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RESEARCH ARTICLE

# AASHTO 1993 Plus: an alternative procedure for the calculation of structural asphalt layer coefficients

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**ABSTRACT**

Many road agencies worldwide still use the empirical AASHTO 1993 pavement design method, which is based on the AASHO Road Test. Since the release of the AASHTO 1993 design guide, many aspects of pavement design have changed, including but not limited to asphalt material (binder and mixture) properties, traffic, and environmental factors. However, many agencies use a single layer coefficient for asphalt mixtures and pavement structures are currently designed with inaccurate values, as proved by many. This paper presents a four-step procedure (called AASHTO 93 Plus method) to determine structural layer coefficients of asphalt mixtures. The AASHTO 93 Plus method considers viscoelasticity and temperature-dependency of asphalt mixtures to estimate the asphalt mixture layer coefficient. Five-hundred-eighteen (518) flexible pavement structures from USA and Canada roads and 1599 HMAs from the Long-Term Pavement Performance (LTPP) database were analyzed for this evaluation. The results showed that the new material-specific layer coefficients were well above the typical value of 0.44 commonly used in USA and worldwide. The total number of 3,726 IRI data records of these 518 pavement structures were converted to PSI and then compared to those obtained from the traditional AASHTO 93 and AASHTO 93 Plus methods for validation. Results confirmed that the conventional AASHTO 93 method (with a single layer coefficient) predicts faster deterioration of pavement conditions over time, while the results of AASHTO 93 Plus method better aligned with the field data.

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## Introduction

Road agencies around the world currently use many different approaches to design flexible pavements. Even though mechanistic pavement analysis methods have improved significantly over the past two decades, empirical methods are still by far the most popular methods used by the highway agencies. A survey conducted in 2013 and published in 2014 by the National Cooperative Highway Research Program (NCHRP) indicated that most responding agencies use the AASHTO 1993 guide (AASHTO 1993) for the design of pavement structures (Pierce and McGovern 2014). Other agencies in the USA adopt the AASHTO guide for design of pavement structures, with 1998 supplement (AASHTO 1998) and the AASHTO interim guide for design of pavement structures (AASHTO 1972). This is principally due to the fact that these procedures have served well for several decades. Since the implementation rate of the mechanistic-empirical method developed under the NCHRP Project 1-37A (ARA Inc. 2004) has been relatively slow, it is reasonable to think that these empirical methods will be used for many years to come.

All versions of the AASHTO 1993 procedure are based on the experimental program developed in Illinois in the late 1950s and early 1960s by the American Association of State Highway Officials (AASHO) (AASHTO 1961). Pavement distresses and surface roughness was periodically measured and used to develop the structural design procedure described in the AASHTO guide. The provided equation links the pavement serviceability to traffic volume, structural capacity, and statistical parameters as shown in Equation (1).

$$\log(W_{18}) = Z_R \cdot S_0 + 9.36\log(SN+1) + \frac{\log\dfrac{\Delta PSI}{4.2-1.5}}{0.40+\dfrac{1094}{(SN+1)^{5.19}}} + 2.32\log M_R - 8.27 \quad (1)$$

where $W_{18}$ is the number of 18 kips equivalent single-axle load (18 kips ESAL), $SN$ is the structural number, $M_R$ is the resilient modulus of the subgrade (psi), and $Z_R$ and $S_0$ are the standard normal deviate and standard deviations error in predicting pavement serviceability, respectively. From Equation (1), designers obtain the required structural capacity ($SN$) for a given combination of traffic volume (ESALs), subgrade soil, terminal serviceability index, and road categories ($Z_R$ and $S_0$) (Fuentes et al. 2021, Sidess et al. 2021). Then, the $SN$ of the final structure must be equal to or greater than the required $SN$. The $SN$ is an abstract number that combines characteristics of layer thicknesses and the materials through the so-called layer coefficients ($a_i$) obtained during the original

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AASHO experiment. Although the AASHTO 1993 guide specifies how those coefficients may vary depending on material properties and type of layer, traditionally, a single value of 0.44 is assigned by designers to all Hot Mix Asphalts (HMA), considering that only one kind of subgrade and one HMA in a single environmental condition were tested during the AASHO Road Tests. Moreover, asphalt technology has advanced since then: more durable materials are produced nowadays (e.g. polymer-modified bitumen), and even mix design processes have changed significantly.

In light of these novelties, several studies have attempted to either calculate structural layer coefficients of specific mixtures, re-calibrate the existing layer coefficients, or provide a framework for including damage-related indices of materials in the calculations of those coefficients, as listed below:

In 1997, Hossain et al. evaluated different pavement sections constructed with crumb-rubber modified HMA in the attempt to assign a proper layer coefficient to this type of HMA mixture. Resilient moduli were back-calculated using FWD measurements, and the resulting layer coefficient was found to be lower than the one used during the pavement design process (Hossain et al. 1997).

The same approach was used in a study commissioned by the Maine Department of Transportation in 2003 for calculating the layer coefficients of recycled materials with foamed asphalt. FWD and laboratory resilient modulus tests lead to a layer coefficient of 0.22 for such materials (Marquis et al. 2003).

Underwood and Zeiada, in 2015, evaluated the layer coefficient of fiber-reinforced HMAs and calculated the resulting potential thickness reduction of the pavement structure. The study was conducted on US-based materials, although the actual application of the fiber-reinforced asphalt concrete was intended for Peru, where the AASHTO 1993 methodology is used for pavement design purposes. The layer coefficient was found to be equal to 0.54, leading to a 16% reduction of the overall pavement thickness (Underwood and Zeiada 2015).

