Development and Implementation of a Mechanistic, Empirically Based Overlay Design Procedure for Flexible Pavements

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In the early 1980s the Washington State Department of Transportation (WSDOT) implemented a new pavement management system (WSPMS) that is project specific and contains detailed construction history and performance data for all projects throughout the state. With the new WSPMS, all resurfacing designs were made with more detailed knowledge of the past performance of each project, and conflicts between design expectations and actual past performance of each project were more obvious. Using the information contained in the WSPMS and deflection data obtained from the falling weight deflectometer, WSDOT decided to develop a mechanistic, empirically based flexible pavement overlay design method and contracted with the University of Washington and the Washington State Transportation Center to cooperatively develop such a method. A general overview of this mechanistic, empirically based overlay design procedure describes subsequent implementation activities and demonstrates the application of the design procedure using a case study in which the WSDOT mechanistic, empirical design procedure is compared with other standard overlay design procedures.

The Washington State Department of Transportation (WSDOT), like most western state DOTs, has been designing all new flexible pavements by a rational, empirically based pavement design method for over 40 years. By the late 1940s, Washington State had adopted a flexible pavement design method that was based on the California bearing ratio (CBR) test. In 1951 the CBR test procedure was abandoned in favor of a method that utilizes the Hveem stabilometer. The design procedure was essentially that originated by Hveem and Carmany (*1*). The principal differences between Washington's design procedure and the original California procedure were modifications in test procedures and in factors used for the base and pavement courses. The modifications were incorporated in the design to reflect field conditions and pavement performance experienced in Washington State. These design procedures and processes were first described by LeClerc in a presentation at the annual Highway Research Board meeting in 1956 (*2*). The basic design procedures were modified in 1957 and in 1966 to provide more detail in the design charts, which resulted in thicker asphalt pavement sections.

In 1974 Washington State established a formal 4-R pavement design process in response to the FHWA requirement for a determination of structural adequacy to qualify for federal aid for resurfacing projects. A component design procedure was adopted similar to that contained in the AASHTO Interim Pavement Design Guide and the Asphalt Institute's design procedures, except that Washington's was based on their Hveem design method. Benkleman beam deflections were collected in some districts and a comparison was made using the Asphalt Institute's rebound deflection-based design.

In the early 1980s WSDOT implemented its new pavement management system (WSPMS), which was project specific and had detailed construction history and performance data for all pavement sections throughout the state. With the WSPMS, all resurfacing designs were made with more detailed knowledge of the past performance of each project; thus, conflicts between design expectations and the actual past performance of each project became more obvious. Also at about this same time WSDOT obtained a falling weight deflectometer (FWD) as part of a federal research study on long-term pavement monitoring. Being somewhat dissatisfied with the adequacy of the current component and deflection-based procedures, plus having a current data base with detailed pavement performance data and a modern field pavement testing device, WSDOT decided to develop a mechanistic, empirically based flexible pavement overlay design method. Further, the general rehabilitation philosophy of WSDOT was to "fix it early, fix it thin." In this regard it was believed that the mechanistic, empirical approach would best provide overlay thicknesses that reflect the "early" treatment of small amounts of fatigue cracking (which is the dominant rehabilitation "trigger" distress for WSDOT). This work began in the mid-1980s when WSDOT contracted with the University of Washington and the Washington State Transportation Center to cooperatively develop a flexible pavement overlay design procedure. The resulting mechanistic, empirically based overlay procedure as developed was described in the final report dated January 1989 (*3*). When the development process began in 1985, the mechanistic, empirical design approach had been developed by other researchers; however, the issues of seasonal moduli changes, appropriate field failure criteria, and backcalculation of layer moduli were still at an early stage. The research team spent considerable effort on these basic issues and placed the entire system into personal computer compatible software.

This paper will provide an overview of the mechanistic, empirically based overlay design procedure, describe subsequent implementation activities, and demonstrate the application of the design procedure using a case study in which the WSDOT mechanistic, empirical design procedure is compared with other standard overlay design procedures.

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DEVELOPMENT OF THE PROCEDURE

The development of a mechanistic, empirical overlay design procedure requires a pavement response model, material characterization, and failure criteria. The associated tasks for the development of the overlay procedure are as follows.

