# Use of the 1993 AASHTO Guide, MEPDG and Historical Performance to Update the WSDOT Pavement Design Catalog

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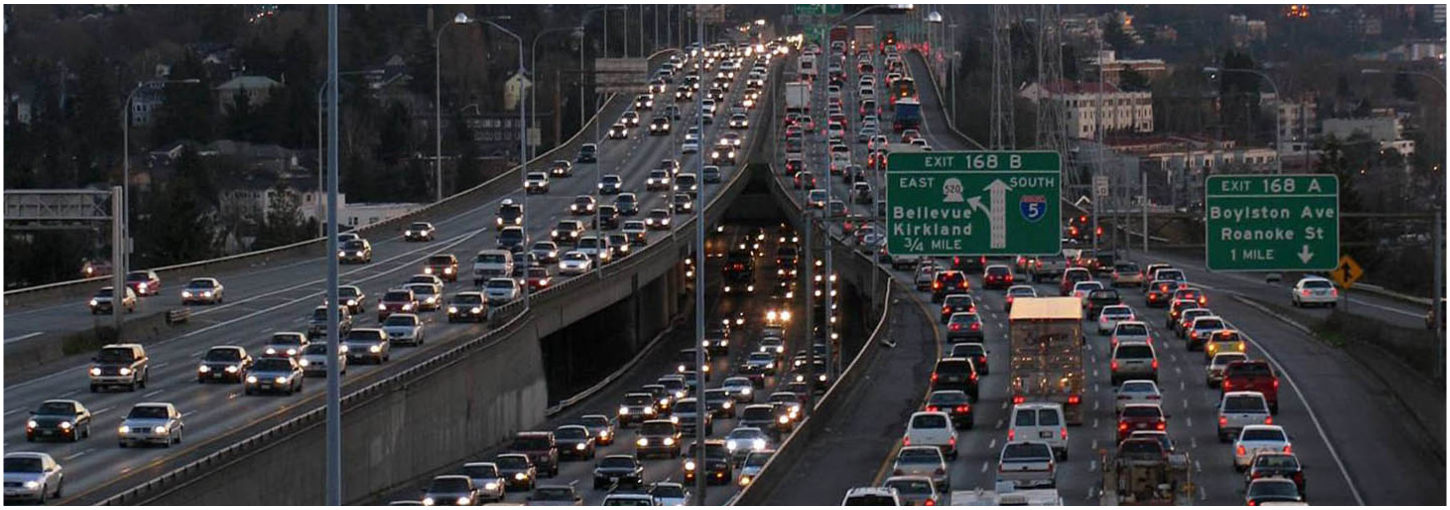
September 2011




Washington State
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WSDOT Research Report

## Research Report

# USE OF THE 1993 AASHTO GUIDE, MEPDG AND HISTORICAL PERFORMANCE TO UPDATE THE WSDOT PAVEMENT DESIGN CATALOG

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Prepared for
The State of Washington
**Department of Transportation**
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<tr><td colspan="5">16. ABSTRACT<br>This report describes the preparation of a revised pavement thickness design catalog for the Washington State Department of Transportation (WSDOT) using the 1993 American Association of State Highway and Transportation Officials (AASHTO) Guide, the Mechanistic-Empirical Pavement Design Guide (MEPDG), and WSDOT historical pavement performance data.</td></tr>
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# TABLE OF CONTENTS

# LIST OF FIGURES



# LIST OF TABLES

# CHAPTER 1 INTRODUCTION

For most state highway agencies, the current primary pavement design tool is the 1993 *AASHTO Guide for Design of Pavement Structures* (the 1993 AASHTO Guide) (*1*). While use of this empirical method has been successful, it has several generally acknowledged shortcomings including being based on a limited number of pavement sections at one location, one climate, limited traffic, and one set of materials (*2*).

Currently, the Washington State Department of Transportation (WSDOT) uses Darwin 3.01 (based on the 1993 AASHTO Guide) as well as a structural thickness catalog developed as a guide for WSDOT pavement designers. For the design tables, which are essentially the first option for state pavement design, pavement thicknesses were based on a series of typical reliability levels, subgrade resilient moduli and equivalent single axle load (ESAL) levels for Washington State (*3*). Since the current table's inception in 1992, changes in materials, mix designs, traffic loading and local practices for both rigid and flexible pavements warrant an update (minor changes have been done in earlier years).