An extensive study on asphalt layer coefficients was also conducted for the Wisconsin Department of Transportation in 2002. Layer coefficients were calculated for new and reprocessed asphaltic mixtures using two approaches. Initially, the traditional calculation based on resilient modulus was performed using laboratory and field test results. Then, a conceptual procedure was investigated in order to include materials' damage in the calculation of the *a*-values. This second approach, although innovative, required a series of laboratory tests for HMA characterisation that are similar to those needed for the implementation of the MEPDG methodology (e.g. HMA rutting and fatigue cracking tests) (Bahia et al. 2000).

Another comprehensive study was performed in 2011 at the National Center of Asphalt Technology (NCAT) by Davis and Timm for the Alabama Department of Transportation (Davis and Timm 2011). In this investigation, data were collected from different test sections built on the NCAT test track. Other than properties of materials, International Roughness Index (IRI) was measured periodically too. These values were then converted into PSI using the equation proposed by Al-Omari and Darter (1994). Then, layer coefficients were back-calculated so that the AASHTO 93-based predicted variation of PSI over time matched the field PSI obtained from IRI conversion. The approach used by Davis and Timm for back-calculation of *a*-values highlighted a fundamental point often disregarded in other studies. As in the case of the MEPDG lab-to-field calibration, also new or revised asphalt layer coefficients shall be validated through actual field conditions (e.g. PSI, PCI, or IRI). A direct correlation between characteristics of materials and layer coefficients without such validation, in fact, may not be appropriate since it ignores other variables.

The Department of Transportation of New Hampshire, in 2019, re-calibrated the asphalt layer coefficients for their pavement design guide. The IRI-to-PSI conversion was performed using data from 17 road sections of their pavement management system. An average *a*-value of 0.58 was associated with the HMAs in those sections. As part of this study, a mixture property-based predictive model for layer coefficients, including rutting, fatigue cracking, and transverse cracking laboratory-based information, was also developed for the base and intermediate HMAs (Dave et al. 2019).

More recently, the idea of including damage characteristics of materials into the calculation of an HMA structural coefficient was implemented by Habbouche et al. (2020) for asphalt mixtures produced with high polymer modified bitumen. The authors used four-point beam bending fatigue tests to calibrate the MEPDG fatigue damage model and then used those calibration factors in the 3D-Move software. The long-term fatigue performance obtained from this analysis was used for the back-calculation of a single value (0.54) indicative of the structural capacity of high-polymer modified HMAs for the AASHTO 93 procedure (Habbouche et al. 2020). In a second study, the same authors used that layer coefficient to design a pavement structure using the AASHTO 93 method and verified that same structure can be obtained using the calibrated mechanistic-empirical methodology with acceptable performance (Habbouche et al. 2021).

The findings of the studies mentioned above indicate that the use of a single value for all HMAs or even for one category of HMAs is not reliable, and the asphalt layer coefficient is dependent on mixture design and composition. However, so far, no procedure is provided to designers to calculate asphalt layer coefficients at the mixture and pavement design stages.

This paper attempts to address this gap by presenting the results of application of a four-step procedure for the determination of asphalt layer coefficients for the AASHTO 93 design of flexible pavements. The procedure was developed based on structural and materials data extracted from 518 pavement sections (1599 HMA mixtures) included in the Long-Term Pavement Performance (LTPP) database (LTPP InfoPave 2022). The total number of 3,753 IRI records were available for these 518 sections which were converted into present serviceability index (PSI) values and used for validating the new procedure for calculating more realistic asphalt layer coefficients.

## Methodology

The methodology followed in this study is illustrated in Figure 1. It consists of three phases: data acquisition, calculation of asphalt layer coefficients ($a_i$) through the AASHTO

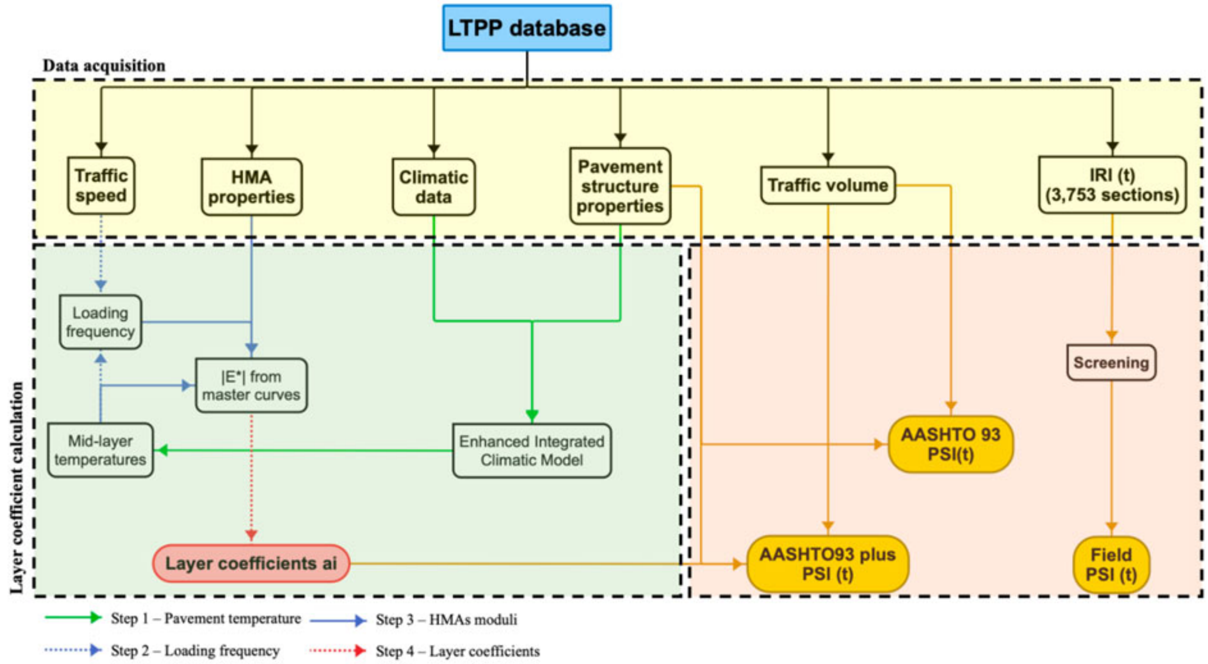


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**Figure 1.** Study methodology illustration.