Test Site Selection

To examine specific pavement performance in Washington State, 16 test sites were selected. These test sites were typical flexible pavement sections and were selected both for their uniformity (construction, distress, and subgrade soil) within each test site and for their variety (age, climate, traffic, structural section, and distress).

Laboratory Testing

Field asphalt concrete cores and unbound disturbed material samples were collected from each test site with the WSDOT Materials Laboratory performing the associated laboratory tests. To determine the modulus of elasticity of the asphalt concrete, the diametral resilient modulus test (ASTM D4123) was conducted at 5°C, 25°C, and 40°C (41°F, 77°F, and 104°F) with a load duration of 100 msec. To determine the resilient modulus of the unbound materials, the samples were remolded and recompacted at a moisture content and density similar to those observed in the field at the time of sampling. A triaxial test was performed on each sample with confining pressures of 7, 14, and 28 kPA (1, 2, and 4 psi) and deviator stresses of 7, 14, 28, 41, and 55 kPa (1, 2, 4, 6, and 8 psi), in accordance with AASHTO T274. Asphalt concrete layer thicknesses were also determined from core samples.

Nondestructive Testing

WSDOT collected pavement surface deflection measurements with the FWD. These measurements were collected in the outer wheelpath nearly every season from 1985 to 1988.

Characterization of Pavement System

Pavement Model

Of several pavement models, the multilayered elastic system has been shown to provide reasonable pavement response solutions, in terms of deflection, stress, or strain caused by an applied load. The multilayered elastic model requires the following assumptions:

1. The material properties of each layer are homogeneous and isotropic;
2. Each layer has a finite thickness, except for the lower layer, and all layers are infinite in the lateral direction, and
3. The materials are characterized by the modulus of elasticity (resilient modulus) and Poisson's ratio (*4*).

Some contradictions to these assumptions include the variability in traffic load intensity; the elliptical or rectangular, rather than the assumed circular, shape of the tire footprint; dynamic rather than static loading; pavement material behavior that is not fully elastic; material properties of a single layer that are somewhat inhomogeneous and anisotropic; and the modulus of a single layer that is an equivalent modulus even though the layer is composed of many different materials. However, a fully monitored pavement experiment showed that the multilayered, linear elastic theory was acceptable (*5*). Several computerized solutions for the analysis of multilayered systems have been developed. This study used CHEVRON *N*-LAYER, which was developed by the Chevron Research Company (*6*). A principal reason for its use was that the software was in the public domain.

Asphalt-Bound Material

The modulus of asphalt concrete depends on its material characteristics and testing conditions (loading time and temperature). The relationship between the resilient modulus and temperature for WSDOT Class B asphalt concrete was found as follows (*7*):

$$\log E_{AC} = 6.4721 - 0.000147362(T_p)^2 \quad (1)$$

where E_{AC} is the resilient modulus of asphalt concrete (psi) and T_p is the pavement temperature (°F).

Unstabilized Materials

The modulus of unstabilized materials depends to a great extent on stress level, dry density, moisture content, degree of saturation, gradation, load duration, and frequency, among which stress level and moisture condition have proven to be the most significant factors. Chou (*8*) has shown a direct relationship between the modulus and the stress state for unstabilized base materials and subgrade soils.

Several resilient modulus tests were conducted on the granular base material (the unbound bases were all produced to the same specifications; hence, some statewide uniformity). Most of the granular base materials were best represented by the following equation (personal communication between J. P. Mahoney and N. C. Jackson, Feb. 5, 1990):

$$E_{BS} = 8{,}500\theta^{0.375} \quad (2)$$

where E_{BS} is the resilient modulus of the coarse-grained materials and soils (pounds per square inch) and, θ is bulk stress (pounds per square inch).

STRUCTURAL EVALUATION OF PAVEMENT

The need for information about the in situ pavement layer properties is readily apparent for pavement overlay design and hence the development of optimal pavement rehabilitation strategies. Material properties can be acquired either by laboratory test of samples or by a nondestructive testing (NDT) evaluation method. Because of the cost and time constraints of the laboratory test, plus the ability to characterize materials as they exist in place, the NDT method is being used more frequently (*9*). At the time of initial development of the design process, NDT evaluations, which generally use the pavement surface deflection basin, were accomplished by either graphic solution or backcalculation. The latter was felt to be more accurate and robust. Thus, the backcalculation procedure for NDT deflection measurements becomes crucial for pavement rehabilitation, and a significant amount of the reported study resources were devoted to backcalculation development. It is ironic that several other backcalculation programs were under way in other states at about the same time.

