Mechanistic-Based Overlay Design Procedure for Washington State Flexible Pavements

WA-RD 170.1

Final Report
January 1989

Washington State Department of Transportation
Planning, Research and Public Transportation Division

in cooperation with the
United States Department of Transportation
Federal Highway Administration

Final Report

Research Project GC8286, Task 29
ACP Overlay Design Implementation

MECHANISTIC-BASED OVERLAY DESIGN PROCEDURE FOR WASHINGTON STATE FLEXIBLE PAVEMENTS

by

Joe P. Mahoney
Sang Won Lee
Newton C. Jackson
David E. Newcomb

Washington State Transportation Center (TRAC)
University of Washington
The Corbet Building, Suite 204
4507 University Way N.E.
Seattle, Washington 98105

Washington State Department of Transportation
Technical Monitor
Newton C. Jackson
WSDOT Pavement and Soils Engineer

Prepared for

Washington State Transportation Commission
Department of Transportation
and in cooperation with
U.S. Department of Transportation
Federal Highway Administration

January 24, 1989

WASHINGTON STATE DEPARTMENT OF TRANSPORTATION

TECHNICAL REPORT STANDARD TITLE PAGE

1. REPORT NO.	2. GOVERNMENT ACCESSION NO.	3. RECIPIENT'S CATALOG NO.
WA-RD 170.1		

4. TITLE AND SUBTITLE	5. REPORT DATE
MECHANISTIC-BASED OVERLAY DESIGN PROCEDURE FOR WASHINGTON STATE FLEXIBLE PAVEMENTS	January 1989
	6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S)	8. PERFORMING ORGANIZATION REPORT NO.
Joe P. Mahoney, Sang W. Lee, Newton C. Jackson, and David E. Newcomb	
9. PERFORMING ORGANIZATION NAME AND ADDRESS	10. WORK UNIT NO.
Washington State Transportation Center (TRAC) University of Washington, JE-10 The Corbet Building, Suite 204; 4507 University Way N.E. Seattle, Washington 98105	11. CONTRACT OR GRANT NO. GC8286, Task 29
12. SPONSORING AGENCY NAME AND ADDRESS	13. TYPE OF REPORT AND PERIOD COVERED
Washington State Department of Transportation Transportation Building, KF-10 Olympia, Washington 98504	Final Report
	14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES

This study was conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration. The FHWA contract manager was Mr. Paul C. Gregson.

16. ABSTRACT

The reported study was used to develop an overlay design procedure for Washington state's flexible pavements based on mechanistic pavement analysis. This design procedure incorporates nondestructive testing (NDT) to estimate layer resilient moduli, seasonal moduli adjustments and failure criteria of asphalt concrete fatigue cracking and subgrade rutting. All necessary computations can be done with two primary microcomputer oriented software packages: EVERCALC to estimate layer moduli from NDT data and EVERPAVE to estimate the needed overlay thicknesses.

17. KEY WORDS	18. DISTRIBUTION STATEMENT
pavement, overlay, mechanistic, resilient modulus, failure criteria, seasonal variation	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22616

19. SECURITY CLASSIF. (of this report)	20. SECURITY CLASSIF. (of this page)	21. NO. OF PAGES	22. PRICE
None	None		

DOT FORM 310-022 (11/86) -1216-

DISCLAIMER

ACKNOWLEDGMENTS

The authors express their appreciation to the Washington State Department of Transportation for supporting this long-term study. We believe this support was critical in developing products which are implementable.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

SUMMARY

The increasing emphasis on pavement maintenance and rehabilitation have prompted the development of more rational and cost effective overlay design procedures. Hence, mechanistic rather than empirical overlay design is emphasized. Of the various mechanistic models, the multilayered elastic analysis of pavement provides reasonable and effective solutions.

There are two primary concerns in the development of a mechanistic based overlay design procedure. One is how to evaluate the existing pavement and the other is identification and integration of pavement design variables.

The existing pavement evaluation is accomplished through back calculation of pavement surface deflection basins, which are measured by nondestructive testing devices. The falling weight deflectometer provides variable and large impulse loadings to the pavement surface and to some degree simulates actual truck traffic. Back calculation also has some drawbacks, such as static interpretation of dynamic load deflection and nonunique solutions, but it provides acceptable material property estimates. A back calculation program, EVERCALC, provided solutions with smaller errors than the natural moduli variations of the pavement test sites studied.

EVERCALC's solutions were verified in two ways. A comparison between back calculated moduli and theoretical moduli showed that more than 90 percent of the solutions for asphalt concrete and the base course had a less than 10 percent error, and all solutions for the subgrade had a less than 5 percent error. The second verification was a comparison of back calculated and laboratory moduli. The results showed the greatest range of differences for the asphalt concrete layer, followed by the base and subgrade moduli. However, back calculation showed low moduli for cracked pavement sections (as should be expected), while laboratory testing showed high moduli (based on uncracked cores). Further, these observed differences were generally much less than the variation of moduli that is generally expected within relatively uniform, short lengths of pavement.

Mechanistic overlay design procedures based on empirical failure criteria have received widespread support. This type of overlay design procedure requires traffic estimates, material properties, and failure criteria. Traffic estimates are usually defined in terms of ESALs. Seasonal

adjustments for the material properties (primarily resilient moduli) are important, since they can vary significantly with season. The adjustment for asphalt concrete can be simply achieved through the relationship between a pavement's moduli and its temperature. Pavement temperature can be obtained by the Southgate method (in analyzing the existing pre-overlay pavement structure) or by Witsczak's method for pavement design. The seasonal variations for unbound materials determined by the comparison of back calculated moduli are shown in Table 11 for two climatic regions, eastern and western Washington. As always, engineering judgment is required.

The two pavement failure criteria used are fatigue and rutting failures. Finn's general model is used for the former (based on the tensile strain at the bottom of the asphalt concrete), and Chevron's method is used for the latter (based on the vertical compressive strain at the top of the subgrade). Finn's model is a parallel shift of Monismith's laboratory model.

EVERPAVE is the overlay design program developed during the study which is based on multilayered elastic analysis and the design criteria of fatigue and rutting failures. It also considers stress sensitive characteristics of pavement materials, seasonal moduli variations, various material properties of the new overlay material, etc. Overlay thicknesses predicted by EVERPAVE were compared with those predicted by the AASHTO and Asphalt Institute methods. These comparisons showed that EVERPAVE's predicted thicknesses and the thicknesses from the other methods were, in general, similar.

In order to validate the EVERPAVE overlay design method, close observation of actual pavement behavior and performance is required. The most significant design factors are the fatigue shift factor, seasonal variation parameters, and traffic volumes, which are input data for EVERPAVE.

INTRODUCTION

BACKGROUND

Pavement maintenance and rehabilitation are increasing because highway pavement ages, traffic volumes, and vehicle weights are increasing. Various efforts to develop more rational pavement rehabilitation (overlay) design procedures have resulted in more realistic pavement modeling, improved nondestructive testing devices, and better knowledge about pavement distress mechanisms.

Overlay design procedures are generally categorized by and based on one of three measurement methods: (1) component analysis, (2) deflection, and (3) mechanistic analysis. Each of these has advantages and disadvantages. While the first two methods are mostly empirical (or based on observations without much regard for theory), the third is more rational (generally taken to mean having a more theoretical basis). Therefore, researchers generally agree that mechanistic procedures with nondestructive testing are desirable since they provide the most comprehensive approach [1].

The Washington State Department of Transportation (WSDOT) has based the design of its flexible pavements on a design method that was originated by Hveem and Carmany in California [2]. For overlay design, WSDOT has adapted a component analysis method as well as deflection-based design procedures. However, these design procedures have not been totally adequate for the task. As more WSDOT resources have been committed to pavement rehabilitation, a more rigorous design procedure has been sought. A key requirement of the new procedure is a greatly enhanced ability to estimate in situ material properties of existing pavement structures.

OBJECTIVE

The objective of the study was to develop a more rational overlay design procedure for Washington state's flexible pavements based on mechanistic pavement analysis, nondestructive testing, and the pavement performance data in Washington state.

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

The following conclusions are offered:

1. The back calculation of layer moduli from measured pavement deflection basins provides reasonable estimates of in situ pavement moduli. Further, moduli can be estimated for cracked asphalt concrete pavement sections.
2. Monismith's general fatigue failure model is appropriate for Washington state, but a shift factor of 3 to 5 is recommended (in lieu of the shift factor of 13 used by Finn).
3. EVERPAVE proved to be versatile and predicts reasonable overlay thicknesses compared to other design methods.
4. The tensile strain at the lowest point of the existing asphalt concrete layer generally controls overlay design.
5. The design philosophy assumes that the pre-overlay pavement surface distress conditions before the overlay are fairly uniform (i.e., presealed). However, EVERPAVE can be used to design the depth of needed overlay at any point within a pavement project.

RECOMMENDATIONS

Even though the research was accomplished based on extensive laboratory tests as well as nondestructive tests, the complex behavior of the pavement structures under traffic loads requires more investigation.

1. Continuous monitoring is needed to define sensitive design factors such as seasonal variation, pavement temperature, and the fatigue shift factor.
2. More study is needed for regions such as mountainous areas and eastern Washington because the coverage for those regions was limited in the reported study.

RESEARCH PLAN

INTRODUCTION

Mechanistic overlay design requires a pavement response model, material characterization, and failure criteria. However, the characteristics of pavement materials are complex, and their response under traffic load is not clearly understood. Pavements have been designed primarily with empirical relationships, engineering judgment, and laboratory tests. Therefore, the research team proposed that the development of the new WSDOT overlay design procedure be based partly on mechanistic analysis of nondestructive field data and partly on laboratory testing, as shown in Figure 1 (see page 49) [3]. The tasks for the development of the overlay procedure were as follows:

1. field test site selection,
2. laboratory testing (field sampling and laboratory tests),
3. field nondestructive testing (NDT),
4. development of a nondestructive testing evaluation method,
5. investigation of pavement design variables,
6. development of the overlay design method, and
7. implementation of the overlay design procedure.

The study was jointly performed by the University of Washington, Department of Civil Engineering, and the WSDOT Materials Laboratory. WSDOT was responsible for collecting field data (sampling and deflection data), laboratory tests and overall project direction. The University of Washington was responsible for the remainder of the tasks (except implementation, which was shared).

TEST SITE SELECTION

In order to examine specific pavement performance in Washington state, 21 test sites were selected on the state routes shown in Figure 2 (see page 50). Each test site was 1000 feet long, with 20 deflection test locations designated every 50 feet. These test sites were typical flexible pavement sections and were selected both for their uniformity (construction, distress, subgrade soil) within each test site and for their variety (age, climate, traffic, structural section, distress). However, five of the test sites were eventually dropped because of the nonuniformity of their pavement thickness and the inconsistency of the NDT data. The descriptions of the test sites used for this study are shown in Table 1 (see page 31).

LABORATORY TESTING

WSDOT obtained field asphalt concrete cores and unbound material samples during the summer of 1985. Three asphalt cores were taken at each of three locations (Stations 0+50, 5+50 and 9+50) at each test site. Disturbed base course and subgrade soil samples were obtained at the pavement shoulder at approximately the middle of each test site (Stations 5+00 or 5+50).

The WSDOT Materials Laboratory in Olympia, Washington, performed laboratory tests (the testing sequence is shown in Figures 3, 4, and 5 (see pages 51, 53, and 54)) for asphalt concrete, base course, and subgrade materials, respectively. To determine the modulus of elasticity (stiffness) of the asphalt concrete, the diametral resilient modulus test (ASTM D4123) was conducted at 41°F, 77°F, and 104°F with a load duration of 100 milliseconds [4]. To determine the resilient moduli of the unbound materials, the samples were remolded and recompacted at a moisture content and density similar to that observed in the field at the time of sampling. A triaxial test was performed on each sample with confining pressures of 1, 2, and 4 psi and deviator stresses of 1, 2, 4, 6, and 8 psi, in accordance with AASHTO T274 [5]. Asphalt concrete layer thicknesses were also determined from core samples.

NONDESTRUCTIVE TESTING

WSDOT collected pavement surface deflection measurements with the falling weight deflectometer (FWD). These measurements were collected in the outer wheel path nearly every season from spring 1985 to summer 1987, as shown in Table 2 (see page 32). The measurements were obtained every 50 feet within each test site, with four load levels at each test stop and two drops at each load level. The load levels were approximately 6,000, 9,000, 12,000, and 15,000 lbs.

CHARACTERIZATION OF PAVEMENT SYSTEM

PAVEMENT MODEL

Flexible pavements are generally composed of an asphalt concrete surface, a stabilized or unstabilized base, and a subgrade. The complex characteristics of these materials, their variations in actual pavements, and the diversity of vehicle loadings make it difficult to model the pavement structure. Of several pavement models, the multilayered elastic system has been shown to provide reasonable pavement response solutions (in terms of deflection, stress, or strain due to an applied load).

The multilayered elastic model of a pavement structure under a circular load is shown in Figure 6 (see page 55). This model can be used to determine pavement responses (deflections, stresses, or strains) for given pavement structure and loading conditions, but it 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 directions; and

(3) the materials are characterized by the modulus of elasticity (resilient modulus) and Poisson's ratio [6].

There are contradictions to these assumptions. Traffic loads are extremely variable in intensity, as well as elliptical or rectangular in shape rather than circular (as assumed) and dynamic rather than static. Pavement material behavior is not fully elastic, and the material properties of a single layer are somewhat inhomogeneous and anisotropic. The modulus of a single layer is an equivalent modulus even though the layer is composed of many different materials, such as in a subgrade, for example, where various kinds of soil layers, including bedrock, can occur.

However, a fully monitored pavement experiment showed that the multilayered, linear elastic theory was acceptable [7]. Since the use of layered elastic analysis in pavement analysis and evaluation offers more advantages than empirical approaches, it is gaining wide acceptance by pavement engineers along with the use of nondestructive testing devices (to characterize the existing pavement structure).

Several computerized solutions for the analysis of multilayered elastic systems have been developed. Some of them are more versatile than others with respect to the number of loads, load direction, and interface friction between layers. This study used the CHEVRON N-LAYER, which was developed by the Chevron Research Company [8].

