US Department
Of Transportation

Federal Highway Administration

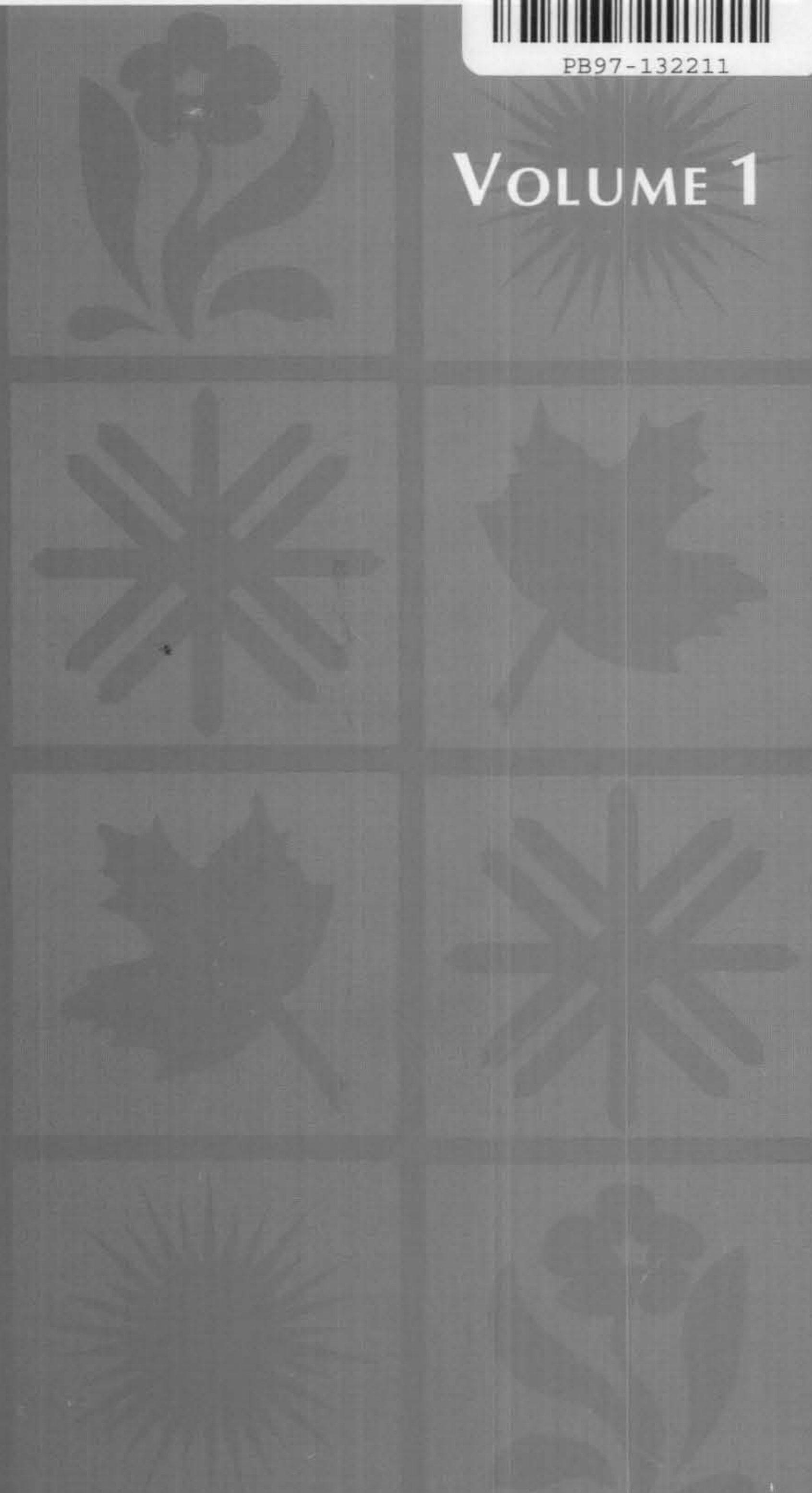
Estimation of Seasonal Effects for Pavement Design and Performance

Prepared by

United States
Department of
Agriculture

Forest Service

Volume 1



Report No. FHWA-FLP-95-006

August 1996

FOREWORD

This study was funded as a part of the Coordinated Federal Lands Highways Technology Implementation Program. It is intended to serve the immediate needs of those who design and construct Federal Lands Highways, but is also made available to all other interested parties.

This report reviews, summarizes, and updates current information on seasonal pavement material properties and responses. Such information can be used directly in various pavement design procedures—both new or reconstruction and rehabilitation.

Thomas O. Edick
Federal Lands Highway Program Administrator
Federal Highway Administration

TECHNICAL REPORT STANDARD TITLE PAGE

1. REPORT NO.
FHWA-FLP-95-006

2. GOVERNMENT
PB97-132211

3. RECIPIENT'S CATALOG NO.

4. TITLE AND SUBTITLE
ESTIMATION OF SEASONAL EFFECTS FOR PAVEMENT DESIGN AND PERFORMANCE—VOLUME I

5. REPORT DATE
August 1996

6. PERFORMING ORGANIZATION CODE

7. AUTHOR(S)
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8. PERFORMING ORGANIZATION REPORT NO.

9. PERFORMING ORGANIZATION NAME AND ADDRESS
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444 East Bonita Avenue
San Dimas, California 91773

10. WORK UNIT NO.
CTIP Study F-9

11. CONTRACT OR GRANT NO.

12. SPONSORING AGENCY NAME AND ADDRESS
U.S. Department of Transportation
Federal Highway Administration
Federal Lands Highway Programs
Washington, D.C. 20590

13. TYPE OF REPORT AND PERIOD COVERED
Final report Feb. 1960 - Feb. 1995

14. SPONSORING AGENCY CODE
HFL-23

15. SUPPLEMENTARY NOTES
This study was part of the Coordinated Federal Lands Highway Technology Implementation Program (CTIP).

16. ABSTRACT

Worldwide, pavement design and analysis is moving toward deflection or mechanistic-empirical procedures. A difficulty with these procedures is that they require some type of adjustment factor to adjust the measured deflections or the layer elastic moduli used in the procedure because of

- the time of day and year (season) when field measurements are taken, and
- the effects of climate related material variations on pavement performance.

The basic objectives of this study were to examine seasonal adjustment factors for deflections and layer moduli and to provide guidelines for selecting seasonal adjustment factors that provide a more realistic pavement design.

Three sets of deflection basin measurements (Japan, WSDOT, and U.S. Forest Service) were used to estimate seasonal moduli. The EVERCALC Version 3.3 backcalculation program was used to estimate these layer moduli. Based on such results, along with those in the literature, a set of moduli ratios were developed (see Chapter 6). Additionally, recommendations on deflection ratios were made.

17. KEY WORDS
Pavement moduli, seasonal, nondestructive testing

18. DISTRIBUTION STATEMENT
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	203	

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3

NOTE: Volumes greater than 1000 l shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

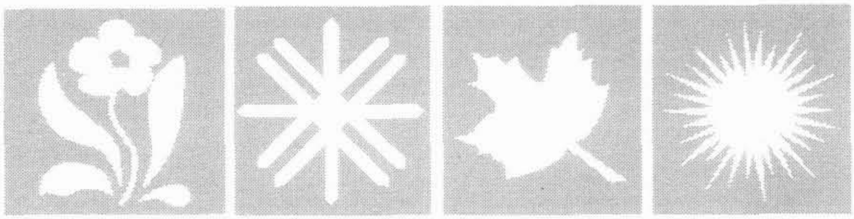
APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

Estimation of Seasonal Effects for Pavement Design and Performance



Volume 1

Report No. FHWA-FLP-95-006

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August 1996

Figure

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ACKNOWLEDGMENTS

The authors sincerely appreciate the contributions by Mr. Peter Bolander and Mr. Gary Evans of the USDA Forest Service for their advice and review, the Washington State Department of Transportation (WSDOT) and specifically Mr. Newton Jackson (retired) and Ms. Linda Pierce of the WSDOT Materials Laboratory.

Mr. Ron Porter of the Washington State Transportation Center (TRAC) coordinated the efforts of the TRAC team in producing the final report.

REPORT SUMMARY

Unlike the design and performance of structures made of concrete or steel, whose pertinent material properties such as strength and stiffness remain relatively constant for the life of the structure, pavement structures involve material properties that change seasonally. These changes are normally attributed to variation in temperature or moisture and must be considered when constructing a new pavement and evaluating an existing pavement for rehabilitation options. The problem is to correctly evaluate the potential changes in pavement material properties within the available resources.

Worldwide, pavement design and analysis is moving toward deflection or mechanistic-empirical procedures. A difficulty with these procedures is that they require some type of adjustment factor to adjust the measured deflections or the layer elastic moduli used in the procedure because of

- the time of day and year (season) when field measurements are taken, and
- the effects of climate related material variations on pavement performance.

The basic objectives of this study were to examine seasonal adjustment factors for deflections and layer moduli and to provide guidelines for selecting seasonal adjustment factors that provide a more realistic pavement design.

Three sets of deflection basins (Japan, WSDOT, and U.S. Forest Service) were used to estimate seasonal moduli. The EVERCALC Version 3.3 backcalculation program was used to estimate these layer moduli. Based on such results, along with those in the literature, a set of moduli ratios were developed (see Chapter 6). Additionally, recommendations on deflection ratios were made.

Several design procedures are described along with how each accommodates seasonal effects. A special emphasis is placed on describing how agencies have designed for frost action.

CHAPTER 1

INTRODUCTION

1. THE PROBLEM

Unlike the design of structures made of concrete or steel, whose pertinent material properties such as strength and stiffness remain relatively constant for the life of the structure, pavement design involves material properties that change frequently during the life of the pavement. These changes are normally attributed to variation in temperature or moisture and should be considered both when a new pavement is constructed, and when an existing pavement is evaluated for rehabilitation options.

Worldwide, pavement design and analysis is moving toward deflection or mechanistic-empirical procedures. A principal difficulty with these procedures is that they require some type of adjustment factor (to adjust the measured deflections or the layer elastic moduli used in the procedure) because of

- the time of day and year (season) when field measurements are taken, and
- the effects of climatic related material variations on pavement performance.

The consequences of not adjusting deflections or layer moduli for such changes can be either under- or overdesigned pavements.

2. BACKGROUND

Numerous Federal Lands Highway projects involve road reconstruction, which often constitutes upgrading from an aggregate surface or bituminous surface treatment (BST) to an asphalt concrete (AC) surface. Common design procedures for the upgrading require either surface deflections or elastic moduli for pavement layers. Often, some type of nondestructive testing (NDT) device, such as a Dynaflect, Road Rater, Falling Weight Deflectometer, or Benkelman Beam, is used to obtain surface deflections. If a deflection basin (several pavement surface measurements taken at the same time and load level) is measured, this information can be used to estimate layer moduli. However, such moduli or surface deflections are generally a function of the time of day and time of year the nondestructive testing takes place.

Table 1.1 lists the input parameters for several common new pavement and overlay design procedures. The specific inputs for each design procedure vary depending on whether the design is for an asphalt concrete overlay or for a new pavement. In most cases the input deflections or layer moduli need adjustment for seasonal variation to trulv characterize the pavement system.

Table 1.1. Design Inputs for Common New Pavement and Overlay Design Procedures

Design Procedure	Modulus Input
AASHTO Guide for Design of Pavement Structures [1]	Subgrade Modulus by Season, Layer Coefficients for Base and Surfacing[1]
Asphalt Institute MS-1 [2]	Subgrade Modulus
Shell Method [3]	Subgrade Modulus, Base Modulus Asphalt Concrete Modulus
Washington State Department of Transportation Mechanistic-Empirical [4]	Subgrade Modulus, Base Modulus Asphalt Concrete Modulus
Asphalt Institute MS-17 [5]	
Deflection Based	Benkelman Beam Maximum Deflection
Effective Thickness	Subgrade Modulus
AASHTO Guide for Design of Pavement Structures — Part III (Pavement Design Procedure for Rehabilitation of Existing Pavements), NDT Method 2 [6]	Subgrade Modulus

Note: 1. AASHTO layer coefficients can be a function of elastic modulus (as well as other test methods such as CBR, R-value, etc.)

The design procedures in Table 1.1 comprise empirical and mechanistic-empirical design approaches. [1, 2, 3, 4, 5] An empirical design is based on the results of experiment or experience. Generally, it requires that a number of observations be made to ascertain the relationships between the variables and the outcomes of results. Firmly establishing the scientific basis for the relationships is not necessary as long as the design limitations are recognized. In some procedures, relying on experience is much more expedient than trying to quantify the exact cause and effect of certain phenomena. The design procedures used in the past were mostly empirical in that their failure criteria were based on a given set of conditions, i.e., traffic, materials, layer configurations, and environment.

The use of mechanistic-empirical approaches for pavement design is increasing as more highway agencies become familiar with methods for determining layer moduli, either by modulus testing or nondestructive techniques. A mechanistic-empirical approach to pavement design incorporates elements of both approaches. The mechanistic component is the determination of stresses, strains, and deflections within the pavement layers to load(s) through the use of mechanical mathematical models. The empirical portion relates these pavement responses to the performance of the pavement structure. For instance, it is possible to calculate the amount of deflection at the surface of the pavement through elastic analysis. If these deflections are related to the life of the pavement, then an empirical relationship can be established between the mechanistic response of the pavement and the number of loads to failure (performance). The basic advantage of a mechanistic-empirical pavement design is that seasonal variations in temperature, freezing and thawing, and moisture effects can be incorporated directly into the design procedure.

3. RESEARCH OBJECTIVES AND METHODOLOGY

The basic objectives of this study were to examine seasonal adjustment factors for deflections and layer moduli and to provide guidelines for selecting seasonal adjustment factors that provide a more realistic pavement design.

This report is organized into six chapters as follows:

CHAPTER 2 — LITERATURE REVIEW

This section discusses how pavements are characterized for seasonal design. Material characterization for pavement layers are explored. Methods and procedures for determining material properties from laboratory and nondestructive testing are described. Finally, a discussion of pavement design procedures reveals the different assumptions for including seasonal effects. Various procedures are reviewed and their assumptions are explained.

CHAPTER 3 — EXAMINATION OF NDT DATA FOR SEASONAL VARIATION IN JAPAN

This case study provides an example of the use of nondestructive data to determine seasonal variation. Seasonal factors from the data obtained from this Japanese study are developed.

CHAPTER 4 — EXAMINATION OF NDT DATA FOR WASHINGTON STATE DOT TEST SITES

Sixteen test sites from over a wide area of Washington state were investigated to determine seasonal variation for layer moduli. This chapter develops seasonal factors for base course and subgrade layer moduli.

The latter portion of Chapter 4 overviews the stress sensitivity response of subgrade and base course materials from several Washington State DOT test sites. Changes observed with stress sensitivity and variation of subgrade and base course moduli with season are compared.

CHAPTER 5 — EXAMINATION OF NDT DATA FOR U.S. FOREST SERVICE TEST SITES

Fifteen test sites located in the Olympic National Forest were investigated to determine seasonal variation for layer moduli. These are summarized and compared to those developed in Chapter 4. Five of these sections are aggregate surfaced and ten are asphalt surfaced.

CHAPTER 6 — SUMMARY AND CONCLUSIONS

Appropriate conclusions and recommendations are made which include a final set of seasonal moduli ratios for aggregate and subgrade layers.

CHAPTER 2

LITERATURE REVIEW

1. INTRODUCTION

Seasonal pavement design requires consideration of the environmental factors that contribute to the deterioration of pavements. By designing for the seasonal changes of temperature, freeze/thaw, and moisture within pavement layers, deterioration can be minimized and enhanced pavement performance will result.

Unfortunately the seasonal variation of pavements is a factor that is not easily expressed in a straightforward pavement design equation. The difficulty lies in that pavements are multilayered systems made up of asphalt, base and subgrade materials. Each layer interacts with the environment. Primarily, asphalt layers respond to climatic changes in temperature while base and subgrade materials respond to changes in moisture conditions.

The seasonal changes that may occur in any geographic location include changes with wet and dry, or warm and cold conditions. The structural response of a pavement often corresponds to these climatic changes. Typically, the stiffness (or strength) of base and subgrade layers is higher during the dryer periods. In contrast during periods of spring thaw the moisture content increases and the base and subgrade stiffness decreases. Asphalt concrete responds to changes in temperature with the colder temperatures providing higher stiffness.

To design for seasonal variations, pavement designers require seasonal strength or pertinent material properties. Most road-owning agencies have limited resources for measuring seasonal material properties. Often, only one measurement for one season of the year is available. To predict stiffness for specific wet and dry periods, seasonal adjustments must be made. The following sections discuss this adjustment process both for current practice and application to design procedures.

Once appropriate stiffness values are determined, one of numerous pavement design procedures can be used. Each design procedure is based on varying assumptions with respect to climatic effects. The final portion of this literature review examines several design procedures and notes how seasonal effects are handled.

2. MATERIAL CHARACTERIZATION

Pavement design procedures require some type of stiffness or other structural value to characterize pavement materials. Agencies across the United States have adopted different parameters for use in design procedures. The three most common are elastic moduli, the California Bearing Ratio (CBR), and Resistance (R-Value). Following is a brief description of each structural value and how each is measured.

2.1 ELASTIC MODULI

Pavement research by Hveem [7] found that pavement performance is strongly dependent upon the pavements deflection under wheel loads. The accumulation of strain cycles induced by deflection was recognized as the cause of fatigue cracking. Engineering mechanics provides that deflection for any structure whether a pavement or a steel bridge is a result of two factors. As load is applied these factors are a structure's geometry, and the material elastic properties. [8] For a pavement, the geometry relates to the thickness of each layer and the elastic properties relate to the modulus of elasticity and Poisson's ratio of individual layer.

