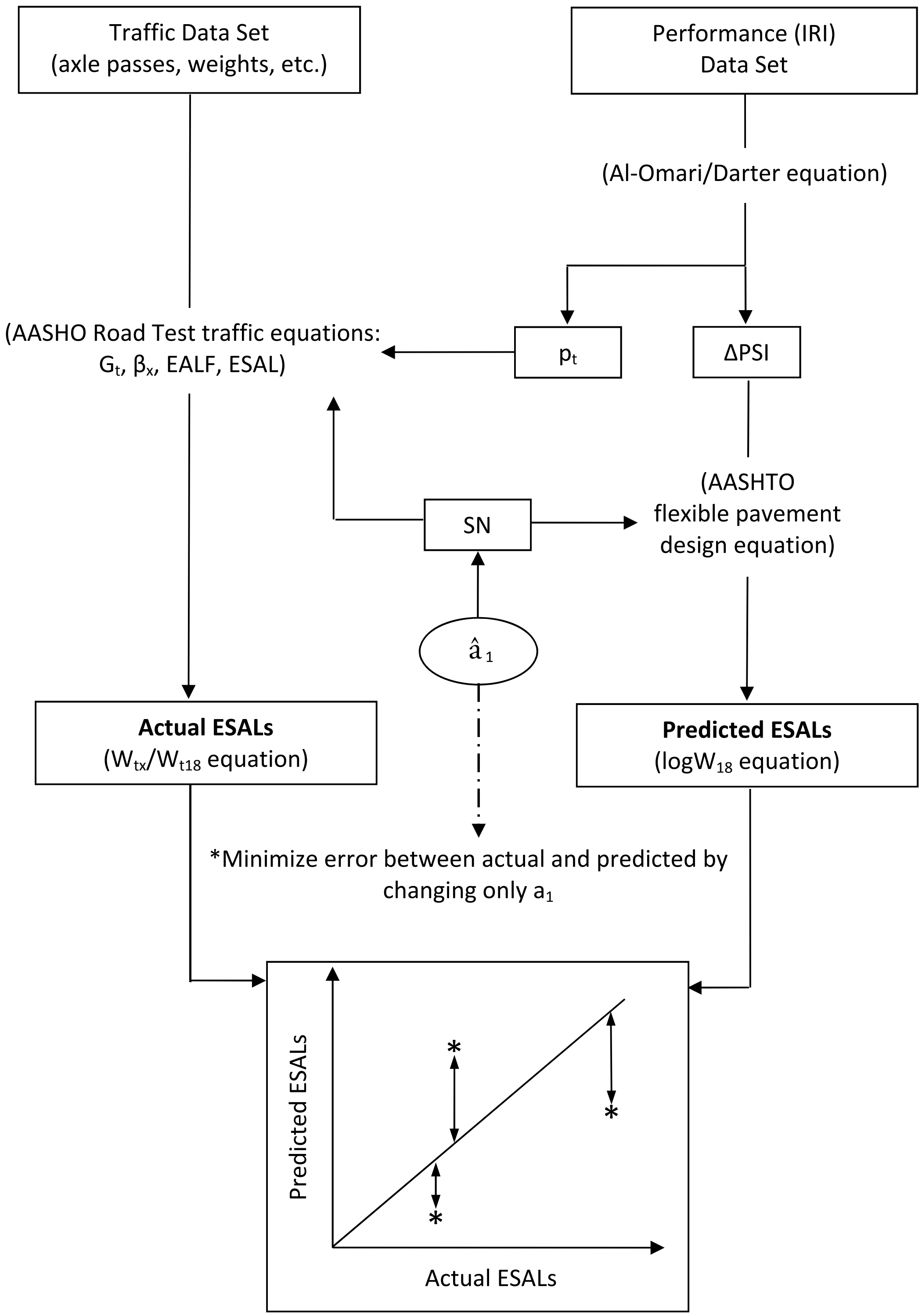


Figure 3.6 Performance-Based Recalibration Procedure (*8*).