The *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures* and its associated software (MEPDG) have been proposed as an advanced pavement design tool. With its basis in empirical field or laboratory observed performance and mechanistic principles, resulting designs are assumed to produce improved thickness estimates over traditional empirical designs (*4*). Since the release of the MEPDG in 2002, many state highway agencies have been involved in data collection, model testing, software calibration and evaluation (*5*, *6*, *7*). WSDOT calibrated the rigid portion of the MEPDG software Version 0.6 in 2005 and the flexible portion of Version 1.0 in 2008 (*8*, *9*). These efforts were primarily focused on the software functionality and model reliability and based on the historical performance data obtained from the Washington State Pavement Management System (WSPMS) (*10*).

The MEPDG has continued to evolve with the AASHTO marketed version (DARWin-ME) released July 1, 2011. Annual licenses for this software range from $5,000 for one user up to $40,000 for an unlimited number of users.

There have been a number of updates for the MEDPG since the first release. The results reported here are based on the software versions previously stated. While it is acknowledged that the MEPDG will continue to be updated and improved, its complexity will, for the foreseeable future, require well-trained personnel to properly use and interpret. Therefore, at least initially, WSDOT plans to use the MEPDG as an analysis tool and train a limited number of users. WSDOT also acknowledges that many pavement design decisions do not require individual analysis using the MEPDG and could thus be better addressed by using a design catalog approach.

This report describes the preparation of a revised pavement thickness design catalog for WSDOT which was prepared for both flexible and rigid pavements. It covers the selection of the design categories and performance criteria, as well as explains how the 1993 AASHTO Guide, MEPDG and historical records are used together to generate the design catalog. Descriptions of the typical design categories and variables are provided. The resulting design catalog has been officially adopted by WSDOT and is included in an update of its pavement policy (*3*).

# CHAPTER 2 DESIGN PROCEDURES

The 1993 AASHTO Guide and MEPDG were used in combination to help develop a revised WSDOT pavement catalog. The underlying design procedure for the revised design catalog remains the 1993 AASHTO Guide. The MEPDG was used to check the 1993 AASHTO Guide thicknesses at all ESAL levels. Depending on the 1993 AASHTO Guide inputs (such as reliability levels or layer coefficients) thicker pavement sections can result. Due to subsequent changes to MEPDG since earlier WSDOT calibration, this process included a recalibration of the software version 1.0. This calibration effort, which was done in an attempt to match the MEPDG outputs with observed WSDOT field performance, as seen in WSPMS, essentially incorporates WSDOT historical performance into the model. Finally, certain minimum layer thicknesses and maximum lift thicknesses controlled by WSDOT's Standard Specifications for Road, Bridge, and Municipal Construction (*11*) and the WSDOT Construction Manual (*12*) were incorporated.

For both AASHTO and MEPDG methods, the following categories and input values were used:

- **Pavement structure types.** Flexible (HMA) and rigid (JPCP).
- **Design period**. Fifty years for both flexible and rigid pavement since this is consistent with the current design policy. The pavement base and shoulders are expected to perform adequately for the full design period, but the surface layers are expected and allowed to be renewed by routine rehabilitation such as HMA overlay for flexible pavements, and diamond grinding for rigid pavements.
- **Traffic level.** The volume and character of traffic was expressed in ESALs rather than load spectra as used in MEPDG because a single number of ESAL provides pavement designers a more intuitive indicator of loading levels than axle load spectra. The traffic levels are expressed by the total ESALs in the 50-year design life for both flexible and rigid pavement design catalog.
- **Base type.** Granular base (GB) was used for flexible pavements. For rigid pavements with ESALs less than 5 million, portland cement concrete (PCC) slabs are placed directly

on a GB. For higher ESAL levels, slabs are placed on a Hot Mix Asphalt (HMA) base over a GB (3). These base types are based on WSDOT experience and policy.

- **Reliability level.** An 85% reliability level was used for two ESAL levels: (1) less than 5 million, and (2) 5 to 10 million. A reliability level of 95% was used for the higher ESAL levels. This too is largely based on existing WSDOT policy.
- **Base layer thicknesses.** These vary depending upon design ESALs but in all cases were pre-determined based on construction practices and WSDOT experience which have exhibited good performance over time. For example, the 4.2 inches GB for rigid pavements is decided by the maximum compaction thickness allowed for a single layer defined by WSDOT Standard Specification (*11*). Therefore they were effectively eliminated as design variables.
- **Subgrade Resilient Modulus.** 10,000 psi is a reasonable assumption based on prior laboratory and field tests statewide (*13*, *14*). Higher subgrade moduli can be achieved but generally only with granular, low fines materials or some type of subgrade stabilization.