The modulus of elasticity is sometimes called Young's modulus since Thomas Young published the concept of elastic modulus in 1807. The modulus of elasticity is defined by the following equation:

$$E = \frac{\sigma}{\varepsilon} \tag{1}$$

where E = modulus of elasticity,

$\sigma = \frac{P}{A}$ = applied stress,

P = applied load,

A = cross sectional area of the sample,

$\varepsilon = \frac{\Delta L}{L}$ = axial strain,

L = gauge length over which the sample deforms, and

ΔL = change in sample length due to applied load.

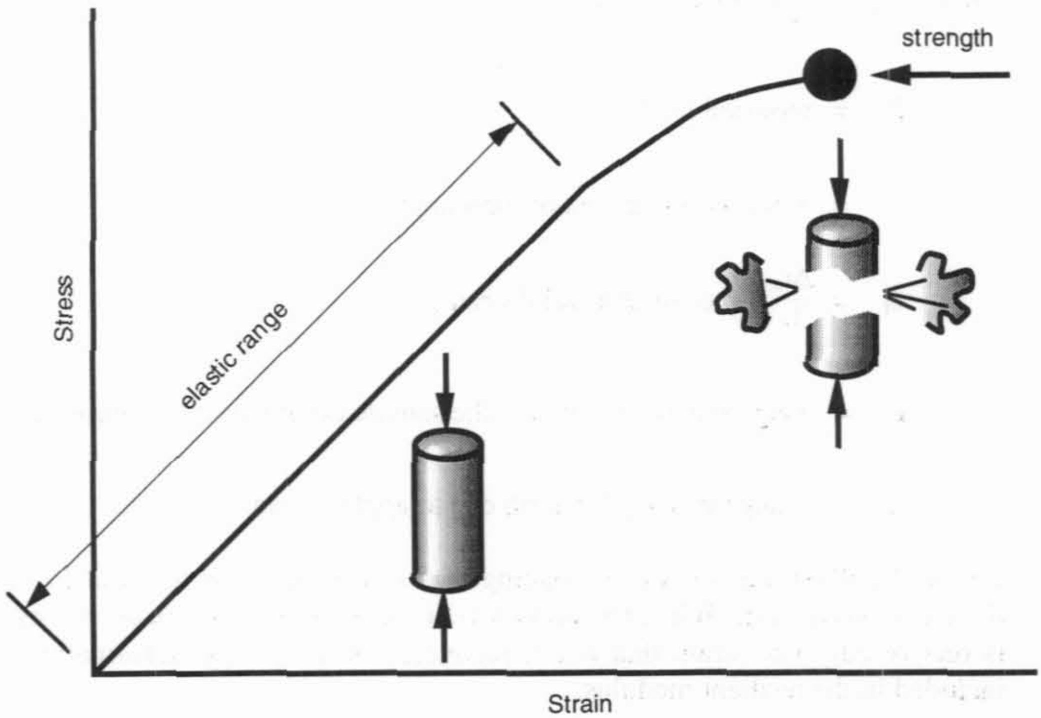


Figure 2.1. Sketch of Stress vs. Strain of a Material in Compression [9]

Modulus of elasticity is measured under laboratory conditions where strain is induced by slowly increasing the stress. For a material in compression, shown in Figure 2.1, the modulus of elasticity is the slope of the linear portion of the stress strain relationship. Modulus of elasticity is often referred to as the "stiffness" of a material.

A distinction needs to be made concerning the strength and stiffness of materials. Strength is defined as the stress needed to break something while stiffness can be measured by the modulus of elasticity.

Resilient modulus is a term that is often confused with the Young's modulus of elasticity. Where Young's modulus is determined by a slowly applied load to a laboratory specimen, resilient modulus is determined by rapidly repeated loads in a triaxial test (at least for most unstabilized pavement materials). The triaxial test, to some degree, resembles the wheel loading of a tire on a pavement. Resilient modulus is based only on the recoverable portion of strain and is defined as:

$$M_R \text{ (or } E_R) = \frac{\sigma_d}{\varepsilon_r} \qquad (2)$$

M_R (or E_R) = resilient modulus,

where $\sigma_d = \frac{P}{A}$ = deviator stress,

P = repeated load,

A = cross sectional area of the sample,

$\varepsilon_r = \frac{\Delta L}{L}$ = recoverable axial strain,

L = gauge length over which the sample deformation is measured, and

ΔL = change in sample length due to applied load.

Figure 2.2 illustrates the cyclic loading of a soil or granular material for a specimen during a triaxial test. When the stress level is decreased the strain decreases but not all is recovered. The strain that is not recovered is the plastic deformation and is not included in the resilient modulus.

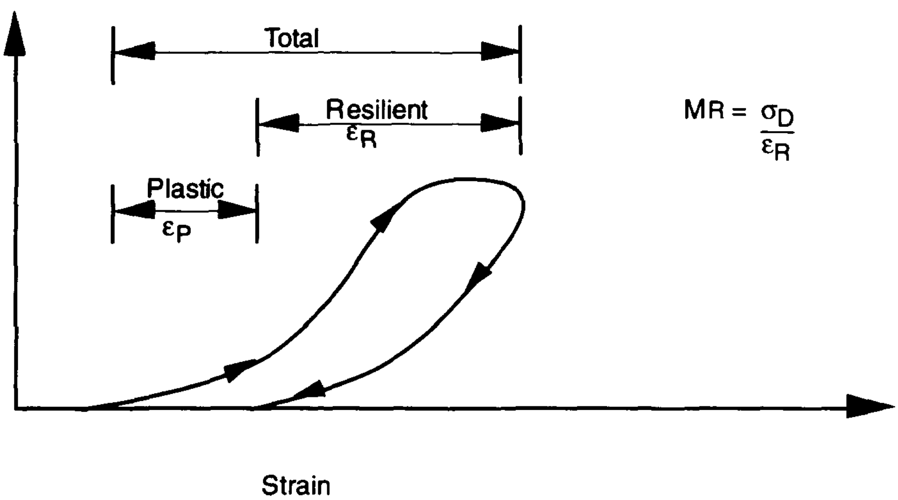


Figure 2.2. Typical Load Response in the Resilient Modulus Test (after Elliott and Thornton [71])

Another important material property is Poisson's ratio. Poisson's ratio can be thought as the ratio of transverse to longitudinal strains of a loaded specimen. When a compressive force is applied to a cylindrical specimens, the material will tend to expand in the direction where no force is applied. This concept is illustrated in Figure 2.3. Generally, "stiffer" materials will have lower Poisson's ratios than "softer" materials. A stiff material such as portland cement concrete has a Poisson's ratio of 0.15 to 0.20 while a soft material such as rubber has a Poisson's ratio of 0.5. [8] Poisson's ratios larger than 0.5 are reported, however, this implies that the material was stressed to cracking or there was experimental error. [9] Typical Poisson's ratios used by Washington State Department of Transportation (WSDOT), Shell, and AASHTO design procedures are reported in Table 2.1.

For this report the nomenclature and symbols from the 1993 *AASHTO Guide For Design of Pavement Structures* [1] (AASHTO Guide) will be used in referring to pavement moduli. For example:

(a) E_{ac} = asphalt concrete elastic modulus

(b) E_{bs} = base course resilient modulus

(c) E_{sb} = subbase course resilient modulus

(d) M_r (or E_{sg}) = roadbed soil (subgrade) resilient modulus

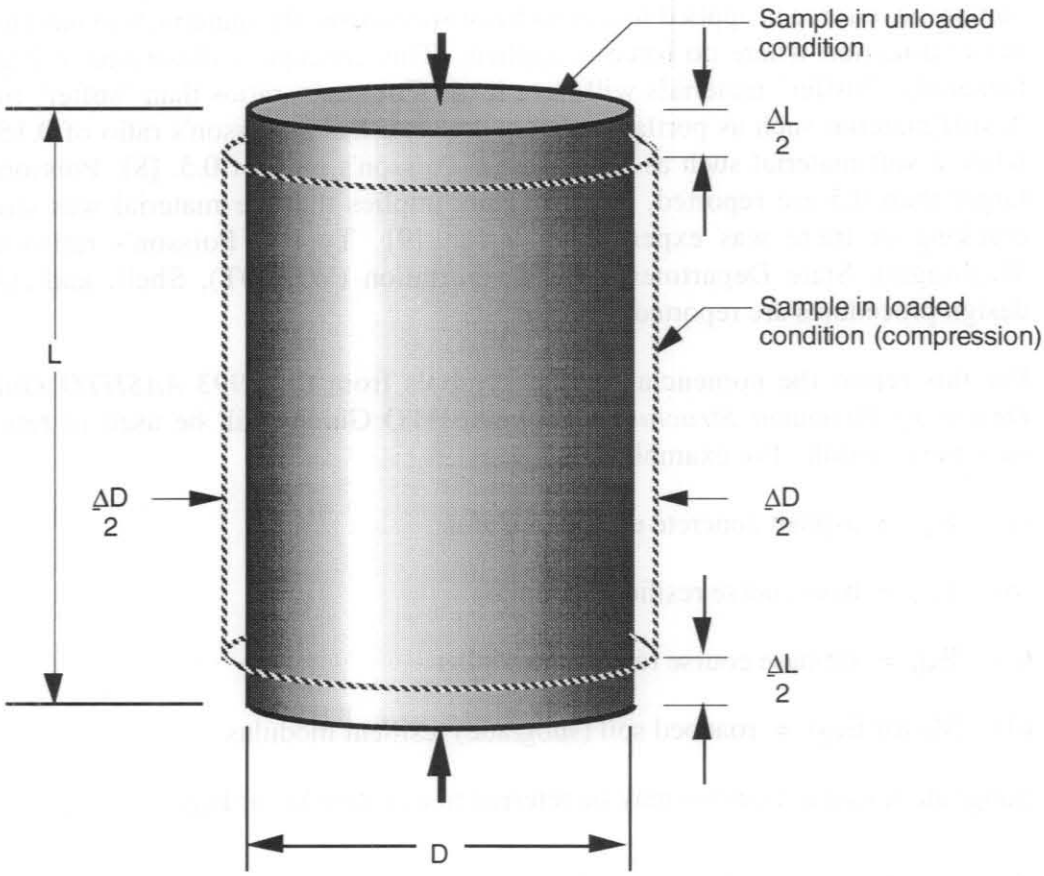
Subgrade resilient modulus may be referred to as either M_r or E_{sg}.

2.1.1 ASPHALT CONCRETE ELASTIC MODULUS

The behavior of asphalt concrete materials depends on temperature and load duration. [10] At low temperatures and short load durations, such as a moving wheel load, asphalt materials act in an elastic manner with lower temperature providing higher elastic modulus. At higher temperatures and longer load duration asphalt materials act viscoelastically and the elastic modulus decreases.

Table 2.1. Typical Poisson's Ratios

	Design Procedure		
Material	WSDOT	AASHTO	Shell
Asphalt	0.35	0.35	0.35
Base	0.40	0.35	0.35
Subgrade	0.45	0.40	0.35



$$\mu = -\frac{\varepsilon_D}{\varepsilon_L}$$

Where

$\mu =$ Poisson's ratio

$\varepsilon_D = \dfrac{\Delta D}{D}$ = strain along the diametrical (horizontal) axis

$\varepsilon_L = \dfrac{\Delta L}{L}$ = strain along the longitudinal (vertical) axis

Figure 2.3. Illustration of Poisson's Ratio [9]

To quantify the temperature dependency on WSDOT Class B asphalt concrete (a traditional, dense mix), Bu-bushait [11] derived a relationship for the elastic modulus (E_{ac}) as a function of temperature. This relationship was developed by testing the resilient modulus of samples of Class B asphalt concrete obtained from test sites located throughout Washington state. The relationship found by combining sites is described by the equation

$$E_{ac} = 10^{[6.47210 - 0.000147362\,(T)^2]} \tag{3}$$

where E_{ac} = asphalt concrete resilient modulus, psi, and

T = temperature in degrees Fahrenheit (°F).

Figure 2.4 illustrates this dependency of asphalt modulus on temperature.

To account for both loading and temperature effects, work by Van der Poel [12], Heukelom [13], and Heukelom and Klomp [14] can be used to predict asphalt concrete elastic modulus.

The Asphalt Institute also developed a method to estimate asphalt concrete elastic modulus. Kallas and Shook [15] began work based on laboratory test results which led to a regression equation [16, 17] that is used to predict asphalt modulus. The equation is a function of numerous parameters and is expressed as:

$$|E^*| = f\,(P_{200}, f, V_v, \eta_{70^\circ F}, T, P_{ac}) \tag{4}$$

where $|E^*|$ = dynamic modulus (stiffness of asphalt concrete), psi,
P_{200} = percent aggregate passing No. 200 sieve,
f = frequency of loading,
V_v = percent air voids,
$\eta_{70^\circ F}$ = original absolute viscosity used in mix at 70° F,
T = temperature, and
P_{ac} = asphalt content, by weight of mix.

For comparison, the asphalt concrete elastic modulus for a WSDOT Class B asphalt concrete [18] was computed based on the WSDOT and Asphalt Institute procedures to predict mix stiffness. The temperatures compared were 40, 70, and 100 °F. Table 2.2 is used to display the results.

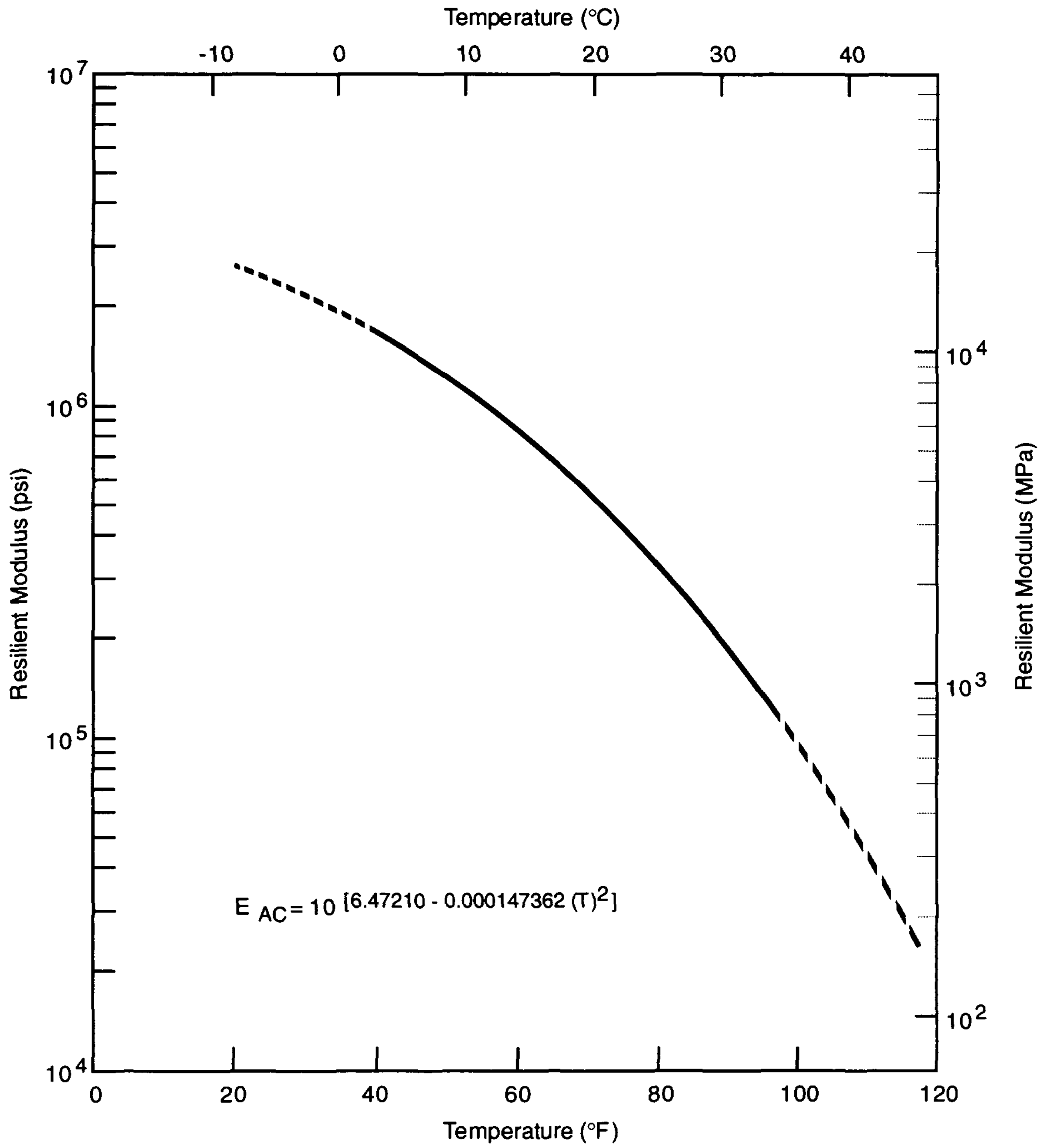


Figure 2.4. General Stiffness-Temperature Relationship for Class B (Dense Graded) Asphalt Concrete in Washington State [9]

Table 2.2. Comparison of Asphalt Modulus Computed by WSDOT and Asphalt Institute Equations

Mean Pavement Temperature °F	Estimated Asphalt Modulus (E_{ac})		
	WSDOT	Asphalt Institute	
	Load Time 100 ms (10 Hz) (ksi)	Load Time 100 ms (10 Hz) (ksi)	Load Time 30 ms (33 Hz) (ksi)
40	1723.1	1275.5	1412.6
70	562.4	475.9	650.2
100	99.7	117.7	190.3

Notes: 1. Determined from WSDOT's stiffness-temperature relationship [11]

2. WSDOT class B mix parameters [19]:
P_{200} = 5% = percent passing the No. 200 sieve
V_v = 7% = percent air voids
$\eta_{70°F} = 10^6$ poises= original absolute viscosity used in the mix at 70 °F
P_{ac} = 5% = asphalt content, by weight of mix

2.1.2 UNBOUND MATERIALS RESILIENT MODULUS (BASE, SUBBASE, AND SUBGRADE)

To quantify the resilient behavior of granular and fine grained materials by changes in dry density, gradation, plasticity, permeability, moisture content, degree of saturation, stress level, and static properties such as shear strength and cohesion is difficult. Combinations of different parameters have unique results on moduli and to separate the effects of each is a large task. Lee [19] noted these properties are all intermingled and it is difficult to delineate the moduli in a simple equation.