Early investigations showed that, except for BISAR (Bitumen Structures Analysis in Roads developed by Koninklijke/Shell-Laboratorium, Amsterdam), some computer programs exhibited truncation errors in computing the deflection of pavement systems with a high stiffness ratio between the layers. A comparison of BISAR and CHEVRON N-LAYER deflection computations disclosed a difference, but it was negligible for the vast majority of flexible pavement cases [9].

ASPHALT BOUND MATERIAL

The modulus of asphalt concrete depends on its material characteristics and testing conditions (loading time and temperature). The Asphalt Institute developed a relationship between the resilient modulus and those parameters based on numerous laboratory tests [10]. Because of the visco-elastic characteristic of asphalt concrete, temperature and loading rate are significant factors in an asphalt concrete's modulus. The relationship between the resilient modulus and temperature for WSDOT Class B asphalt concrete, shown in Figure 7 (see page 56), was found as follows [11]:

$$\log E_{ac} = 6.4721 - 0.000147362\,(T_p)^2 \qquad \text{(Equation 1)}$$

where:

E_{ac} = the resilient modulus of asphalt concrete (psi)

T_p = pavement temperature (°F).

Adjustments of temperature can be accomplished by using either the above relationship or the Asphalt Institute equation.

Loading time differences can be adjusted by using the relationship between modulus and loading rate, which are included in the Asphalt Institute equation. In order to find the effects of the different loading times of the falling weight deflectometer test and WSDOT's standard laboratory test (about 33 and 100 milliseconds, respectively), the material parameters of WSDOT's Class B asphalt concrete were substituted into the Asphalt Institute equation, and the moduli were predicted for various temperatures and the two loading conditions. The ratio of the modulus for a 33 millisecond loading to that for a 100 millisecond loading for various temperatures was regressed as follows [9]:

$$LAF = 0.791 + 0.00813\,(T_p) \qquad \text{(Equation 2)}$$

where:
LAF = the loading time adjustment factor from FWD to laboratory, and
T_p = pavement temperature (°F).

This relationship adjusts the "field" back calculated asphalt concrete modulus from the FWD deflection data to an equivalent laboratory test condition by multiplying the back calculated modulus by 1/LAF.

As shown in Equations 1 and 2, temperature affects the modulus of asphalt concrete significantly; thus reasonably accurate pavement temperatures must be obtained. Generally, pavement temperatures are higher than air temperatures by approximately 5°F (this value is much higher in the warmer months). For pavement evaluation, pavement temperature is usually determined either by direct measurement or by Southgate's method [12]. Southgate's method can be used to estimate pavement temperature by use of pavement surface temperature, the previous five-day mean air temperature, and pavement thickness, as shown in Figure 8 (see page 57) [12]. The latter method was used for the study.

For the purpose of pavement design, pavement temperature can be determined by Witczak's method, which uses monthly mean air temperature (MMAT). The equation is as follows [13]:

$$MMPT = MMAT\,\{1 + 1/(z+4)\} - 34/(z+4) + 6 \qquad \text{(Equation 3)}$$

where:
MMPT = mean monthly pavement temperature (°F),
MMAT = mean monthly air temperature (°F), and
z = depth below pavement surface (inches).

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 proved to be the most significant factors. Investigations [14] have shown a direct relationship between the modulus and the stress state for unstabilized base materials and subgrade soils. The relationships are generally as follows:

E_{bs} = $K1\theta^{K2}$ for coarse-grained materials and soils (psi), (Equation 4)

M_R = $K3\sigma_d^{K4}$ for fine-grained soils (psi), (Equation 5)

where : E_{bs} = resilient modulus of coarse-grained materials and soils (psi),

M_R = resilient modulus of fine-grained soil (psi),

θ = bulk stress (psi),

σ_d = deviator stress (psi), and

K1, K2, K3, K4 = regression constants

K1 and K3 depend primarily on moisture contents, which can change with season. K2 and K4 are related to material or soil type -- either coarse-grained or fine-grained. Generally, K2 is positive and K4 is negative [14, 15].

STRUCTURAL EVALUATION OF THE PAVEMENT

INTRODUCTION

The need for information about 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 tests of samples or by an NDT evaluation method. Because of the cost and time constraints of the laboratory tests, the NDT method is being used more frequently [42]. NDT evaluations, which generally use the pavement surface deflection basin, are accomplished either by graphic solution or back calculation. The latter is more complicated but more accurate. Thus, the back calculation procedure for NDT deflection measurements becomes crucial for pavement rehabilitation, and a significant amount of the reported study resources were devoted to back calculation development.

FALLING WEIGHT DEFLECTOMETER (FWD)

Of the various nondestructive testing devices available, the FWD was chosen to be the primary focus of this study. There are a number of reasons for this. First, FWD is WSDOT's primary deflection measuring equipment. Second, it can provide variable and large impulse loadings to the pavement surface that, to some degree, simulate actual truck traffic [16].

With the FWD (Dynatest Model 8000), as shown in Figure 9 (see page 58), a transient impulse load is applied through a set of rubber cushions, which results in a load pulse of 25 to 33 milliseconds. The pavement deflections are measured at up to seven locations with velocity transducers.

As with any NDT device, the FWD has a few (but generally minor) drawbacks [17, 18, 19]. For example, the depth to a "rigid layer" in a pavement may affect the deflection basin and hence the back calculation solution. The acceleration of the FWD load is higher than that of a moving wheel load; thus, the inertia of the pavement mass can affect the results. Overall, the FWD has been shown to be a powerful, if not the best, NDT device currently available [20].

Because the Benkelman Beam (BB) has been widely used, previous empirical pavement design procedures have been developed to incorporate BB deflection measurements. Because of this extensive, previous use of BB deflections, the maximum deflections of the FWD were compared with BB deflection measurements for various types of the pavement structures in Washington state. The correlation was found as follows:

$$D_{BB} = 1.33269 + 0.93748\, D_{FWD} \qquad \text{(Equation 6)}$$

$(R^2 = 0.86, n = 713)$

where: D_{BB} = deflection measured by BB (mils)

D_{FWD} = deflection measured by FWD (mils) at the center of the load plate

PAVEMENT DEFLECTION ANALYSIS PROGRAM (EVERCALC)

EVERCALC [21] 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 ones in each iteration. The iteration process continues until the summation of the absolute differences in the theoretical (calculated) and measured deflections fall within an allowable tolerance, usually 10 percent or less. (Analyses performed up to May 1988 show that a tolerance level of about 5 percent is preferred.)

There are currently two versions of the program (EVERCALC 2.0 and EVERCALC 2.1). The primary version used by WSDOT is EVERCALC 2.1. This version of the program is capable of evaluating a flexible pavement structure containing up to three layers and can be run with or without a "rigid base." The program makes an initial, rough estimate of modulus ("seed modulus") for each layer using internal regression equations and then back calculates to determine a "final" modulus for each pavement layer. It 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 (refer to Equations 4 and 5) and then normalizes the asphalt concrete modulus to the WSDOT standard laboratory condition (which is 77°F and load time of 100 milliseconds). Equations 1 and 2 are used to adjust the back calculated moduli to a "standard" laboratory condition.

The seed moduli are estimated with internal regression equations, which were developed from the relationships among the layer moduli, surface deflections, applied load, and pavement thicknesses [22]. The regression equations need six surface deflection measurements at 0-, 8-, 12-, 24-, 36- and 48-inch offsets from the center of the FWD load.

VERIFICATION OF THE PROGRAM

NDT pavement evaluation includes potential sources of error such as (1) errors associated with pavement modeling (pavement responses), (2) those associated with deflection measurements, and (3) those associated with the back calculation process. The first type of error occurs when the pavement structure is modeled as a multilayered elastic system, as described earlier. The second type of error includes the mechanical error of the NDT device and the possible distortion of the deflection basin as a result of a shallow rigid base (or rock layer). This problem is negligible when the rigid base is about 35 feet (or more) deep [19]. The third type of error is associated mainly with the nonuniqueness of back calculated solutions and the deflection basin convergence, which are somewhat related to the accuracy of the seed modulus estimate.

The basic EVERCALC program was verified in two ways. Back calculated moduli were compared with theoretical ones, and back calculated moduli were compared with laboratory moduli test results.

The first approach was to quantify the convergence problem and the similarity of solutions by comparing theoretical and back calculated moduli for a range of three-layer pavements (432 cases), as shown in Table 3 (see page 33). Basically, this process involved selecting a range of pavement cases and computing their theoretical surface deflection basins for a 9,000 lb. load applied on a circular plate of the same size as the FWD. The average and standard deviation of the differences are shown in Table 4 (see page 34), and the cumulative percentage versus the error is shown in Figure 10 (see page 59). EVERCALC showed that 90 percent of the solutions were within a 10 percent error band and 95 percent of the solutions were within a 15 percent error band. The solutions for the subgrade were the most accurate (all solutions had less than a 5 percent error) and those for base course the least accurate (93 percent of the solutions had less than a 20 percent error). Thus the solutions of EVERCALC were reasonably accurate.

The second verification attempt compared back calculated and laboratory moduli. These results are shown in Table 5 for the asphalt concrete (see page 35), Table 6 for the base (see page 37), and Table 7 for the subgrade layers (see page 38). The asphalt concrete layer had the greatest range of differences, followed by the base and subgrade materials; however, large differences between the back calculated and the laboratory results should be expected for sites with extensive cracking. The observed differences between the back calculated and laboratory moduli did not offer a true verification, since the laboratory test procedures do not necessarily provide a reference (or true) moduli. Further, these observed differences were generally less than the natural variation of moduli usually expected within relatively uniform, short lengths of pavement (1,000 ft).

The stress sensitivity constants determined by the WSDOT triaxial laboratory tests and back calculation are shown in Table 8 for the base course (see page 39) and Table 9 for the subgrade materials (see page 40). The constants for the base course material were generally better correlated than those of the subgrade material, which were poorly correlated.

OVERLAY DESIGN PROCEDURE

INTRODUCTION

Traffic load repetitions and environment are two of the 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. For overlay design, the relationships among pavement performance (hence distress), pavement material properties, and pavement layer thicknesses have been used. Numerous studies of pavement distress have shown that pavement performance is related to pavement response parameters (such as stress and strain), which are determined through mechanistic pavement analysis.

This section explains the design criteria and a mechanistic based overlay design procedure computer program, EVERPAVE.

DESIGN CRITERIA

The principal distresses associated with traffic load repetition are fatigue cracking and rutting. Investigations have shown that fatigue failure is best related to the horizontal tensile strain at the bottom of the asphalt bound layer and one type of rutting can be related to the vertical compressive strain at the top of the subgrade [10].

The models for fatigue failure criterion are generally a function of the tensile strain and the modulus of the asphalt bound material. Monismith's laboratory model, one of the most widely used models, is as follows [24]:

$$\log N_f = 14.82 - 3.291 \log (\epsilon_t) - 0.854 \log (E_{ac}/1000) \qquad \text{(Equation 7)}$$

where:

N_f = loads to failure,

ϵ_t = initial tensile strain (10^{-6} in/in), and

E_{ac} = the modulus of asphalt bound material (psi).

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. Since 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 (SHF). The resulting predictive equation becomes

$$N_{field} = (N_{lab})(SHF).$$

However, a wide range of shift factors were found in the literature, as shown in Table 10a (see page 42) [3]. 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. For thick asphalt concrete pavement, early investigations [7, 25] found that the maximum tensile strain occurred at mid-depth of the asphalt concrete layer rather than at the bottom of the layer. The shift factors developed from thin asphalt concrete pavements are not directly applicable to thick asphalt concrete pavements. As the asphalt concrete ages, its modulus increases and its fatigue behavior can be different from that of new asphalt concrete materials.

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 failure distress (Table 10b). The pavements' service lives were 10 to 13 years and the thicknesses of the asphalt concrete ranged from 4 to 10 inches. The moduli of the original asphalt concrete were estimated based on engineering judgment and the results of laboratory tests on pavement cores. The moduli of the unbound materials were obtained through back calculation 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 (i.e., the N_{lab} estimate from Monismith's model is multiplied by 0.1 to 6.0 to estimate N_{field}). However, the lower shift factors were for thick asphalt concrete (about 8 to 9 inches). Thus, more reasonable shift factors of about 3.0 to 6.0 were found for sections with asphalt concrete thicknesses of 5 to 7 inches.

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 of the performance relationships that are developed for new asphalt concrete. Some design procedures consider the strain at the bottom of a new asphalt concrete layer; others consider the strain only at the bottom of the existing asphalt concrete layer or both strains [26, 27, 28, 29, 30]. Both strains (bottom of new overlay layer and bottom of the existing (pre-overlay) asphalt concrete) were considered in this study and are used in EVERPAVE.

Rutting occurs because of permanent deformation of the asphalt concrete layer and unbound layers. However, since the permanent deformation of asphalt concrete is not as yet well defined, the failure criterion equations are expressed as a function of vertical compressive strain at the top of the subgrade (therefore only rutting in the subgrade soils is considered). The project team concluded (for now) that rutting in the asphalt concrete is primarily a mix design issue. The Chevron equation was used to estimate rutting in the subgrade because it is slightly more conservative than the others [31]. It is as follows:

$$N_r = 1.077 \times 10^{18} (\epsilon_{vs})^{-4.4843} \qquad \text{(Equation 8)}$$

where: N_r = number of loads needed to cause approximatly a 0.75 inch depth rut,

ϵ_{vs} = vertical compressive strain at the top of the subgrade (10^{-6} in/in).

PAVEMENT ENVIRONMENTAL EFFECTS IN WASHINGTON STATE

Temperature and precipitation are the two primary environmental factors that induce significant changes in pavement materials [32]. Therefore, the consideration of environmental conditions is essential in mechanistic pavement design. Seasonal adjustment for asphalt-bound materials is generally obtained simply from the relationship between the modulus and temperature. However, for unbound (unstabilized) materials that process is not simple because of the complex interaction between the unbound materials and the environment. This section covers only unbound materials.

In the state of Washington, the Cascade Mountain range is a topographic and climatic barrier separating the state into "eastern" and "western" Washington. Two types of climate prevail in Washington, a marine type in the west and a continental climate east of the Cascade Mountains [33]. The mean monthly air temperatures for the climatic regions [34], shown in Figure 11 (see page 60), reflect these climates. 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, as shown in Figure 12 (see page 61) [35].