93 Plus procedure, and validation. Details for each of these phases will be provided in the following subsections.

## Data acquisition (LTPP database)

The properties and field IRI measurements for 518 different flexible pavement sections were extracted from the Long-Time Pavement Performance (LTPP) database. A variety of asphalt mixtures were included in the extracted pavement sections, such as recycled mixtures, warm-mix HMAs, gap/open-graded mixtures, etc. This was the maximum number pavement sections in the LTPP for which the required analysis data were available. The following raw data were extracted for each pavement section:

- Project location;
- Traffic volume;
- Posted traffic speed;
- Pavement structure (thickness of layers);
- Materials properties (unit weight, thermal conductivity, heat capacity, HMA master curve coefficients, etc.);
- IRI measurements during pavement life.

Based on the extracted data, the experienced traffic, expressed in Equivalent Standard Axle Load (ESALs), was reported annually for each pavement section. However, it should be noted that the missing traffic data at some years were interpolated. Given the experienced traffic, the design traffic of the pavement sections can be calculated over the pavement life. Overall, 23% of the pavement sections experience a traffic volume of fewer than 1 million ESALs (mESALs), 32% between 1 and 3 mESALs, 37% from 3 to 10 mESALs, and 8% above 10 mESALs.

The extracted pavement sections were spatially distributed in 46 states in USA and Canada as their distribution is shown in Figure 2. This relatively wide spatial distribution of the pavement sections was selected in order to evaluate the comprehensiveness of the proposed AASHTO 93 Plus methodology over a wide range of the climatic, traffic, and material circumstances.

The IRI measurements for each of the selected pavement sections were available for at least three different times during the pavement life, which resulted in the total number of 3,726 IRI measurement records. The progression of the IRI over the years is affected by maintenance or resurfacing works. The characteristics of the pavement structures and the magnitude of the IRI can be significantly different before and after a maintenance cycle. For this reason, IRI data were cross-checked with the date of maintenance works provided by LTPP, and data points after the maintenance cycle were discarded.

## Asphalt layer coefficient calculation procedure

The four steps bellow illustrate the proposed methodology of this study for calculation of the structural coefficient of bituminous layers. For this purpose, the pavement structures and the material properties extracted from the LTPP were utilised.

### Step 1 – pavement temperature spectrum

A climatic model, called MClim, was developed to predict the pavement temperature profile with depth based on an improved climatic-materials-structure (CMS) model, which was originally developed by Dempsey (1969). The main improvement is on the computation of downwelling short-wave radiation (D-SWR) from the sun, downwelling long-wave radiation (D-LWR) from the atmosphere, and upwelling

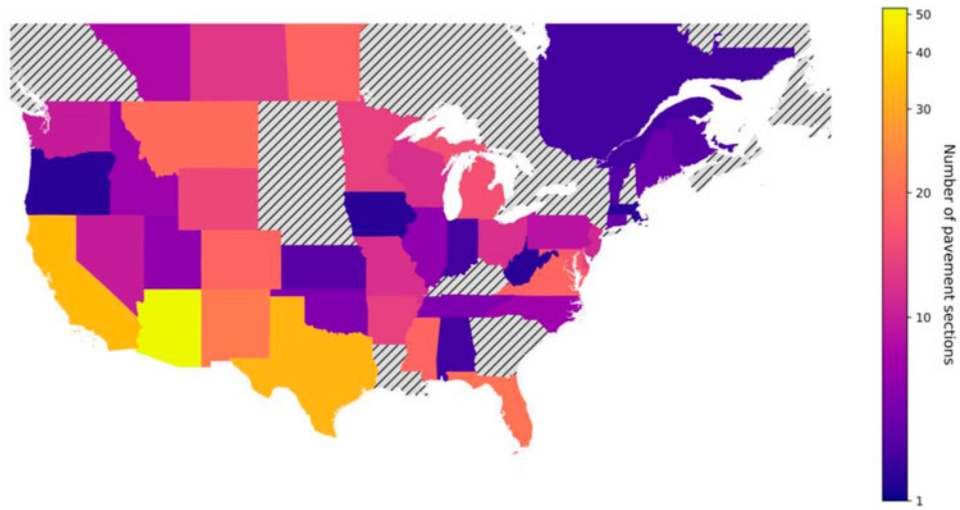




**Figure 2.** Spatial distribution of pavement sections extracted from the LTPP and used in this study (no data were available for areas states with parallel lines).

long-wave radiation (U-LWR) (Alam-Khan et al. 2021). Another improvement is on the calculation of sunrise and sunset times, which are important in calculation of net radiation flux at the pavement surface. The same model was used in the Mechanistic-Empirical Asphalt Pavement Analysis (MEAPA) web-application (Kutay and Lanotte 2020, Ghazavi et al. 2022). MClim was used to compute the temperature profile within 518 pavement structures (Dempsey 1969). The algorithm provided by Dempsey for the calculation of sunrise and sunset times in different days of the year at multiple geographic locations was substituted with a new one. Data from the MERRA database (Global Modeling and Assimilation Office (GMAO) 2022) were used for the calculation of the hourly pavement temperatures. Then, the monthly average temperatures at the middle of each layer were calculated for the next steps of the analysis.