## 2.1 THE 1993 AASHTO GUIDE SPECIFICS

The prior use of the 1993 AASHTO Guide by WSDOT used the following basic inputs (note: some of the inputs for the new WSDOT design table were changed and these are described later in the report).

### 2.1.1 INPUTS—FLEXIBLE PAVEMENTS

- The difference of the serviceability indexes (PSI) from the construction to the end of the pavement design life was set to 1.5. This implies a $p_o = 4.5$ and $p_t = 3.0$.
- The combined standard error of the traffic prediction and performance prediction ($S_o$) was set to 0.5.
- The layer coefficients for the HMA were 0.44 and 0.13 for GB.
- Drainage coefficient (*m*) was set to 1.0 for all layers. WSDOT does not increase layer thicknesses due to expected poor drainage; they instead attempt to address this issue with positive drainage design when necessary.

### 2.1.2 INPUTS—RIGID PAVEMENTS

- ΔPSI is the same as for flexible pavements.
- $S_o$ was set at 0.4.
- The load transfer coefficient J was 3.2 with dowels.
- The elastic modulus of doweled Jointed Plain Concrete Pavement (JPCP) slabs was 4,000,000 psi.
- Modulus of rupture $S_c$' was 650 psi.
- The drainage coefficient $C_d$ was 1.0.
- The modulus of subgrade reaction k was 200 pci for granular base, and 400 pci for HMA base paved over granular base. These values have been in use by WSDOT for at least 20 years and appear to be reasonable.

### 2.1.3 OUTPUTS

Outputs are listed in Table 1. Surface layer thicknesses were calculated for the upper limits of each ESAL level. The base layer thicknesses were predetermined as inputs according to WSDOT construction practices as previously noted. WSDOT is aware, like many highway agencies, that an improved understanding of its pavements and their performance suggests that the 1993 AASHTO Guide is generally conservative, given the inputs noted above.

**Table 1 Pavement Layer Thicknesses by the 1993 AASHTO Guide**

| 50-year ESALs | Reliability Level | Flexible Pavement* | | Rigid Pavement* | | |
|---|---|---|---|---|---|---|
| | | HMA** | Base | PCC | Base | |
| 5,000,000 | 85% | 7.5 | 6 | 9.5 | GB only | 4.2 |
| 10,000,000 | 85% | 8.5 | 6 | 10.0 | HMA over GB | 4.2 + 4.2 |
| 25,000,000 | 95% | 11.2 | 6 | 12.5 | HMA over GB | 4.2 + 4.2 |
| 50,000,000 | 95% | 12.3 | 7 | 14.0 | HMA over GB | 4.2 + 4.2 |
| 100,000,000 | 95% | 13.3 | 8 | 15.5 | HMA over GB | 4.2 + 4.2 |
| 200,000,000 | 95% | 14.5 | 9 | 17.0 | HMA over GB | 4.2 + 4.2 |

*Note: * Thicknesses are in inches and rounded to the 0.1 inches.*
*** The HMA layer coefficient is 0.44.*

## 2.2 THE MECHANISTIC-EMPIRICAL PAVEMENT DESIGN GUIDE (MEPDG)

The mechanistic-empirical approach uses both mathematical models and experimental results to estimate the future pavement distress under a defined pavement condition of climate, traffic patterns, materials and structure. However, the accuracy of the model estimation is dependent on calibration (*4*, *15*).

### 2.2.1 INPUTS

The MEPDG requires three categories of input data: traffic, climate and pavement structure (*4*). The values used in this analysis were generally taken from typical WSDOT designs.

- This analysis used the typical axle load spectra created by prior WSDOT studies of WIM station data (*16*). It includes traffic volume adjustment factors, vehicle class distribution and axle load distribution factors. The Annual Average Daily Truck Traffic (AADTT), traffic growth rates were obtained from the historical traffic data files in the Washington State Traffic Data Office. To match the format of the design catalog and compare

MEPDG design with the AASHTO results, these detailed traffic data were converted to ESALs over the entire design life by using a fixed ESAL number for each truck type.

- The default climate data of weather stations located in Washington State have proved acceptable for use with the MEDPG (*8*, *9*).
- The detailed material properties and structural information were obtained from WSDOT original design files, material tests, the WSDOT Standard Specifications for Road, Bridge and Municipal Construction (*11*), and the WSDOT Construction Manual (*12*).