3.2.1 Performance (IRI) Data

The AASHTO procedure requires pavement performance expressed in terms of present serviceability index (PSI). However, many agencies do not collect PSI values as part of pavement management activities, but rather IRI. Therefore, there is typically a need to convert IRI to PSI so the data may be used within the AASHTO system. While there are a number of equations available in the literature (*e.g., 17-19*), one recommended for use by the National Highway Institute and used in a previous investigation (*8*) was developed by Al-Omari and Darter (*20*) and recommended for use in this document. It was based on studying pavements from 5 different states and yielded an R^2 of 0.81 (*20*):

$$PSI = 5e^{(-0.0038 \cdot IRI)} \qquad \text{(Equation 12)}$$

where:
PSI = present serviceability index (0-5 scale)
IRI = International Roughness Index, in./mile

Once IRI data versus time for a pavement section have been obtained, it is straightforward to convert from IRI to PSI using Equation 12 and develop performance curves, such as those shown in Figure 3.7. Note in the figure that the data are separated by wheelpath representing the left (LPSI), right (RPSI) and average (AvgPSI) serviceability ratings. If datasets from both wheelpaths are available, it is recommended to use the data set representing the worst performance in the recalibration process to be conservative. At this stage, it is important to establish the initial serviceability (p_o) and terminal (p_t) calibration points. These will be used to establish ΔPSI values and points in time corresponding to cumulative ESAL applications at those times.

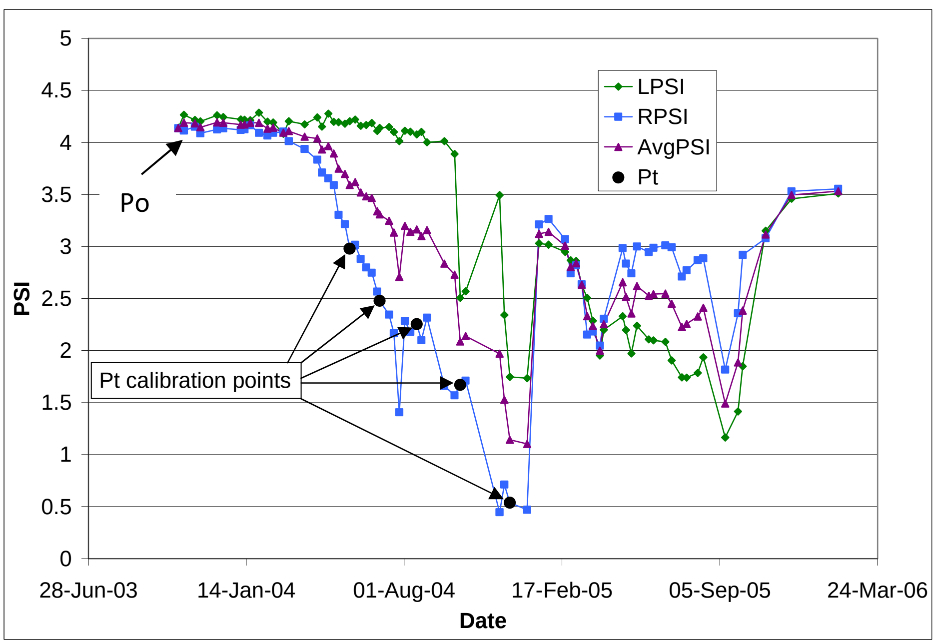


Figure 3.7 PSI Data Obtained from IRI Data (*8*).

3.2.2 Traffic Data and Actual ESALs

It is imperative that reasonably accurate historical traffic records are obtained for this recalibration procedure. Information regarding axle types (single, tandem, tridem), axle weights and volume of axles is critical in computing the actual ESALs applied. This information may come from weigh-in-motion or static scale sites. After assembling the necessary information, the total actual ESALs must be computed as detailed below.