Pavement Deflection Analysis Program (EVERCALC)

EVERCALC (*10*) is a pavement analysis computer program that is based on the multilayered elastic pavement analysis program CHEVRON *N*-LAYER. The program is primarily for the analysis of flexible pavement using FWD deflection measurements. A reverse-solution technique is used to determine elastic modulus from the deflection measurements. (Actually, the pavement surface deflections at a known load and assumed Poisson's ratio and known thickness of each layer are required.) The theoretical deflections are compared with the measured deflections in each iteration. When the discrepancies in the calculated and measured deflections, as characterized by the root mean square error, or the changes in moduli fall within the allowable tolerance, or the number of iterations has reached the specified limit, the program terminates. The current version of the program (EVERCALC 3.3) is capable of evaluating a flexible pavement structure containing up to five layers and can run with or without a "stiff layer." The program makes an initial, rough estimate of modulus ("seed modulus") for each layer using internal regression equations and then backcalculates to determine a "final" modulus for each pavement layer. The program also determines the coefficients of stress sensitivity for unstabilized materials when the deflection data for two or more load levels are available at a given point and then normalizes the asphalt concrete modulus to the WSDOT standard laboratory condition [which is 25°C (77°F)]. The seed moduli are estimated with internal regression equations, which were developed from the relationship among the layer moduli, surface deflections, applied load, and pavement thicknesses (*11*).

OVERLAY DESIGN PROCEDURE

Traffic load repetitions and environment are two primary factors that induce pavement distress (other factors include construction variation and age). Of the various kinds of distress, fatigue cracking and rutting are the two primary distresses found in flexible pavements in Washington State (mostly fatigue cracking). Numerous studies of pavement distress have shown that pavement performance is related to pavement response parameters (such as strain), which are determined through mechanistic pavement analysis.

This section reviews the design criteria (failure criteria) and a mechanistic, empirically based overlay design procedure computer program, EVERPAVE.

Design Criteria

Investigations have shown that fatigue failure is best related to the horizontal tensile strain at the bottom of the asphalt-bound layer and that rutting can be best related to the vertical compressive strain at the top of the subgrade (*12*).

The models for fatigue failure criteria generally are a function of the tensile strain and the modulus of the asphalt-bound material. Monismith's laboratory model, one of the most widely used, is as follows (*13*):

$$\log N_f = 14.82 - 3.291\log(\varepsilon_t) - 0.854\log(E_{AC}/1{,}000) \qquad (3)$$

where

N_f = loads to failure,
ε_t = initial tensile strain (10^{-6} in./in.), and
E_{AC} = modulus of asphalt bound material (psi).

This model was developed for asphalt concrete mixes quite similar to those used by WSDOT. However, the model raises two concerns for overlay design. One is the adjustment of this laboratory relationship to field conditions, and the other is the consideration of the existing asphalt concrete layer.

Because differences exist between the laboratory and actual pavement in the definition of failure and loading mode, laboratory fatigue models need to be adjusted to field conditions. To do this, the laboratory model is multiplied by a shift factor (*SF*). The resulting predictive equation becomes $N_{\text{field}} = (N_{\text{lab}})(SF)$. The shift factor depends on asphalt concrete properties such as void ratio, asphalt cement content, and viscosity and other factors such as layer thickness and pavement loading conditions. An investigation of the shift factor, using Monismith's laboratory model for initial pavement performance (i.e., N_{lab}), was attempted at six test sites in Washington State that showed fatigue distress, as shown in Table 1. The service lives of the pavements were 10 to 13 years, and the thicknesses of the asphalt concrete ranged from about 100 to 250 mm (4 to 10 in.). The moduli of the original asphalt concrete were estimated on the basis of engineering judgment and the results of laboratory tests on pavement cores. The moduli of the unbound materials were obtained through backcalculation and seasonal material modulus variations, which will be discussed later. The shift factor ranged from 0.1 to about 6.0, depending on the asphalt-bound layer thickness. However, the lower shift factors were for thick asphalt concrete [about 203 to 229 mm (8 to 9 in.)]. Thus, shift factors of about 6.0 were found for sections with asphalt concrete thickness of 90 to 130 mm (4 to 5 in.). We generally observe that for asphalt concrete pavement thicknesses of about 200 mm (8 in.) or greater, fatigue cracking primarily occurs in the upper wearing course (top-down cracking). For thicker asphalt concrete sections [200 mm (8 in.) or more], structural overlay designs are rarely needed (EVERPAVE is not used for overlay design on thicker asphalt concrete sections). Treatments such as milling and replacement of the distressed surface course are generally recommended.