One approach used in pavement design to account for moduli variation in unbound materials is to express the material variation in terms of stress sensitivity. A number of studies [1, 9, 10, 11, 19] have characterized moduli by stress sensitivity and it is for this reason this method is pursued.

Much research was performed during the 1960's by researchers [20, 21] at the University of California at Berkeley who explored the stress sensitivity nature of unbound materials. Findings showed that when granular or fine grained samples were placed in a repeated-load triaxial test device and subjected to various confining pressures and deviator stresses, the resulting resilient moduli were found to be a function of the applied stress state. The relationships for granular and fine grained materials follow.

2.1.2.1 Granular Materials

Granular materials of a flexible pavement are confined between the asphalt and subgrade layers. These materials in a confined state develop interparticle friction with increased loading. The increased interparticle friction increases the resilient modulus. Other factors affecting the resilient modulus response include, degree of saturation, gradation, and dry density. [22, 23] Granular materials are often modeled as follows [10]:

$$E_{bs} = K_1 \theta^{K_2} \text{ for coarse grained soils} \tag{5}$$

where E_{bs} = resilient modulus of coarse grain soils,
θ = bulk stress (sum of principal stresses, $(\sigma_1 + \sigma_2 + \sigma_3)$), and
K_1, K_2 = regression coefficients.

This stress sensitivity relationship is illustrated in Figure 2.5 and typical K_1 and K_2 values are shown in Tables 2.3 and 2.5.

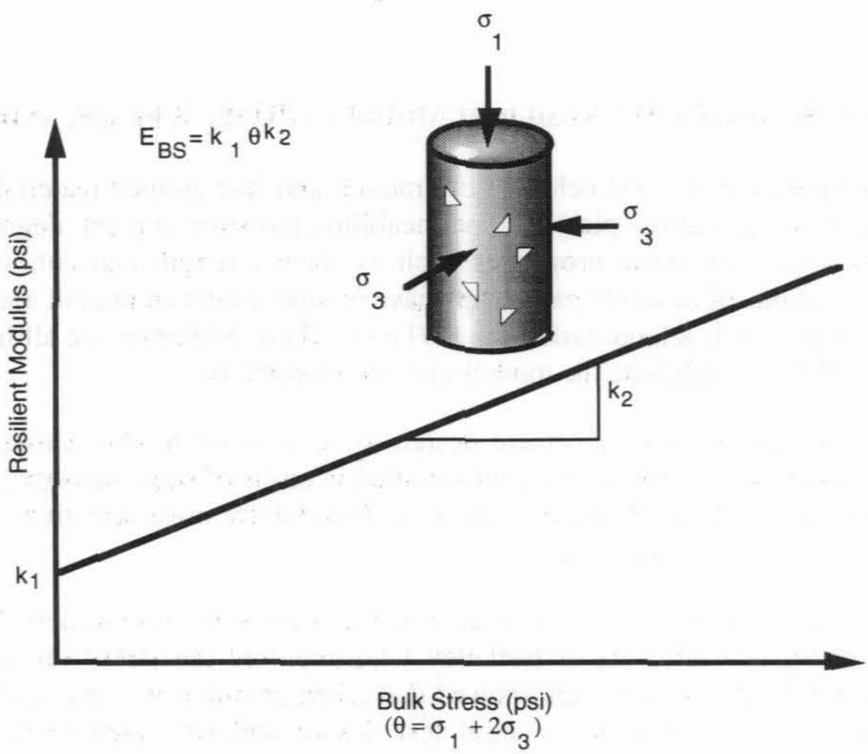


Figure 2.5. Resilient Modulus vs. Bulk Stress for Unstabilized Coarse Grained Materials [9]

Table 2.3. Summary of Repeated Load Triaxial Compression Laboratory Test Data for Untreated Granular Materials

Investigator	Material(s)	Regression Constants[3] K_1	Regression Constants[3] K_2	Moduli Range[4] (ksi)
Hicks[1]	Partially crushed gravel, crushed rock	1600 - 5000	.57 - .73	10.0-52.4
Hicks/Finn[1]	Untreated base	2100 - 5400	.61	15.0-38.5
Allen[1]	Gravel, crushed stone	1800 - 8000	.32 - .70	5.0-51.4
Kalcheff/Hicks[1]	Crushed Stone	4000 - 9000	.46 - .64	17.6-70.6
Boyce/Brown/Pell [1]	Well graded crushed limestone	8000	.67	69.1
UC Berkeley [1]	Base and subbase material	2900 - 7750	.46 - .65	12.7-62.8
Rada/Witczak [23]	Silty sands, sand gravel, sand-aggregate blend, crushed stone, limerock, slag	9240	.53	50.9
Mahoney [9]	Crushed rock	8500	.38	28.9
AASHO Road Test [1]	Unbound materials - base:			
	Dry	6000 - 10000	.50 - .70	30.0-95.1
	Damp	4000 - 6000	.50 - .70	20.0-57.1
	Wet	2000 - 4000	.50 - .70	10.0-38.1
AASHO Road Test [1]	Unbound materials — subbase:			
	Dry	6000 - 8000	.40 - .60	21.7-55.2
	Damp	4000 - 6000	.40 - .60	14.5-41.4
	Wet	1500 - 4000	.40 - .60	5.4-27.6
Thompson[2]	Wide range of granular materials	1620-7210	.45 - .62	6.9-53.1

Notes: [1]After Shook et al. [17]

[2]After Siddharthan [24]

[3]Results in moduli with psi units.

[4]Used $\theta = 25$ psi

Table 2.4. Comparison of Dry and Wet Resilient Moduli for Granular Base Course Materials

Material	Modulus E_{bs}								
	Bulk Stress = 20 psi			Bulk Stress = 30 psi			Bulk Stress = 40 psi		
	Dry (ksi)	Wet (ksi)	Reduction (percent)	Dry (ksi)	Wet (ksi)	Reduction (percent)	Dry (ksi)	Wet (ksi)	Reduction (percent)
Dense Graded Aggregate[1] (Limestone)	47	30	36	57.5	35	39	66.5	39.5	41
Crushed Rock[1] (Slag)	57	25.5	55	66	29.5	55	72.5	32.5	55
Sand Aggregate Blend[1]	27	24.5	9	33	32.5	2	38	39.5	—
Bank Run Gravel[1]	26.5	16.5	38	31	21.5	31	35	26.5	24
Crushed Aggregate [2]	18.1	15.1	17	24.1	19.9	17	29.6	24.1	19

Notes: [1]After Rada/Witczak [23]
Dry condition is defined as a moisture content < 60% of saturation.
For the wet condition saturation moisture content > 85 % saturation.

[2]After Hicks [22]
Based on dry and partially saturated specimens.

Table 2.5. Summary of K_1 and K_2 Statistics for 271 Samples Grouped by Aggregate Class (after Rada and Witczak [23])

Aggregate Class	No. of Data Points	Mean K_1	Standard Deviation	Range K_1	Mean K_2	Standard Deviation	Range K_2
Silty sands	8	1620	780	710-3830	0.62	0.13	0.36 - 0.80
Sand gravel	37	4480	4300	860 - 12840	0.53	0.17	0.24 - 0.80
Sand-aggregate blends	78	4350	2630	1880 - 11070	0.59	0.13	0.23 - 0.82
Crushed stone	115	7210	7490	1705 - 56670	0.45	0.23	-0.26 - 0.86
Limerock	13	14030	10240	5700 - 83860	0.40	0.11	0.00 - 0.54
Slag	20	24250	19910	9300 - 92360	0.37	0.13	0.00 - 0.52
All data	271	9240	11225	710 - 92360	0.52	0.17	-0.16 - 0.86

Note: All K_1 and K_2 values result in moduli with units in psi.

Numerous researchers have performed resilient modulus tests to investigate K_1 and K_2 values for granular materials. Table 2.3 summarizes many of the results found in the literature. The values are for various types of granular materials including crushed stone, gravels, sands and sand-aggregate mixes. For comparison of the different K_1 and K_2 values, the moduli shown in Table 2.3 are calculated for a bulk stress level of 25 psi. The 25 psi value corresponds approximately to the bulk stress that may result in a base layer for a 18,000 pound equivalent axle load (typical for a pavement with a six to eight inch base and three to four inches of asphalt concrete).

Saturation level plays an influential role in the resilient moduli of granular materials. With increased moisture the aggregate to aggregate contact is lubricated which increases slippage and deformation with load [25] resulting in a reduced resilient modulus. In a study by Maree et al. [26], an increase in pavement deflection was noted when the base materials were saturated with water. Table 2.4 shows results from Rada and Witczak [23] and Hicks [22] for both dry and saturated materials at bulk stress levels of 20, 30, and 40 psi. The degree of saturation reduced the resilient modulus response anywhere from 2 to 55 percent when compared to dry conditions. Results of studies by Shifley, Kallas and Riley, and Hicks, as summarized by Chou [10] seem to indicate K_1 values decrease and K_2 values remain relatively constant with increases in saturation level.

Also noted in Table 2.4 is an illustration of the significance of stress level in determining the resilient modulus. For the crushed aggregate the resilient modulus for the dry material increased from 18,100 to 29,600 psi by increasing the confining, or bulk stress, from 20 to 40 psi. The wet material increased from 15,100 to 24,100 psi for the same stress range. In the extreme case, when granular layers are confined between two very stiff layers and subjected to very high confining pressures resilient moduli in excess of 100,000 psi have been reported by Maree et al. [27]

In a study by Rada and Witczak [23], 271 individual resilient modulus tests from 11 state, federal and private agencies were collected and compared. When the aggregates were classified into six categories the results as shown in Table 2.5 were obtained. As can be seen in Table 2.5, for each aggregate class, a wide range of K_1 and K_2 values were obtained. Rada and Witczak noted that because of the large range, moisture-density conditions of the aggregates are critical in evaluating K_1 and K_2 values for design.

2.1.2.2 Fine Grain Materials

Fine grained materials can be classified as either cohesive or noncohesive. Fine grained cohesive materials are illustrated. The resilient modulus for cohesive materials shows a stress sensitivity that is reversed from that of granular materials. The stress sensitivity is negative and is expressed as:

$$E_{sg} = K_3\, \sigma_d^{K_4} \quad \text{for fine-grained material} \tag{6}$$

where E_{sg} = resilient modulus for fine-grained soils,
σ_d = deviator stress ($\sigma_1 - \sigma_3 = \sigma_d$), and
K_3, K_4 = regression coefficients

The significance of negative stress sensitivity is that with increasing load (deviator stress) the resilient modulus is reduced. Increased moisture content decreases the resilient modulus. [21, 28] Figure 2.6 illustrates the stress sensitivity for fine grained materials.

2.1.3 Measuring Resilient Modulus By Laboratory Tests

Laboratory testing may be used to determine the elastic properties of asphalt concrete and unbound materials and offers the advantage over nondestructive techniques in that the test environment can be controlled. Changes in moisture content, density, and temperature are elements that are easily monitored with laboratory testing. [29]

The disadvantages of laboratory testing include the cost of conducting the test [29] and the requirement of recompaction to model density and moisture conditions of in situ materials (assuming "disturbed" samples are used). Parker [30], notes that sample disturbance in recompacting to obtain laboratory specimens destroys any cementation or thixotropic strengthening that may have already existed.

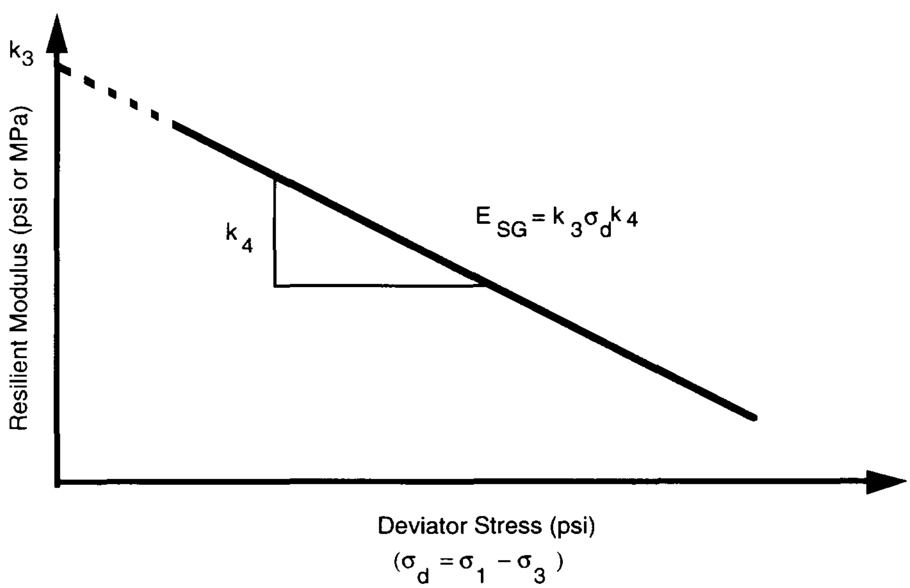


Figure 2.6. Resilient Modulus vs. Deviator Stress for Unstabilized Fine Grained Materials [9]

The following are some of the laboratory tests which can be used for determining asphalt and unbound materials resilient modulus.

2.1.3.1 Asphalt Materials

Resilient modulus for asphalt materials can be measured in accordance with ASTM D4123-82, Indirect Tension Test for Resilient Modulus of Bituminous Mixtures. [31] The test places a compressive load on a asphalt concrete core or laboratory sample. Typically the sample size is 4 inches in diameter and 2.5 inches thick or 6 inches in diameter by 3 inches thick. Figure 2.7 illustrates the application of a compressive load which in turn produces a relatively uniform tensile strain across the vertical diameter, Figure 2.8. The horizontal deformation is measured with linear variable differential transducers (LVDTs) across the diameter of the sample shown in Figure 2.9. The asphalt resilient modulus is calculated from the relationship:

$$E_{ac} = \frac{P(\mu + 0.27)}{(t)(\Delta H)} \tag{7}$$

where E_{ac} = asphalt concrete resilient modulus, psi,
P = repeated load, lb,
μ = Poisson's ratio (usually assumed),
t = thickness of the sample, in., and
ΔH = recoverable horizontal deformation, in.

The standard test temperatures are 41, 77, and 104 °F. A measurement and recording system capable of measuring deformations of 0.00001 inch is needed. Loads are measured with an electronic load cell.

2.1.3.2 Unbound Materials

For unbound materials the standard AASHTO test method was AASHTO T 274 (currently designated AASHTO T 294(I) following modification). Either laboratory compacted or undisturbed samples are used. Undisturbed samples are preferable but are difficult to obtain as discussed previously. Individual specimen sizes are normally 4 inches in diameter and 8 inches high as shown in Figure 2.10.