As described by Peters-Davis and Timm (*8*), the AASHTO Design Guide (*2*) quantifies pavement damage using Equivalent Axle Load Factors (EALFs), which are used to find the number of ESALs. An EALF quantifies the damage done per pass of any axle relative to the damage done per pass by a standard axle (typically an 18-kip single axle). This equation comes from the results of the AASHO Road Test (*1*), and is expressed as follows according to Huang (*21*):

$$EALF = \frac{W_{t18}}{W_{tx}} \qquad \text{(Equation 13)}$$

where:
W_{tx} = number of x axle load applications at time t
W_{t18} = number of 18 kip axle load applications at time t

Equations 14b and 14c are used within 14a to generate the W_{tx}/W_{t18} value from which the EALF value may be determined from Equation 13 for any axle type relative to the standard (*21*). Equation 14a represents the fourth-power relationship presented in Figure 2.1, though it is impossible to clearly see the fourth-power trend in the equation due to its complexity.

$$\log\left[\frac{W_{tx}}{W_{t18}}\right] = 6.1252 - 4.79\log(L_x + L_2) + 4.33\log L_2 + \frac{G_t}{\beta_x} - \frac{G_t}{\beta_{18}} \qquad \text{(Equation 14a)}$$

$$G_t = \log\left[\frac{4.2 - p_t}{4.2 - 1.5}\right] \qquad \text{(Equation 14b)}$$

$$\beta_x = 0.40 + \frac{0.081 \cdot (L_x + L_2)^{3.23}}{(SN + 1)^{5.19} \cdot L_2^{\;3.23}} \qquad \text{(Equation 14c)}$$

where:
L_x = axle group load in kips
L_2 = axle code (1 for single, 2 for tandem and 3 for tridem)
SN = structural number
W_{tx} = number of x axle load applications at time t
W_{t18} = number of 18 kip axle load applications at time t
β_x = a function of design and load variables

β_{18} = value of β_x when L_x is equal to 18 and L_2 is equal to one
p_t = terminal serviceability determined from IRI data converted to PSI
G_t = a function of serviceability levels

The EALFs for each axle load group, determined by equations 13 and 14 above, are used to find the total damage done during the design period, which is defined in terms of passes of the standard axle load (ESALs), as shown in the following equation (*21*):

$$ESAL = \sum_{i=1}^{m} EALF_i n_i \qquad \text{(Equation 15)}$$

where:
ESAL = actual ESALs
m = number of axle load groups
$EALF_i$ = EALF for the *i*th axle load group
n_i = number of passes of the *i*th axle load group during the design period

It is important to note that Equation 14b requires a terminal serviceability value, p_t. This comes directly from the discussion above in subsection 3.2.1. Also, Equation 14c requires an SN value, which may be computed for the pavement as previously defined:

$$SN = \hat{a}_1 * D_1 + a_2 * m_2 * D_2 + a_3 * m_3 * D_3 \qquad \text{(Equation 16)}$$

It is assumed for the purposes of this recalibration procedure that the thicknesses are known and the non-AC structural and drainage coefficients are known. Therefore, $\hat{a}_1$ is the only unknown and is the value that will be adjusted, as noted in Figure 3.6, to arrive at the best match between actual and predicted ESALs.

3.2.3 Predicted ESALs

The predicted ESAL computation is made directly by the AASHTO pavement design equation presented earlier (Equation 1) and repeated here as Equation 17. A primary input to the equation is the pavement performance characterized by ΔPSI obtained through the IRI data set.

$$\log W_{18} = Z_R S_0 + 9.36 \log(SN+1) - 0.20 + \frac{\log\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.4 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \log M_R - 8.07 \qquad \text{(Equation 17)}$$

where:
$\log W_{18}$ = predicted ESALs
Z_R = standard normal deviate for a given reliability
S_0 = standard deviation
ΔPSI = difference between initial and terminal serviceability at time *t*
M_R = resilient modulus of the subgrade, psi

SN = structural number (Equation 16)

Equation 17 is normally used for design, where ESALs (W_{18}) are input, SN is computed and a reliability in excess of 50% is used to act as a safety factor when determining the required structural number. However, in the case of recalibration, the objective is to closely match predicted and actual ESALs applied without this design safety factor applied. Therefore, reliability should be set at 50% (average), which yields a standard normal deviate equal to 0 and the $Z_R S_0$ term drops out of Equation 17.