The second concern is how the performance of the existing asphalt concrete layer is incorporated into the overlay design procedure. This situation poses difficulties because the fatigue failure criteria were developed for new asphalt concrete. Some design procedures consider the strain at the bottom of the new asphalt concrete layer; others consider the strain only at

TABLE 1 Washington State Shift Factors

Test Site	AC Thickness (mm)	ESAL (10^3)	Age (yrs)	Shift Factor
1	132	640	11	5.8
3	213	466	13	0.4
8	185	700	12	2.5
10	229	389	12	0.1
13	244	2,135	12	0.5
15	91	332	10	5.6

Note: The modulus of the original asphalt concrete was assumed to be 2,756 MPa. ESALs accumulated from the original construction date to the time of fatigue cracking.

the bottom of the existing asphalt concrete layer, and others consider both of these strains (*14–18*). Both strains (bottom of the new overlay layer and the bottom of the existing (pre-overlay) asphalt concrete) were considered in this study and are used in EVERPAVE. The rationale for this is straightforward in that strain (or other movement) in the existing asphalt concrete layer must be controlled to reduce the potential for reflection cracking into the new overlay layer.

Rutting occurs because of permanent deformation of the asphalt concrete layer and unbound layers. However, since the permanent deformation of asphalt concrete is more a construction, materials issue instead of a structural thickness issue, the failure criterion is expressed as a function of vertical compressive strain at the top of the subgrade. The Chevron equation was used to estimate rutting in the subgrade, as follows:

$$N_r = 1.077 \times 10^{18}(\varepsilon_{vs})^{-4.4843} \quad (4)$$

where N_r is the number of loads needed to cause a rut depth of approximately 19 mm (0.75 in.) and ε_{vs} is the vertical compressive strain at the top of the subgrade (10^{-6} in./in.)

Because there have been no visible signs of rutting in the 16 test site locations, validation of the rutting model has not been possible. In addition, for Washington State, fatigue instead of rutting typically is the controlling pavement failure criteria.

Pavement Environmental Effects in Washington State

The consideration of environmental conditions (as a function of resilient modulus) is essential in mechanistic, empirical pavement design. Seasonal adjustment for asphalt-bound materials is obtained from the relationship between the modulus and temperature. However, for unbound (unstabilized) materials that process is not so straightforward.

Two types of climate prevail in the state of Washington: a marine type in the west and a continental climate in the east. In western Washington, there are two distinct seasons, a warm and dry summer and a wet and mild winter. Eastern Washington experiences a hot and dry summer and a cold winter; thus spring thaw problems can exist. The predominant roadbed soils are mostly silts and various types of glacial till and clay.

The seasonal variations of soil moduli are primarily induced by variations in soil moisture content, which depend on precipitation, temperature, soil gradation and permeability, surface distress level, and drainage conditions (*19*). Seasonal variations for each of the two regions were investigated over two distinct seasons (wet or dry) that were based on the backcalculated moduli from 4 years of FWD data and climate data obtained from published climatological information. The ratios of the moduli of various seasons were determined and are indicated in Table 2. The seasonal variations were reevaluated in June 1992 and were found to be unchanged from the values determined in the original study.

Several studies in Washington State have examined the various aspects of seasonal pavement material changes. The observation that the base course changes the most came as no surprise. This change is probably the result of the larger layer moisture changes in the base course layer, compared with those of subgrade layer.

Traffic in Washington State

The equivalent single-axle load (ESAL) concept was adopted for the traffic input [80 kN (18,000 lb)]. The primary concern is how to quantify the mixed traffic for a design period to use in pavement design. FHWA's W-4 loadometer tables were used to determine ESALs for various truck types from 1950 to 1983. Structural numbers of 3.0 and 5.0 were assumed for the pavements built before and after 1963, respectively. A summary of the determined ESAL factors are listed in Table 3.