To perform the test the sample is enclosed vertically by a thin rubber membrane and on both ends by rigid plates (Figure 2.11). The specimen is placed in a triaxial cell and then a confining pressure (σ_3) is applied to the specimen as shown in Figure 2.12. Repeated axial load pulses ($\sigma_d=\sigma_1-\sigma_3$) are next applied as shown in Figure 2.13 to simulate the wheel loading on a pavement and the effect it has on an unbound material. To calculate the resilient modulus the following equation is used:

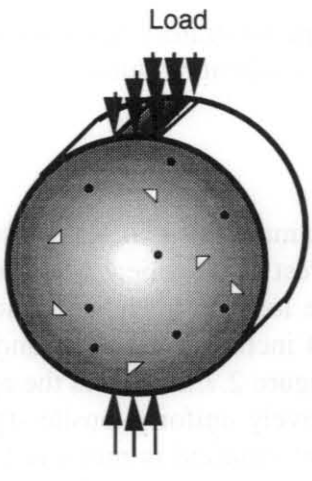


Figure 2.7. Vertical Loading of a AC Core or Laboratory Prepared Specimen for Determining Diametral Resilient Modulus [9]

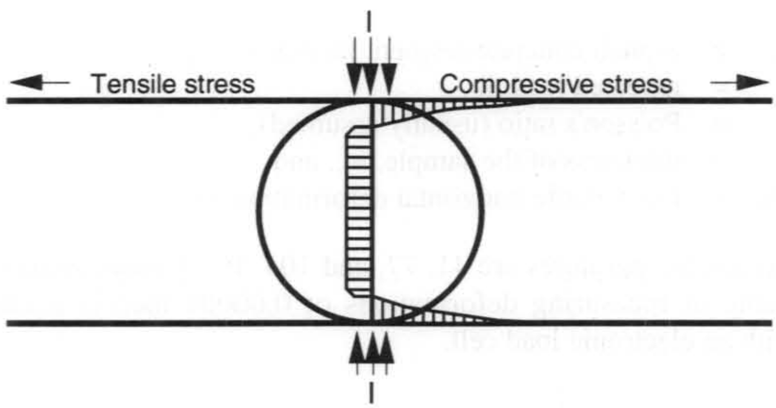


Figure 2.8. Vertical Loading Produces a Relatively Uniform Tensile Stress Across the Vertical Diameter [9]

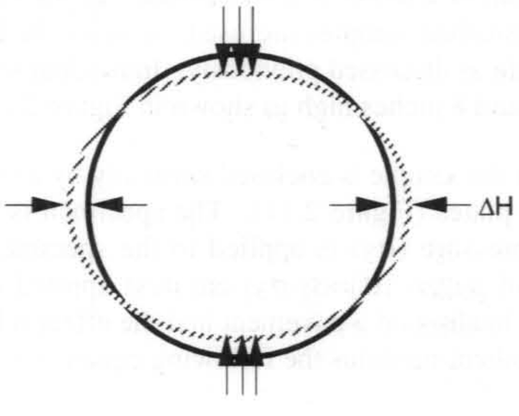


Figure 2.9. Measurement of Horizontal Deformation in the Diametral Resilient Modulus Test [9]

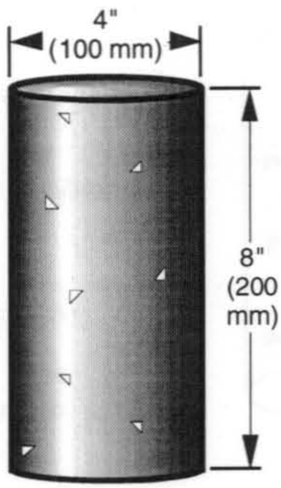


Figure 2.10. Basic Triaxial Specimen Configuration [9]

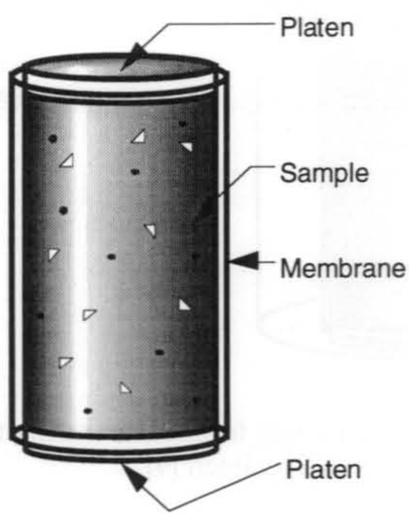


Figure 2.11. Enclosure of Triaxial Specimen [9]

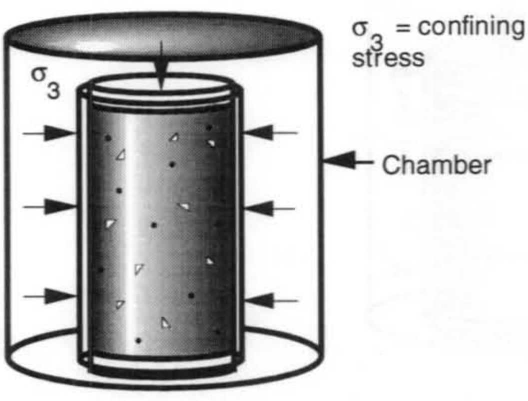


Figure 2.12. Triaxial Specimen in Pressure Chamber [9]

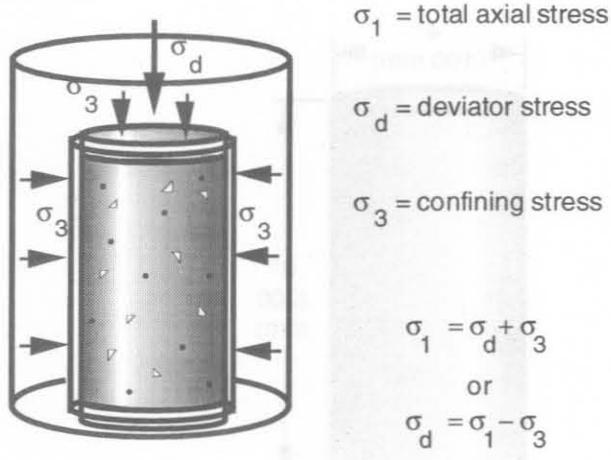


Figure 2.13. Stresses Acting on Triaxial Specimen [9]

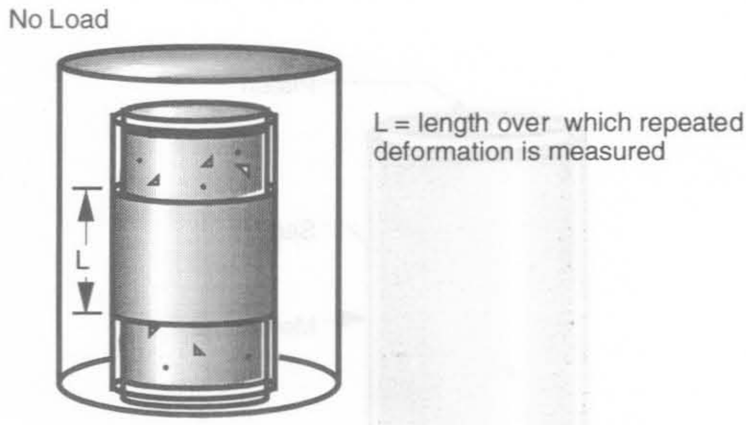


Figure 2.14. Gage Length for Measurement of Strain on Triaxial Specimen [9]

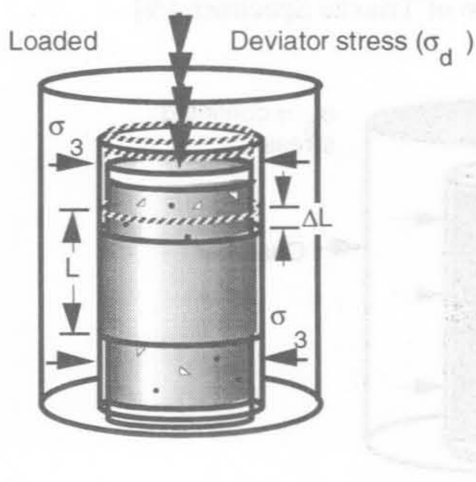


Figure 2.15. Deformation of Triaxial Specimen Under Load [9]

$$E_r = \frac{\sigma_d}{\varepsilon_r} \tag{8}$$

where E_r = resilient modulus,

$\sigma_d = \sigma_1 - \sigma_3 = \frac{P}{A}$ = deviator stress

σ_1 = total applied stress

σ_3 = confining pressure

P = repeated load,

A = cross sectional area of the sample,

$\varepsilon_r = \frac{\Delta L}{L}$ = recoverable axial strain,

L = gauge length over which the sample deformation is measured (see Figure 2.14), and

ΔL = change in sample length due to applied load (see Figure 2.15).

2.1.4 Measuring Resilient Modulus by Nondestructive Field Tests

Nondestructive field testing for resilient modulus often makes use of a Falling Weight Deflectometer (FWD). An advantage of this nondestructive technique is that the impulse load transmitted to the pavement by the FWD closely resembles the load transferred by a actual wheel load. [32, 33] As the impulse load deflects the pavement surface, transducers measure the vertical movement at various distances from the load thus providing a deflection basin. To simulate different loading conditions drop heights of the FWD mass system are varied.

Two FWD models are mostly used in the United States. The Dynatest Model 8000 imparts impulse loads between 1,500 and 27,000 pounds while the KAUB 150 range is 2,700 to 33,700 pounds. Detailed comparisons between FWD equipment are offered by Hudson [33] and Smith. [32] Standard test methods can be found in ASTM 4694-87, Standard Test Method for Deflections with a Falling Weight Impulse Device [34], and ASTM D4695-87, Standard Guide for General Pavement Deflection Measurement. [35]

The deflection basins determined by FWD testing can be used in one of many backcalculation computer programs to determine resilient modulus values for the layers. A typical program is based on multilayered elastic theory and requires an inverse solution technique to match measured deflection basins with theoretical basins. [36, 37] Estimates of elastic moduli are made until measured deflections and theoretical measurements fall within specified tolerances. Since the solution is achieved through an iterative technique there is not an unique solution. Estimated moduli need examination to be sure the results are reasonable. Often times the depth to a stiff layer such as bedrock or even the water table will effect backcalculated results. Techniques for estimating depth to a stiff layer can be found in References 36, 38, and 39.

The advantages of using FWD and backcalculation techniques to determine elastic moduli include no disturbance of the roadway section, ease of obtaining test results, and low operational cost. [29]

2.1.5 CORRELATION OF NONDESTRUCTIVE FIELD TESTING TO LABORATORY TESTING

Laboratory and field (nondestructive) moduli results are difficult to compare. For unbound materials, possible reasons are provided by Newcomb et al. [40] and Houston et al. [29] One reason is that laboratory samples are disturbed and then subjected to recompaction to estimate the in situ moisture and compaction level. As a result the laboratory sample may have a higher or lower modulus than exists in the field. The second reason is that laboratory samples are taken from a layer surface representing a specific point in the pavement structure. The FWD does not test a specific point but rather a stress bulb resulting from the distribution of the impact loading. A third reason offered by Parker [30] is that the stress level used in laboratory derived equations often do not truly represent the confining stress effects caused by overburden confinement, or horizontal residual confining stresses developed by traffic or compaction.

In a study of eight test sites by Parker [30], the moduli ratios of FWD backcalculated moduli to laboratory moduli for base materials ranged from 0.80 to 8.57. Parker noted the FWD moduli were higher than the laboratory moduli as the mean value of the ratios was 3.03 with a standard deviation of 1.99. Presented in Table 2.6 is the results of FWD and laboratory tested base resilient modulus for five sites studied by Newcomb et al. [40] The moduli ratios of FWD backcalculated moduli to laboratory moduli were 0.63 to 1.00 to with a mean at 0.81 and the standard deviation at 0.14. With Newcomb the laboratory results were higher.

In the study by Parker [30], subgrade moduli showed moduli ratios of FWD to laboratory tested moduli of 0.62 to 2.57. The average was 1.42 with a standard deviation of 0.53. The moduli ratios for the subgrade material in Newcomb's study [40] are shown in Table 2.7. The range of ratios for FWD to laboratory testing was 0.55 to 2.50. The average was 1.22 with a standard deviation of 0.47. Considering the difficulties in correlating unbound material moduli, Newcomb noted the overall results of the base and subgrade materials as shown in Tables 2.6, and 2.7 show fairly good agreement.

Table 2.6. Comparison of Laboratory and Backcalculated Moduli for Base Layer (after Newcomb et al. [40])

Test Site	Base Thickness (inch)	Field E_{sg} (ksi)	Lab E_{sg} (ksi)	% Difference (percent)	E_{sg}Field/E_{sg}Lab
1	28.8	23	23	0	1.00
4	9.0	45	53	18	0.85
5	6.6	38	60	60	0.63
11	21.0	21	25	22	0.84
15	11.4	22	31	36	0.71
					Mean = 0.81
					Std. Dev. = 0.14

Table 2.7 Comparison of Laboratory and Backcalculated Subgrade Moduli (after Newcomb et al. [40])

Test Site	Field E_{sg} (ksi)	Lab E_{sg} (ksi)	% Difference (percent)	E_{sg}Field/E_{sg}Lab
1	25	20	-21	1.25
2	21	16	-23	1.31
3	15	20	32	0.75
4	27	49	84	0.55
5	36	32	-11	1.13
6	29	15	-47	0.93
7	39	33	-14	1.18
8	9	5	-36	1.80
9	37	32	-14	1.16
10	39	26	-32	1.50
11	26	28	8	0.93
12	36	35	-2	1.03
13	36	42	17	0.86
14	40	42	4	0.95
15	20	12	-42	1.67
16	20	8	-59	2.50
				Mean = 1.22
				Std. Dev. = 0.47

Asphalt concrete moduli, as stated previously, vary as a function of temperature. The backcalculated moduli must be first adjusted to the laboratory conditions. Relationships such as established by Bu-bushait [11] can be used for this temperature adjustment.

Another adjustment required to correlate FWD backcalculated moduli to laboratory tested moduli is that of adjusting laboratory moduli for differences in loading. The FWD rate of loading (25-35 ms) is faster than the laboratory tested load rate (100 ms). Parker indicated without the load rate adjustment the backcalculated moduli should be larger than the laboratory moduli. [30]

Table 2.8 are comparisons of FWD backcalculated and laboratory tested results (asphalt concrete) for sites provided by Newcomb. [40] These results are not adjusted for load rate and as Parker suggested the backcalculated results are not necessarily higher. Other reasons such as the presence of fatigue cracking can alter backcalculated results. Sites 8 and 10 in Table 2.8 had signs of fatigue cracking. [40] The corresponding backcalculated laboratory moduli showed the greatest discrepancy. On the other hand Sites 3, 7, 13, and 14 had the thickest asphalt and the best agreement.

2.2 CALIFORNIA BEARING RATIO (CBR)

The California Bearing Ratio (CBR) was developed in the 1930s by the California State Highway Department. [41] The procedure was soon adopted by numerous states, counties, and U.S. federal agencies. Essentially empirical in nature, the test is a comparative measure of the shearing resistance of a soil. The CBR obtained from laboratory tests compares the material being tested with the bearing of a well-graded crushed stone. For a high quality crushed stone base material the CBR should be about 100 percent. The test is widely used as it is quick and offers a means of characterizing qualitatively the bearing capacity of soils, sands, and unbound base course materials. When used in a design procedure, CBR values allow pavement designers to quickly compare pavement designs when a variety of materials are available.

The procedure for determining CBR is provided in ASTM D1883-87, Bearing Ratio of Laboratory Compacted Soils [42], and AASHTO T 193-81, The California Bearing Ratio. [43] The test is basically a penetration test where a 3 in^2 penetration piston is forced into a sample compacted in a 6 inch diameter mold. For testing, prior to penetration, the specimen is soaked in water for typically 96 hours. Surcharge weights are placed on top of the soaked sample to provide a degree of confinement as is experienced for materials in a pavement structure. A load rate of 0.05 inch per minute is applied to the piston and total loads are recorded at 0.025 inch increments.

Table 2.8 Comparison of Laboratory and Backcalculated Asphalt Concrete Moduli (after Newcomb et al. [40])

Test Site	Asphalt Thickness (inch)	Field E_{ac}	Lab E_{ac}	% Difference (percent)	E_{ac}Field/E_{ac}Lab
1	5.2	713	427	-40	1.67
2	4.9	570	407	-29	1.40
3	10.9	395	408	3	0.97
4	3.5	450	268	-40	1.68
5	3.4	588	286	-51	2.06
6	11.2	658	355	-46	1.85
7	13.0	598	590	-1	1.01
8	7.3	79	214	170	0.37
9	16.4	563	144	-74	3.91
10	9.0	253	664	162	0.38
11	6.8	272	305	12	0.89
12	6.3	274	379	38	0.72
13	9.6	280	286	2	0.98
14	9.6	245	239	-2	1.03
15	6.2	387	466	20	0.83
16	8.5	281	177	-37	1.59
					Mean = 1.33 Std. Dev. = 0.85

The CBR value is determined for the 0.1 and 0.2 inch penetrations and is simply the ratio of the test bearing value to the standard bearing value for a well -graded crushed stone. The standard bearing values are 1,000 psi for the 0.1 inch penetration and 1,500 psi for the 0.2 inch penetration. While the CBR value at 0.1 inch penetration is usually considered standard, some agencies will select the higher of the two CBR values after verifying the results with a re-test when the 0.2 inch penetration CBR is higher. Table 2.9 lists some typical CBR ranges for coarse grain and fine grained materials [9] using the Unified Soil Classification System.

Table 2.9. Typical CBR ranges using the Unified Soil Classification System [9]

	Soil Type	CBR Range
Coarse - grained soils	GW	40 - 80
	GP	30 - 60
	GM	20 - 60
	GC	20 - 40
	SW	20 - 40
	SP	10 - 40
	SM	10 - 40
	SC	5 - 20
Fine grained soils	ML	15 or less
	CL	15 or less
	OL	5 or less
	MH	10 or less
	CH	15 or less
	OH	5 or less

CBR values are used by many highway agencies who do not use resilient modulus testing equipment. Since design procedures such as AASHTO requires resilient modulus, a conversion to resilient modulus must be made. A widely used empirical relationship developed by Heukelom and Klomp [44] and used in the 1993 AASHTO Guide is:

$$M_R = 1500 \times CBR \tag{9}$$

where M_R = estimated resilient modulus, psi, and
CBR = California bearing ratio.

This equation is restricted to fine grain materials with soaked CBR values of 10 or less. [9] One problem of using such a correlation as noted by Drumm [45] and Rada and Witczak [23] is that the CBR value is a measure of shear strength while E is stiffness prior to shearing. A relation between CBR and E does not necessarily have to exist for all soils. In addition, the CBR value does not recognize the materials stress sensitivity. The AASHTO guide suggests that any correlation to resilient modulus be performed according to well planned experiments for a range of soil types, saturation levels, and soil densities. [1]

2.3 RESISTANCE (R-VALUE)

Another test developed by the California State Highways is the Resistance Value (R-Value) test. [41] The test is used to evaluate treated and untreated base, subbases and subgrade soils. The test procedure was developed by Hveem and Carmany [46] and was first reported in the late 1940s. Yoder and Witczak [47] notes how the method is based on the properties of cohesion and friction for pavement materials. In a sense the test is a type of triaxial test.

To determine R-value a device called a stabilometer is used. The test methods include ASTM D2844-89, Resistance R-Value and Expansion Pressure of Compacted Soils [48], and AASHTO T 190-90, Resistance R-Value and Expansion Pressure of Compacted Soils. [49] The test procedure is basically one where the material's resistance to deformation is expressed as a function of the ratio of the transmitted lateral pressure to that of 160 psi applied vertical pressure. The relationship used for R-Value is:

$$R = 100 - \frac{100}{(2.5/D)[(Pv/Ph) - 1] + 1} \tag{10}$$

where R = resistance value,

Pv = applied vertical pressure (160 psi),

Ph = transmitted horizontal pressure at Pv = 160 psi, and

D = displacement of stabilometer fluid necessary to increase horizontal pressure from 5 to 100 psi.