The ΔPSI term should be obtained as described in Section 3.2.1, and the SN term is as defined in Equation 16 based on the iterative $\hat{a}_1$ term. The soil resilient modulus (M_R) should be calculated from falling weight deflectometer (FWD) testing of the section at a 9,000 lb load level. Refer to Section 3.1.1 and Equation 2 for the AASHTO recommendations regarding determination of subgrade soil modulus. AASHTO further recommends, when using Equation 17, that the M_R value determined through backcalculation be divided by three to account for differences in how testing was conducted during the AASHO Road Test versus modern FWD testing (*2*). This AASHTO (*2*) recommendation is only for fine-grained cohesive soils and no recommendation is made for granular, coarse-grained soils. Finally, if the soil modulus changes appreciably with the seasons, it is recommended that the AASHTO procedure for adjusting soil modulus to reflect these seasonal changes be followed (*2*). This requires testing at multiple times during the course of a year to establish the seasonal trends and using another AASHTO empirical equation that relates pavement damage to soil modulus to compute a weighted average soil modulus based on seasonal duration and damage potential (*2*).

3.2.4 Determination of $\hat{a}_1$

For a given pavement section, the outcome of subsections 3.2.1 through 3.2.3 is a simple table listing the predicted and actual ESALs at specific points in time for a particular $\hat{a}_1$ value. For example, Table 3.2 shows the actual and predicted ESALs for an NCAT Test Track section assuming 0.44 as the asphalt structural coefficient. Notice that the predicted ESALs far underestimate the actual ESALs applied by 46% to 65%. The objective is now to improve the prediction by adjusting $\hat{a}_1$ such that the error is minimized. It is recommended to follow a least-squares regression procedure to minimize the error.

Table 3.2 Example ESAL Differences Assuming a_1 = 0.44 (*8*)

Predicted ESALs	Actual 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%

Following standard statistical regression procedures (*22*), the differences between actual and predicted ESALs must be squared and summed to obtain the error sum of squares (*SSE*), which is defined as:

$$SSE = \sum_i \left(PredictedESAL_i - ActualESAL_i \right)^2 \qquad \text{(Equation 18)}$$

Next, the mean should be obtained for the actual ESALs ($\overline{\text{ActualESAL}}$) and the difference between that mean and each predicted ESAL level must be squared. The sum of these values represent the total sum of squares (*SST*):

$$SST = \sum_i \left(PredictedESAL_i - \overline{ActualESAL} \right)^2 \qquad \text{(Equation 19)}$$

The Pearson's coefficient of determination (R^2) may be calculated from the SSE and SST as a measure of how well the predicted and actual ESALs match (*22*):

$$R^2 = 1 - \frac{SSE}{SST} \qquad \text{(Equation 20)}$$

To perform the regression, it is recommended to use the Solver add-in within Excel. Solver may be set to minimize the SSE term while only changing the AC layer coefficient ($\hat{a}_1$). 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 actual and predicted ESALs change. This is because both of these values are calculated using the structural number (SN), which is calculated using the layer coefficient ($\hat{a}_1$). However, Excel should automatically converge to a final least-squares solution.

Figure 3.8 summarizes the before and after calibration results for the example shown in Table 3.2. This regression resulted in a HMA layer coefficient of 0.50 and an R^2 equal to 0.74. There is a noticeable improvement in the actual vs. predicted ESAL differences after the regression was completed. It is also important to note that the errors were not completely eliminated. The objective of the regression procedure is to minimize error and bias, not to eliminate these factors.