Overlay Design Program (EVERPAVE)

EVERPAVE (*20*) is a mechanistic, empirically based overlay design program. The pavement analysis is accomplished through use of EVERSTRS (used as a subroutine), which can account for the stress-sensitive characteristics of the unbound mate-

TABLE 2 Seasonal Variations of Unbound Material Moduli for Washington State

	Base		Subgrade	
Region	Wet/Thaw	Dry/Other	Wet/Thaw	Dry/Other
Eastern Washington	0.65	1.00	0.95	1.00
Western Washington	0.80	1.00	0.90	1.00

TABLE 3 Summary of ESAL Factors

Highway System	Single Units	Combination Units	Buses	Individual Axle	Overall Truck*
Interstate	0.30	1.25	1.30	0.25	1.20
Non-Interstate Rural	0.50	1.50	1.30	0.25	1.40
Non-Interstate Urban	0.25	1.20	1.30	0.25	1.00

* Excludes Buses

rials (*8*). A flow chart of the EVERPAVE program is presented in Figure 1.

The program can analyze a pavement system up to five layers, including the new overlay. The pavement responses under single or dual wheel loads are determined from the analysis of a pavement system, as shown in Figure 2. The responses include the failure criteria for fatigue and rutting, which are a function of the tensile strains at the bottom of the overlay asphalt concrete and that of the existing asphalt concrete layer, and the compressive strain at the top of the subgrade. The program calculates overlay thickness by comparing the pavement performance lives for fatigue and rutting with the projected design traffic volume (ESALs). When the minimum repetitions of the two failure criteria are greater than the traffic volume, the final overlay thickness is produced. Otherwise, the overlay thickness is increased by increments (an input data requirement) and the analysis is repeated. This process continues until the maximum distress performance period exceeds the design traffic volume.

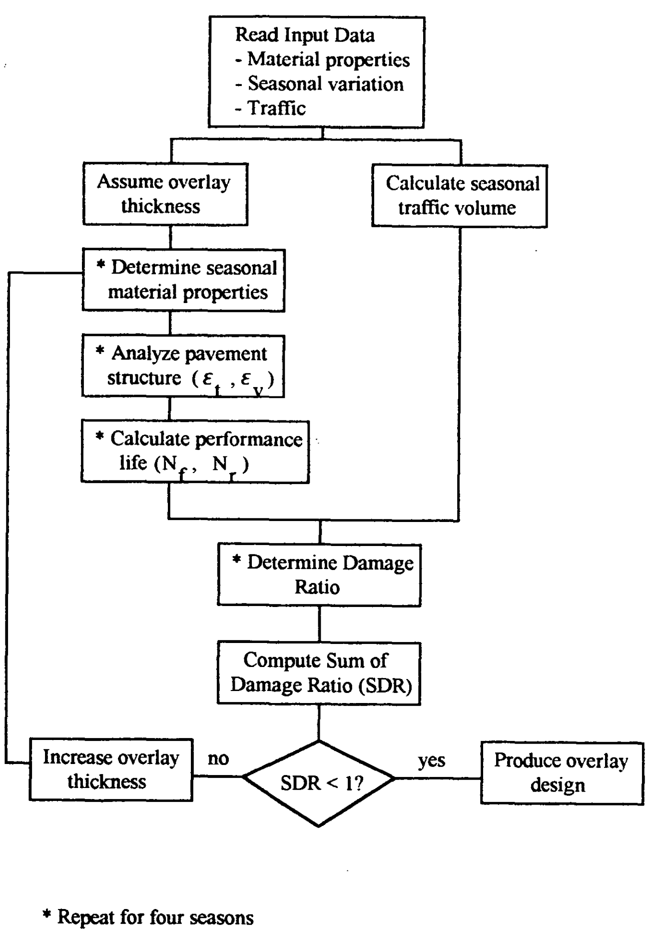

FIGURE 1 Overlay design procedure by EVERPAVE.