### 2.2.2 PAVEMENT CONDITION GOALS

Since the MEPDG output is pavement condition over time, end-of-life pavement condition goals must first be defined in order to determine pavement structural thicknesses. These condition goals are:

- **HMA Pavements**
    - *Total structural life*. WSDOT HMA pavements can to serve 50 years with proper rehabilitation—this is based on results from pavement design systems and, importantly, observed pavement performance.
    - *Surface life*. WSPMS data indicate that WSDOT typically overlays HMA pavements on about 12 to 16 year intervals (less often in Western Washington (WW) more often in Eastern Washington (EW)). This means that on average, the surface layer is able to serve about 12 to 16 years before the next overlay. Therefore, to reflect the performance of the surface course only and thus the analysis period for flexible pavement was set to 16 years. Specific MEPDG inputs for terminal conditions were: (1) IRI = 220 inch/mile, (2) longitudinal cracking = 1,000 ft/mile, (3) fatigue cracking = 50%, (4) transverse cracks (thermal fracture) = 1,000 ft/mile, and (5) permanent deformation = 0.5 inches (*17*).
- **Rigid Pavements**
    - *Total structural life*. WSDOT doweled JPCP can serve 50 years with proper rehabilitation—this is based on results from pavement design systems and, importantly, observed pavement performance. Specific MEPDG inputs for terminal

conditions were: (1) IRI = 220 inch/mile, (2) transverse cracks as % of total slabs = 50%, and (3) mean joint faulting = 0.24 inches.

  - *Surface life*. Over the structural life, diamond grinding is normally planned every 25 years to address studded tire wear (*18*).

A reasonable pavement structure thickness design should be able to keep (1) the related surface layer distresses, as noted above, within the range of the pavement condition goals for two analysis periods: 16 years for the surface layer flexible pavements, and 50 years for rigid pavements; and (2) the base layers in an assured good condition throughout the 50-year structure design life.

### 2.2.3 RECALIBRATION

Previously, WSDOT calibrated the rigid portion of the MEDPG Version 0.6 and the flexible portion of Version 1.0. Both bench testing and model analysis have been performed (*8*, *9*). However, additional recalibration was required for this work due to subsequent changes to the MEPDG software. The recalibration has been accomplished by comparing the model outputs with WSDOT historical pavement performance data according to the *Recommended Practice for Local Calibration of the ME Pavement Design Guide* (*15*).

The MEPDG software is designed to evaluate one pavement design at a time; the user provides a set of input values and the damage over time is estimated. Based on the acceptability of these results, the user modifies the calibration factor values until an acceptable damage progression is estimated. Because this process only allows for the evaluation of one pavement section at a time, a full calibration of all WSDOT flexible pavements (which allows for the simultaneous calibration of multiple pavement sections) is not possible. Instead, a single pavement section must be selected, evaluated with the MEPDG software and the resulting damage estimates compared to the actual pavement performance. This method requires that the "calibration sections" be carefully chosen to represent typical design parameters and pavement condition data for a larger group of WSDOT's pavements.

Typical calibration sections were selected from all WSDOT pavements based on:

- **Pavement type.** Flexible (HMA) and rigid (JPCP).

- **Traffic level**. Defined by 50 year design ESALs.
- **Subgrade Resilient Modulus.** 20,000 psi, 10,000 psi and 5,000 psi.
- **Climate.** Eastern Washington (EW) and Western Washington (WW).

A set of calibration factors is chosen and then the design software is run on each of the two selected calibration sections. Based on the results, the calibration factors are adjusted in order of high to low elasticity and the design software is run again. When this process converges on an acceptable set of calibration factors it is essentially repeated on the validation sections. Calibration coefficients were changed between iterations, by comparing observed versus predicted distress.

With the broad range of validation sections, models do not provide precise predictions for each section, but rather approximate field performance. The calibrated models were tested against each individual validation section. The group of calibration factors with the least standard errors between the MEPDG prediction and WSPMS measures on all calibration and validation sections was determined as the final calibration results (Table 2).