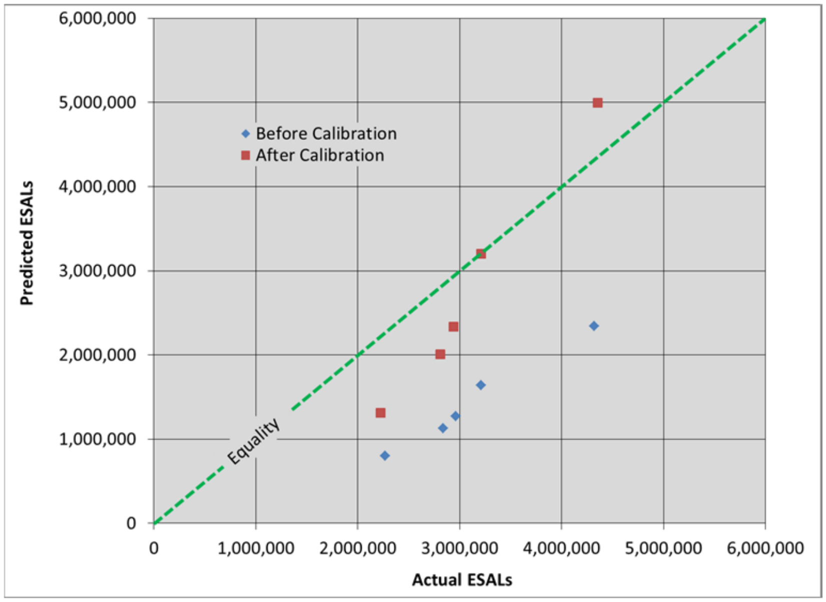


Figure 3.8 Actual vs. Predicted ESALs Before and After Calibration.

The procedure described above formed the basis of the ALDOT newly-recommended asphalt structural coefficient equal to 0.54 (*8*). Figure 3.9 shows the range of values obtained in that investigation, which found all the values were within the range originally calculated at the AASHO Road Test (*1*) that varied from 0.33 to 0.83. When conducting recalibration, it is important to check the final results for reasonableness against the original values. It is also important to select a range of pavement sections that exhibited a range of performance to avoid biasing the recalibrated coefficient toward conservative or liberal designs. In the case of the Test Track sections depicted in Figure 3.9, they represented a range of cross-sections that included a variety of thicknesses (5 inches to 14 inches of AC), two subgrade types (AASHTO A-4 and A-7-6) and different types of aggregate base. They also included a range of AC materials that included unmodified and SBS-modified asphalt binder, SMA, and mixtures designed as a rich-bottom (2% air voids). This variety of cross-sections resulted in a wide range of performance histories that included bottom-up fatigue cracking, surface rutting, substructure rutting, top-down cracking and, in some cases, no measurable distress (*8*).

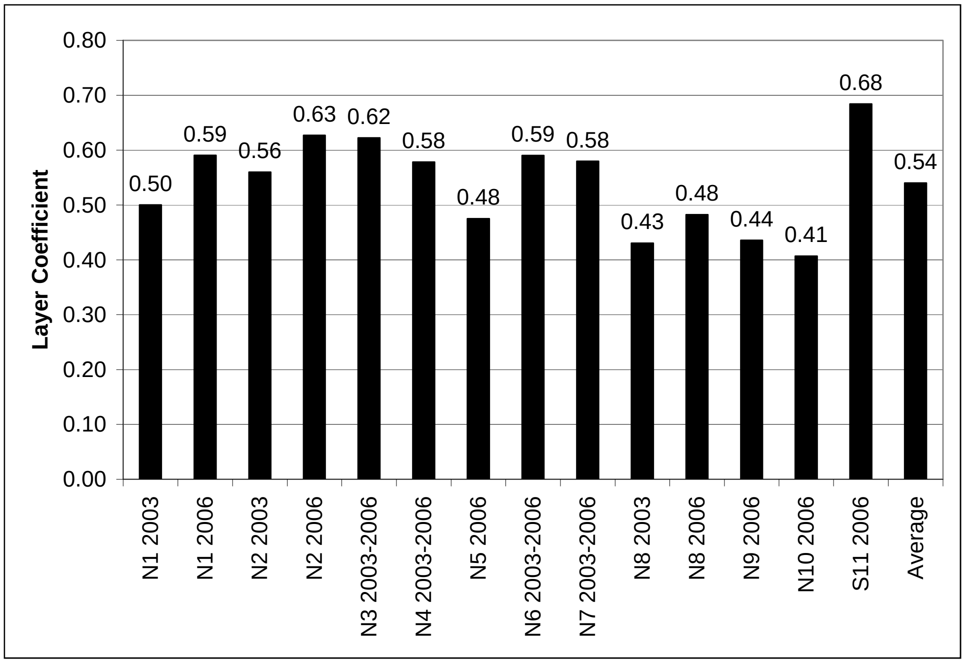


Figure 3.9 NCAT Test Track Asphalt Layer Coefficients (*8*).