Typical R-Values include 80+ for well graded (dense gradation) crushed stone base, and 15-30 for MH silts. [9] As with CBR values, R-values can be input directly to many pavement design procedures. A correlation developed by the Asphalt Institute is:

$$M_R \text{ (psi)} = A + (B \times \text{R-value}) \tag{11}$$

where A = 772 to 1155, and
B = 369 to 555.

For the 1993 AASHTO Design Guide, the equation used is reduced to:

$$M_R \text{ (psi)} = 1000 + 555 \times \text{(R-value)} \tag{12}$$

where M_R (psi) = estimated resilient modulus.

Use of this equation is restricted to fine grained clay soils with R-Values 20 or less. [9]

3. PAVEMENT MATERIALS VARIATION

3.1 ASPHALT MATERIALS

Climatic variation for asphalt concrete is the simplest to quantify. Since asphalt concrete stiffness is primarily a function of temperature, an accurate estimate of elastic moduli can be determined based on mean pavement temperature. Estimates of mean pavement temperatures can be obtained from relationships such as those developed by Witczak [50] or Southgate and Deen. [51] The relationship suggested by Witczak between monthly mean pavement temperature (MMPT) and monthly mean air temperature is as follows:

$$\text{MMPT} = \text{MMAT}\left(1+\left(\frac{1}{z+4}\right)\right)-\left(\frac{34}{z+4}\right)+6 \qquad (13)$$

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

The Asphalt Institute [5] provides a complete procedure for determining pavement temperature based on the Southgate procedure. Figure 2.16 is the replotted Asphalt Institute chart to estimate pavement temperature based on pavement surface temperature plus the five day mean temperature.

To illustrate the seasonal variation of asphalt concrete consider a typical roadway section with the mean temperature distribution shown in Table 2.10. The pavement is located in Eastern Washington on I 90 M.P. 208.9. The asphalt concrete layer is approximately 10 inches thick. Monthly mean pavement temperature is easily computed from Witczak's method with the monthly mean pavement temperature shown in Table 2.10. Also shown in Table 2.10 is the corresponding asphalt modulus as determined from the asphalt-temperature developed by Bu-bushait. [11]

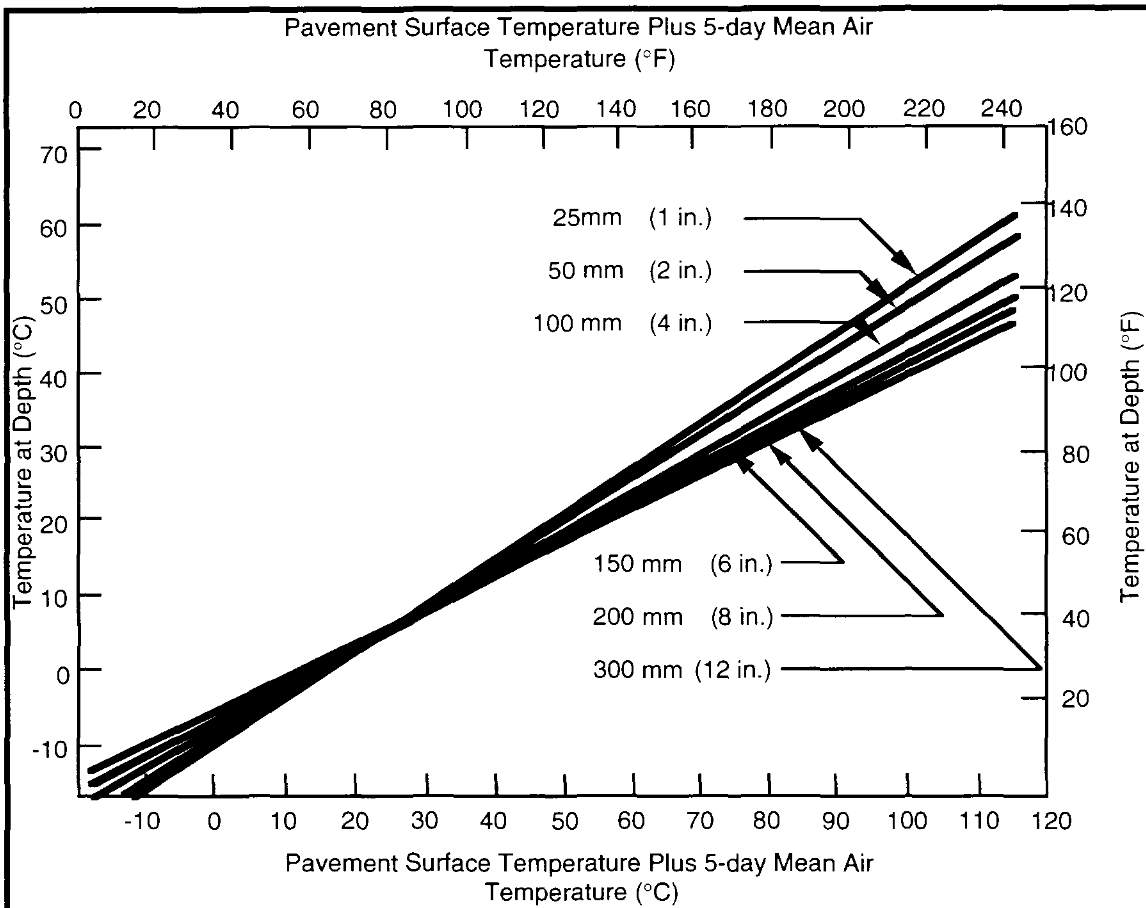


Figure 2.16. Estimation of Pavement Temperature

3.2 BASE AND SUBGRADE LAYERS

The variation in base and subgrade layers are most pronounced with changes in moisture content. Increases in moisture content can be caused by many factors, the most obvious is precipitation brought about by seasonal rains. Surface cracks are a primary means that surface water infiltrates the pavement structure. [52] Other avenues for increased moisture is water movement from external sources such as the shoulders or lateral flowing water, fluctuations in the water table, and capillary rise. [52, 53]

Temperature has minimal influence on the unstabilized materials except during periods of freezing and thawing. During periods of freeze/thaw the pavement structures become both the strongest and the weakest. With freezing temperatures, moisture within base and subgrade layers freezes causing an increased stiffness of the base and subgrade layers. Load capacity is increased above that capable in non freezing conditions.

Table 2.10. Monthly Mean Air Temperature, Monthly Mean Pavement Temperature and Corresponding Monthly Mean Asphalt Modulus for I 90, MP 208.9 in Eastern Washington

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Mean Monthly Air Temperature[1] (°F)	26.9	34.0	39.5	46.8	54.8	62.5	69.9	68.5	60.9	49.4	36.5	30.3
Monthly Mean[2] Pavement Temperature (°F)	32.1	40.0	46.1	54.2	63.1	71.7	79.9	78.3	69.9	57.1	42.8	35.9
Asphalt Modulus[3] (ksi)	2090.5	1723.1	1441.9	1094.5	768.0	518.2	339.9	370.4	565.1	980.8	1592.8	1915.0

Notes:
1. After Lee [19]
2. Monthly mean pavement temperature determined by Witczak's method [50]
   Pavement thickness is 10 inches, z = 5 inches
3. Asphalt modulus determined by Bu-Bushait's relationship [11]

On the other hand a weakened pavement results when the pavement thaws. [52, 54, 55] If the thawing occurs from the top of pavement toward the subgrade, the base layer can become saturated as thawed water is trapped between the impervious asphalt concrete and the frozen subgrade. The base layer becomes weaker as drainage through the structure is not possible. It is during this freezing and thawing period that seasonal variation in the stiffness of pavements is most dramatic. [56]

Several studies have provided data which shows seasonal variation in base and subgrade materials. In a study by Lary, et al. [57], for WSDOT, six sites were monitored to measure the variation of base and subgrade moisture content, frost depth and location, and pavement deflection. A FWD was utilized to measure deflections several times during the year. Effort was made to collect data during periods of the spring thaw.

The BISDEF [58] computer program was used to backcalculate base and subgrade resilient modulus from the FWD data. Tables 2.11 and 2.12 show the backcalculated results for three sites, all of which are in Eastern Washington which has winter ground freezing. Observation of the tables shows more seasonal change occurred in the base layer than the subgrade. For SR 2, MP 159.6 the base modulus decreased 39% from August 1983 to March 1984 while the subgrade only changed 8 percent. Many of the test data for all sites showed the subgrade changing by less than 1,000 psi. Frozen sections are reflected for January 1984 deflection tests. Of particular interest is the magnitude of the moduli for frozen layers. The frozen base layer moduli on SR 172 and SR 2 (Sunnyslope) ranged from 57,300 to 377,900 psi. Subgrade moduli showed frozen moduli of 17,600 to 59,700 psi.

In a study by Chandra, six farm to market roads in Texas were studied to determine the effects of temperature and moisture on the load response of low volume roads. [59] The climate for the sites was typically mild, dry to humid winters, with warm to very hot summers.

The roads had a surface treatment and a granular base. Each site had two subgrade soils, one sandy, and one clayey. A FWD was used to obtain deflection basins once a month for a year. The data was backcalculated with the LOADRATE [60] program.

Table 2.13 shows the ranges of resilient moduli obtained for base and subgrade materials. The modulus of the base is the composite of the surface treatment and the base thus accounting for the higher moduli. Greater variation is seen in the base than the subgrade layer which stayed relatively constant for the entire year. Chandra [59] noted all the test sections had good surfaces with no cracks and that the water table was well below the pavement surface.

Table 2.11. Seasonal Changes in Base and Subgrade Moduli (after Lary et al. [57])

Site: SR 2, Sunnyslope				Site: SR 2, MP 159.6			
Date	Pavement Surface Temp. (°F)	Base E_{bs} (ksi)	Subgrade E_{sg} (ksi)	Date	Pavement Surface Temp. (°F)	Base E_{bs} (ksi)	Subgrade E_{sg} (ksi)
2/23/83	40	16.3	14.0	2/24/83	50	19.7	13.3
3/4/83	48	21.9	13.3	3/3/83	45	24.3	13.0
3/9/83	46	20.2	15.5	3/9/83	47	26.4	13.0
3/18/83	58	23.1	14.1	3/13/83	60	26.3	12.6
3/24/83	62	20.9	13.5	3/24/83	40	18.4	12.8
8/16/83	99	25.9	13.6	8/17/83	72	28.8	11.1
1/11/84	34	57.3	17.6	2/21/84	42	21.8	12.6
1/31/84	34	106.4	17.9	3/1/84	48	17.5	12.2
2/21/84	50	24.3	14.9	3/9/84	60	28.4	12.3
2/29/84	51	27.2	13.0	3/21/84	49	29.9	12.7
3/6/84	60	26.4	12.8				
3/19/84	50	28.6	12.5				

Table 2.12. Seasonal Changes in Base and Subgrade Moduli (after Lary et al. [57])

Site: SR 172, MP 2.0			
Date	Pavement Surface Temp. (°F)	Base E_{bs} (ksi)	Subgrade E_{sg} (ksi)
2/24/83	50	17.8	5.7
3/3/83	38	28.6	5.9
3/9/83	47	32.1	6.7
3/17/83	39	23.4	6.6
8/17/83	75	26.7	6.6
1/10/84	34	377.9	59.7
3/1/84	46	32.8	4.8
3/7/84	60	21.2	5.6
3/21/84	50	27.7	5.8

Table 2.13. Backcalculated Base and Subgrade Moduli (after Chandra et al. [59])

Road	Yearly Rainfall (inches	Base			Subgrade		
		United Classification System	Modulus E_{bs} Max (ksi)	Modulus E_{bs} Min (ksi)	Unified Classification System	Modulus E_{sg} Max (ksi)	Modulus E_{sg} Min (ksi)
FM 1235	16 - 27	GP	119.8	55.4	CH	7.2	5.2
FM 1983	16 - 27	SW	74.0	32.2	SC	7.7	6.6
FM 2864	45	SW-SM	44.3	22.7	CH	12.0	9.5
SH 7	45	SP-SM	147.8	92.6	SP-SM	7.2	6.1
FM 491	19 - 27	GW	57.3	34.3	SC	5.2	5.1
FM497	19 - 27	SP-SM	58.5	42.5	SC	5.3	5.2

In North Carolina, Ali and Khosla [61] made a study comparing four backcalculation programs with results of laboratory testing. They found that two of the backcalculation programs (ELMOD and VESYS) were more suitable in predicting layer moduli. Three highways with two sites each were considered.

Shown on Tables 2.14 and 2.15 are the results from the three sites. Moisture contents of the in situ material is shown. For all sites, the backcalculated results followed a trend of increasing moduli with decreasing moisture. As with the two previous studies, greater variation is seen in the base moduli than in the subgrade. Chu, et al. [62] noted that subgrades after construction remain fairly stable in moisture condition.

A final comparison offered from Tables 2.14 and 2.15 is that estimated moduli vary depending upon the backcalculation procedure used. For instance, during April 1985 the ELMOD program estimated a base modulus of 34.5 ksi for US 64 sub-section 01. For the same date, location, and base moisture condition the VESYS program estimated a base modulus of 26.0 ksi. Similar differences were seen in the base layer for other routes and test dates. The maximum subgrade difference between computer programs was 2.8 ksi, which also occurred April 1985 on US 64, sub-section 01.

To quantify the seasonal variation of base or subgrade layers, seasonal adjustment factors are typically developed. Adjustment factors are determined by first obtaining sufficient resilient moduli to represent a geographic region for a yearly period. The moduli are normalized by selecting one of the seasons as a base value. Typically a dry period (summer), when resilient modulus is the highest, is chosen.

Table 2.14. Changes in Base and Subgrade Moduli Backcalculated from ELMOD Program (after Ali and Khosla [61])

Route	Date	Sub-Section 01				Sub-Section 11			
		Base Moisture percent	Base E_{bs} (ksi)	Subgrade Moisture percent	Subgrade E_{sg} (ksi)	Base Moisture percent	Base E_{bs} (ksi)	Subgrade Moisture percent	Subgrade E_{sg} (ksi)
US 64	April '85	5.3	34.5	20.1	6.3	5.3	18.0	18.9	4.4
	Aug '85	4.6	49.5	19.2	6.8	4.6	32.8	17.9	5.0
I 40	May '85	5.6	24.5	10.0	10.3	5.6	32.0	18.5	5.5
	Sept '85	4.8	29.3	11.0	9.5	4.8	29.3	20.4	3.5
US 19	May '85	6.2	22.0	18.4	4.0	6.2	27.8	14.8	4.9
	Sept '85	5.8	25.5	18.0	4.8	5.8	34.8	15.0	4.2

Table 2.15. Changes in Base and Subgrade Moduli Backcalculated from VESYS Program (after Ali and Khosla [61])

Route	Date	Sub-Section 01				Sub-Section 11			
		Base Moisture percent	Base E_{bs} (ksi)	Subgrade Moisture percent	Subgrade E_{sg} (ksi)	Base Moisture percent	Base E_{bs} (ksi)	Subgrade Moisture percent	Subgrade E_{sg} (ksi)
US 64	April '85	5.3	26.0	20.1	9.1	5.3	26.0	18.9	6.0
	Aug '85	4.6	28.0	19.2	9.5	4.6	28.0	17.9	7.3
I 40	May '85	5.6	25.8	10.0	9.8	5.6	25.8	18.5	5.8
	Sept '85	4.8	27.4	11.0	9.2	4.8	27.4	20.4	5.2
US 19	May '85	6.2	25.0	18.4	6.1	6.2	25.0	14.8	7.5
	Sept '85	5.8	27.9	18.0	6.0	5.8	27.9	15.0	6.7

An example of seasonal adjustment factors is shown in Table 2.16. This table was derived from results of a study by Finn, et al. [63] who assigned different moduli values to different seasons for the materials of the AASHO Road Test. The results are summarized in Table 2.17. Finn, et al. [63] reports that the subgrade values of Table 2.17 were selected by using a iterative approach to adjust the subgrade modulus to fit a measured deflection during selected base and subbase seasonal values. The calculations were made by using the seasonal values shown for the base and subbase of Table 2.17. Finn, et al. reported a certain amount of judgment as well as laboratory test results were used to select base and subbase moduli.

Table 2.16 was derived from Table 2.17 by normalizing the K_1 coefficient of the other seasons with respect to the summer period. [11] The K_2 coefficient was considered constant. The winter base and subgrade moduli (Dec. - Feb.) were considered frozen and set at 50,000 psi. The winter seasonal factor of 2.0 for base course results when the summer modulus is evaluated at a bulk level of 25 psi. Likewise, the factor of 31.9 results when the deviator stress for the summer subgrade moduli is evaluated at 10 psi.

Two additional studies have provided examples of seasonal adjustment factors. Thompson and Hoffman [64] provided the adjustment factors for a variety of subgrade materials as shown in Table 2.17. The adjustment factor for the summer-fall period had a value of 1.0. Mahoney, et al. [65] determined some adjustment factors for eastern and western Washington. Eastern Washington is characterized by cold winters and hot and dry summers. Western Washington has two primary periods, warm and dry summers, and wet and mild winters. The factors in Table 2.19 show more seasonal variation in the base than in the subgrade layer.