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 [36]. Seasonal variations for each of the two regions was investigated over two distinct seasons (wet or dry) that were based on the back calculated moduli from three years of FWD data (spring 1985 to spring 1987) and climatic data obtained from published climatological information. The ratio of the moduli of different seasons were determined and are shown in Table 11 (see page 43).

Seasonal variations in the base layer were greater than those in the subgrade. This may be due, in part, to the equivalent stiffness concept of pavement modeling. Although the subgrade can consist of various layers, it is usually assumed to be homogeneous and semi-infinite in depth for the pavement modeling. Thus, the back calculated subgrade modulus is the equivalent modulus of the whole depth. Obviously, seasonal change in modulus occurs to a certain depth, which generally includes the whole depth of the base course and the upper subgrade. Therefore, the application of the seasonal variation of a shallow subgrade (where severe seasonal variation can occur) to a semi-infinitely deep subgrade may lead to a result that is not conservative.

The effect of pavement surface cracking on the modulus change in unbound materials was found to be significant. After two severely cracked pavement test sites were overlayed, the back calculated moduli of the unbound layers increased about 30 percent.

Care should be taken in applying seasonal variation adjustments because micro-climate conditions vary significantly in both location and time. The distress condition of the pavement also should be taken into account.

TRAFFIC IN WASHINGTON STATE

Traffic is a significant parameter for pavement design and maintenance. The effect on the pavement depends on vehicle type and volume (i.e., axle configuration and weight), tire pressure, contact area between tire and pavement, and axle repetitions [3].

The primary concern is how to quantify the mixed traffic for a design period to use in pavement design. Traffic volume is generally expressed in terms of 18 kip equivalent single axle loads (ESAL). AASHTO's load equivalency factors [32], developed from the AASHO Road Test, have been used extensively for mixed traffic conversion in the United States. Traffic load information is usually accumulated in the format of the Federal Highway Administration's W-4 loadmeter tables, which include the number of axles observed with a series of load groups and the number of vehicles within each category.

During this study, the W-4 tables were used to determine the 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. The results are shown in Table 12 (see page 44). The number of axles and truck weights have been increasing, and changes (increases) have occurred since 1976, when the federal regulation of the maximum single axle weight was changed from 18,000 to 20,000 lbs. and tandems from 32,000 to 34,000 lbs. [11].

Design traffic volume is usually determined from average daily traffic (ADT), truck percentage (single units and combinations), and the ESALs per truck for each road section.

OVERLAY DESIGN PROGRAM (EVERPAVE)

EVERPAVE [37] is a mechanistic based overlay design program. The pavement analysis is accomplished by use of EVERSTRS (used as a subroutine), which can account for the stress sensitive characteristics of the unbound materials [42]. A flow chart of the EVERPAVE program is shown in Figure 13 (see page 62). Most variables were assigned as input data because their use requires engineering judgment. The input data include design traffic volume, seasonal variations of material properties, temperatures, the shift factor for fatigue failure, the minimum overlay thickness, and thickness increment and material properties. The material property data for the moduli are the stress sensitive coefficients K1, K2, K3, and K4, included in Equations 3 and 4.

The program can analyze a pavement system of up to five layers, including a new overlay. The pavement responses under dual wheel loads are determined from the analysis of a pavement system, as shown in Figure 14 (see page 63). 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 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 is 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 minimum distress performance period exceeds the design traffic volume.

As previously stated, the traffic volume is estimated in terms of ESALs. Further, mean monthly air temperatures (MMAT) are converted to mean monthly pavement temperatures (MMPT) using Equation 3.

EXAMPLE COMPARISON OF OVERLAY THICKNESS

INTRODUCTION

A comparison of the overlay thicknesses produced by different methods was used to examine the new overlay design procedure (EVERPAVE). Since every overlay design method has its own peculiar design parameters, the comparisons were limited to the AASHTO and Asphalt Institute methods, which are currently widely used. The comparison was based on a pavement section in eastern Washington overlayed in 1987 on SR 195, mileposts 19.50 to 23.5 (near Pullman, Washington).

The original pavement section contained 4.2 inches of asphalt concrete and a 10-inch, unbound granular base course. The overlay design traffic volume was 1 million ESALs. The pavement section was split into five subsections (three northbound and two southbound) based on surface deflections. Pavement material moduli were determined by EVERCALC using FWD surface deflection measurements. The moduli and the maximum deflections for each section are shown in Table 13 (see page 45), in which the design moduli are determined from the lowest 80 percentile values of the back calculated moduli.

OVERLAY THICKNESS DESIGN

EVERPAVE

EVERPAVE used the overlay design parameters for eastern Washington and the stress sensitivity characteristics of the base and subgrade were ignored. The input data used were as follows:

1. The shift factor for fatigue failure was 4.0.
2. The design load was dual tire with 4500-lb per tire load (total of 9,000 lb.), 100-psi tire pressure and 15-inch center-to-center spacing.
3. Seasonal variations were as follows:

Category	Spring	Summer	Autumn	Winter
Base material	0.65	1.00	1.00	1.00
Subgrade	0.95	1.00	1.00	1.00
Air Temp (°F)	45.2	63.5	48.3	31.4

4. The modulus of new asphalt concrete was 400,000 psi.

AASHTO Method

AASHTO's Overlay Design Method 1 was used for this study. The Structural Number was determined based on the relationship between the material coefficients and the material moduli. The assumptions were as follows:

1. the PSI for the new pavement and the overlay was 4.5;
2. the terminal PSI was 2.5;
3. drainage factors were ignored;
4. the material layer coefficient for the new asphalt concrete was 0.45; and
5. the reliability was set at 90 percent and the standard deviation was 0.35.

Asphalt Institute Method

Because the Asphalt Institute's overlay design procedure was developed to use data from the Benkelman Beam (BB), FWD deflections were converted to BB deflections by using Equation 6. The calculation of representative rebound deflection (RRD) required temperature and seasonal adjustment factors. These factors, the RRDs, and overlay thicknesses are shown in Table 14 (see page 46).

The overlay thicknesses determined by the three methods are shown in Table 15 (see page 47). Overall, all the methods produced generally similar thicknesses except for section NB-1. Since the overlay thicknesses were somewhat similar, the value of using EVERCALC lies, in part, in determining (hence examining) the individual layer moduli -- a capability heretofore not available. Presumably, this will help pavement designers better understand the root causes of the pavement deterioration which "triggers" the need for an overlay.

IMPLEMENTATION REQUIREMENTS

The overlay design procedure, EVERPAVE, contains unproved aspects because the pavement behavior under traffic load repetitions in ambient climatic conditions has not yet been clarified. The implementation of the new overlay design method requires close observation of the actual performance of overlayed pavement and the significant design variables.

Contrary to the generally accepted concept that most damage occurs during the spring thaw period in regions with seasonal frost, the mechanistic based overlay design process as used thus far in Washington state has shown that more pavement damage occurs in warm periods (summer/fall) when the asphalt concrete stiffness is at its lowest because of high temperature. (However, the winter/spring wet conditions for asphalt concrete are <u>not</u> accounted for.) Thus, it is very important to observe pavement behavior and performance closely during critical periods: spring thaw periods and summer in eastern Washington, and later winter or early spring and summer in western Washington. The significant variables for mechanistic based overlay design procedures are the shift factor for the fatigue failure criterion, seasonal variations of unbound materials, the temperature change in asphalt concrete, and traffic volume.

The monitoring should include (to the extent possible) pavement surface deflection, the level of distress, traffic volume, air and pavement temperature, precipitation and soil moisture condition, frost penetration, and thaw.

The remaining issues to be accomplished by the implementation process include the following:

1. Train District personnel to use the elastic layered analysis (a process that was started during the reported study in all six WSDOT districts).
2. Exercise the overlay design program through about 100 projects to obtain a general feeling of its reasonableness.

3. Set up a data collection process to check the range of the shift factor. For example, is the

    a. factor higher for high traffic (and hence age effects)?

    b. factor higher in western Washington and hence lower in eastern Washington (with its more severe climate)?

4. Look at risk and reliability and a possible tie to the fatigue shift.

5. Investigate the effects of pre-overlay pavement surface distress on overlay performance.

REFERENCES

1. F.N. Finn and C.L. Monismith, "Asphalt Overlay Design Procedures," National Cooperative Highway Research Program, Synthesis of Highway Practice 116, Transportation Research Board, Washington D.C., December 1984

2. Washington State Transportation Commission, "Design Manual," Washington State Department of Transportation, Olympia, Washington, 1987.

3. D.E. Newcomb, A.A. Bu-bushait, J.P. Mahoney and J. Sharma, "State-of-the-Art on Pavement Overlay Design Procedures,"Vol. 1, Report No. WA-RD 65.1, Washington State Department of Transportation, Olympia, Washington, 1983.

4. "Annual Book of ASTM Standards," Vol. 04.03, American Society for Testing and Materials, 1984.

5. "Methods of Sampling and Testing", Part II, American Association of State Highway and Transportation Officials, Washington, D.C., 1986.

6. D.M. Burmister, "The Theory of Stress and Displacements in Layered System and Application to the Design of Airport Runways," Proceedings, Highway Research Board, Washington D.C., 1943.

7. W.G. Bleyenberg, A.I.M. Claessen, F.V. Gorkum, W. Heukelom and A.C. Pronk, "Fully Monitored Motorway Trials in the Netherlands Corroborate Linear Elastic Design Theory," Proceedings, Vol. 1, Fourth International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, Michigan, 1977, pp. 75-90.

8. J. Michelow, "Analysis of Stress and Displacements in an n- Layered Elastic System Under a Load Uniformly Distributed on a Circular Area," California Research Corporation, Richmond, California, 1963.

9. S.W. Lee, J.P. Mahoney and N.C. Jackson, "A Verification of Backcalculation of Pavement Moduli," Paper Accepted for Publication, Transportation Research Board, Washington D.C., 1988.

10. The Asphalt Institute, "Research and Development of The Asphalt Institute's Thickness Design Manual (MS-1) Ninth Edition," Research Report No. 82-2, The Asphalt Institute, College Park, Maryland, August 1982.

11. A.A. Bu-bushait, "Development of a Flexible Pavement Fatigue Model for Washington State," Ph.D. Dissertation, Department of Civil Engineering, University of Washington, Seattle, 1985.

12. H.F. Southgate and R.C. Deen, "Temperature Distributions in Asphalt Concrete Pavements," Highway Research Record No. 549, Highway Research Board, Washington D.C., 1975, pp. 39-46.

13. M.W. Witczak, "Design of Full Depth Asphalt Airfield Pavements." Proceedings, Third International Conference on the Structural Design of Asphalt Pavement, Proceedings, Vol. 1, London, England, 1972, pp. 550-567.

14. Y.T. Chou, "Engineering Behavior of Pavement Materials: State of Art," Report No. FAA-RD-77-37, US Army Engineer Waterways Experiment Station, Vicksburg, Miss., February 1977.

15. M.W. Witczak, "Comprehensive Evaluation of Laboratory Resilient Moduli Results for Granular Material," Transportation Research Record No. 810, Transportation Research Board, Washington D.C., 1981, pp. 23-33.

16. M.S. Hoffman and M.R. Thompson, "Backcalculating Nonlinear Resilient Moduli from Deflection Data," Transportation Research Record No. 853, Transportation Research Board, Washington D.C., 1982, pp. 42-51.

17. M.S. Mamlouk, "Use of Dynamic Analysis in Predicting Field Multilayer Pavement Moduli," Transportation Research Record No. 1043, Transportation Research Board, Washington D.C., 1985, pp. 113-121.

18. R.E. Smith and R.L. Lytton, "Synthesis of Study of Nondestructive Testing Devices for Use in Overlay Thickness Design of Flexible Pavements," Report No. FHWA-RD-83/097, Federal Highway Administration, Washington D.C., April 1984.

19. W. Uddin, A.H. Meyer and W.R. Hudson, "Rigid Bottom Considerations for Nondestructive Evaluation of Pavements," Transportation Research Record No. 1070, Transportation Research Board, Washington D.C., 1986, pp. 21-29.

20. O. Tholen, J. Sharma and R.L. Terrel, "Comparison of Falling Weight Deflectometer with Other Deflection Testing Devices," Transportation Research Record No.1007, Transportation Research Board, Washington D.C., 1985, pp. 20-26.

21. "EVERCALC User's Guide," Washington State Transportation Center, University of Washington, Seattle, Washington, April 1987.

22. D.E. Newcomb, "Development and Evaluation of Regression Methods to Interpret Dynamic Pavement Deflections," Ph.D. Dissertation, Department of Civil Engineering, University of Washington, Seattle, Washington, 1986.

23. K. Majidzadeh and G.J. Ilves, "Flexible Overlay Design Procedures Vol. 1. Evaluation and Modification of the Design Methods," Report No. FHWA-RD-81/032, Federal Highway Administration, Washington, D.C., 1981.

24. C.L. Monismith and J.A. Epps, "Asphalt Mixture Behavior in Repeated Flexure," Institute of Transportation and Traffic Engineering, University of California, Berkeley, California, 1969.

25. C.L. Monismith, "Fatigue Characteristics of Asphalt Paving Mixtures and Their Use in Pavement Design," Paper prepared for Symposium, Fatigue of Asphalt Concrete Pavement, University of New Mexico, Albuquerque, New Mexico, January 1981.

26. R.C. Koole, "Overlay Design Based on Falling Weight Deflectometer Measurements," Transportation Research Record No. 700, Transportation Research Board, Washington D.C., 1979, pp. 59-72.

27. M.S. Mamlouk and B.E. Sebaaly, "Overlay Thickness Design for Flexible Pavement," Transportation Research Record No. 993, Transportation Research Board, Washington D.C. 1986, pp. 63-66.

28. R.E. Smith, M.I. Darter and R.L. Lytton, "Mechanistic Overlay Design Procedures Available to the Design Engineer," Transportation Research Record No. 1060, Transportation Research Board, Washington D.C., 1986.