3.3 Mechanistic-Empirical Procedures

The last procedure to consider relies upon using the MEPDG, locally calibrated with performance data, to establish pavement layer thicknesses from which the structural coefficients may be determined. This comprehensive approach, developed and used in Washington (*11*), is conceptually straightforward but very time and data intensive. Agencies should consider this approach if efforts are already in progress toward calibrating the MEPDG or if local calibration has been completed. The general steps are as follows and discussed in the following subsections:

1. Locally calibrate the MEPDG.
2. Use the locally calibrated MEPDG to generate pavement thickness designs.
3. Recalibrate a_1 to match AASHTO empirical designs to MEPDG designs.

3.3.1 MEPDG Local Calibration

Local calibration of the MEPDG is no easy task and full discussion of this topic is outside the scope of this document. However, detailed procedures were published in 2010 by AASHTO (*23*) that should be followed to execute local calibration of the MEPDG. As a brief summary, the MEPDG local calibration procedure involves identifying candidate pavement sections that have:

- as-built material property characterization
- performance data in terms of cracking, rutting and ride quality
- traffic history data characterized as load spectra
- detailed climate records

Once each of these data sets has been developed, pavement sections are simulated in the MEPDG software with trial calibration coefficients, which predict performance over time.

Comparisons between the MEPDG predictions and actual performance are made, from which the calibration coefficients may be adjusted to reduce the bias and error so that the MEPDG makes realistic predictions of measured pavement performance. The outcome of the MEPDG calibration procedure is a new set of calibration coefficients specific to a state or region for various distress predictions. For example, Table 3.3 lists the calibration coefficients obtained by the WSDOT study (*11*). It is important to emphasize that these coefficients are specific to WSDOT as they were calibrated to performance data in the Washington State Pavement Management System (WSPMS).

Table 3.3 WSDOT MEPDG Calibration Results (*data from 11*)

Distress	Coefficient	Original (National Calibration) Value	Local Calibration Value
AC Fatigue	β_{f1}	1	0.96
	β_{f2}	1	0.945
	β_{f3}	1	1.055
Longitudinal Cracking	C_1	7	6.42
	C_2	3.5	3.8
	C_3	0	0
	C_4	1,000	1,000
Alligator Cracking	C_1	1	1
	C_2	1	1
	C_3	6,000	6,000
AC Rutting	β_{r1}	1	1.05
	β_{r2}	1	1
	β_{r3}	1	1.06
Subgrade Rutting	β_{s1}	1	0

3.3.2 Use MEPDG to Generate Pavement Thicknesses

Once the MEPDG has been well calibrated, it is possible to execute pavement designs under a variety of conditions to determine the required asphalt concrete thickness. In the WSDOT study, for example, Li et al. (*11*) developed pavement thicknesses under a range of traffic levels and corresponding reliabilities as part of updating the WSDOT pavement design catalog. They fixed the aggregate base thickness according to WSDOT construction practice and experience and determined the required AC thickness using the local-calibration coefficients listed above in the MEPDG (*11*). Table 3.4 summarizes the required AC thickness for the six traffic and reliability levels in the WSDOT design catalog.

Table 3.4 WSDOT Design Comparisons (*data from 11)*

50-Year ESALs, Millions	Reliability	Base Thickness, in.	AC Thickness by Method		
			MEPDG	AASHTO 1993	
				a_1=0.44	a_1=0.50
5	85%	6	6	7.5	6.5
10	85%	6	7.4	8.5	7.5
25	95%	6	9.0	11.2	9.9
50	95%	7	11.2	12.3	10.8
100	95%	8	12.1	13.3	11.8
200	95%	9	13.2	14.5	12.8

3.3.3 Recalibrate a_1 to Match MEPDG Thicknesses

After establishing AC thicknesses for a range of pavement conditions with the MEPDG, the AASHTO empirical procedure is used to determine corresponding AC thicknesses. Table 3.4 shows thicknesses resulting from the 1993 AASHTO Guide (*2*) assuming 0.44 as the default structural coefficient. On average, using 0.44 results in pavements overdesigned by 1.4 inches. Li et al. (*11*) recalibrated a_1 to 0.50 resulting in an average difference of 0.07 inches, which was considered negligible. In other words, 0.50 better reflects the performance of asphalt materials in Washington as characterized by actual pavement performance data and modeling within the MEPDG.