**Table 2 Calibration Results of MEPDG**

| Calibration Factor* | | | Elasticity | Default | Recalibration Results |
|---|---|---|---|---|---|
| Flexible Pavement | AC Fatigue | $B_{f1}$ | -3.3 | 1 | 0.96 |
| | | $B_{f2}$ | -40 | 1 | 0.945 |
| | | $B_{f3}$ | 20 | 1 | 1.055 |
| | Longitudinal cracking | $C_1$ | -0.2 | 7 | 6.42 |
| | | $C_2$ | 1 | 3.5 | 3.8 |
| | | $C_3$ | 0 | 0 | 0 |
| | | $C_4$ | ≈0 | 1,000 | 1,000 |
| | Alligator cracking | $C_1$ | 1 | 1 | 1 |
| | | $C_2$ | 0 | 1 | 1 |
| | | $C_3$ | ≈0 | 6,000 | 6,000 |
| | AC Rutting | $B_{r1}$ | 0.6 | 1 | 1.05 |
| | | $B_{r2}$ | 20.6 | 1 | 1 |
| | | $B_{r3}$ | 8.9 | 1 | 1.06 |
| | Subgrade Rutting | $B_{s1}$ | - | 1 | 0 |
| Rigid Pavement | Cracking | $C_1$ | -7.579 | 2 | 1.93 |
| | | $C_2$ | -7.079 | 1.22 | 1.177 |
| | | $C_3$ | 0.658 | 1 | 1 |
| | | $C_4$ | -0.579 | -1.98 | -1.98 |

**Note: Default values are suggested and used for the calibration factors not listed in the table.*

The following observations are made relative to the MEPDG **flexible pavement distress models**:

- **Alligator cracking model**. The MEPDG model assumes bottom up fatigue cracking. For thicker WSDOT HMA pavements, this model does not reflect historical performance; most alligator cracking is top down (i.e., the origin of the cracking is within the wearing course) (*19*). However, it still produces a reasonable rate of cracking albeit the wrong type. This is a major issue that has yet to be fully resolved by WSDOT since it essentially relegates the MEPDG to an empirical model; no different than the existing 1993 AASHTO Guide in its inability to predict the correct type of cracking. Also, the progression of cracking in the MEPDG output is questionable and illustrates markedly different trends for different traffic loadings.
- **Longitudinal cracking model.** The model is recalibrated and able to reasonably estimate WSDOT longitudinal cracking conditions.
- **Transverse cracking model**. The default calibration factors are able to reasonably estimate WSDOT transverse cracking conditions and were used.
- **Rutting model**. The models were calibrated for the surface rutting only since WSDOT does not typically experience rutting in the base and subgrade layers.
- **Roughness model.** The model cannot be calibrated due to MEPDG software bugs within the versions uses. Further, the default roughness model underestimates observed WSDOT roughness, so the model cannot be used for now. However, WSDOT flexible pavement rehabilitation projects are mostly triggered by cracking or rutting, but not roughness. Therefore, the pavement failure is defined by cracking and rutting for this study.

For **rigid pavement distress models**, default calibration factors for the faulting and roughness models were used resulting in estimates that matched well with the WSPMS data in both magnitude and trend. Pavement wear is observed on Washington State rigid pavements due to studded tires, but it is not modeled in MEPDG. Since faulting seldom happens on doweled JPCP, and the distresses of wear and roughness can be addressed by diamond grinding which is normally planned at least once throughout the 50-year design life, slab failure is defined only by the cracking condition.

The MEPDG had model issues and software bugs during the time these computer runs and calibrations were done. However, it was able to estimate reasonable values of the major pavement distresses for WSDOT through the calibration efforts.

### 2.2.4 OUTPUTS

The MEPDG outputs are the surface layer distresses over the analysis periods. With all other inputs fixed, the structural thicknesses vary in each MEPDG run as a design input. Then, the estimated surface layer distresses are compared to the defined pavement condition goals in the analysis periods (16 years for HMA layer and 50 years for PCC slabs). The layer thicknesses which can match all related surface distress to the condition goals throughout the analysis period are chosen as the design results.

The 1993 AASHTO Guide design thicknesses (Table 1) were used as input values for the MEPDG in an effort to check these thicknesses for reasonableness against WSDOT historical performance. As previously stated, the MEPDG recalibration suggests that MEPDG results reflect historical performance as observed in WSPMS. As expected, these checks confirmed that the 1993 AASHTO Guide produced overly thick pavement designs for high ESAL levels (depending on the selected reliability levels and layer coefficients). Table 3 shows the results of MEPDG revised pavement thicknesses.