Another type of seasonal adjustment factor are those that are applied to deflection measurements. Many design procedures require a critical period maximum deflection. This corresponds to when the pavement is the weakest. If measurements are taken during a different period of the year, measurements require adjustment to the critical season. In areas of freeze/thaw the critical season is generally taken as the spring. During this period the serviceability loss to the pavement structure may equal or exceed the loss during the remainder of the year. [56]

Table 2.20 is a summary [66] of some typical Forest Service deflection measurements for paved roads in the Willamette National Forest located in the Cascade mountains of Oregon. Several sites are shown for deflections that were taken with a FWD and then converted to Benkelman Beam representative rebound deflections. These deflections were then normalized to a temperature of 70 °F, as required by the Asphalt Institute overlay design procedure. [5] The critical period was chosen as the wet season. The c-value used to convert from the dry to the wet period varies from site to site and is therefore site dependent. In fact, the value depends on the period of the year, subgrade soils, thickness of the pavement, environmental considerations, and the material that makes up the pavement structure. [56, 67]

Table 2.16. Seasonal Ratios Between Summer Resilient Modulus to Other Season Resilient Modulus for AASHO Road Test Base and Subgrade Materials (after Finn et al. [63])

Material	Ratios Between Fall Modulus to Other Seasons Modulus for Base and Subgrade Materials			
	Fall	Spring	Summer	Winter
Base	1.1	0.9	1.0	2.0
Subgrade	1.5	0.4	1.0	31.9

Table 2.17. Seasonal Variations in Elastic Moduli for AASHO Road Test Materials (after Finn et al. [63])

Material	Seasonal Moduli (psi			
	Fall Sept, Oct, Nov	Spring March, April	Summer May, June, July, Aug	Winter Dec, Jan, Feb
Asphalt Concrete E_{ac}, (psi)	450,000	710,000	230,000	1,700,000
Temperature °F	70	59	85	30
Base, E_{bs}, (psi)	$4000\theta^{.60}$	$3200\theta^{.60}$	$3600\theta^{.60}$	50000
Subbase, E_{sb}, (psi)	$5400\theta^{.60}$	$4600\theta^{.60}$	$5000\theta^{.60}$	50000
Subgrade, E_{sg}, (psi)	$27000\sigma_d^{-1.06}$	$8000\sigma_d^{-1.06}$	$18000\sigma_d^{-1.06}$	50000

Notes: Winter period modulus is considered frozen and fixed at 50,000 psi.
θ = bulk stress, σ_d = deviator stress, E = Resilient modulus

Table 2.18. Subgrade Climatic Adjustment Factors (after Thompson and Hoffman [64])

USDA Soil Types	USDA Internal Drainage Class			
	Well Drained or Better		Other USDA Drainage Classes	
	Freeze-Thaw	No-Freeze-Thaw	Freeze-Thaw	No-FreezeThaw
Silt, silt loam loam, sandy loam	0.70	0.85	0.50	0.60
Silty clay loam clay loam, sandy clay loam, sandy clay, silty clay, clay	0.65	0.85	0.50	0.75

Notes 1. E_{sg} for Summer/Fall period is assigned a factor of 1.0 (no adjustment required)

2. To predict Summer/Fall E_{sg} from Spring data, divide the Spring E_{sg} by the appropriate adjustment factor from Table 2.18

3. To predict Spring E_{sg} from Summer/Fall data, multiply by the appropriate adjustment factor from Table 2.18

Table 2.19. Seasonal Variation of Unbound Materials Moduli Ratios for Washington State [65]

	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 2.20. Typical Forest Service Wet vs. Dry Weather Representative Rebound Deflections for Selected Roads Located in the Willamette National Forest of Oregon (after "Willamette National Forest Wet Season Testing" [66])

Road Number	April 1991		Sept 1990	
	Wet Season RRD[1] (mils)	Critical c-Value	Dry Season RRD[1] (mils)	RRD ratio[2]
15	30.12	1.00	25.67	1.17
46	32.16	1.00	24.55	1.31
1506	50.59	1.00	36.50	1.39
2266	52.03	1.00	30.77	1.69
2643	65.35	1.00	51.84	1.28
2000-68	79.02	1.00	75.30	1.05

Notes: 1. Representative rebound deflections - not adjusted for season

2. RRD ratio = wet season RRD divided by dry season RRD

Table 2.21 is a summary of Benkelman Beam deflections obtained during 1985 on Forest Road No. 92 in the Kootenai National Forest. The Seasonal Factors (ratio of seasonal deflection to average summer deflection — similar to Asphalt Institute c-value) are quite large ranging from about 3 to 8 (depending on test location). These are substantially larger than those observed for various roads in the Willamette National Forest (Table 2.20). This further reinforces the view that seasonal deflection ratios are quite site specific.

A rather complete method of quantifying Dynaflect deflection c-values was summarized by Bandyopadhyay. [56] In this study 14 sites were monitored for two years in six regions of Kansas. Dynaflect deflections and pavement temperatures were recorded. Material types for each region were determined and a table of c-values corrected for temperature was made to reflect adjustment factors for different months of the year. Table 2.22 is a summary of the c-values for the Kansas sites showing both monthly values and the soil type. Variability in adjustment factors (c-values) is seen with both the material type and zone for the adjustment factors.

Additional deflection adjustment factors were developed in Pennsylvania by Bhajandas et al. [68] by using data from eight test sections monitored over a three year period. Deflections were corrected for temperature and were plotted against calendar day. The deflection factors in Table 2.23 are the result of linear regression equations obtained from the data. Bhajandas reported the deflection factors developed for the clay

Table 2.21. Typical Forest Service Freeze/Thaw Seasonal Factors for Selected Sites Located on Forest Road No. 92 in the Kootenai National Forest of Montana

	FOREST ROAD NO. 92 - MILEPOST																			
	17.05		23.95		30.60		35.55		38.00		45.20		49.40		49.95		50.20		50.70	
1985 (date)	DEFL (mils)	SF	DEFL (mils)	SF	DEFL (mils)	SF	DEFL (mils)	SF	DEFL (mils)	SF	DEFL (mils)	SF	DEFL (mils)	SF	DEFL (mils)	SF	DEFL (mils)	SF	DEFL (mils)	SF
3/12	~	~	37	1.17	21	1.00	~	~	~	~	~	~	~	~	~	~	~	~	~	~
3/22	~	~	141	4.48	73	3.48	~	~	~	~	~	~	~	~	~	~	~	~	~	~
3/29	59	1.53	102	3.24	50	3.38	44	1.96	~	~	~	~	~	~	~	~	~	~	~	~
4/5	144	3.74	44	1.40	38	1.81	49	2.18	53	3.21	124	7.75	93	3.51	~	~	~	~	~	~
4/8	184	4.78	38	1.21	36	1.71	51	2.27	73	4.42	125	7.81	79	2.98	~	~	~	~	~	~
4/11	98	2.55	38	1.21	36	1.71	76	3.38	78	4.73	66	4.12	131	4.94	~	~	~	~	~	~
4/17	82	2.13	43	1.37	33	1.57	69	3.07	37	2.24	41	2.56	130	4.91	~	~	~	~	~	~
4/22	62	1.61	45	1.43	33	1.57	54	2.40	33	2.00	40	2.50	81	3.06	~	~	~	~	~	~
4/26	85	2.21	46	1.46	~	~	53	2.36	35	2.12	39	2.44	68	2.57	~	~	58	2.83	~	~
4/30	68	1.77	~	~	~	~	47	2.09	~	~	35	2.19	55	2.08	~	~	63	3.07	61	2.49
5/3	78	2.03	39	1.24	~	~	46	2.04	~	~	33	2.06	54	2.04	~	~	58	2.83	69	2.82
5/6	64	1.66	~	~	~	~	39	1.73	~	~	~	~	53	2.00	~	~	48	2.34	48	1.96
5/14	64	1.66	~	~	~	~	34	1.51	~	~	~	~	46	1.74	43	2.21	50	2.44	58	2.37
5/20	~	~	~	~	~	~	~	~	~	~	~	~	37	1.40	30	1.54	34	1.66	43	1.76
5/24	54	1.40	33	1.05	27	1.29	28	1.24	21	1.27	20	1.25	32	1.21	24	1.23	28	1.37	37	1.51
6/20	53	1.38	43	1.37	29	1.38	31	1.38	18	1.09	22	1.38	33	1.25	27	1.38	27	1.32	40	1.63
7/11	49	1.27	30	0.95	19	0.90	20	0.89	15	0.90	15	0.94	27	1.02	19	0.97	21	1.02	27	1.10
9/3	28	0.73	33	1.05	23	1.10	25	1.11	18	1.09	17	1.06	26	0.98	20	1.03	20	0.98	22	0.90
PEAK SF		4.78		4.48		3.48		3.38		4.73		7.81		4.94		2.21		3.07		2.82

Notes:
1. Deflections: temperature corrected per Asphalt Institute MS-17.
2. SF = Seasonal Factor = Ratio of seasonal deflection to average summer deflection (avg. of 7/11 and 9/3 deflections).
3. Deflections corrected to 9,000 lb dual tire load.

Table 2.22. Seasonal Adjustment Factors for Dynaflect Maximum Deflection (DMD) for Six Zones in Kansas [56]

Zone	Soil Type	Annual Rainfall (inch)	Mar	Apr	May	June	July	Aug	Sept	Oct
1	Clay Loams	34	1.21	1.21	1.16	1.00	1.17	1.22	1.18	1.22
2	Silt Loams Clay Loams	36	1.12	1.27	1.00	1.00	1.00	1.00	1.00	1.00
3	Silt Loams Silty Clays	28	1.26	1.28	1.15	1.12	1.14	1.00	1.25	1.00
4	Sandy Loams	34	1.00	1.12	1.18	1.20	1.28	1.25	1.18	1.00
5	Silt Loams	21	1.13	1.11	1.10	1.10	1.10	1.00	1.00	1.00
6	Silt Loams	18	1.24	1.15	1.10	1.14	1.15	1.20	1.18	1.00

subgrade compared well to trends in deflections for pavement built on clays, silts, and sands in the State of Minnesota. [68] The deflection factors for pavements on sand and silts listed in Table 2.23 are extended from the trends experienced in Minnesota. The month of March is the critical period as indicated by the adjustment factor of 1.0.

The Roads and Transportation Association of Canada (RTAC) surface deflection based method for pavement evaluation and new design uses the "maximum spring Benkelman Beam rebound value" [95]. This is calculated as the mean plus two standard deviations. Thus, only about two percent of the deflections would be higher. If the Benkelman Beam measurements are obtained between September 1 to October 15, the measured values are converted to "maximum spring values" by use of a ratio of 2.5 if site specific information is not available. This ratio of 2.5 is a bit higher than generally observed in the U.S. possibly reflecting the generally more severe winter and thaw periods.

Note that seasonal adjustment may be based on either "best" or "worst" conditions, resulting in seasonal factors being greater than 1.0 in some studies and less than 1.0 in others. The reader is cautioned to always note which season is assigned a value of 1.0 when comparing seasonal adjustment factors.

Table 2.23. Deflection Adjustment Factors for Pennsylvania Test Sections with Various Types of Subgrade Soils [68]

Date of Test	Deflection Factors[2,3]		
	Soil Type		
	Sand	Clay	Silt
March[1]	1.00	1.00	1.00
April	1.20	1.10	1.20
May 1 - May 15	1.20	1.15	1.25
May 16 - May 31	1.20	1.20	1.35
June 1 - June 15	1.20	1.25	1.40
June 16 - June 30	1.20	1.30	1.45
July 1 - July 15	1.20	1.35	1.50
July 16 - July 31	1.20	1.40	1.55
August	1.20	1.48	1.63
September	1.20	1.55	1.70
October	1.20	1.60	1.75
November	1.20	1.67	1.82
December[1]	1.20	1.75	1.90

Notes:
1. Pavement structure unfrozen
2. March is chosen as the critical month deflection. The deflection factors were obtained by dividing the March deflection by the deflection obtained during the specific periods.
3. To predict the critical deflection, multiply the measured deflection from a specific period by the appropriate deflection factor

4. CONSIDERATION IN DESIGN PROCEDURES

The following sections discuss the considerations given for seasonal variation in some of the contemporary pavement design procedures. Assumptions concerning climatic effects used to develop deflection based, component analysis or mechanistic empirical design procedures are highlighted.

4.1 NEW PAVEMENT DESIGN PROCEDURES

Many pavement design procedures recognize the seasonal variation of pavement materials, and the assumptions used to include seasonal variation are discussed in the following sections.

4.1.1 AASHTO DESIGN PROCEDURE

The AASHTO design procedure as described in the 1993 *Guide For the Design of Pavement Structures* [1] considers seasonal variation of the subgrade and to a limited degree the variation of the base or subbase layers. This is an improvement over the previous procedure as outlined the *AASHTO Interim Guide for Design of Pavement Structures* [69] where seasonal variation was provided by use of a regional factor. Use of the regional factor was arbitrary and amounted to adjusting layer thicknesses based on climatic conditions more or less severe than those of the AASHO Road Test.

The AASHO Road Test, which occurred from 1959-1961, provided a comprehensive study of the relationship of performance, structural thickness, and traffic loadings. [70] An empirical relationship was developed that is known as the performance equation. The equation provided a means to design layer thickness, but due to its empirical nature there are many limitations.

One major limitation as related to seasonal effects is that the AASHO Road Test is represented by only one type of subgrade soil. For use with other subgrade conditions a relative scale was initially adopted which was termed the soil support value. As summarized by Elliott and Thornton [71] the scale was not based on any particular method of test and highway agencies were required to establish relationships between their testing methods and the soil support scale. A second limitation was that the road test was performed in an accelerated two year period for a single environment. Extrapolation is therefore required for 10 - 20 year (or more) designs. As mentioned previously, different climatic regions were dealt with by the use of a regional factor.

The 1986 AASHTO Guide adopted resilient modulus to characterize pavement materials, in part, because it provided a way to characterize the seasonal variation. Resilient moduli are measurable and reflects stiffness changes in the pavement. The performance equation was revised to include subgrade resilient modulus and both the soil support value and regional factor parameters were deleted.

Subgrade resilient modulus is introduced to the guide by use of the "effective roadbed soil resilient modulus." [1] The effective roadbed soil resilient modulus is the subgrade soil strength value that represents the resilient modulus throughout the year (Figure 2.17).

The recommended procedure to determine the effective resilient modulus is provided in the AASHTO Guide. [1] Time periods of approximately equal repetitive loading corresponding to seasonal changes in the subgrade soil are considered. Estimates of monthly or bimonthly resilient modulus are made and recorded on a chart provided by AASHTO and shown in Figure 2.18.

Resilient moduli for separate months have different effects on the performance of the pavement. Subgrade values with low resilient moduli allow more damage to the pavement structure than those with high moduli. The relative damage is accounted for by assigning damage factors to the monthly or bimonthly moduli.

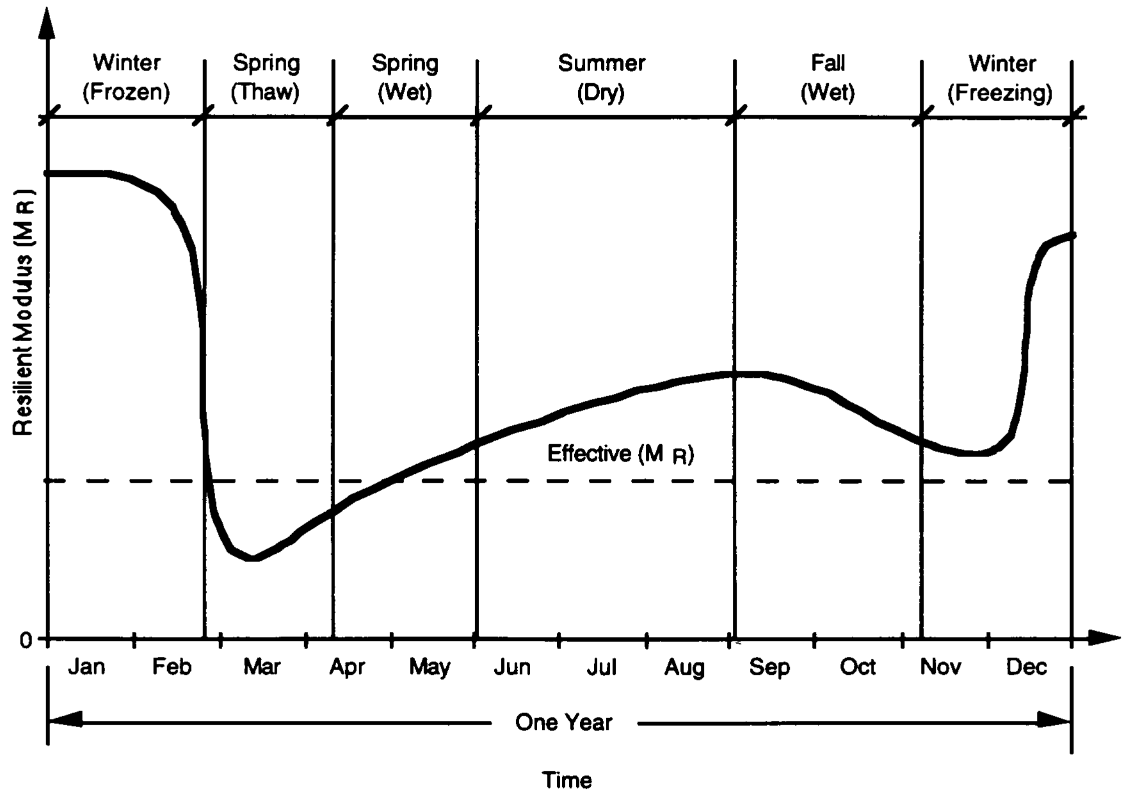


Figure 2.17. Concept of Seasonal Roadbed Soil Variation

Month	Roadbed Soil Modulus, M_R (psi)	Relative Damage, u_f
Jan	20,000	0.01
	20,000	0.01
Feb	20,000	0.01
	5,000	0.31
Mar	3,000	1.01
	4,000	0.52
April	4,000	0.52
	4,000	0.52
May	6,000	0.20
	7,000	0.14
June	7,000	0.14
	8,000	0.10
July	9,000	0.08
	10,000	0.06
Aug	11,000	0.05
	11,000	0.05
Sept	12,000	0.04
	12,000	0.04
Oct	10,000	0.06
	9,000	0.08
Nov	7,000	0.14
	6,000	0.20
Dec	20,000	0.01
	20,000	0.01
Summation: $\sum u_f =$		4.31

$$\text{Average: } \bar{u}_f = \frac{\sum u_f}{n} = \frac{4.31}{24} = 0.18$$

Effective Roadbed Soil Resilient Modulus, M_R (psi) = 6,300 (corresponds to $\bar{u}_f$)

$$u_f = (1.18 \times 10^8)(M_R)^{-2.32}$$

$$M_{R_{eff}} = (3005)(\bar{u}_f)^{-0.431}$$

Figure 2.18. Chart for Estimating Roadbed Soil Resilient Modulus for Flexible Pavements

Damage factors are determined from the equation shown in Figure 2.18. The damage factors are next averaged to obtain an average damage factor. The design resilient modulus is found from the scale by matching the average value to the corresponding resilient modulus. The effective resilient modulus represents a weighted modulus based on the damage caused by the seasonal variations in the subgrade resilient modulus.