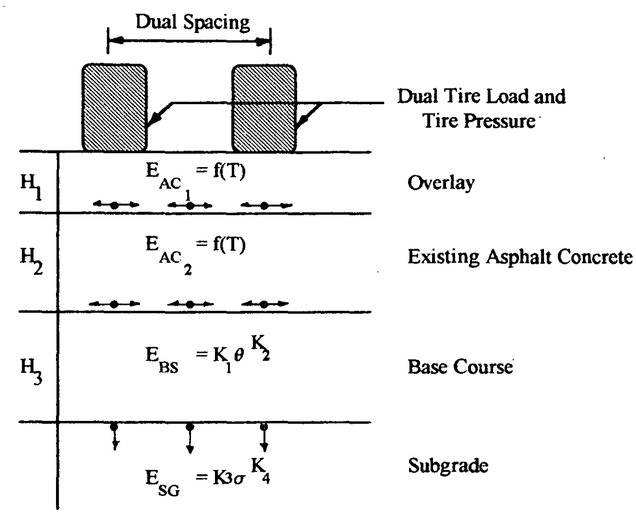

FIGURE 2 Pavement system for overlay design for four layers.

IMPLEMENTATION OF THE PROCEDURE

A resurfacing report is prepared by the district materials engineer for all asphalt concrete (AC) overlays and is even recommended for bituminous surface treatments in which structural problems are evident (*21*). The resurfacing report includes pavement deflection data (which are collected by the headquarters materials branch), descriptions and photographs of typical pavement conditions, road life history, pavement cores, surfacing and subgrade samples, and a review of drainage features. The resurfacing report is oriented toward analyzing the existing roadway conditions so that a reasonable definition of the special problems and structural needs of the roadway may be defined.

Upon submittal of the resurfacing report to the headquarters materials branch, the materials branch reviews the contents of the report, performs a mechanistic, empirical design analysis (when FWD data are available), and submits a final headquarters overlay design.

The mechanistic, empirical design analysis is composed of two distinct parts: materials analysis and overlay design. Materials analysis, which may include pavement layer backcalculation, has been purposely separated from the overlay design process so that the backcalculated material properties can be characterized before input for overlay design.

A comparison of the overlay thicknesses produced by various methods was used to examine the new overlay design procedure (EVERPAVE). Because every overlay design method has its own peculiar design parameters, the comparisons were limited to the revised AASHTO (*22*) and the Asphalt Institute methods (*23*). To illustrate how the process works, a typical project is presented as a case study.

CASE STUDY

State Route 500 (Milepost 2.00 to Milepost 3.05) is a section of urban undivided multilane highway located in southwestern Washington (near Vancouver, Washington). The soil consists largely of fluvial deposited (deposited under water) sands and silts. The cuts through this material intercepted considerable perched water flowing in the sandy soils between the silt lenses at varying depths through the cut. The environment is typical of the Pacific Northwest coast, relatively mild temperatures, both in summer and winter, with an annual rainfall of about 1,000 mm (40 in.) per year, mostly occurring from midfall through late spring.

The roadway was constructed in 1983 with 3.7-m (12-ft) lanes and 1.2-m (4-ft) inside shoulders and 3-m (10-ft) outside shoulders. The pavement section consists of 108 mm (4.25 in.) of asphalt concrete, over 70 mm (2.75 in.) of crushed rock base, over 127 mm (5.0 in.) of gravel base. This roadway was a staged construction, which called for placing a second-stage AC overlay 5 to 6 years after the initial construction. The original construction included an extensive system of drains to mitigate erosion and to intercept the perched water before it reached the subgrade. It is clear that the drainage design did not control the subsurface water as well as expected, as was evident when the pavement began to fatigue crack within 4 to 5 years after construction. Subsequent investigations indicated that the sandy subgrade soils have stabilized at a moisture content of about 3 to 5 percent over optimum, resulting in a structurally underdesigned pavement section. As a result of the staged pavement design and the wet subgrade, the pavement began to experience both longitudinal and fatigue cracking in approximately 10 percent of the total area of the wheelpaths by the spring of 1988. The estimated ESALs for the next 10 years is 2 million.

FWD data were collected for this project in April 1988. The average center deflection (D_0) is 508 μm (20.0 mils), with values ranging from 371 to 678 μm (14.6 to 26.7 mils). The subgrade modulus ranges from 79 to 166 MPa (11,537 to 24,100 psi), and averaging 120 MPa (17,351 psi) with a standard deviation of 30 MPa (4,420 pis) (*24*).