29. M.R. Thompson and M.S. Hoffman, "Concepts for Developing a NDT Based Asphalt Concrete Overlay Thickness Design Procedure", Paper Presented at the 1983 Annual Meeting of the Transportation Research Board, Transportation Research Board, Washington, D.C., 1982.

30. J.H. Tenison, Jr., "Proposed New Mexico State Highway Department Elastic-Layered Overlay Evaluation and Design Procedure for Asphalt Concrete Pavements," Research Report No. MB-RR-83/2, Materials Laboratory Bureau, New Mexico State Highway Department, Santa Fe, New Mexico, 1983.

31. J.F. Shook, F.N. Finn, M.W. Mitczak, and C.L. Monismith, "Thickness Design of Asphalt Pavements -- The Asphalt Institute Method," Proceedings, Vol. 1, Fifth International Conference on the Structural Design of Asphalt Pavement, The Netherlands, 1982, pp. 17-44.

32. "Proposed AASHTO Guide for Design of Pavement Structures" NCHRP Project 20-7/24, American Association of State Highway and Transportation Officials, March 1985.

33. J.A. Ruttner, "Climates of the States Vol. 2," National Oceanic and Atmospheric Administration, Gale Research Company, 1985.

34. "Climatological Data Washington," National Climatic Center, National Oceanic and Atmospheric Administration, 1985-1987.

35. R.W. Chapin, "General Soil Map Washington," United States Geologic Survey, May 1968.

36. J.A. Lary, J.P. Mahoney, C.A. Bell and R.G. Hicks, "Seasonal Environmental Effect on the Strength of Pavement Structures," Research Report for the U.S. Forest Service, University of Washington, Seattle, May 1983.

37. "EVERPAVE User's Guide," Washington State Transportation Center, University of Washington, Seattle, 1987.

38. S.F. Brown and P.S. Pell, "A Fundamental Structural Design Procedure for Flexible Pavement," Proceedings, Vol. 1, Third International Conference on the Structural Design of Asphalt Pavements, London, England, 1972.

39. W. Van Dijk, "Practical Fatigue Characterization of Bituminous Mixes," Proceedings, Association of Asphalt Paving Technologists, Vol. 44, 1975.

40. D.E. Pickett, D. Saylak, R.L. Lytton, W.E. Conger, D.E. Newcomb and R.A. Schapery, "Extension and Replacement of Asphalt Cement with Sulfur," Report No. FHWA-RD-78-95, Federal Highway Administration, Washington, D.C., March 1978.

41. F. Finn, C. Saraf, R. Kulkarni, K. Nair, W. Smith, and A. Abudullah, "The Use of Distress Prediction Subsystems for the Design of Pavement Structures," Proceedings, Vol. 1, Fourth International Conference on the Structural Design of Asphalt Pavements, London, England, 1977.

42. Federal Highway Administration, "Summary of the AASHTO Questionnaire on The Guide for Design of Pavement Structures, 1986," Federal Highway Administration, Office of Highway Operations, Pavement Division, Washington, D.C., April 1988.

Table 1. Test Site Descriptions

Test Site No.	Test Site Route No.	Test Site Milepost	Year Original Construction (overlay year)	ACP Thick-ness	Base Thick-ness	Observed Surface Distress (if any)
1	SR 11	20.85	72	5.2	28.8	Long. cracking
2	SR 20	53.50	73	4.9	4.8	Long. cracking
3	SR 20	77.50	68 (85)	10.9	6.6	---
4	SR 20	108.20	78	3.5	9.0	---
5	SR 20	140.80	72	3.4	6.6	---
6	SR 167	17.80	68 (80)	11.2	---	---
7	SR 202	30.12	78	13.0	---	---
8	SR 410	9.60	68	7.3	3.6	Fatigue cracking
9	SR 5	35.80	73	16.4	---	---
10	SR 14	18.15	73	9.0	3.6	Long. cracking
11	SR 411	18.05	79	6.8	21.0	---
12	SR 500	3.20	79	6.3	8.4	---
13	SR 90	208.65	73	9.6	8.4	Long. & Trans. crack.
14	SR 90	208.85	73	9.6	8.4	Trans. cracking
15	SR 195	7.24	70 (85)	6.2	11.4	---
16	SR 195	63.80	76	8.5	12.0	---

Table 2. Falling Weight Deflectometer Measurement Dates

TS	1985 SPR	1985 SUM	1985 FAL	1986 SPR	1986 SUM	1986 FAL	1987 WIN	1987 SPR
1	05-21	08-08			07-16	10-09	01-27	06-01
2	05-21	08-08			07-15	10-09	01-27	06-03
3	05-21	08-27			07-15	10-08	01-26	06-02
4	05-21	08-28			07-15	10-08	01-26	06-02
5		08-28	10-10		07-15	10-08		06-02
6	05-29	07-23			07-17	10-10	01-28	
7	06-06	08-19				10-21	01-28	05-21
8	05-15	07-24		04-28	07-24	10-14	01-30	05-12
9	05-13	08-06		04-24	07-23	10-13	01-29	05-11
10	05-14	08-22		04-29	07-23	10-13	01-29	05-12
11	05-13	08-06		04-28	07-23	10-13	01-29	05-11
12	04-10	07-30		04-02	07-14	10-06		03-19
13	04-10	07-30		04-02	07-14	10-06		03-19
14	04-16	07-17	10-09	03-26	07-10	10-01		03-18
15	04-18	07-17	10-09	03-27	07-10	10-01		03-17

Table 3. Hypothetical Pavement Sections
Used for Verification of EVERCALC

Layers	Category	Properties
ACP	Thickness (in)	3, 5, 8
	Modulus (ksi)	100, 300, 500, 800
	Poisson's Ratio	.35
Base	Thickness (in)	6, 12, 24
	Modulus (ksi)	10, 20, 40
	Poisson's Ratio	.40
Subgrade	Thickness (in)	Semi-infinite
	Modulus (ksi)	5, 10, 20, 30
	Poisson's Ratio	.45

Table 4. Calculated Errors from Comparison of Backcalculated and Theoretical Layer Moduli

Surface Thick. (in.)	Layer	Error (%) Theoretical Elastic Modulus (psi) 100,000 Mean	100,000 Std	300,000 Mean	300,000 Std	500,000 Mean	500,000 Std	800,000 Mean	800,000 Std	Average Mean	Average Std
3.0	ACP	4.3	10.8	-3.2	8.9	0.0	11.5	-1.0	5.1	0.0	9.1
	Base	1.8	3.5	1.7	3.7	-0.3	7.4	-0.5	5.4	0.7	5.0
	Subgrade	0.2	1.3	0.0	0.9	0.0	1.3	0.0	1.5	0.1	1.3
5.0	ACP	-3.8	6.1	-2.4	6.1	-1.8	8.9	-2.6	6.9	-2.7	7.0
	Base	2.7	6.7	3.3	8.0	4.0	15.2	2.8	14.6	3.2	11.1
	Subgrade	0.1	0.9	-0.3	1.2	-0.3	1.3	0.0	1.2	-0.1	1.2
8.0	ACP	0.1	8.6	-0.5	5.5	-0.8	4.5	-1.8	3.9	-0.8	5.6
	Base	3.4	8.6	6.5	17.2	4.7	10.7	4.5	14.6	4.8	12.8
	Subgrade	-0.3	1.0	-0.4	1.4	-0.2	1.5	-0.2	1.7	-0.3	1.4
Avg.	ACP	0.2	8.9	-2.0	7.1	-0.9	8.8	-1.8	5.5	-1.1	7.2
	Base	2.6	6.6	3.8	11.3	2.8	11.8	2.3	12.5	2.9	9.6
	Subgrade	0.0	1.1	-0.2	1.2	-0.2	1.4	-0.1	1.5	-0.1	1.3

Note 1: Error (%) = $\left|\left(\frac{\text{Backcalculated modulus - theoretical modulus}}{\text{theoretical modulus}}\right)(100)\right|$

Table 5. Comparison of Backcalculated and Laboratory Asphalt Concrete Moduli

Test Site No.	Solution Method	Modulus (psi) Station 0 + 50	Station 5 + 50	Station 9 + 50	Average
1	NDT/FWD	868,000	668,000	604,000	713,000
	Lab	426,000	469,000	387,000	427,000
	Difference (%)	-51	-30	-36	-40
2	NDT/FWD	725,000	498,000	487,000	570,000
	Lab	633,000	355,000	234,000	407,000
	Difference (%)	-13	-29	-52	-29
3	NDT/FWD	246,000	373,000	568,000	395,000
	Lab	333,000	557,000	334,000	408,000
	Difference (%)	+36	+50	-41	3
4	NDT/FWD	684,000	472,000	194,000	450,000
	Lab	346,000	215,000	244,000	268,000
	Difference (%)	-49	-54	+26	-40
5	NDT/FWD	531,000	494,000	740,000	588,000
	Lab	357,000	147,000	354,000	286,000
	Difference (%)	-33	-70	-52	-51
6	NDT/FWD	722,000	741,000	510,000	658,000
	Lab	171,000	489,000	464,000	355,000
	Difference (%)	-76	-34	-21	-46
7	NDT/FWD	460,000	577,000	757,000	598,000
	Lab	431,000	596,000	743,000	590,000
	Difference (%)	-6	+3	-2	-1
8	NDT/FWD	102,300	42,000	94,000	79,400
	Lab	200,000	227,000	215,000	214,000
	Difference (%)	+96	+438	+130	+170

Table 5. (cont.)

Test Site No.	Solution Method	Modulus (psi) Station 0 + 50	Station 5 + 50	Station 9 + 50	Average
9	NDT/FWD	455,000	657,000	578,000	563,000
	Lab	135,000	117,000	181,000	144,000
	Difference (%)	-70	-82	-69	-74
10	NDT/FWD	131,000	289,000	340,000	253,000
	Lab	779,000	687,000	527,000	664,000
	Difference (%)	+495	+137	+55	+162
11	NDT/FWD	215,000	243,000	358,000	272,000
	Lab	344,000	352,000	219,000	305,000
	Difference (%)	+60	+45	-39	+12
12	NDT/FWD	311,000	242,000	270,000	274,000
	Lab	343,000	414,000	380,000	379,000
	Differemce (%)	+10	+71	+41	+38
13	NDT/FWD	264,000	232,000	344,000	280,000
	Lab	198,000	343,000	318,000	286,000
	Difference (%)	-25	+47	-8	+2
14	NDT/FWD	260,000	218,000	256,000	245,000
	Lab	289,000	240,000	188,000	239,000
	Difference (%)	+11	+10	-27	-2
15	NDT/FWD	404,000	262,000	495,000	387,000
	Lab	375,000	605,000	419,000	466,000
	Difference (%)	-7	+131	-15	+20
16	NDT/FWD	307,000	214,000	321,000	281,000
	Lab	202,000	166,000	164,000	177,000
	Difference (%)	-34	-22	-49	-37

Table 6. Comparison of Backcalculated and Laboratory Base Course Moduli

Test Site No.	Modulus (psi)			Moisture Content (%)	
	NDT/FWD	Lab	Diff. (%)	Field	Lab
1	23,000	15,000	35	3.7	4.0
4	45,000	47,000	4	4.4	5.2
5	38,000	28,000	26	5.0	5.4
11	21,000	11,000	48	4.2	4.7
15	22,000	10,000	55	4.4	5.3
Average	30,000	22,000	27		

Table 7. Comparison of Backcalculated and Laboratory Subgrade Soil Moduli

Test Site No.	Modulus (psi)			Moisture Content (%)	
	NDT/FWD	Lab	Diff. (%)	Field	Lab
1	26,000	9,000	64	5.6	6.1
2	21,000	25,000	19	2.4	4.9
3	15,000	14,000	7	3.7	4.2
4	27,000	28,000	3	3.8	5.1
5	36,000	25,000	30	3.5	3.6
6	29,000	15,000	47	9.6	6.0
7	39,000	33,000	14	5.6	5.5
8	9,000	5,000	44	21.5	17.8
9	37,000	14,000	62	12.2	10.5
10	39,000	18,000	54	7.8	8.1
11	26,000	28,000	8	6.9	11.1
12	36,000	10,000	72	8.2	10.5
13	36,000	19,000	47	10.4	8.9
14	40,000	19,000	52	10.4	8.9
15	20,000	11,000	45	13.6	14.1
16	20,000	5,000	75	11.8	12.5
Average	29,000	17,000	40		

Table 8. Base Course Stress Sensitivity Coefficients

Test Site No.	Solution Method	Stress Sensitivity Coefficients K_1	K_2	R^2 (%)
1	NDT/FWD	4,680	0.68	98
	Lab	7,840	0.38	87
4	NDT/FWD	1,150	1.16	92
	Lab	9,810	0.33	87
5	NDT/FWD	280	1.44	96
	Lab	7,840	0.35	86
11	NDT/FWD	1,590	1.10	96
	Lab	4,770	0.44	90
15	NDT/FWD	11,700	0.32	2
	Lab	6,010	0.45	88

Table 9. Subgrade Soil Stress Sensitivity Coefficients

Test Site No.	Solution Method	Stress Sensitivity Coefficients* K_3	K_4	R^2 (%)
1	NDT/FWD	34,160	-0.24	89
	Lab	5,280	0.50	83
2	NDT/FWD	7,600	0.31	72
	Lab	5,280	0.53	60
3	NDT/FWD	20,610	-0.19	72
	Lab	6,220	0.48	95
4	NDT/FWD	59,050	-0.32	87
	Lab	10,990	0.35	89
5	NDT/FWD	48,710	-0.12	99
	Lab	8,550	0.35	91
6	NDT/FWD	48,670	-0.38	58
	Lab	11,750	0.20	14
7	NDT/FWD	49,176	-0.19	58
	Lab	27,360	0.16	42
8	NDT/FWD	11,910	-0.12	79
	Lab	2,110	0.36	32
9	NDT/FWD	47,750	-0.21	59
	Lab	8,130	0.32	88
10	NDT/FWD	37,270	0.02	20
	Lab	12,090	0.19	17
11	NDT/FWD	44,730	-0.44	97
	Lab	23,750	0.15	34
12	NDT/FWD	21,030	0.15	76
	Lab	6,100	0.20	18
13	NDT/FWD	65,390	-0.19	94
	Lab	14,890	0.12	8

*Note: If relationship is positive, bulk stress (θ) is used and if negative, deviator stress (σ_d).