AASHTO provides estimates of subgrade resilient moduli for the design of low volume roads. A low volume road is classified as one where the design equivalent single axle loads (ESALs) are less than 1 million. The estimate of subgrade resilient modulus is based on six climatic regions of the United States. The regions include:

- Wet, no freeze
- Wet, freeze - thaw cycling
- Wet, hard - freeze, spring thaw
- Dry, no freeze
- Dry, freeze - thaw cycling
- Dry, hard freeze, spring thaw

Table 2.24 shows the seasonal lengths that AASHTO uses to represent each of the climatic regions.

Suggested seasonal subgrade moduli as a function of the relative quality of the subgrade material are listed in Table 2.25. By combining the suggested subgrade resilient moduli and the seasonal lengths of Table 2.24 and using the procedure outlined previously, effective resilient moduli were determined. These are listed in Table 2.26. The resilient moduli suggested by AASHTO are only estimates and engineering judgment is required with use.

Table 2.24. Suggested Seasonal Lengths (months) for the Six U.S. Climatic Regions [1]

U.S. Climatic Region		Seasonal Length in Months			
		Winter (Roadbed Frozen)	Spring-Thaw (Roadbed Saturated)	Spring/Fall (Roadbed Wet)	Summer (Roadbed Dry)
I.	Wet, no freeze	0.0	0.0	7.5	4.5
II.	Wet, freeze - thaw cycling	1.0	0.5	7.0	3.5
III.	Wet, hard - freeze, spring thaw	2.5	1.5	4.0	4.0
IV.	Dry, no freeze	0.0	0.0	4.0	8.0
V.	Dry, freeze - thaw cycling	1.0	0.5	3.0	7.5
VI.	Dry, hard freeze, spring thaw	3.0	1.5	3.0	4.5

Table 2.25. Suggested Seasonal Subgrade Resilient Modulus, E_{sg} (psi), as a Function of the Relative Quality of the Subgrade Material [1]

Relative Quality of Roadbed Soil	Seasonal Subgrade Modulus, E_{sg} (psi)			
	Winter (Roadbed Frozen)	Spring-Thaw (Roadbed Saturated)	Spring/Fall (Roadbed Wet)	Summer (Roadbed Dry)
Very Good	20000	2500	8000	20000
Good	20000	2000	6000	10000
Fair	20000	2000	4500	6500
Poor	20000	1500	3000	4900
Very Poor	20000	1500	2500	4000

Table 2.26. Effective Subgrade Resilient Modulus, E_{sg} (psi), That May be Used in Design of Flexible Pavements for Low Volume Roads [1]

U.S. Climatic Region		Effective Resilient Modulus E_{sg}, (psi)				
		Very Poor	Poor	Fair	Good	Very Good
I.	Wet, no freeze	2800	3700	5000	6800	9500
II.	Wet, freeze - thaw cycling	2700	3400	4500	5500	7300
III.	Wet, hard - freeze, spring thaw	2700	3000	4000	4400	5700
IV.	Dry, no freeze	3200	4100	5600	7900	11700
V.	Dry, freeze - thaw cycling	3100	3700	5000	6000	8200
VI.	Dry, hard freeze, spring thaw	2800	3100	4100	4500	5700

The AASHTO procedure does not specifically address seasonal variation in the base layer. At first glance seasonal variation in the base seems to be provided for by the drainage provision of modifying the base layer (with an m-value) depending upon how well the material drains and how long the base layer remains wet. This is adequate for considering the structural capacity of the entire pavement. However, in design of layer thickness the AASHTO Guide states:

It should be recognized that for flexible pavements, the structure is a layered system and should be designed accordingly. First, the structural number required over the roadbed soil should be computed. In the same way, the structural number required over the subbase layer and the base layer should also be computed, using the applicable strength values for each. [1]

The difficulty in considering seasonal variation in a subbase or base layer is in determining what the appropriate stiffness or strength value should be.

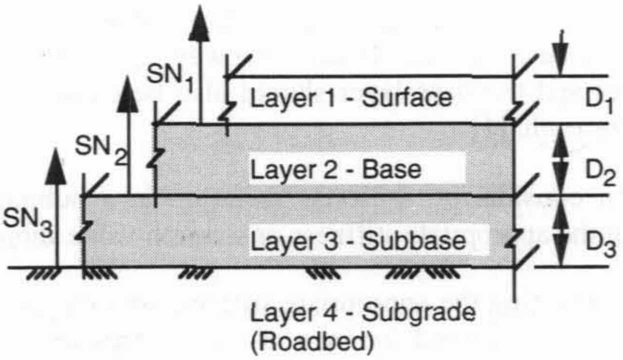
One approach in selecting the appropriate stiffness or strength value is to assume that seasonal variation is accounted for by a m-value. This approach modifies the layer coefficient which increases or decreases the structural capacity based on the drainage quality of the base material and the period of time the material nears saturation. A brief explanation of layer coefficients is discussed in following paragraphs.

When designing a pavement composed of a surface, base and subgrade the AASHTO procedure in effect requires the design of two pavements. The first design is a surface course to protect the base layer. The second pavement consists of a thickness which includes the base and surface layer to protect the subgrade (Figure 2.19). Each of these thicknesses are defined by the Structural Number (SN). The SN to protect the subgrade is:

$$SN = a_1 D_1 + a_2 D_2 \tag{14}$$

where SN = Structural Number
a_i = layer coefficient for i^{th} layer (structural value)
D_i = thickness of i^{th} layer

The SN is the thickness of a hypothetical material with a layer coefficient of 1.0 (originally termed the "Thickness Index"). The layer coefficient is an empirical number that relates the actual thickness of the layer to the SN. Typical layer coefficients used in the AASHTO Guide are 0.44 for asphalt, 0.14 for base, and 0.11 for subbase material. If for instance an asphalt layer had an SN requirement of 1.7 , and the layer coefficient was 0.42, then the thickness of the asphalt would be about 4 inches.



$$D_1^* \geq \frac{SN_1}{a_1}$$

$$SN_1^* = a_1 D_1^* \geq SN_1$$

$$D_2^* \geq \frac{SN_2 - SN_1^*}{a_2 m_2}$$

$$SN_1^* + SN_2^* \geq SN_2$$

$$D_3^* \geq \frac{SN_3 - (SN_1^* + SN_2^*)}{a_3 m_3}$$

* indicates value actually used which must be equal to or greater than the required value.

Figure 2.19. AASHTO Conceptual Flexible Pavement Layer Determination

Layer coefficients are chosen from charts provided by AASHTO. These charts provide correlation of strength or stiffness values such as CBR, R-value, or E with the appropriate layer coefficient (separate charts for asphalt, base, and subbase materials). The sketch shown as Figure 2.20 is used to illustrate the chart in the AASHTO Guide which provides correlations for granular base materials.

Modified layer coefficients are introduced into the SN equation to account for drainage conditions different from those experienced at the AASHTO Road Test. The SN with consideration of m-values is expressed as:

$$SN = a_1 D_1 + a_2 D_2 m_2 \tag{15}$$

The asphalt layer is not modified, as it is not assumed to be influenced by drainage conditions. Table 2.27 lists the recommended m-value ranges as shown in the AASHTO Guide.

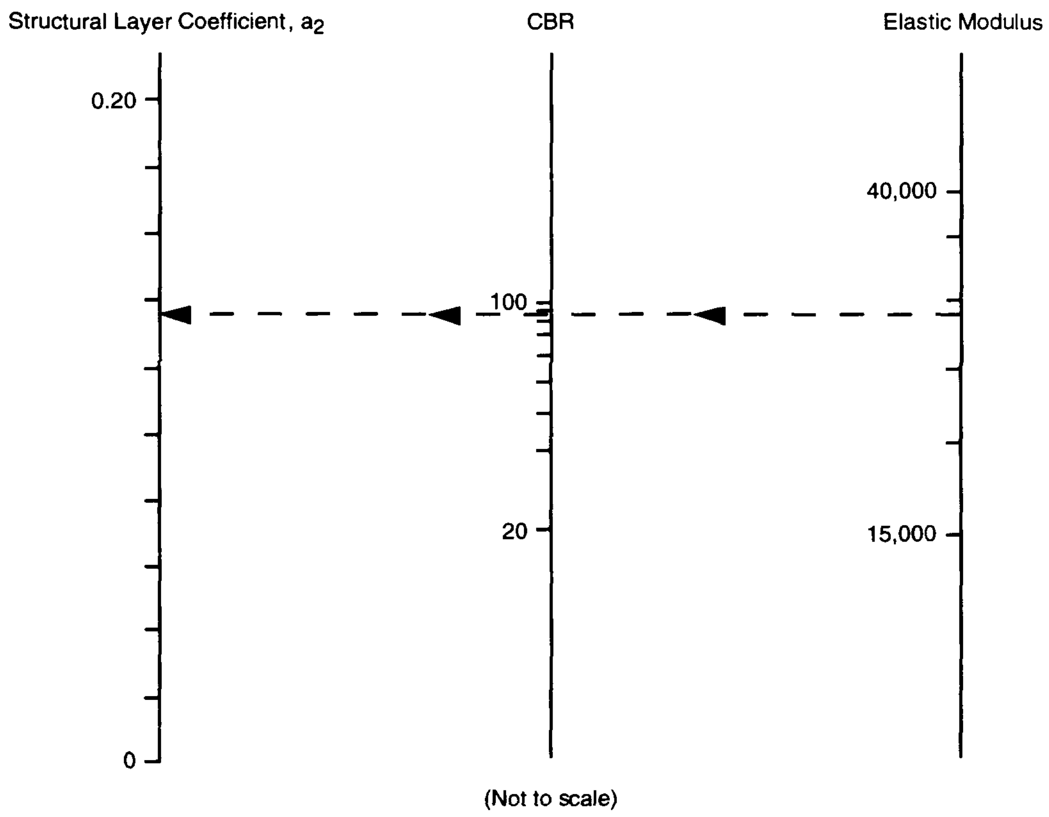


Figure 2.20. Sketch Illustrating the Determination of Structural Layer Coefficient for Base Materials [after Ref. 1]

Table 2.27. Recommended m-Values for Modifying Structural Layer Coefficients of Untreated Base and Subbase Materials in Flexible Pavements [1]

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	Less than 1%	1 - 5%	5 - 25%	Greater than 25%
Excellent	1.40 - 1.35	1.35 - 1.30	1.30 - 1.20	1.20
Good	1.35 - 1.25	1.25 - 1.15	1.25 - 1.00	1.00
Fair	1.25 - 1.15	1.15 - 1.05	1.00 - 0.80	0.80
Poor	1.15 - 1.05	1.05 - 0.80	0.80 - 0.60	0.60
Very Poor	1.05 - 0.95	0.95 - 0.75	0.75 - 0.40	0.40

Table 2.28. AASHTO Criteria for Selecting m-Values [1]

Quality of Drainage	Water Removed Within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very Poor	water will not drain

The AASHO Road Test is used as the standard when determining the m-values. Conditions at the AASHO Road Test were rated "fair" for quality of drainage and the m-value was set at 1.0. Engineering judgment is required for selection of quality and length of time the material remains wet. Table 2.28 provides criteria in selection of the quality of drainage. The m-values can be less than or greater than 1.0 with greater than 1.0 values indicating that drainage conditions are better than that of the AASHO Road Test materials. The difficulty in assigning m-values is that most users of the AASHTO Guide do not have a basis to compare materials with those of the AASHO Road Test.

To compare the effect of using modified layer coefficients consider the example shown in Tables 2.29, 2.30 and 2.31. Two base materials representing Good to Excellent

Table 2.29. Estimated Monthly Moisture Conditions for Base A and Base B

	Base A	Base B
Drainage Quality	Good - Excellent	Poor - Very Poor
Months Wet	1	3
Months Damp	5	3
Months Dry	6	6
Drainage m-value	1.2	0.7

Table 2.30. Estimated Resilient Modulus and Corresponding Layer Coefficients for Base A and B

Base Moisture Condition	Base A		Base B	
	Estimated Resilient Modulus (ksi	Layer Coefficient (a_2)	Estimated Resilient Modulus (ksi)	Layer Coefficient (a_2)
Wet	23.5	0.113	23.0	0.110
Damp	27.0	0.128	26.0	0.124
Dry	28.0	0.132	28.0	0.132

Table 2.31. Effect on Base Resilient Modulus in Using Drainage m-Values for Base A and B Conditions

Base A		Base B	
Drainage Consideration	Resilient Modulus E_{bs} (ksi)	Drainage Consideration	Resilient Modulus E_{bs} (ksi)
None	28.0	None	28.0
Modified Layer Coefficient $(a_2)(m_2)$=0.1584	35.0	Modified Layer Coefficient $(a_2)(m_2)$=0.0924	19.0

Notes: 1. Base A modified layer coefficient = (a_2)(m-value) = (0.132)(1.2) = .1584

2. Base B modified layer coefficient = (a_2)(m-value) = (0.132)(0.7) = .0924

(Base A) and Poor to Very Poor (Base B) drainage quality are shown. The moisture condition for Base A shows 1 month of wet, 5 months of damp and 6 months of dry materials. Materials for Base B show 3 months wet, 3 months of damp and 6 months of dry conditions. The m-value chosen for Base A is 1.2 based on the 1 month (8 percent of year) of saturated conditions. The m-values for Base B is 0.7 as the material is saturated 25 percent of the year.

Table 2.30 show estimated moduli based on engineering judgment and the seasonal moisture conditions represented by the sites. The dry season resilient modulus for Base A and B was selected as 28,000 psi. Layer coefficients corresponding to the estimated base moduli were selected from the AASHTO Guide and illustrated by Figure 2.20.

The base modulus to represent Base A and B for design was selected as the modulus that is most typical over a yearly period. By use of Tables 2.29 and 2.30, the dry season modulus (28,000 psi) occurs for 6 months for both bases. Multiplying the dry season layer coefficient (Table 2.31) by the m-value selected previously gives the modified layer coefficient. The original figure from the AASHTO Guide (as represented by the sketch shown as Figure 2.20) can then be used (converting from layer coefficient to moduli) to determine the resilient modulus that is represented by the modified layer coefficient. Table 2.31 summarizes the effect of applying m-values to base materials. For the same dry season base modulus (28,000 psi), Base A with improved drainage conditions is increased to 35,000 psi by use of a m-value. Base B has the opposite effect as a result of poor drainage conditions. The dry season modulus was reduced from 28,000 to 19,000 psi. In both instances a large increase or decrease is observed.

The reduction of the Base B resilient modulus means a thicker base will be required as opposed to a decreased base thickness for Base A. The AASHTO Guide does not directly address the possibility that thicker base sections do not necessarily solve water related problems such as drainage. [9]

Since the needed inputs for Table 2.27 (or Table 2.4 in the AASHTO Guide) may be difficult to determine or estimate, approximate levels of m-values were made a function of moduli ratios. These moduli ratios are the ratios of "seasonal" moduli to "summer" moduli. Two equations were developed from the original assumptions used by Seeds and Hicks [105] and m-values contained in Table 2.27 to relate layer modulus to m-value. These equations are for saturated layer conditions less than 25 percent of the time and greater than 25 percent (percentages taken on an annual basis). Further, the maximum base modulus used was 30,000 psi which is somewhat typical for as-constructed crushed stone bases. Layer moduli and m-values were regressed for the following data:

	m-value	
Base Modulus (psi)	Saturation < 25%	Saturation > 25%
30,000	1.0	0.8
20,000	0.7	0.6
10,000	0.4	0.4

The following equations result:

- For $E_{BS} \leq 30{,}000$ psi and saturated conditions less than 25 percent of the time

$$m = 0.1 + 0.00003\,(E_{BS}) \tag{16a}$$

- For $E_{BS} \leq 30{,}000$ psi and saturated conditions more than 25 percent of the time

$$m = 0.2 + 0.00002\,(E_{BS}) \tag{16b}$$

These equations (16a and 16b) can be used to develop m-values as a function of moduli ratios (using the basic assumption that the "summer" modulus is 30,000 psi) as follows:

	Approximate m-value	
Moduli Ratio	Time Saturated <25%	Time Saturated > 25%
1.00	1.00	0.80
0.95	0.96	0.77
0.90	0.91	0.74
0.85	0.86	0.71
0.80	0.82	0.68
0.75	0.78	0.65
0.70	0.73	0.62
0.65	0.68	0.59
0.60	0.64	0.56
0.55	0.60	0.53
0.50	0.55	0.50

Thus, the above m-values associated with a specific moduli ratio can be used to adjust the base course thickness.

As previously noted, the use of m-values to adjust unstabilized base and subbase layer thicknesses may not be an adequate solution where severe subsurface moisture problems are not corrected in the initial pavement design and construction process. Further, the approximate method described above for selecting an m-value should be used with caution and judgment. It is, at best, a very approximate method. Where saturated conditions exist for less than 5 percent of the time, Table 2.27 should be used.

4.1.2 SHELL METHOD

The Shell pavement design method is a mechanistic based procedure which can consider seasonal variation. The three failure criteria considered are vertical compressive strain at the top of subgrade, horizontal tensile strain at the bottom of asphalt, and permanent deformation of the asphalt layer. [72, 73] Design curves [3] which satisfy the failure criteria were developed.