Asphalt Institute's Deflection Procedure

Representative rebound deflection (RRD)

= 671 μm (26.4 mils)

Overlay thickness = 0.0 mm

Asphalt Institute's Effective Thickness Procedure

Effective thickness of existing pavement (108 mm AC)(0.75)

= 81 mm (3.2 in.)

Effective thickness of base (197 mm base)(0.20)

= 39 mm (1.6 in.)

Total effective thickness = 120 mm (4.7 in.)

1. Milepost (MP) 2.05

M_R = 83 MPa (12,000 psi)

Design thickness = 235 mm (9.3 in.)

Effective thickness = 120 mm (4.7 in.)

Overlay thickness = 115 mm (4.5 in.)

2. MP 2.55

M_R = 76 MPa (11,000 psi)

Design thickness = 241 mm (9.5 in.)

Effective thickness = 120 mm (4.7 in.)

Overlay thickness = 121 mm (4.8 in.)

Revised AASHTO

The overlay was determined using the FWD deflection basins at MP 2.05 and MP 2.55 and using only the NDT method.

1. MP 2.05

D_0 = 678 μm (26.7 mils)
D_{36} = 140 μm (5.5 mils)
C = 1.00, 0.50, and 0.33
D = 305 mm (12 in.)
p_i = 4.5
p_t = 2.5
S_0 = 0.49
R = 50 percent
M_R = (C = 1.00) 75 MPa (10,909 psi)
(C = 0.50) 38 MPa (5,455 psi)
(C = 0.33) 25 MPa (3,636 psi)

Overlay thickness (C = 1.00) = 40 mm (1.6 in.)
(C = 0.50) = 89 mm (3.5 in.)
(C = 0.33) = 122 mm (4.8 in.)

2. MP 2.55

D_0 = 572 μm (22.5 mils)
D_{36} = 158 μm (6.2 mils)
C = 1.00, 0.50, and 0.33
D = 305 mm (12 in.)
p_i = 4.5
p_t = 2.5
S_0 = 0.49
R = 50 percent
M_R = (C = 1.00) 67 MPa (9,677 psi)
(C = 0.50) 33 MPa (4,893 psi)
(C = 0.33) 22 MPa (3,226 psi)

Overlay thickness (C = 1.00) = 28 mm (1.1 in.)
(C = 0.50) = 77 mm (3.0 in.)
(C = 0.33) = 112 mm (4.4 in.)

where

D_0 = FWD center deflection,
D_{36} = FWD deflection at 36 in. from the load plate,
C = adjustment factor: "The recommended method for determination of the design M_R from NDT back-calculation requires an adjustment factor (C) to make the value calculated consistent with the value used to represent the AASHTO subgrade. A value for C of no more than 0.33 is recommended for adjustment of backcalculated values to design M_R values." (*23*),
D = actual pavement structure thickness,
p_i = initial serviceability index,
p_t = terminal serviceability index,
S_0 = overall standard deviation of normal distribution of errors associated with traffic prediction and pavement performance,
R = reliability level, and
M_R = effective roadbed resilient modulus.

WSDOT Mechanistic, Empirically Based Overlay Design

The EVERCALC backcalculation results (*24*) were examined and the following conditions were determined:

1. Existing AC modulus: 1654 MPa (240,000 psi), which represents a partially cracked AC pavement condition, 3858 MPa (560,000 psi), which represents an unfatigued AC pavement condition.
2. Base course modulus conforms to $E_{BS} = 8{,}500\theta^{0.375}$
3. Subgrade modulus (using stiff layer): 76 and 83 MPa (11,000 and 12,000 psi)
4. AC overlay modulus: 2756 MPa (400,000 psi)
5. Fatigue criterion relationship: $SF = 5$

The results are as follows:

1. MP 2.05
E_{AC} = 1654 MPa (240,000 psi)
M_R = 83 MPa (12,000 psi)
Overlay thickness = 91 mm (3.6 in.)
2. MP 2.55
E_{AC} = 3858 MPa (560,000 psi)
M_R = 76 MPa (11,000 psi)
Overlay thickness = 611 mm (2.4 in.)

The overlay design that was selected for this project is 76 mm (3.0 in.) AC pavement after repairing the worst of the fatigue cracked pavement. Table 4 summarizes the overlay design for this case study.