**Table 3 Pavement Layer Thicknesses by the MEPDG**

| 50-year ESALs | Reliability Level | Flexible Pavement | | Rigid Pavement | | |
|---|---|---|---|---|---|---|
| | | HMA | Base | PCC | Base | |
| 5,000,000 | 85% | 6.0 | 6 | 7.8 | GB only | 4.2 |
| 10,000,000 | 85% | 7.4 | 6 | 9.0 | HMA over GB | 4.2 + 4.2 |
| 25,000,000 | 95% | 9.0 | 6 | 10.0 | HMA over GB | 4.2 + 4.2 |
| 50,000,000 | 95% | 11.2 | 7 | 11.3 | HMA over GB | 4.2 + 4.2 |
| 100,000,000 | 95% | 12.1 | 8 | 12.2 | HMA over GB | 4.2 + 4.2 |
| 200,000,000 | 95% | 13.2 | 9 | 13.3 | HMA over GB | 4.2 + 4.2 |

*Note: Thicknesses are in inches.*

### 2.2.5 DISCUSSION

Several observations were noted while checking the 1993 AASHTO Guide results with the MEPDG:

- The adjustment to the MEPDG revised structure design thicknesses for HMA pavements averages about **1.4 inches less** than obtained by 1993 AASHTO Guide. For JPCP, the reverse occurs in that the **1993 AASHTO Guide produces slab thicknesses, on average, 2.8 inches thicker** than results from the MEPDG would suggest.
- For heavy traffic levels, such as 100 million to 200 million ESALs, the estimated rutting for a HMA surface layer is predicted to be as high as 0.6 inches in 16 years which exceeds WSDOT's rutting limit of 0.5 inches. This indicates the need to choose other binders than WSDOT base performance grades (PG) (for the MEDPG runs PG 58-22 was used for WW and PG 64-28 for EW) or possibly other mix types, such as stone matrix asphalt (SMA), for high ESAL levels. Increasing the PG high temperature binder grade is normally done by the WSDOT pavement designers according to standard WSDOT procedures.

- Longitudinal cracking is observed in WSDOT rigid pavements (especially in Western Washington on Interstate-5) but is not modeled in the MEPDG. However, longitudinal cracking alone rarely triggers rehabilitation because (1) multiple cracks on the same panel are required to do so, and (2) it is rare to see multiple longitudinal cracks on the same panel without at least one transverse crack. Therefore, prediction of transverse cracking is likely more critical for design of PCC slabs.
- WSDOT allows studded tires from November to April which causes surface wear. A WSDOT test showed that PCC pavements with an IRI of 145 to 155 inch/mile in the wheelpath (where studded tire wear would have the greatest effect) only had IRIs of 80~100 inches/mile outside of the wheelpaths (*18*). This type of roughness is not modeled in the MEPDG for rigid pavements, but can be estimated according to WSPMS historical data. Even so, since only slab cracking is used to define failure, this issue is of limited concern for structural design.

# CHAPTER 3 HISTORICAL PAVEMENT PERFORMANCE

While the most direct influence of historical performance comes from its use in calibrating the MEPDG, two other historical performance factors influenced the revised design catalog: selection of a 50-year performance life and base layer selection for rigid pavements.

## 3.1 FIFTY YEAR DESIGN LIFE

Both pavement types can perform adequately for at least 50 years or more with proper surface layer renewal techniques. Figure 1 show that a large percentage of WSDOT's Interstate routes have pavement structures that are at least 30 years old (greater than 94% of the flexible and 63% of the rigid). This performance is due to reasonable design decisions and superior local materials (in the case of the PCC) over the past several decades.

Interstate HMA pavements (based largely on data from I-90) constructed mostly during the 1960s were generally thick—with original constructed HMA thicknesses averaging 14.5 inches in WW to 9.5 inches in EW (the ranges were 13.8 to 18.6 inches in WW and 6.0 to 13.9 inches in EW). These sections have performed well and are no known structural failures of these HMA pavements. Naturally, periodic HMA overlays have been placed to renew the wearing course. Interstate undoweled JPCP pavements constructed in the same timeframe averaged 9.3 inches thick in WW and 9.1 inches in EW (the ranges were 8.0 to 12.0 inches in WW and 8.0 to 11.0 inches in EW). The undoweled JPCP typically survived 25 to 45 years of Interstate level traffic with the shorter lives occurring in EW and longer lives in WW. Early performance of the thick doweled PCC pavement (10 to 13 inch thick slabs) in Washington State suggests that these thicknesses should also perform well over a long time span.