The mechanistic-empirical approach to recalibration using the MEPDG is the most data intensive procedure. However, for states in the process of calibrating and implementing the MEPDG, it may be a viable option. It also serves the dual purpose of providing similar pavement design results with both the older empirical and newer M-E procedures. This is desirable since even when the new system is adopted, there may be many scenarios that do not warrant its use and the empirical design system will be employed. It is important that the empirical design system accurately reflect modern pavement performance.

4. CONCLUSIONS AND RECOMMENDATIONS

Though mechanistic-empirical pavement design may gain widespread use across the U.S. in the coming years, there is a need to update the AASHTO empirical pavement design system to account for advances in pavement materials, construction and performance. Updating the structural coefficient can help optimize asphalt pavement cross sections leading to better use of financial and natural resources. This recalibration document described three general approaches to recalibrating the asphalt structural coefficient, which are summarized in Table 4.1. Based on the information provided in this document, the following conclusions and recommendations are made:

1. The asphalt layer coefficient originally recommended by AASHO in 1962 (*1*) is not necessarily applicable in all situations. Studies in Alabama (*8*) and Washington (*11*) found a higher value better reflected actual performance. The values in each state (Alabama = 0.54; Washington = 0.50) were remarkably similar despite geographical

distance and different approaches taken in the recalibration process. State agencies might consider evaluating their value with respect to actual pavement performance.

2. Deflection-based approaches can provide structural coefficients in a relatively short time with relatively little data required. However, regression equations were developed from past pavement performance observations that may not accurately reflect the material under investigation. In the absence of historical performance records, deflection-based approaches may be considered to provide provisional structural coefficients until the new coefficient is validated with material-specific performance data.
3. The performance-based method used by Alabama (*8*) most closely replicates the process used to develop the original AASHO structural coefficient. Though historical traffic and performance records (i.e., IRI) are needed, the data are often readily available and collected as part of routine pavement management activities in many states.
4. The MEPDG approach is the most time and data-intensive procedure to follow. It should only be undertaken if MEPDG calibration activities are already in process or completed. One could view this approach as an additional useful output of the MEPDG-calibration process, as it allows states to continue using the AASHTO empirical procedure and produce pavement designs consistent between the MEPDG and empirical approach.
5. The results of any recalibration investigation should be checked against the range of original AASHO values and other investigations. The fact that the Alabama and Washington coefficients after recalibration were so similar, despite very different conditions and recalibration procedures, lends confidence to using the new values.
6. Local agencies or municipalities that may not have all the information required for recalibration could still perform recalibration by utilizing existing information available through state or other local agencies for similar roadways in their geographic regions.

Table 4.1 Summary of Methods

Procedure Type	General Process	Advantages	Disadvantages
Deflection-Based	Conduct deflection testing on existing pavement section. Use deflection data to backcalculate pavement properties. Correlate backcalculated properties to structural coefficients using pre-existing equations.	Relatively rapid procedure. Requires only short-term data sets. Relatively little deflection testing needed.	Does not correlate to section-specific performance. Relies primarily on past correlation studies.
Performance-Based	Pavement ride quality data are used to quantify changes in pavement serviceability over time. These changes are correlated to measured traffic levels (Actual ESALs) and the structural number equation is used to provide predicted traffic levels (Predicted ESALs). The structural coefficient is used as a calibration coefficient to minimize the error between actual and predicted ESALs.	Most closely replicates how the original AASHO layer coefficients were determined. Calibrates to actual pavement performance. Relatively simple method, once traffic and performance records have been compiled.	Historical performance data needed. Historical traffic data (ESALs) needed.
Mechanistic-Empirical	The MEPDG is locally calibrated and used to generate pavement thickness designs. The asphalt layer coefficient is then recalibrated to provide thicknesses that match the MEPDG thicknesses.	Calibrates both empirical and M-E approaches. Calibrates to actual pavement performance. Provides continuity between design systems.	Most intensive procedure in terms of required data. Requires calibration of the MEPDG, which is a costly and time-consuming process.