### *Step 2 – assessment of the load frequency*

The equivalent loading frequency was calculated from the average vehicle speed (Losa and Natale 2012) as shown in Equation (2) and (3).

$$f = 0.043\frac{V}{2a}\cdot\exp\left(-2.65z + \beta(T)\right) \quad (2)$$

$$\beta(T) = 1.25\times10^{-5}\times T^3 - 1.6\times10^{-3}\times T^2 + 9.2 \times 10^{-2}\times T \quad (3)$$

where: $f$ is the load frequency (Hz), $V$ the vehicle speed (m/sec), $a$ is the radius of tire pressure (m), $z$ is the depth from surface to the center of the AC layer (m), and $\beta$ is a function of the average pavement temperature, $T$ (°C). Twelve frequency values, one for each month of the year, were calculated for mid-layer location of each bituminous pavement layer.

### *Step 3 – determination of dynamic modulus (|E*|)*

The calculated temperatures and frequencies at the mid-layer locations of the bituminous layers were then used for estimating the dynamic modulus ($|E^*|$) values from the $|E^*|$ master curves. The master curve coefficients for each bituminous layer were acquired from the LTPP database.

### *Step 4 – assessment of HMA layer coefficients*

Monthly dynamic moduli ($|E^*|$) computed in the previous step were converted to layer coefficients using Equation (4) (AASHTO 1993).

$$a_1 = 0.171\ln\left(|E^*|\right) - 1.784 \quad (4)$$

where $|E^*|$ is the HMA dynamic modulus (psi). The $a$ coefficient values were calculated for each month of the year and each bituminous layer. These values were averaged to obtain a single specific layer coefficient for each bituminous layer as an effective value for the layer.

## *Validation procedure*

For validation purposes, Equation (1) was rearranged as shown in Equation (5).

$$\Delta PSI = 2.7\times 10^{\left[(\log W_{18} - Z_R\cdot S_0 - 9.36\log(SN+1) - 2.32\log M_R - 8.27)\left(0.40+\frac{1094}{(SN+1)^{5.19}}\right)\right]} \quad (5)$$

The structural number ($SN$) for each section was first calculated as per the traditional AASHTO 93 procedure by assigning the layer coefficient from the LTPP database (almost 0.44 for all HMAs). Then, PSI values were calculated using the accumulated design traffic at each year during the pavement life. The same steps were repeated using the layer coefficients obtained from using the developed AASHTO 93 Plus procedure. Finally, considering the initial PSI of 4.2, the PSI over the pavement life were calculated.

To evaluate the validity of the AASHTO 93 Plus procedure, the IRI values from the LTPP database were converted to PSI for a direct comparison with the results of the AASHTO 93 and AASHTO 93 Plus procedures. Five conversion equations were considered for converting the field IRI values to the PSI, as shown in Table 1.

Given the assumption of AASHTO 93 procedure, the initial PSI in Equation (1) was fixed to 4.2 for both AASHTO 93 and AASHTO 93 Plus procedures. However, none of the equations listed above could ever return such a value at the beginning of the pavement service life. Hence, all data series were normalised accordingly for better evaluating the PSI trend over time. An example of the IRI conversion procedure with the five conversion models is provided in Figure 3.

The accuracy of the predictions was evaluated based on the average of the errors (Bias) and the standard error of the estimates (SEE) indices, defined as in Equations (6) and (7).

$$Bias = \sum \frac{PSI_{model} - PSI_{field}}{N} \tag{6}$$

$$SSE = \sqrt{\frac{\sum (PSI_{model} - PSI_{field})^2}{N}} \tag{7}$$

**Table 1.** Conversion models for IRI to PSI.

| IRI-to-PSI model | IRI unit | Reference |
|---|---|---|
| $PSI = 9\exp(-0.008784\,IRI)$ | m/km | (Gulen et al. 1994) |
| $PSI = 5\exp(-IRI/5.5)$ | m/km | (Paterson 1986) |
| $PSI = 5\exp(-0.24\,IRI)$ | m/km | (Al-Omari and Darter 1994) |
| $PSI = 5\exp(0.198 - 0.261\,IRI)$ | m/km | (LCI TM Inc. 1995) |
| $PSI = 5 - 0.2397x^4 + 1.771x^3 - 1.4045x^2 - 1.5803x$, $x = \log(1 + 2.2704\,IRI^2)$ | m/km | (Hall and Munoz 1998) |

where $N$ is number of data point, $PSI_{field}$ is the field PSI (converted from IRI), and $PSI_{model}$ is the PSI predicted via either AASHTO 93 or AASHTO 93 Plus procedures. Therefore, the positive average of the errors means that the AASHTO 93 (Plus) procedure overestimated the PSI condition of the field, while the negative average of the errors indicated the conservative case, where the AASHTO 93 (Plus) model underestimated the field condition.

## Results and discussion

### AASHTO 93 plus asphalt layer coefficients

The average temperature at the mid-depth of all asphalt layers were calculated using the climatic model, as shown in Figure 4. The error bars show the statistical variation of the calculated temperature within each state. This figure shows a reasonable trend in the calculated temperatures and the climatic condition of the states.

The average frequency at the mid-depth of all asphalt layers were calculated and presented in Figure 5, in which the error bars show the statistical variation of the calculated frequencies for each state. As this figure shows, the calculated loading frequencies were considerably different from the common assumption of 10 Hz, which is used in the original AASHTO 93 procedure. Given these calculated temperature and frequencies, as well as master curve and shift model coefficients for each asphalt mixture, the corresponding $|E^*|$ were calculated.