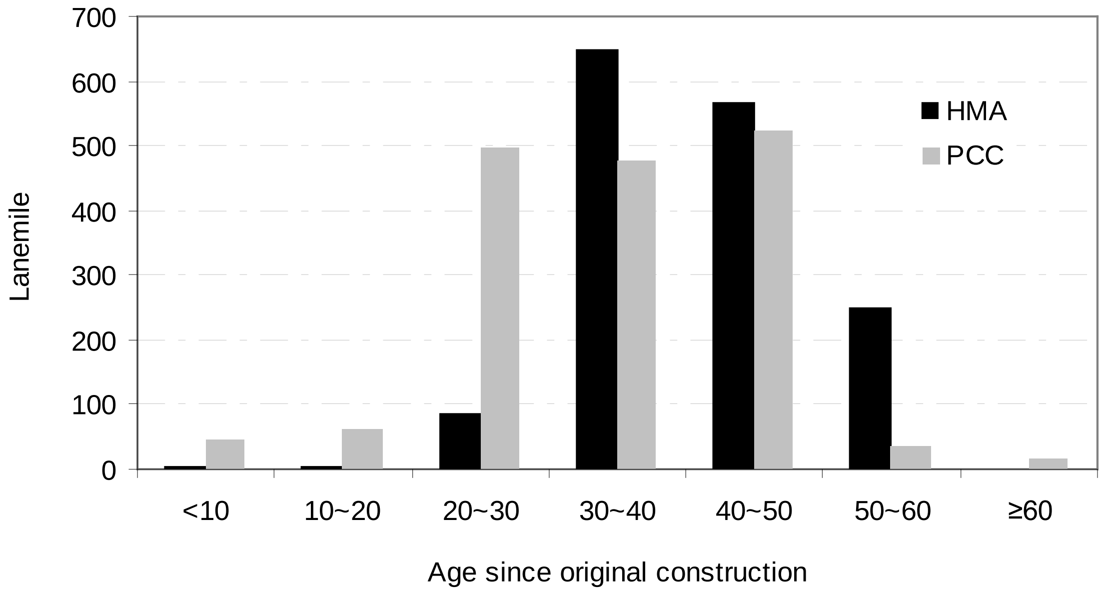


**Figure 1 Age of HMA and PCC pavements for WSDOT Interstate highways in 2008.**

### 3.2 RIGID PAVEMENT BASE MATERIAL

In the past, base depths under rigid pavements were determined primarily by the requirement for support of construction traffic. Currently, it is recognized that the base course directly beneath PCC slabs is a critical element in the performance of PCC pavement because of the large influence of the slab support. HMA is the required base material on high traffic roadways (greater than 5 million ESALs) in order to minimize the risk of failure. GB is used under the HMA layer to provide a stable construction surface for the HMA.

### 3.3 THICKNESS ADJUSTMENTS

The MEPDG was recalibrated and used as a tool to reflect the pavement performance in WSPMS. The base layer thicknesses were predetermined according to WSDOT construction practice and experience. The MEPDG outputs are shown in Table 3, and the design thicknesses are further rounded to the nearest half inch. Table 4 is the revised pavement thickness design catalog. This revised design catalog is applied to WSDOT's typical pavement design cases with similar categories and parameters as described in Chapter 2.

Thickness adjustments can be made to catalog designs if they are based on site-specific data that warrant change. Two cases for table value adjustment stand out. First, reductions in the total HMA thickness can be made by constructing flexible pavements on a stiffer subgrade. In these cases, pavement designers may choose to use an approved design procedure rather than follow the design catalog as published. Second, the primary surface renewal technique for rigid pavements is diamond grinding which reduces the PCC slab thickness (*3*, *19*). To reduce the impacts of the reduced slab thickness by diamond grinding, 0.5 to 1.0 inches of slab thickness can be added to PCC slab thicknesses in anticipation that it would be removed by future diamond grind(s).

**Table 4 Revised Pavement Thickness Design Catalog for WSDOT**

| 50-year ESALs | Reliability Level | Flexible Pavement | | Rigid Pavement | | |
|---|---|---|---|---|---|---|
| | | HMA | Base | PCC | Base | |
| ≤ 5,000,000 | 85% | 6 | 6 | 8 | GB only | 4.2 |
| 5,000,000 to 10,000,000 | 85% | 8 | 6 | 9 | HMA over GB | 4.2 + 4.2 |
| 10,000,000 to 25,000,000 | 95% | 10 | 6 | 10 | HMA over GB | 4.2 + 4.2 |
| 25,000,000 to 50,000,000 | 95% | 11 | 7 | 11 | HMA over GB | 4.2 + 4.2 |
| 50,000,000 to 100,000,000 | 95% | 12 | 8 | 12 | HMA over GB | 4.2 + 4.2 |
| 100,000,000 to 200,000,000 | 95% | 13 | 9 | 13 | HMA over GB | 4.2 + 4.2 |