ADDITIONAL STUDIES

With the development of the mechanistic, empirically based overlay design procedure, the need has evolved for additional studies to validate and modify the various aspects in both the EVERCALC and the EVERPAVE computer programs. Currently, WSDOT, in conjunction with the University of Washington, has conducted several studies to validate program conditions.

TABLE 4 Summary of Overlay Design Thickness

	MP 2.05	MP 2.55
Asphalt Institute		
Deflection Procedure	0	0
Effective Thickness Procedure	115 mm	121 mm
AASHTO		
C = 1.00	40 mm	28 mm
C = 0.50	89 mm	77 mm
C = 0.33	122 mm	112 mm
WSDOT EVERPAVE	91 mm	61 mm
Constructed Overlay	76 mm	76 mm

DEPTH TO STIFF LAYER

The depth to stiff layer is estimated using the scheme reported by Rohde and Scullion (*25*) and has been incorporated into the current version of EVERCALC. The original development of this scheme requires the use of one of four separate regression equations that are dependent on ranges of the AC layer thickness [less than 50 mm (2.0 in.)], between 50 and 100 mm (2.0 to 4.0 in.), between 100 and 150 mm (4.0 to 6.0 in.) and greater than 150 mm (6.0). It has been found that the regression equations are not continuous across the AC layer ranges. Therefore, as time allows, this issue (and equations), will be revisited.

The effect that a water table may have on the calculated layer moduli is currently being examined. If the depth-to-stiff-layer option is used, a moduli and Poisson's ratio for the stiff layer must be selected. This layer has a significant effect on the calculated layer moduli and root-mean-square error. Several locations are being evaluated where FWD and boring logs are available. In some cases it has been found that the depth-to-stiff layer is actually the depth to the water table or an area of saturated soils. Generally, it appears that lower moduli are appropriate for the "stiff layer" where this condition exists.

SHIFT FACTOR

The EVERCALC/EVERPAVE process has been in use in Washington State since 1988. WSDOT evaluates about 100 projects a year as part of the basic overlay design process. With some additional work the actual fatigue performance (shift factor) can be estimated. It is the intent of WSDOT to further evaluate these past overlay designs, reevaluate the originally determined shift factors, and build a fairly large and comprehensive data base documenting the range of fatigue performance experienced in the state of Washington.

RELIABILITY

Currently the EVERPAVE program uses a 50 percent reliability. As the modifications to the AASHTO Design Guide become available and are further evaluated, a range of reliability values will also be incorporated into the EVERPAVE program.

SUMMARY

The mechanistic, empirical flexible pavement overlay design procedure developed for WSDOT has been overviewed. It uses somewhat traditional failure criteria for estimating fatigue cracking and rutting. These criteria were simply adaptations of prior work that were "shifted" to accommodate conditions found in Washington State. Further, the seasonal effects of changing moduli were investigated and incorporated as seasonal moduli ratios.

The required layer moduli for design are assumed (as for the new AC overlay material) or estimated via laboratory tests or NDT analysis. The primary use of the NDT data (from the FWD) is for backcalculation. After several years of experience obtaining and using backcalculated pavement moduli, we believe that this fundamental approach is valid. However, we continue to gain experience mostly through empirical evidence on how to improve our backcalculation process.

The mechanistic, empirical design process has many clear advantages over conventional (empirically based) design processes. Our experience has shown that it is not only a more rational design process; it also more adequately addresses unique conditions, such as the following:

1. Overlays of granular surfacing with thin surface seals, which are very strain critical.
2. Overlays of thick AC in which the existing AC is moisture sensitive and has lower stiffness and tensile strength than normal AC.
3. Overlays with materials that have different fatigue properties from normal AC, such as open-graded emulsion mixes, or some of the new AC mixes with modified AC binders.

The design process is sensitive to the material properties selected for input. Effective use of the design process requires a rigorous and accurate analysis of the in situ material properties. Most of the effort required to use the design process is in analyzing and characterizing the in situ material properties.

The overall goal of designing AC overlay thickness that are appropriate for "early" distress to "extreme" distress was accomplished by the design process described in this paper. The design process is not unlike that developed by numerous other researchers; however, what may be somewhat unique is the development process, whereby WSDOT and the Washington State Transportation Center (University of Washington) worked together, as a team, throughout various research efforts. Early results were trial implemented and subsequently refined. The end results are a fully implemented design system.

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