Structural coefficients of bituminous layers calculated using the proposed procedure of AASHTO 93 Plus are presented in Figure 6, categorised based on the states. The error bars show

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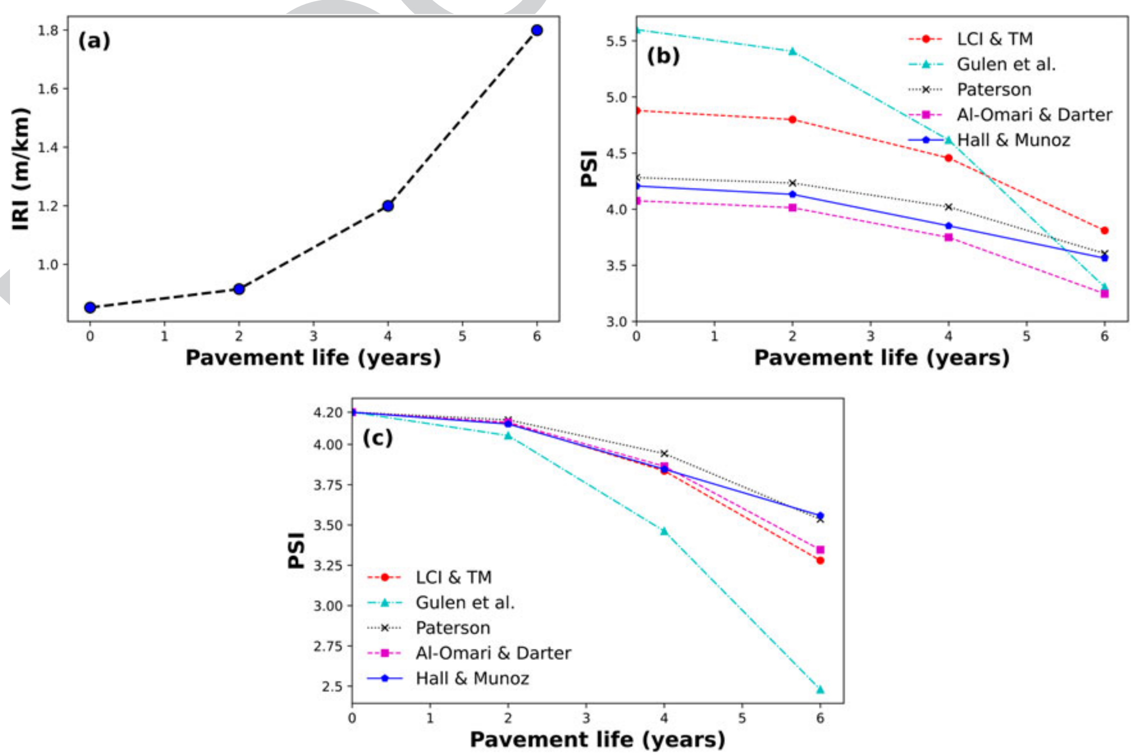


**Figure 3.** Example for IRI trend and corresponding PSI: (a) measured IRI values, (b) estimated PSI values, and (c) estimated PSI values normalised to 4.20.