Table 9. (cont.)

Test Site No.	Solution Method	Stress Sensitivity Coefficients*		
		K_3	K_4	R^2 (%)
14	NDT/FWD	39,260	-0.03	29
	Lab	14,890	0.12	8
15	NDT/FWD	28,760	-0.29	98
	Lab	18,050	-0.29	56
16	NDT/FWD	34,840	-0.30	96
	Lab	3,890	0.15	35

*Note: If relationship is positive, bulk stress (θ) is used and if negative, deviator stress (σ_d).

Table 10. Summary of Fatigue Shift Factors

(a) Prior Studies

Ref. No.	Researcher	Relationship
38	Brown and Pell	$N_{field} = 20\ N_{lab}$
39	Van Dijk	$N_{field} = 3\ N_{lab}$
40	Pickett, et al.	$N_{field} = K\ N_{lab}$, where $K = 0.516 \times 10^{0.0147T}$, T = Temperature, °F
41	Finn, et al.	$N_{field} = 13.0\ N_{lab}$
31	Santucci	$N_{field} = N_{lab} \times 10^m$, where $m = 4.48 \frac{V_b}{V_v + V_b} - 0.69$ V_b = asphalt volume V_v = air voids volume

(b) Washington State Shift Factors

		Performance		
Test Site	AC Thickness (in.)	ESALs(10^3)	Age (years)	Shift Factor
1	5.2	640	11	5.8
3	8.4	466	13	0.4
8	7.3	700	12	2.5
10	9.0	389	12	0.1
13	9.6	2,135	12	0.5
15	3.6	332	10	5.6

Notes:

1. The modulus of the original asphalt concrete was assumed to be 400,000 psi.
2. ESALs accumulated from original construction date to time of fatigue cracking.

Table 11. 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 12. 18 Kip Equivalences for Washington State

	18 kips Eq./Truck					
Year	Inter-States Rural	Other Main Rural	All Rural	Urban Station	All Systems	18 K Eq. per Axle
1950	00000	00000	00000	00000	.4590	.1624
1951	00000	00000	00000	00000	.4810	.1568
1952	00000	00000	00000	00000	.5146	.1637
1953	00000	00000	00000	00000	.5293	.1670
1954	00000	00000	00000	00000	.5665	.1754
1960	.4820	.7425	.7039	.3435	.6479	.1795
1961	.6091	.7132	.6955	.3096	.6337	.1748
1962	.6308	.5376	.5593	.2688	.5143	.1404
1963	.6324	.8786	.7797	.3522	.7145	.2103
1964	.6303	.7530	.7000	.3279	.6401	.1907
1965	.3828	.8017	.6949	.3720	.6633	.1737
1970	.5829	.7997	.6094	.4101	.5708	.1477
1971	.6514	.7881	.6707	.4704	.6283	.1615
1972	.7211	.8161	.7366	.4235	.6400	.1623
1973	.6889	.7987	.7100	.5337	.6504	.1652
1974	.6747	.7884	.6874	.4686	.6399	.1638
1975	.7077	.9738	.7355	.5217	.6860	.1736
1976	.7920	1.0805	.8205	.5757	.7722	.1956
1978	.8174	1.1980	.8664	.5757	.8284	.1964
1979	.9471	1.0090	.9538	.5757	.9000	.2127
1980	.9218	.9519	.9258	.7336	.8901	.2110
1983	.8435	1.0110	.9095	.6989	.8294	.1876

Table 13. Backcalculated Moduli and Maximum Deflections for Overlay Thickness Comparisons (SR-195)

Section. No.	Route & Milepost		Modulus (ksi)* Mean (Std. Dev.)			Max. Deflection (mils) Mean (Std. Dev.)
NB-1	SR195	19.78 - 19.93	ACP Base Subgrade	601.9 40.4 17.4	(273.1) (9.4) (2.6)	13.07 (4.20)
NB-2	"	20.03 - 25.58	ACP Base Subgrade	118.6 6.1 56.9	(73.4) (3.0) (3.6)	48.50 (19.11)
NB-3	"	22.63 - 23.60	ACP Base Subgrade	427.4 15.2 28.3	(203.1) (8.6) (7.3)	19.82 (15.71)
SB-4	"	22.30 - 22.10	ACP Base Subgrade	207.0 6.5 14.6	(16.4) (.5) (1.9)	29.67 (3.87)
SB-5	"	22.00 - 19.90	ACP Base Subgrade	111.3 6.3 11.4	(81.1) (2.8) (3.0)	48.98 (19.58)

*80th percentile values

Table 14. Overlay Design by the Asphalt Institute Method (SR 195)

SECTION		D_{FWD}	D_{BB}	F	C	RRD	T(OVL)
NB-1	MEAN	13.07	13.58	1.40	1.10	33.05	0.00
	STD.	4.20	3.94				
NB-2	MEAN	48.50	46.80	1.40	1.10	133.58	6.00
	STD.	19.11	19.97				
NB-3	MEAN	19.82	19.91	1.40	1.10	76.03	4.00
	STD.	15.71	14.73				
SB-4	MEAN	29.67	29.15	1.22	1.10	48.86	2.50
	STD.	3.87	3.63				
SB-5	MEAN	48.98	47.26	1.22	1.10	112.70	5.50
	STD.	19.58	18.36				

Note : 1. $D_{BB} = 1.333 + 0.93748 \times D_{FWD}$

2. $RRD = (\bar{\chi}m + 2s) \times F \times C$

Table 15. Comparison of Overlay Thicknesses (inches)
for SR-195

Section.	Route & Milepost		EVERPAVE	AASHTO	ASP. INST.
NB-1	SR195	19.78 - 19.93	1.2	0.0	0.0
NB-2	"	20.03 - 25.58	5.2	5.8	6.0
NB-3	"	22.63 - 23.60	2.7	2.1	4.0
SB-4	"	22.30 - 22.10	3.4	3.6	2.5
SB-5	"	22.00 - 19.90	5.1	5.7	5.5