*Note: Thicknesses are in inches.*

## 3.4 NEW HMA LAYER COEFFICIENTS OF 0.50

Currently, WSDOT is using the 1993 AASHTO Guide which requires a layer coefficient for the HMA layer. In the past, WSDOT has used a HMA layer coefficient of 0.44 which underestimates the load sharing contribution of the HMA layers (Table 1). As Table 5 shows, a HMA layer coefficient of 0.50 generates similar thickness designs as the MEPDG and WSDOT historical pavement performance data especially for the higher ESAL loads. Therefore, a HMA layer coefficient of 0.50 is recommended for WSDOT typical pavement designs using the 1993 AASHTO Guide. This increase is a bit less than the recommended 0.54 resulting from the recent NCAT Test Track study (*20*).

**Table 5 HMA Layer Thickness Design by the 1993 AASHTO Guide**

| 50-year ESALs | Reliability Level | HMA Layer Thickness* | | |
|---|---|---|---|---|
| | | a**=0.44 | a**=0.50 | Final Revised (Table 4) |
| 5,000,000 | 85% | 7.5 | 6.5 | 6 |
| 10,000,000 | 85% | 8.5 | 7.5 | 8 |
| 25,000,000 | 95% | 11.2 | 9.9 | 10 |
| 50,000,000 | 95% | 12.3 | 10.8 | 11 |
| 100,000,000 | 95% | 13.3 | 11.8 | 12 |
| 200,000,000 | 95% | 14.5 | 12.8 | 13 |

*Note:* Thicknesses are in inches.*
***a is the HMA layer coefficient for the 1993 AASHTO Pavement Design Guide.*

# CHAPTER 4 CONCLUSIONS AND RECOMMENDATIONS

WSDOT's revised pavement design catalog reflects a combination of the 1993 AASHTO Guide, the MEPDG, and historical pavement performance. Major observations in developing this catalog are:

1. The 1993 AASHTO Guide provides reasonable structural designs for flexible pavements by use of a more realistic layer coefficient for HMA. Rigid pavements exhibit excessively thick slabs for the given inputs in 1993 AASHTO Guide. WSDOT has been aware of this for some time but has attempted to address the issue during the development of the new pavement policy and associated design catalog.

2. The MEPDG was recalibrated based on WSDOT historical data obtained through WSPMS. While this recalibration seems to create reasonable output values, there are still several distress types that routinely occur in WSDOT pavements that are not adequately modeled. These unmodeled behaviors make it difficult to rely solely on the MEPDG for pavement design. It should be noted that the MEPDG was not used as a design tool in this analysis, but as a tool to reflect WSPMS historical performance data under various traffic, reliability level, climate, and pavement structures. Specific MEPDG issues are:

   - The MEPDG model assumes bottom up fatigue cracking. For thicker WSDOT HMA pavements, this model does not reflect historical performance, since most alligator cracking is top down in WSDOT flexible pavements. Also, the MEPDG estimates of cracking progression are questionable and show markedly different trends for different traffic loadings.

   - The flexible pavement roughness models cannot be calibrated due to software bugs.

   - The MEPDG does not model studded tire wear, thus, wheelpath wear on WSDOT rigid pavements due to studded tire wear cannot be estimated.

   - The MEPDG does not have a longitudinal cracking model for rigid pavements, but longitudinal cracking is observed on WSDOT rigid pavements.

3. A new pavement thickness design catalog was created based on the 1993 ASSHTO Guide as modified by checks with the MEPDG and typical WSDOT design conditions,

local practices and construction guidance. Deviations from this table are possible for any number of conditions but should be analyzed by the 1993 AASHTO Guide and MEPDG along with the historical pavement performance data.

It is difficult to base statewide pavement design on models alone. While the 1993 AASHTO Guide has been in existence for quite some time, it is now being used to design pavements for ESAL levels far in excess of the empirical evidence upon which it was based. It is a testament to this model's robustness and the ingenuity of those who have worked on it that it can still be used to design reasonable pavement structures. Nevertheless, most users of the 1993 AASHTO Guide recognize its shortcomings and have developed work-arounds to get results that match field performance. While the MEPDG is, in many respects, a much more sophisticated and detailed design approach, it too has limitations. For WSDOT, the MEPDG does not model all the desired distress types and is unable (due to bugs or otherwise) to predict others adequately. Ultimately it is historical performance that provides the missing information and the assurance that model outputs are likely to be achieved during actual performance. Thus, the new WSDOT design catalog is developed based on a combination of old models, new models and historical performance.

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