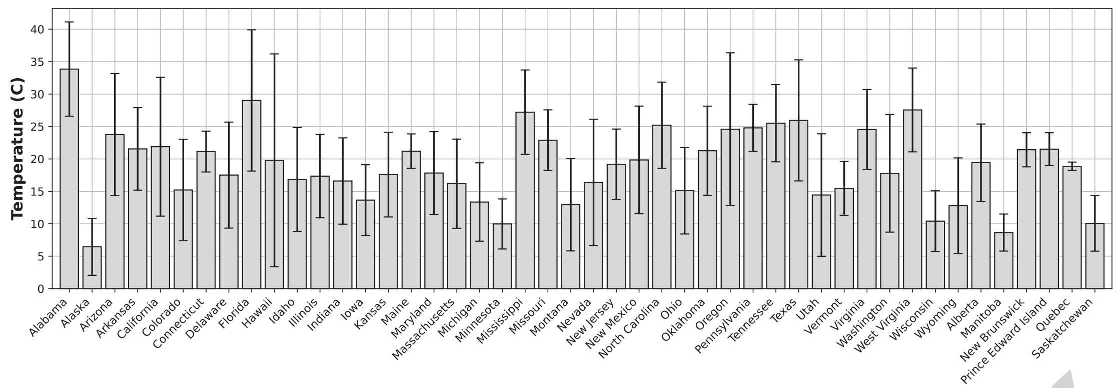


**Figure 4.** Temperatures at the mid-depth of all asphalt layers calculated with AASHTO 93 plus.

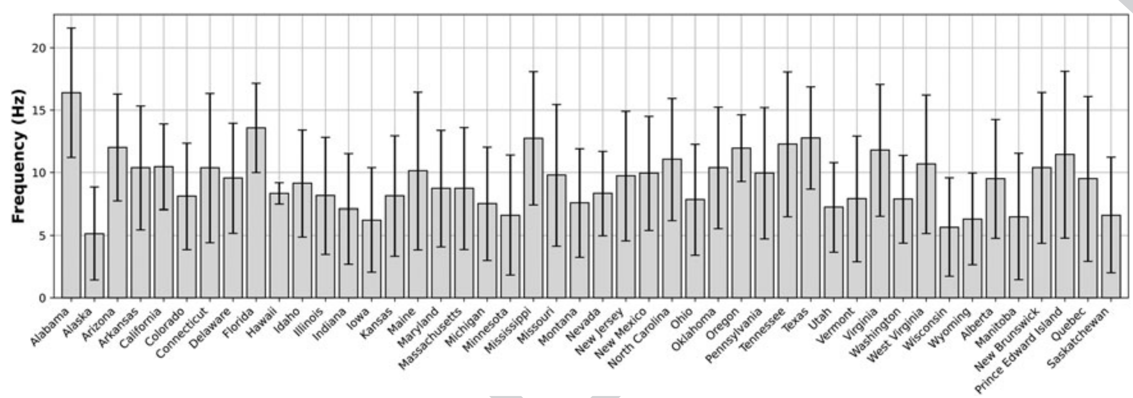


**Figure 5.** Frequencis at the mid-depth of all asphalt layers calculated with AASHTO 93 plus.

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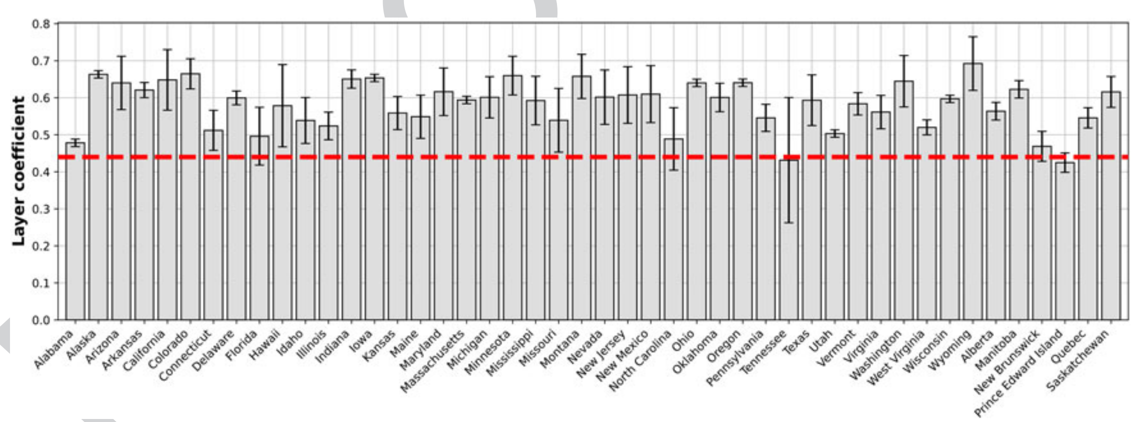


**Figure 6.** Structural layer coefficients calculated with AASHTO 93 plus.

the statistical variation of the calculated $a$-values in each state. The red dashed line indicates, for comparison purposes, the layer coefficient of 0.44, which is the sole value used in many states around the world for all types of HMAs. With the only exception of the mixtures from Tennessee (USA) and Prince Edward Island (Canada), all coefficients were above 0.44, which indicates that the bearing capacity of the HMAs is currently underestimated. Moreover, the variability of the new values demonstrates the ability of the procedure to discern the behaviour of different materials under the naturally different loading and climatic conditions that an HMA might experience in various project locations.

As shown in Figure 6, higher structural coefficients calculated by AASHTO 93 plus in most of the states can be translated in terms of lower thicknesses of asphalt layers, which is a potentially promising results for the asphalt agencies. In this regard, Lanotte and Kutay (2017) utilised similar methodology for designing three pavement structures and came up with lower asphalt layer thicknesses compared with the original AASHTO 1993 procedure. These thickness reductions were then verified with a more advanced mechanistic-empirical methods.

### *Validation process*

Figure 7 graphically summarises the results obtained from the converted field IRI measurements into PSI values against those calculated using the traditional AASHTO 93 ($a = 0.44$) and AASHTO 93 Plus procedures for the state of Arizona in USA. There were 52 pavement sections with the total of 336 measured IRI data records for this state in LTPP data base. Similar graphs were developed for all US states and Canadian provinces but not shown here for brevity.

According to the Figure 7, data points above the equality line indicate that the evolution of the PSI over time predicted by the selected design procedure is slower than the one observed in the field. Hence, that pavement structure is under-designed by that design procedure. On the contrary, data points below the equality line indicate that the structure is over-designed. Both cases should be avoided for financial and environmental reasons. Still, it is well-known that many other factors play a role in the pavement damage evolution (e.g. poor construction procedure), and no perfect fit can be achieved by any pavement design procedure. Thus, it is essential that the cloud of points associated with a design methodology shows good overall accuracy of correlation between predicted and field PSI values. In this regard, it can be visually concluded from Figure 7 that for the state of Arizona, AASHTO 93 Plus design procedure resulted in a better correlation to the line of equality compared the traditional AASHTO 93 one, regardless of the IRI-to-PSI conversion model.

The performance of AASHTO 93 and AASHTO 93 Plus design procedures were quantitatively evaluated based on the correlations with the field PSI values shown in Figure 7 using Bias and SEE statistic indices. In this regard, Figure 8 and Figure 9 shows the SEE and Bias indices, respectively, for each state using the IRI-to-PSI model of Al-Omari and Darter (1994). The same graphs for other IRI-to-PSI models were provided in the supplementary document. Moreover,