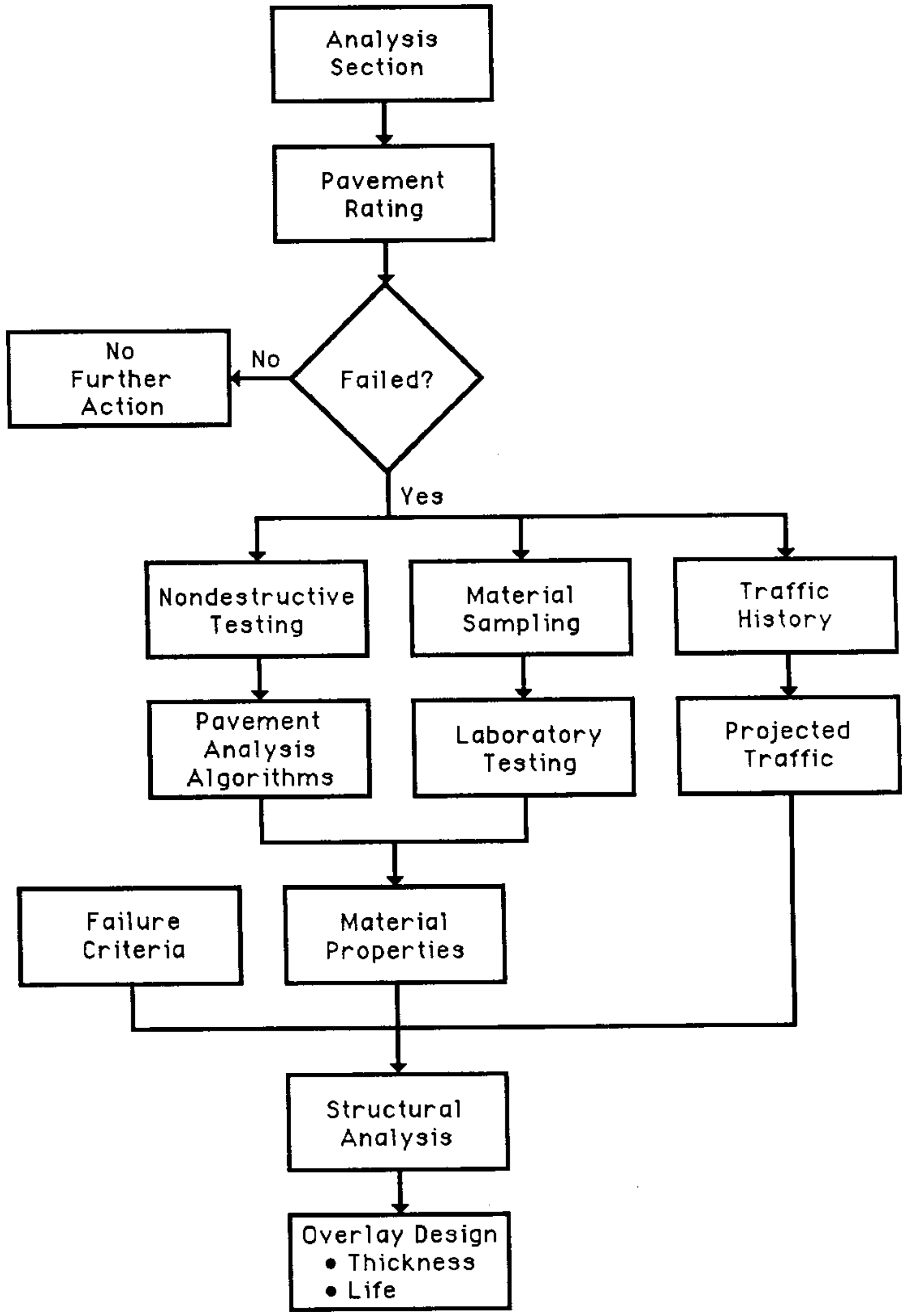

Figure 1. Pavement Overlay Design Concept

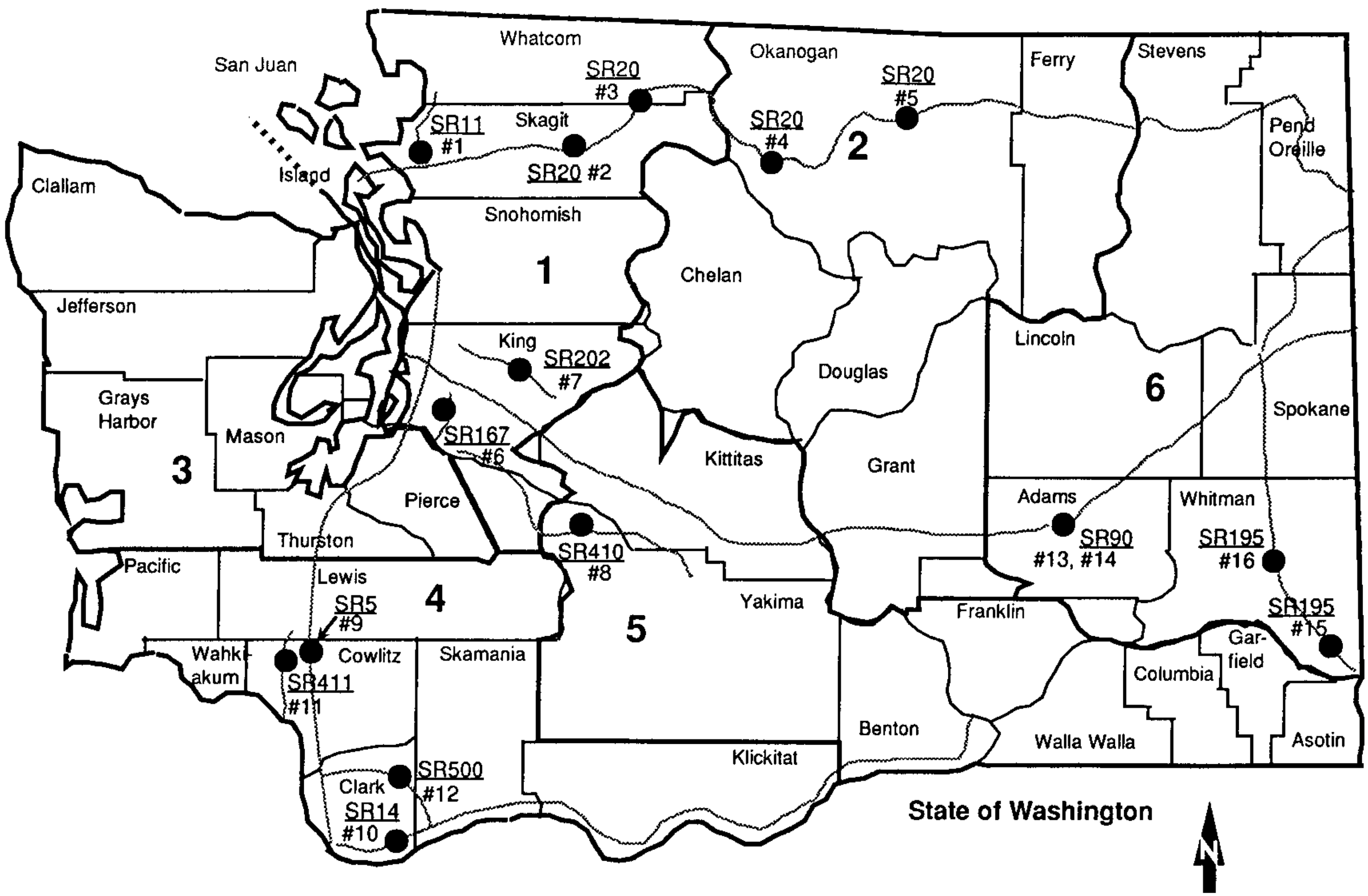

Figure 2. Location of Test Sites

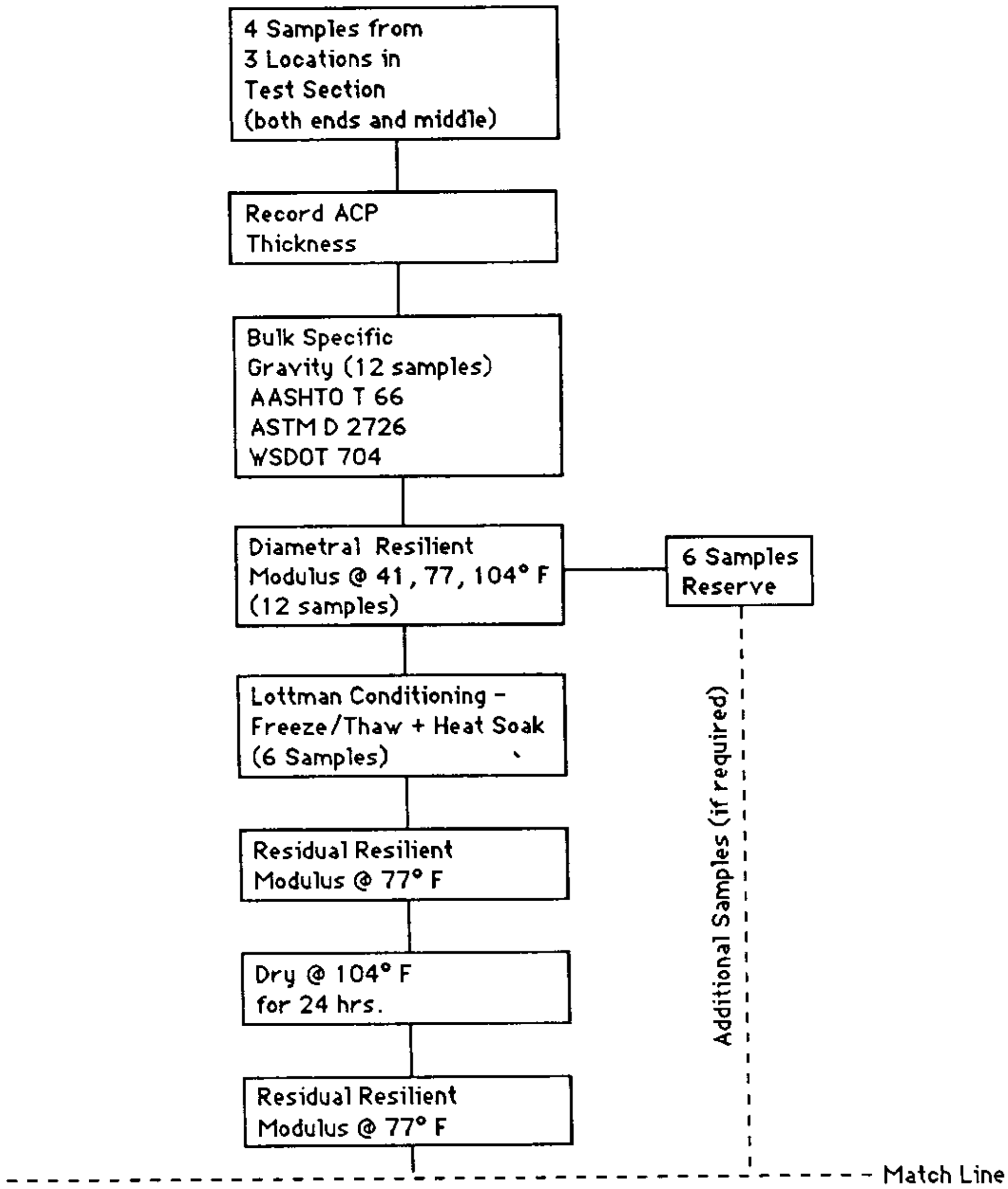

Figure 3. Asphalt Concrete Sampling and Testing

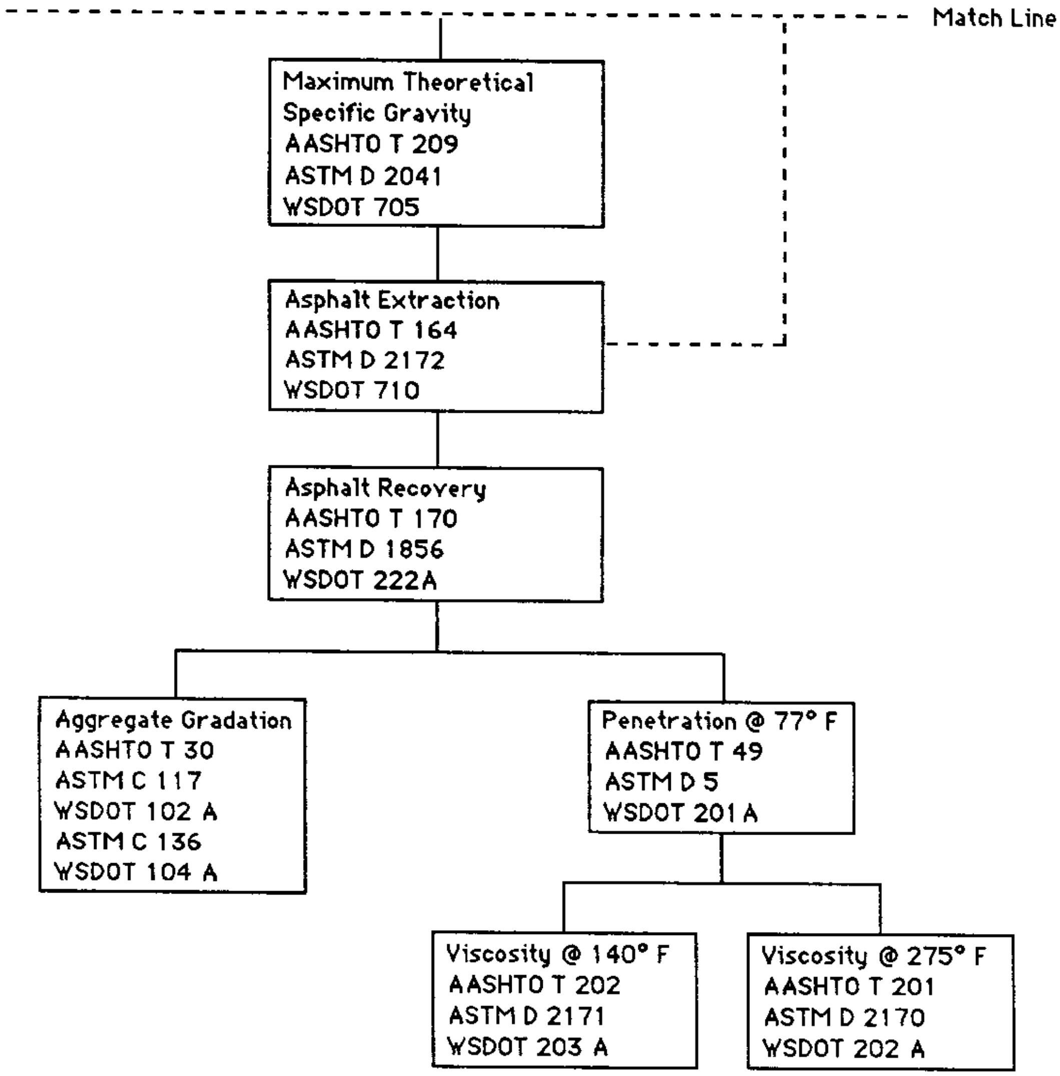

Figure 3. (cont.)