5. REFERENCES

1. Highway Research Board, "The AASHO Road Test", Report 5, Pavement Research Special Report 61E, National Academy of Sciences – National Research Council, Washington, DC, 1962.
2. AASHTO Guide for Design of Pavement Structures. Washington D.C.: American Association of State and Highway Transportation Officials, 1993.
3. Timm, D.H., M.M. Robbins, N. Tran and C. Rodezno, "Flexible Pavement Design – State of the Practice," National Asphalt Pavement Association, 2014.
4. AASHTO, Mechanistic-Empirical Pavement Design Guide, A Manual of Practice, Interim Edition, July 2008.
5. Pierce, L.M. and G. McGovern, "Implementation of the AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) and Software," Third Draft, NCHRP Project 20-05, Topic 44-06, October, 2013.
6. George, K.P., "Structural Layer Coefficient for Flexible Pavement," ASCE Journal of Transportation Engineering, Vol. 110, No. 2, 1984, pp. 251-267.
7. AASHO, "AASHO Interim Guide for Design of Pavement Structures-1972," Washington, D.C., 1972.
8. Peters-Davis, K. and D.H. Timm, "Recalibration of the Asphalt Layer Coefficient," Report No. 09-03, National Center for Asphalt Technology, Auburn University, 2009.
9. Timm, D.H. and A.L. Priest, "Material Properties of the 2003 NCAT Test Track Structural Study," Report No. 06-01, National Center for Asphalt Technology, Auburn University, 2006.
10. Davis, K. and D. Timm, "Structural Coefficients and Life Cycle Cost," Proceedings, T&DI Congress 2011: Integrated Transportation and Development for a Better Tomorrow, Proceedings of the First T&DI Congress 2011, American Society of Civil Engineers, Chicago, IL, 2011, pp. 646-655.
11. Li, J., J.S. Uhlmeyer, J.P. Mahoney and S.T. Muench, "Use of the 1993 AASHTO Guide, MEPDG and Historical Performance to Update the WSDOT Pavement Design Catalog," WA-RD 779.1, Washington State Department of Transportation, 2011.
12. Kuennen, T., "How Alabama Gets More Bang for Its Asphalt Buck," Volume 15, No. 1, Hot Mix Asphalt Technology, January/February 2010, 30-35.
13. FHWA, "LTPP Manual for Falling Weight Deflectometer Measurements Operational Field Guidelines," Version 3.1, August 2000.
14. Timm, D.H., A. Vargas-Nordcbeck, "Structural Coefficient of Open Graded Friction Course," Transportation Research Record 2305, Transportation Research Board, 2012, pp. 102-110.
15. Hossain, M., A. Habib and T.M. LaTorella, "Structural Layer Coefficients of Crumb Rubber-Modified Asphalt Concrete Mixtures," Transportation Research Record No. 1583, Transportation Research Board, 1997, pp. 62-70.
16. Ullidtz, P., "Pavement Analysis," Elsevier, N.Y., 1987, pp. 221-223.
17. Gulen, S., R. Woods, J. Weaver, and V.L. Anderson, Correlation of Present Serviceability Ratings with International Roughness Index. Transportation Research Record 1435, Transportation Research Board, Washington, D.C. 1994.

18. Holman, F., Guidelines for Flexible Pavement Design in Alabama. Alabama Department of Transportation, 1990.
19. Hall, K.T., and C.E.C. Munoz, Estimation of Present Serviceability Index from International Roughness Index. Transportation Research Record 1655, Transportation Research Board, Washington, D.C. 1999.
20. Al-Omari, B. and M.I. Darter, Relationships between International Roughness Index and Present Serviceability Rating. Transportation Research Record 1435, Transportation Research Board, Washington, D.C. 1994.
21. Huang, Y.H., Pavement Analysis and Design. 2nd ed. New Jersey: Prentice Hall, 2004.
22. McClave, J.T. and F.H. Dietrich II, Statistics, Sixth Edition, MacMillan College Publishing Company, New York, New York, 1994.
23. AASHTO, "Guide for the Local Calibration of the Mechanistic-Empirical Pavement Design Guide," American Association of State Highway and Transportation Officials, Washington, D.C., 2010.