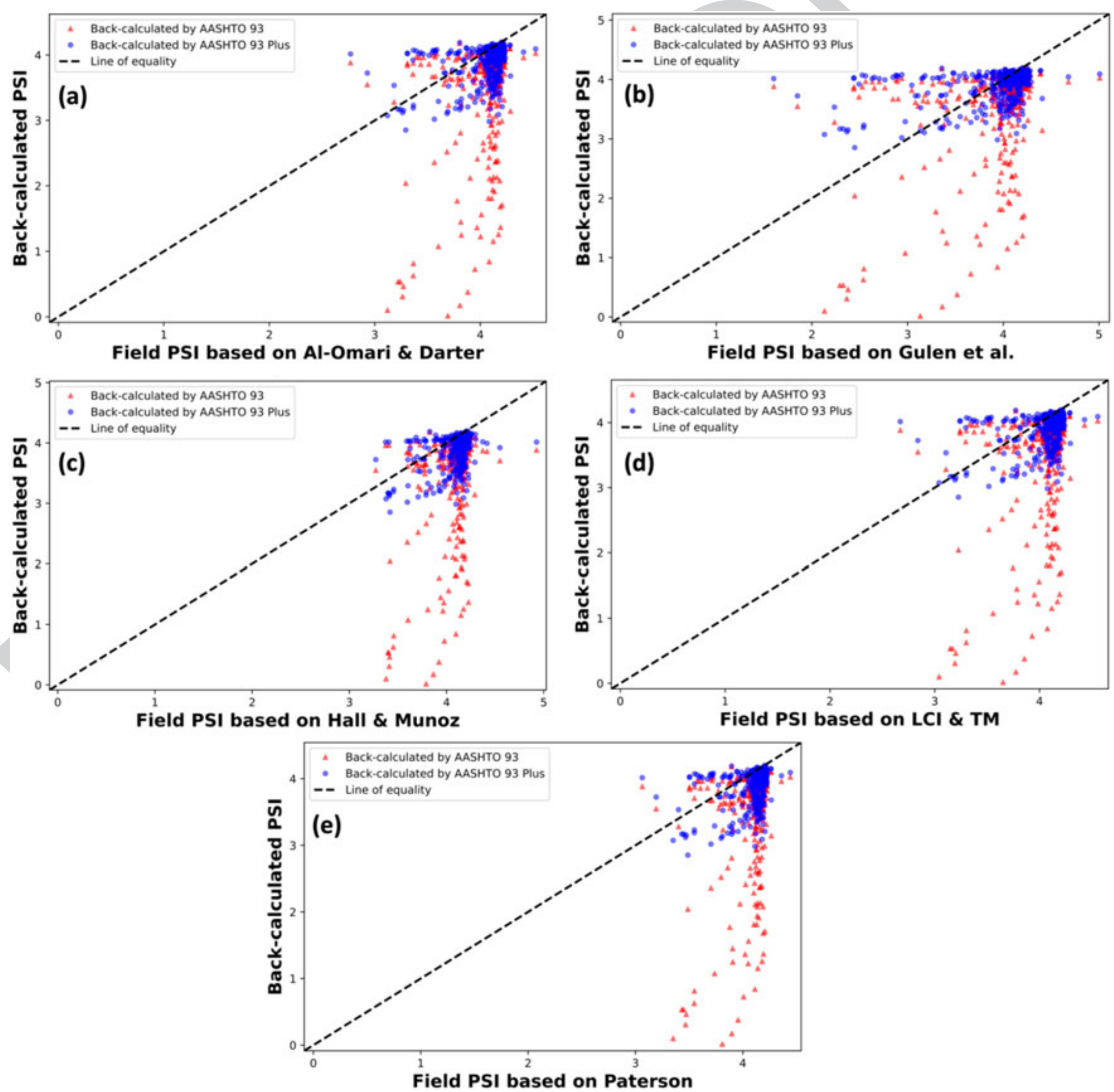


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**Figure 7.** Comparison between PSI calculated from AASHTO 93 and AASHTO 93 Plus versus field PSI converted from IRI for Arizona using (a) Al-Omari and Darter, (b) Gulen et al., (c) Hall and Munoz, (d) LCI TM Inc., and (e) Paterson conversion equations.

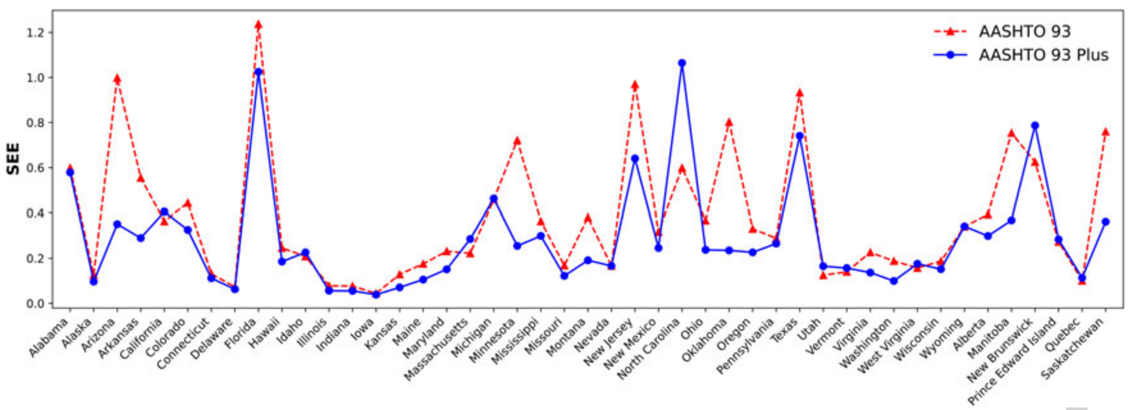


**Figure 8.** Performance of AASHTO 93 and AASHTO 93 Plus design procedures (SEE statistic) for each state using IRI-to-PSI model of Al-Omari and Darter (1994).



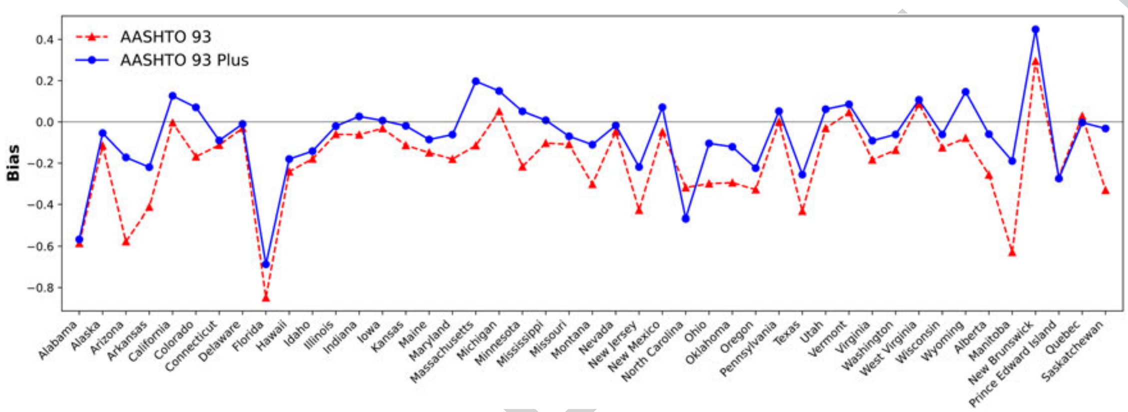


**Figure 9.** Performance of AASHTO 93 and AASHTO 93 Plus procedures (Bias statistic) using IRI-to-PSI model of Al-Omari and Darter (1994).



the weighted average of the SEE and Bias statistical indices are also presented in Table 2 and Table 3.