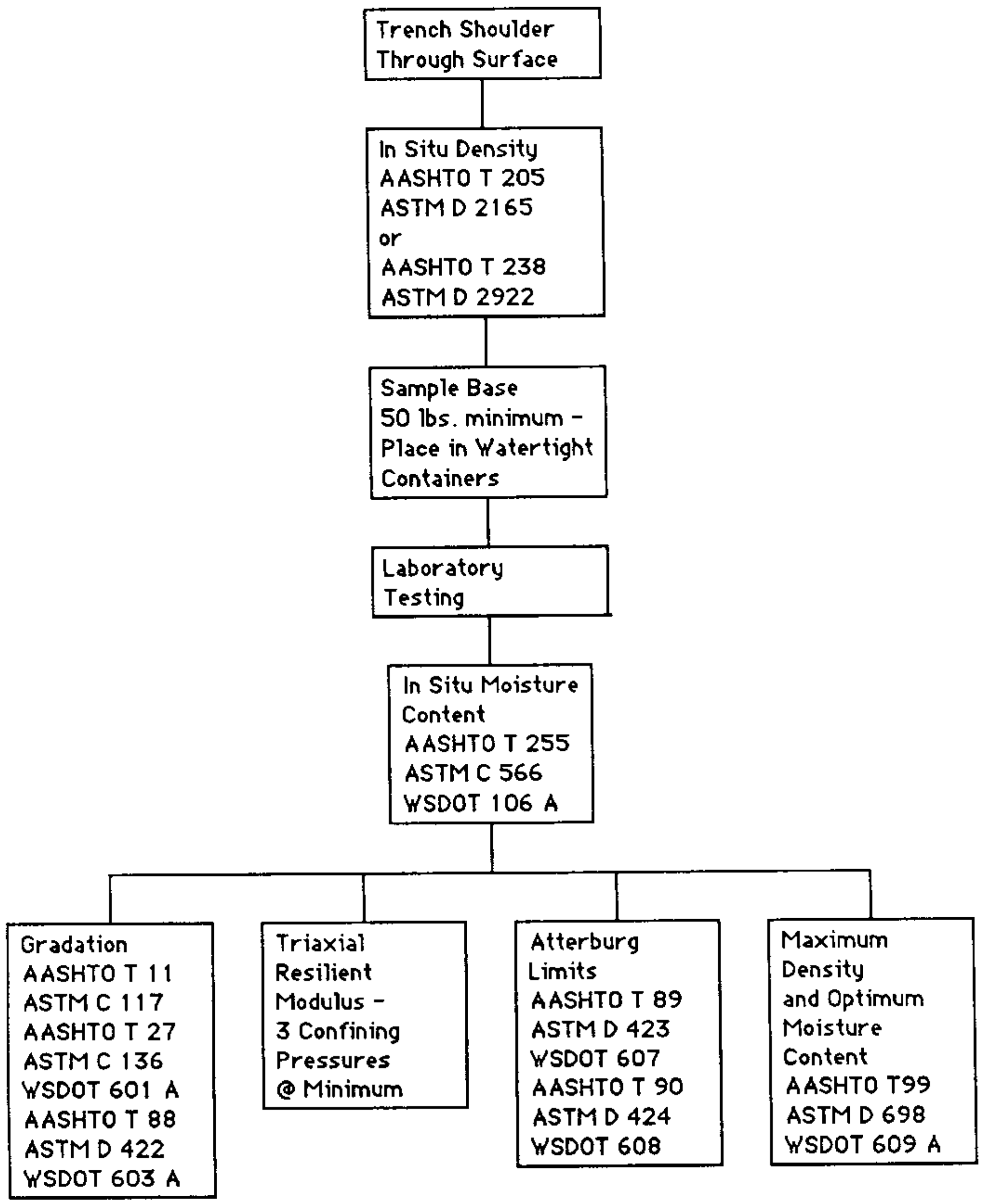

Figure 4. Granular Base Sampling and Testing

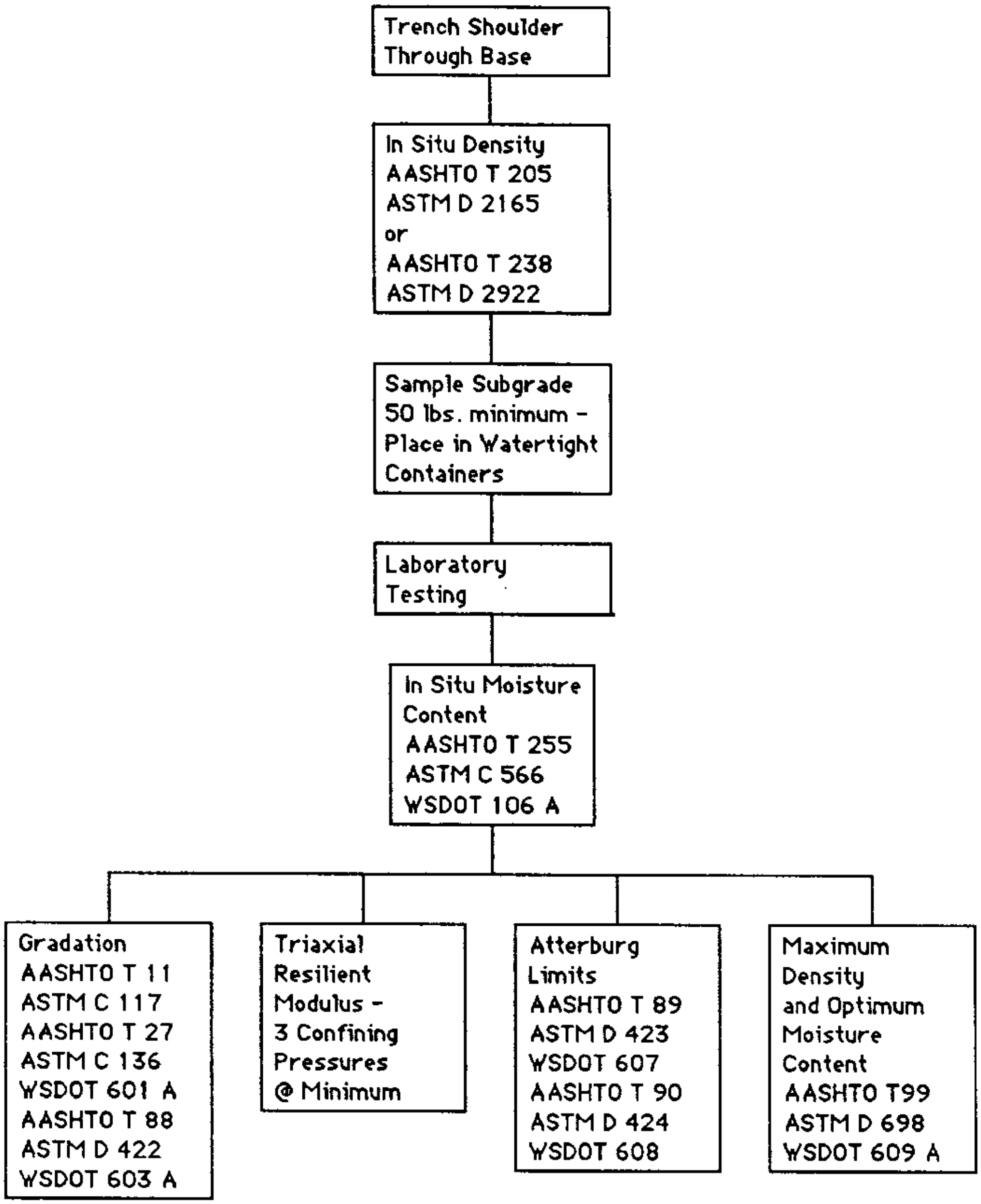

Figure 5. Subgrade Sampling and Testing

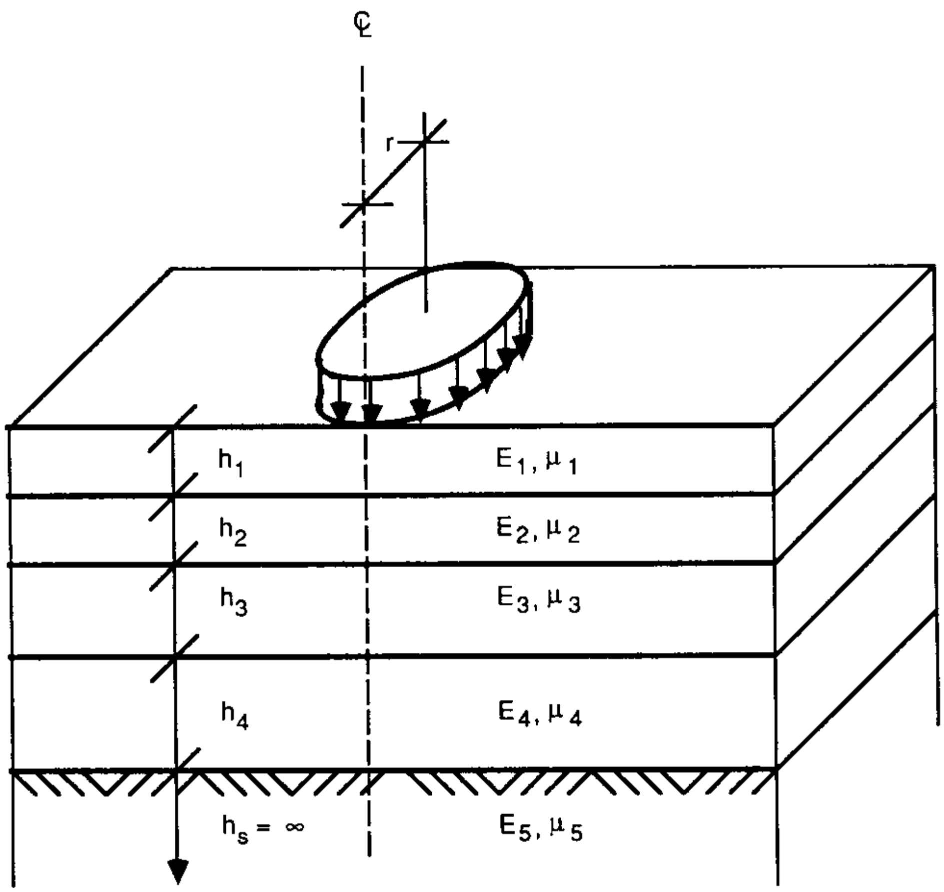

Figure 6. Multilayered Elastic System

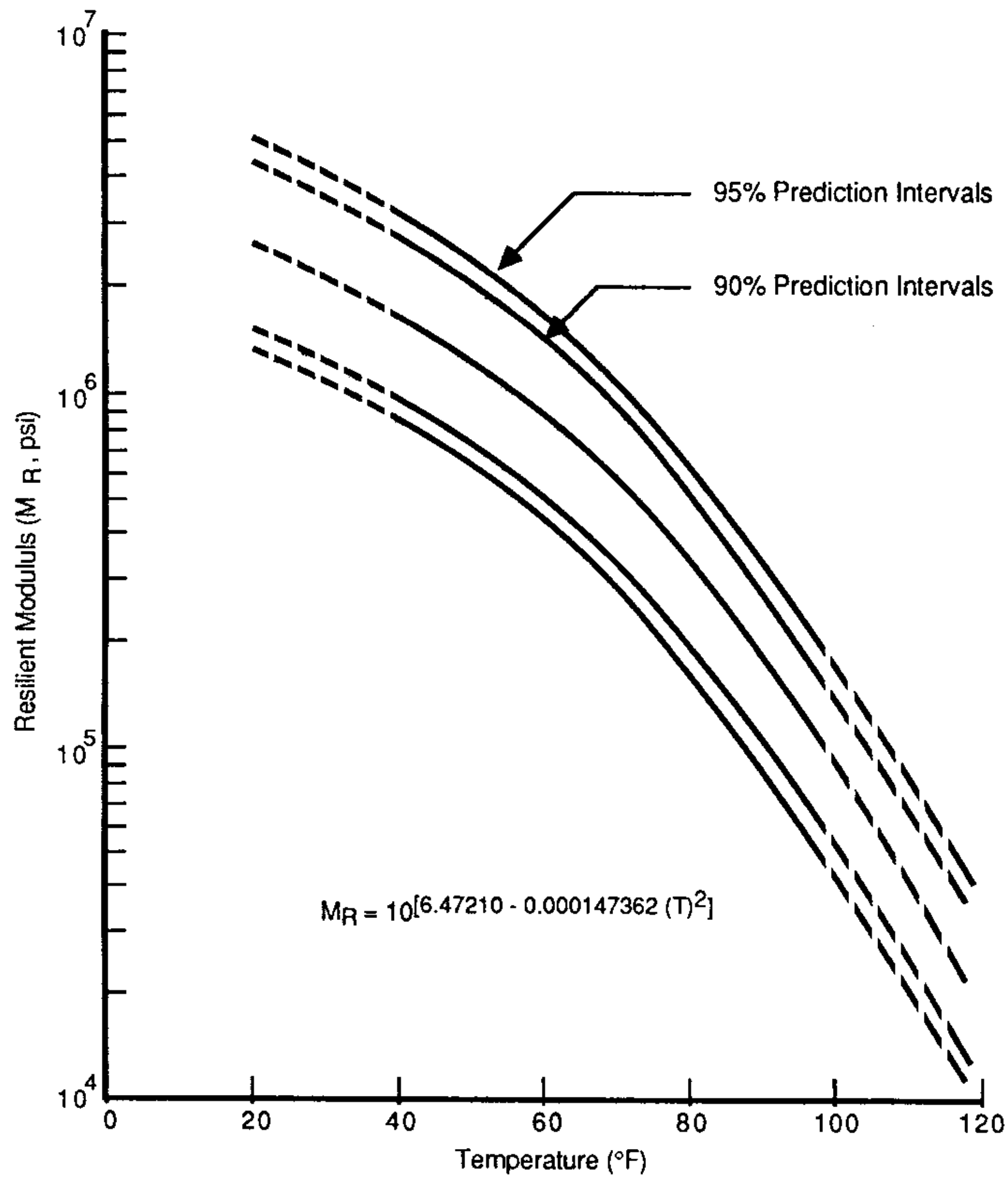

Figure 7. General Stiffness-Temperature Relationship with 90 and 95% Prediciton Intervals for Class B Asphalt Concrete in Washington State [11]

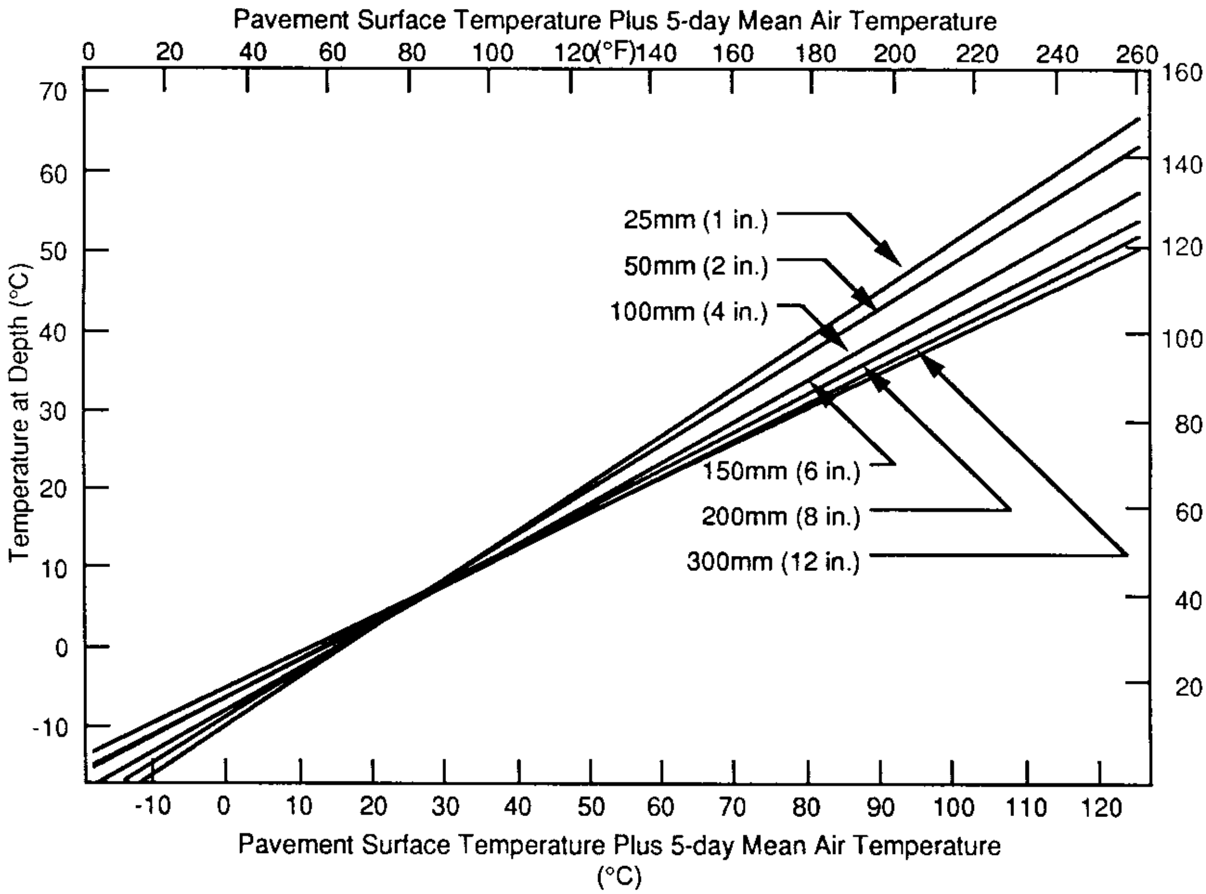

Figure 8. Estimation of Pavement Temperature
[12]