According to Figure 8 and Figure 9, the SEE and Bias indices of the AASHTO 93 Plus were generally lower than those of AASHTO 93 design procedure, which is in agreement with the results listed in Table 2 and Table 3. The average of errors (Bias) is indicative of a systematic error in the prediction of PSI. For this study, a negative value of the Bias implies that the systematic error leans more towards the over-design of pavement structure and vice versa. According to Table 2, except when the IRI-to-PSI model of Gulen et al. is used,

**Table 2.** Average of errors (Bias) based on IRI-to-PSI model used.

| IRI-to-PSI model | AASHTO 93 | AASHTO 93 plus | Percent difference |
|---|---|---|---|
| LCI & TM | −0.231 | −0.067 | −71% |
| Gulen et al. | −0.058 | 0.105 | +281% |
| Paterson | −0.281 | −0.117 | −58% |
| Al-Omari & Darter | −0.244 | −0.080 | −67% |
| Hall & Munoz | −0.324 | −0.160 | −51% |

**Table 3.** Standard error of estimates (SEE) based on IRI-to-PSI model used.

| IRI-to-PSI model | AASHTO 93 | AASHTO 93 plus | Percent difference |
|---|---|---|---|
| LCI & TM | 0.583 | 0.418 | −28% |
| Gulen et al. | 0.655 | 0.608 | −7% |
| Paterson | 0.591 | 0.390 | −34% |
| Al-Omari & Darter | 0.584 | 0.409 | −30% |
| Hall & Munoz | 0.622 | 0.392 | −37% |

both AASHTO 93 and AASHTO 93 Plus lead to over-designed (more conservative). Still, the average of errors using the AASHTO 93 Plus procedure was about 60% lower than those of traditional AASHTO 93 procedure. Moreover, the extent of such over-design is significantly different, and it can be evaluated through the standard error of the estimates (SEE), which shows the accuracy of the design predictions as defined by better correlation with field IRI data. According to Table 3, the SEE values obtained by the AASHTO 93 plus are approximately 30% lower than those given by the traditional AASHTO 93 procedure. This demonstrated a significant improvement of the design accuracy with the new design methodology proposed in this study, AASHTO 93 Plus. It is also worth noting that when the IRI-to-PSI model of Gulen et al. is used, the application of the AASHTO 93 Plus methodology does not significantly affect the design accuracy. The reasons why the application of the Gulen et al. model provides inconsistent indications might attribute in the equation itself and the database used for its development.

## Summary and conclusion

The empirical AASHTO 93 pavement design procedure is the most widely used all around the world. However, since its development, structural layer coefficients of HMAs have not been updated to accommodate advances of the modern asphalt materials and construction capabilities. This paper presented a four-step procedure, named AASHTO 93 Plus, to estimate the asphalt layer coefficients based on material properties and local climatic as well as traffic conditions. In the proposed method, the coefficients are calculated considering HMA loading time and temperature dependency as well as site-specific parameters (e.g. climatic conditions and posted traffic speed). The procedure can accommodate modifications related to the calculation of the dynamic modulus of the asphalt mixtures. Four steps of the proposed AASHTO 93 Plus method can be summarised as follows:

- Step 1: Calculating the pavement temperature spectrum during the design period using any finite difference or finite element method. As such an analytical tool may not be easily available, one can use the free climatic model in MEAPA software (Kutay and Lanotte 2020), available at https://www.paveapps.com/meapa.
- Step 2: Estimating the loading frequency at the middle of the bituminous layers.
- Step 3: Estimating the dynamic modulus of each bituminous layer from its $|E^*|$ master curve using the estimated temperature and loading frequency.
- Step 4: Calculating the structural coefficient of each bituminous layer using the AASHTO 93 equation (Equation 4).

In this study, HMA $|E^*|$ master curve coefficients reported by LTPP used for this purpose, but agencies can use other predictive models or data from a materials library. For the 1599 HMAs analyzed in this study, the proposed AASHTO 93 Plus design procedure returned layer coefficients often well above the typical value of 0.44, commonly used in many design agencies worldwide. This underestimation of layer coefficients leads to over design and using layers thicker than is needed to handle the traffic estimated for the design life. This statement was validated by a comparative analysis between the field PSI, converted from actual IRI measurements, and the PSI evolution over time based on the AASHTO 93 and AASHTO 93 Plus procedures. It is shown that the use of traditional AASHTO 93 design procedure returns PSI estimates that were generally lower than those back-calculated from the actual pavement condition.

It should be noted that the AASHTO 93 Plus methodology is not intended to be a competitor to the more advanced (and appropriate) design methodologies such as the one described in the NCHRP 1-37A Mechanistic-Empirical Pavement Design Guide (MEPDG). The layer coefficient estimation method presented herein is only based on the 'modulus' of the asphalt layer and does not include any damage or aging characteristics. It is very important for agencies that has the software and have well trained staff, to move towards Mechanistic-Empirical approaches to optimise pavement design for materials and traffic conditions of a region. Given the fact that Mechanistic-Empirical approaches require local calibration studies that need pavement management system data (e.g. field cracking, rutting, IRI etc.) as well as laboratory-measured material properties, many countries and states may not be able to implement Mechanistic-Empirical approaches in the near future but should consider better estimate of the layer coefficients as significant funds are invested today in better and more modified asphalt mixtures. Therefore, AASHTO 93 Plus methodology (with minimal additional effort) can be used by the road agencies where the MEPDG implementation and calibration efforts are still not available.

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