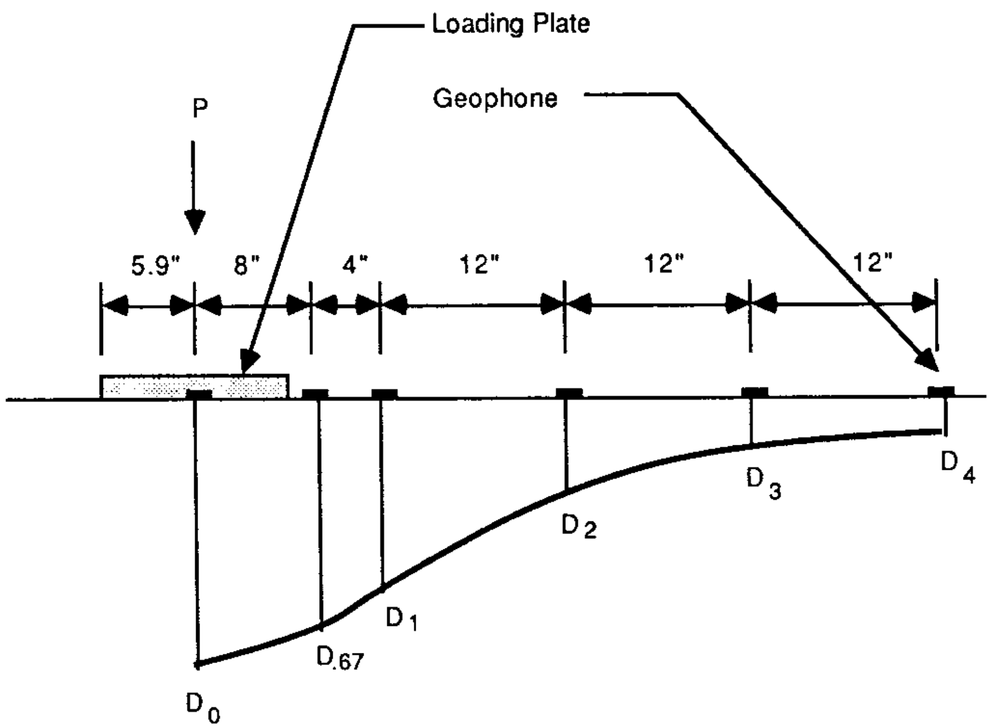

Figure 9. Present Configuration of WSDOT FWD

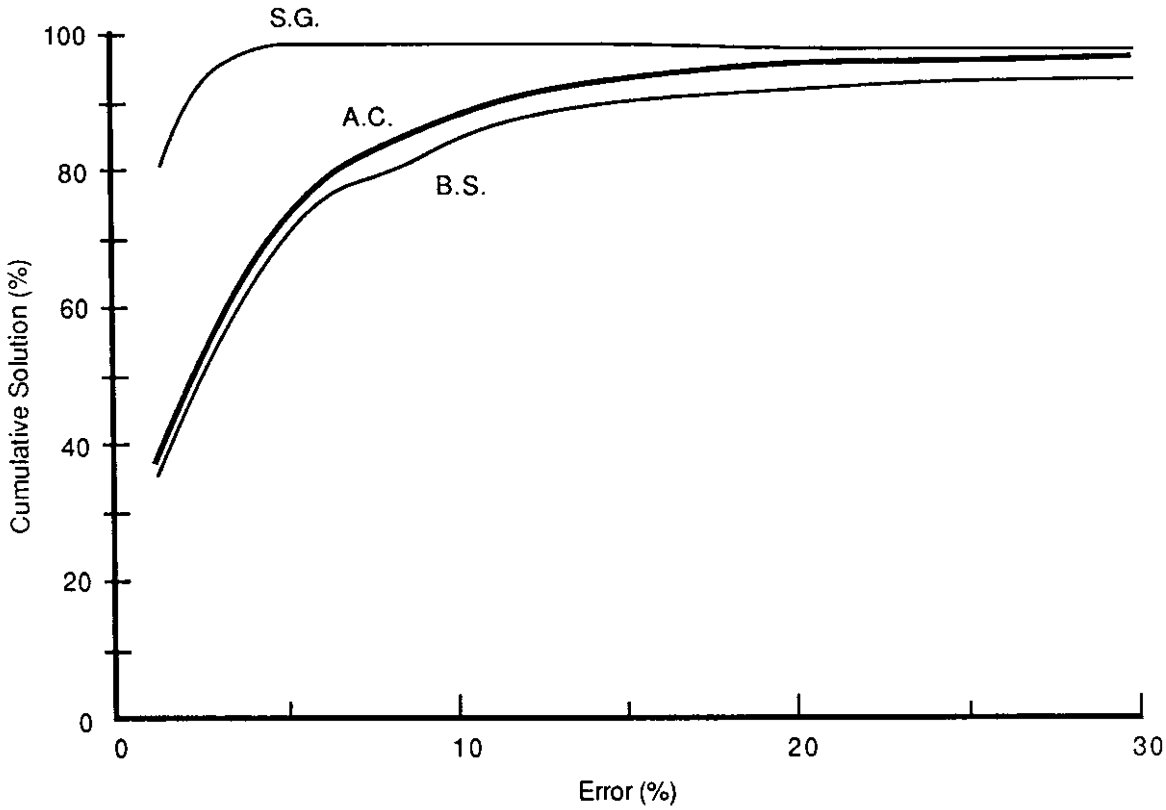

Figure 10. Accuracy of Backcalculation

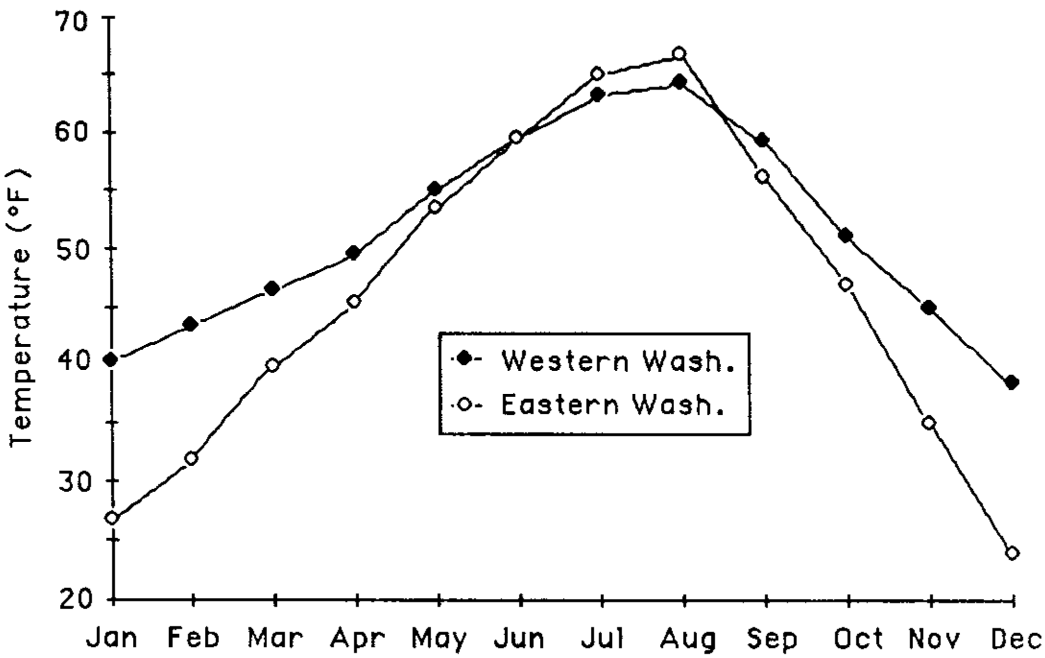

Figure 11. Monthly Average Temperature for Western and Eastern Washington [11]

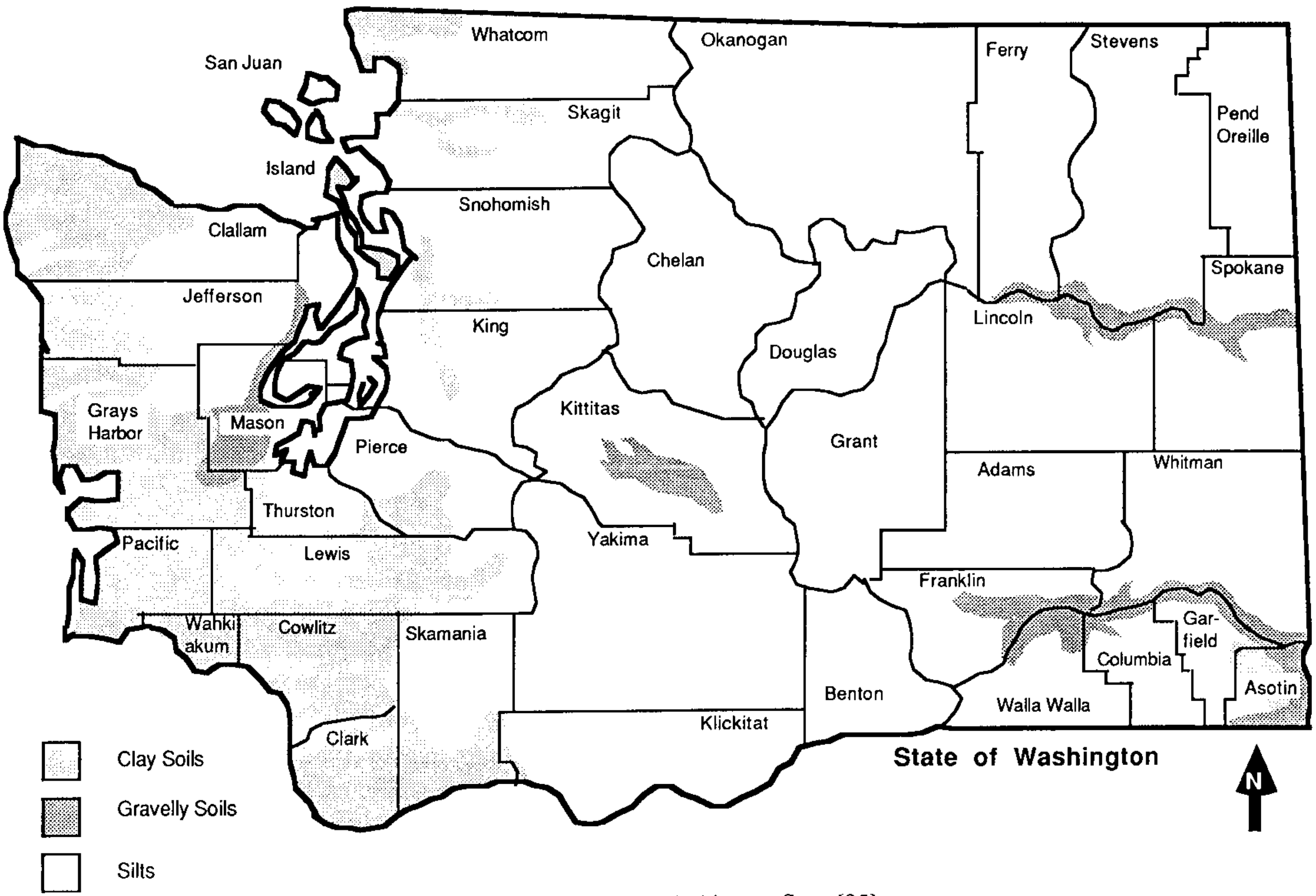

Figure 12. Soil Types in Washington State [35]

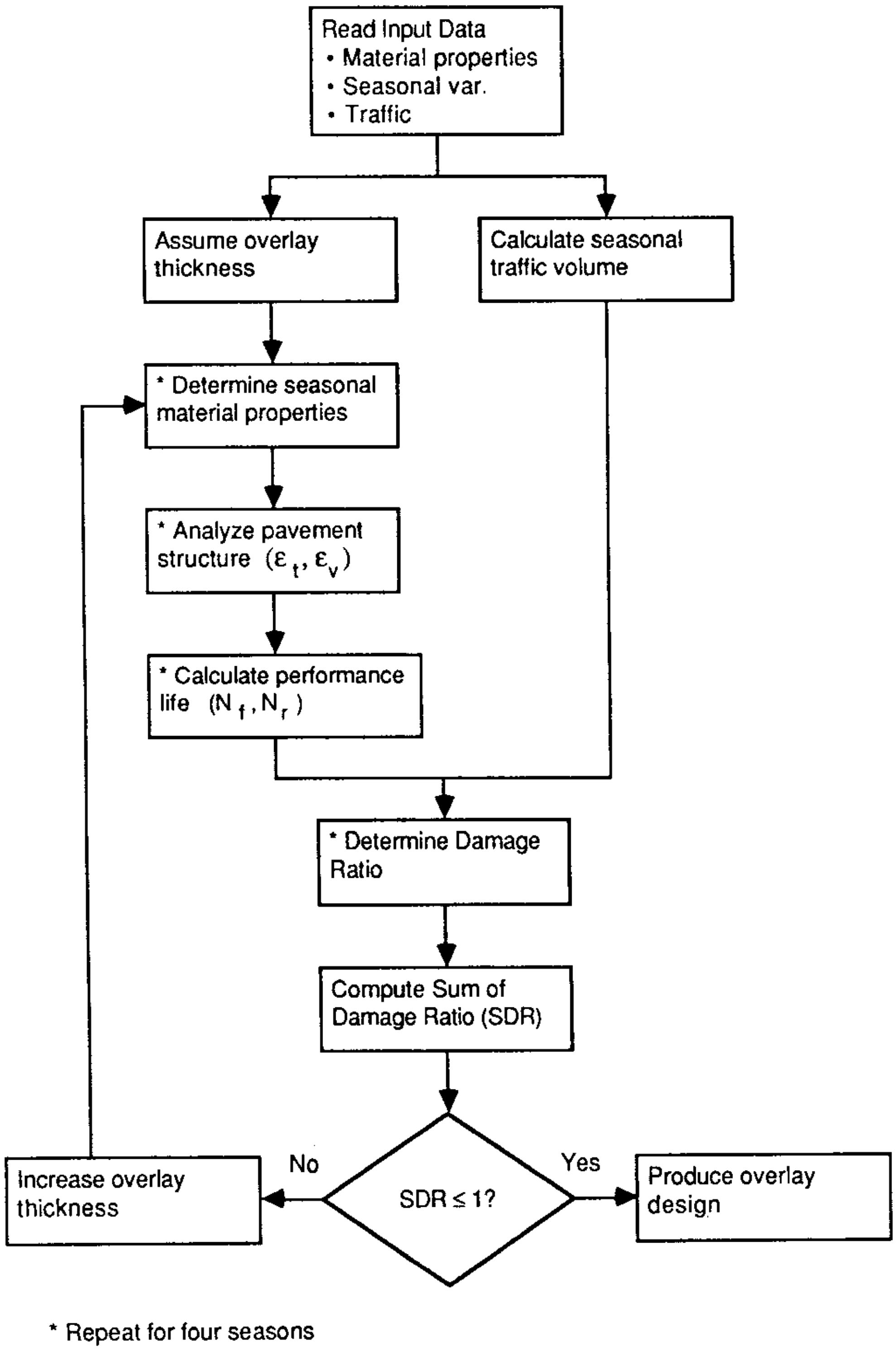

Figure 13. Overlay Design Procedure by EVERPAVE

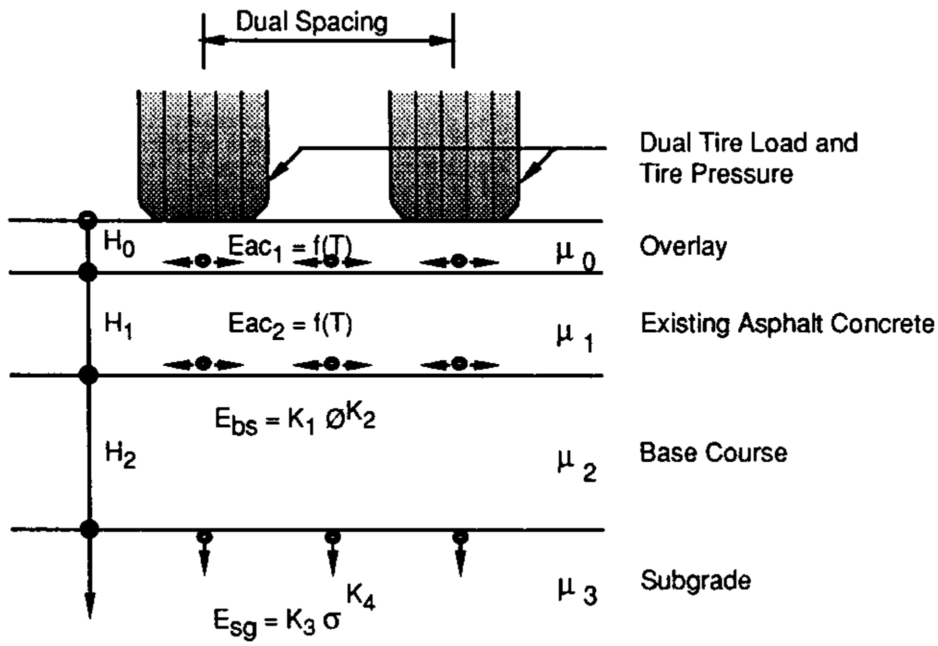

Figure 14. Pavement System for Overlay Design for Four Layers