The seasonal variation recognized by the Shell procedure is the temperature dependency of asphalt concrete and the resulting elastic modulus. The Shell method provides a procedure for converting Mean Monthly Air Temperatures (MMAT) to a Weighted Mean Annual Air Temperature (w-MAAT). The w-MAAT takes into account the variation in monthly temperature and is used to compute an effective asphalt modulus. The four temperatures considered are 39, 54, 68, 82 °F. The effective asphalt modulus depends upon both the temperature and the thickness of the asphalt concrete.

Any seasonal variation occurring in the base layer is largely a function of the subgrade layer. The relationship [74] used to model the base layer in development of design curves is provided by:

$$E_2 = 0.2 h_2^{0.45} E_3 \tag{17}$$

where E_2 = modulus of the unbound base layer,
E_3 = modulus of the subgrade layer, and
h_2 = thickness of the base layer.

As seen in the equation the base layer moduli is dependent on the subgrade moduli and the base thickness. The E_2/E_3 ratio has a limitation between two and four to limit tensile strains in the base layer. The design charts provide the minimum base or subbase moduli required for the design of a structural section.

Three subgrade resilient moduli were used to generate the design curves. The moduli considered represented a range of subgrade resilient modulus and included 3,600, 7,250, and 14,500 psi. The design procedure requires interpolation between curves if other subgrade moduli are required.

Using the design charts directly from the Shell Pavement Design Manual does not account for seasonal variation in subgrade moduli. To design for varying subgrade moduli that result from seasonal influences, a cumulative damage approach is required. Each season is separated into similar resilient moduli and then treated as a separate pavement design. Miner's hypothesis [75, 76] of fatigue damage accumulation is then used. Miner's hypothesis is expressed as:

$$\sum_{i=1}^{r} \frac{n_i}{N_i} = 1 \qquad (18)$$

where n_i = actual number of cycles of stress or strain applied to the pavement,

N_i = allowable number of cycles to failure based on failure criteria (such as fatigue or rutting), and

r = number of loading conditions considered.

For a proper design the total damage as expressed by Miner's hypothesis should be equal to one. If the value is greater than one, a thicker pavement is required.

4.1.3 ASPHALT INSTITUTE THICKNESS DESIGN PROCEDURE

The Asphalt Institute MS-1 design procedure [2] incorporated seasonal variation of the asphalt concrete, base course and subgrade layers into the design charts. Using the charts is relatively straightforward as the basic inputs to obtain a design thickness are, MMAT, traffic volumes, and design subgrade modulus.

To characterize asphalt concrete, the temperature dependency of the asphalt modulus was considered. This was accomplished by considering the climatic conditions of three temperature profiles representative of the United States. Table 2.32 shows both the MMAT and MAAT for New York, South Carolina, and Arizona. The mean annual air temperatures corresponding to each respectively are 45, 60, 75° F. The asphalt moduli used are based on the mean monthly air temperatures for a selected region and are calculated based on an the extensive study performed by the Asphalt Institute to model temperature and asphalt properties. [2, 16, 17]

Subgrade modulus is modeled by considering monthly variation in subgrade strength. Figure 2.21 shows the representation of yearly subgrade modulus with regard to normal, freezing, and thawing periods. During periods of freeze the resilient modulus was increased and during periods of thaw the resilient modulus was reduced. The Asphalt Institute design procedure does not specifically state what determines the normal period modulus. The normal period modulus appears to be represented by subgrade that is not frozen and has recovered from weakened conditions caused by thaw.

Table 2.32. Mean Monthly Air Temperatures (°F) Used in Asphalt Institute (MS-1) Design [16]

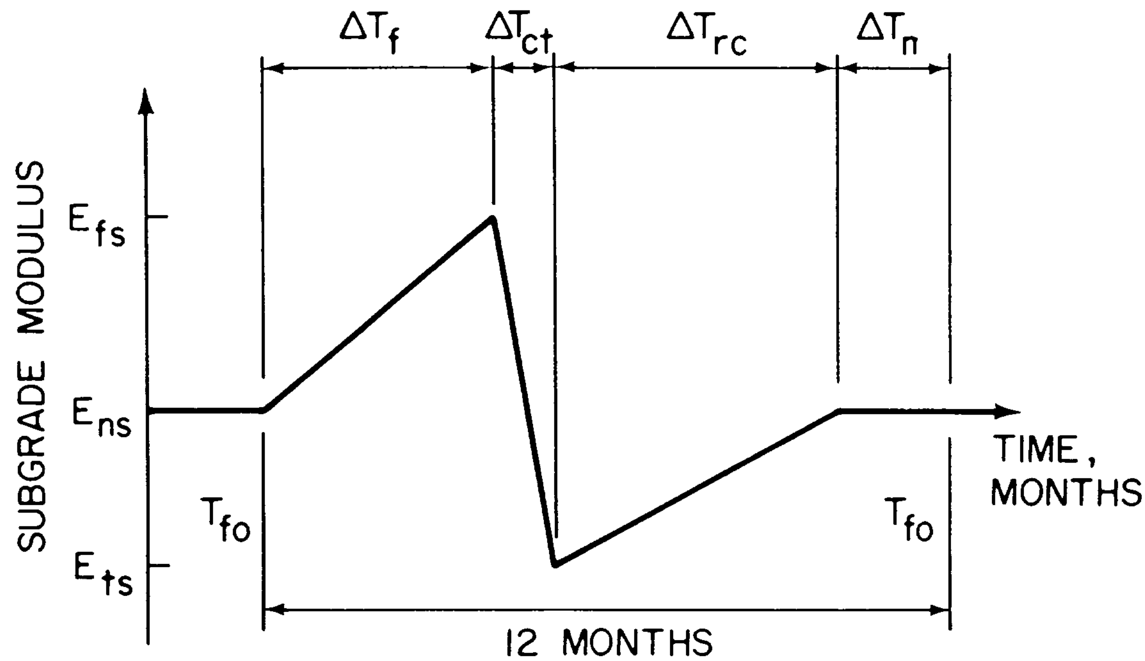
Mean Annual Air Temp (MAAT)	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec
45 °F - New York	24	25	14	27	42	48	61	69	65	55	48	41
60 °F - South Carolina	45	38	43	15	56	70	78	81	78	73	58	54
75 °F - Arizona	55	61	61	73	90	91	92	93	93	86	72	55

Table 2.33. Subgrade Conditions Used to Develop Asphalt Institute's (MS-1) Design Curves [16]

Mean Annual Air Temperature °F	Unfrozen E_{ns} (ksi)	Frozen E_{fs} (ksi)	Reduction r_t percent	E_{ts} (ksi)	T_{fo} Month Freeze Started	ΔT_f Months Frozen	ΔT_{ct} Months Thaw	ΔT_{rc} Months Recovery	ΔT_n Months Normal
45	4500	50000	20	900	Dec.	4	1	5	2
	12000	50000	50	6000	Dec.	4	1	5	2
	22500	50000	70	15800	Dec.	4	1	5	2
60	4500	50000	30	1350	Jan.	2	1	4	5
	12000	50000	60	7200	Jan.	2	1	4	5
	22500	50000	80	18000	Jan.	2	1	4	5

Note: Definitions for Asphalt Institute's nomenclature:

E_{ns} = normal subgrade modulus
E_{fs} = frozen subgrade modulus
r_t = thaw reduction factor
E_{ts} = thaw (reduced) subgrade modulus = $(r_t)(E_{ns})$
T_{fo} = month freeze started
ΔT_f = time of freeze
ΔT_{ct} = time of critical thaw
ΔT_{rc} = time of thaw recovery
ΔT_n = time-normal subgrade condition



$E_{ts} = r_t E_{ns}$

E_{fs} = Frozen subgrade modulus

E_{ns} = Normal subgrade modulus

E_{ts} = Thaw (reduced) subgrade modulus

r_t = Thaw reduction factor

T_{fo} = Month freeze started

ΔT_f = Time of freeze

ΔT_{ct} = Time of critical thaw

ΔT_{rc} = Time of thaw recovery

ΔT_n = Time-normal subgrade condition

Figure 2.21. Representation of Subgrade Stiffness (Modulus) Variations Throughout the Year [16]

The minimum subgrade modulus is determined by applying a thaw reduction factor to the normal period. The reduction factors and resulting thaw or frozen moduli as reported by Shook et al. [17] are shown in Table 2.33. A constant resilient modulus of 50,000 psi was assumed for freezing periods. Table 2.33 shows a range of assumed normal period moduli, the length in months for the freeze, thaw, normal and transition periods. Table 2.34 shows the corresponding subgrade moduli used for the 45 and 60 °F MAAT conditions. Frozen subgrade as late as March and April is indicated.

Base materials were adjusted similarly to those of subgrade. Base materials were considered to be stress sensitive and the range of monthly K_1 and K_2 values used are shown in Table 2.35.

The design charts were developed with the use of a computer program named DAMA. The program uses elastic layer theory to calculate critical tensile strains at the bottom of asphalt and vertical compressive strains at the top of subgrade. Using subgrade vertical tensile strain and asphalt tensile strain criteria [63, 77] the number of allowable load repetitions are determined. A cumulative damage approach is used to sum the damage obtained from the failure criteria. Cumulative damage is computed based on monthly traffic repetitions until a damage value of one is obtained.

The reliability in this design procedure comes in part from choosing the design subgrade modulus. The basic procedure is to adjust the subgrade modulus based on traffic levels. For greater traffic the subgrade modulus is reduced more than for lower traffic levels. Essentially this adjustment accounts for the variability in the range of resilient modulus that may be encountered during testing of the site conditions.

The design subgrade resilient modulus is chosen as the resilient modulus that is less than 60, 75, or 87.5 percent of all subgrade modulus test values in a given section. Traffic levels corresponding to these limits are as shown in Table 2.36. A specific example is shown in Tables 2.36, 2.37 and Figure 2.22. An explanation follows.

The subgrade moduli obtained by either nondestructive testing or laboratory testing (E, CBR, R-value) are first ordered in decreasing order. From the order the number equal to or greater than are listed for each test. Next the percent equal to or greater are computed. Example data is then plotted as the percent equal to or greater vs. resilient modulus as shown in Figure 2.22. The resilient design resilient modulus is then selected based on the projected traffic. The design subgrade modulus for each of the traffic levels are shown on Table 2.36.

The Asphalt Institute design charts are entered by using the design traffic ESALs and the environmental condition (MAAT) that applies to the region where the roadway is designed. Table 2.38 as provided in the Thickness Design Manual [2], is a guide in selecting the appropriate mean annual air temperature with respect to frost effects.

Table 2.34. Subgrade Moduli Used in Development of the Asphalt Institute's (MS-1) Design Curves [16]

Mean Annual Air Temperature	Normal Period[1] E_{ns}	Subgrade Modulus (by month)					
		Jan (ksi)	Feb (ksi)	Mar (ksi)	Apr (ksi)	May (ksi)	Jun (ksi)
45 (°F)	4.5	15.9	27.3	38.7	50.0	0.9	1.6
	12.0	21.5	31.0	40.5	50.0	6.0	7.2
	22.5	29.4	36.3	43.1	50.0	15.8	17.1
		Jul (ksi)	Aug (ksi)	Sept (ksi)	Oct (ksi)	Nov (ksi)	Dec (ksi)
	4.5	2.3	3.1	3.8	4.5	4.5	4.5
	12.0	8.4	9.6	10.8	12.0	12.0	12.0
	22.5	18.5	19.8	21.2	22.5	22.5	22.5
60 (°F)		Jan (ksi)	Feb (ksi)	Mar (ksi)	Apr (ksi)	May (ksi)	Jun (ksi)
	4.5	4.5	27.3	50.0	1.4	2.1	2.9
	12.0	12.0	31.0	50.0	7.2	8.4	9.6
	22.5	22.5	38.3	50.0	18.0	19.1	20.3
		Jul (ksi)	Aug (ksi)	Sept (ksi)	Oct (ksi)	Nov (ksi)	Dec (ksi)
	4.5	3.7	4.5	4.5	4.5	4.5	4.5
	12.0	10.8	12.0	12.0	12.0	12.0	12.0
	22.5	21.4	22.5	22.5	22.5	22.5	22.5

Note: 1. E_{ns} = normal period modulus

Table 2.35. Monthly Values for K_1 and K_2 Stress Sensitivity Coefficients for Granular Base [16]

Mean Annual Air Temperature	K_1 (normal)	Monthly Value for K_1 (10^3)											
		Dec	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov
45 (°F)	8000	8.0	12.0	16.0	20.0	24.0	2.0	3.2	4.4	5.6	6.8	8.0	8.0
	12000	12.0	18.0	24.0	30.0	36.0	3.0	4.8	6.6	8.4	10.2	12.0	12.0
60 (°F)	8000	8.0	8.0	16.0	24.0	2.0	3.5	5.0	6.5	8.0	8.0	8.0	8.0
	12000	12.0	12.0	24.0	36.0	3.0	5.3	7.5	9.8	12.0	12.0	12.0	12.0

Notes: 1. $E_{bs} = K_1\ \theta^{K_2}$

2. K_2 Value is .5 and E_{bs} and θ are in psi units

Table 2.36. Subgrade Design Traffic Levels, Design Subgrade Value Percent, and Design Subgrade Modulus Example

Equivalent Axle Loads (18,000 lb)	Design Subgrade Value Percent	Example Design Subgrade[1] E_{sg} (psi)
10^4 or less	60.0	9.7
10^4 to 10^6	75.0	8.4
10^6 or more	87.5	7.4

[1]Refer to Table 2.37 (example data) and Figure 2.22 (plot of example data)

Table 2.37. Example of the Determination of Design Subgrade Modulus for the Asphalt Institute (MS-1) Design (data plotted in Figure 2.22)

Subgrade Test Values E_{sg} (ksi)	Number Equal to or Greater Than	Percent Equal to or Greater Than Percent
18.2	1	13
16.5	2	25
12.3	3	38
9.6	4	50
9.5	5	63
9.3	6	75
7.0	7	88
6.9	8	100

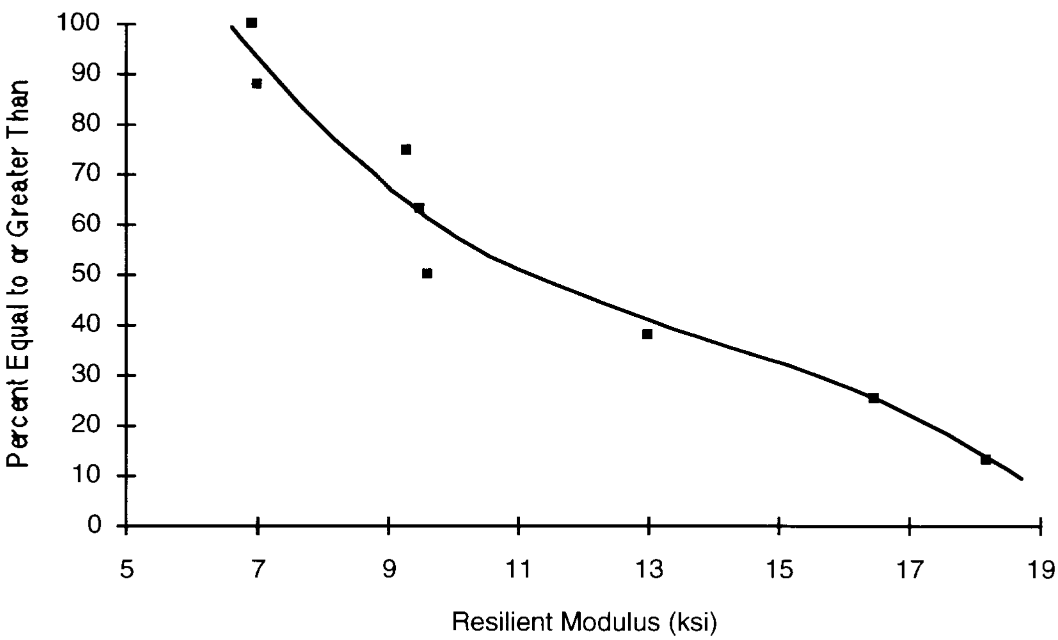


Figure 2.22. Plot to Determine Design Subgrade by the Asphalt Institute Thickness Design Procedure (from example data shown in Table 2.37)

Table 2.38. Environmental Conditions for Selecting a MAAT for Asphalt Institute (MS-1) Design [2]

Mean Annual Air Temperature	Frost Effects
45 °F	Yes
60 °F	Possible
75 °F	No

4.2 OVERLAY DESIGN PROCEDURES

4.2.1 ASPHALT INSTITUTE DEFLECTION PROCEDURE

The Asphalt Institute Deflection Method as described in [5] is a empirical procedure that relates pavement deflection to performance. The basic procedure involves taking a recommended 20 deflection measurements per mile using a Benkelman Beam placed between dual tires for an 18,000 lb. single axle load. Data is reduced to obtain a representative rebound deflection (RRD) and is described by:

$$RRD = (\bar{x} + 2s)(f)(c) \tag{19}$$

where RRD = representative rebound deflection (in),

$\bar{x}$ = mean of the individual deflections (in),

s = standard deviation of the deflections (in),

f = temperature adjustment factor, and

c = critical period adjustment factor (where c = 1 if deflection tests made during the most critical period (highest pavement deflections)).

In an area subject to freeze/thaw the most critical deflection will likely occur in the spring although Madden [79] suggests this in not always the case. Areas that do not experience freezing conditions may experience the maximum deflection when rainfall is the greatest. The "c" value allows adjustment for the most critical period.

Although the Asphalt Institute design may give reasonable design thicknesses, there are instances where single deflection measurements may not fully describe the weakened condition of a pavement. This can occur, for example, when a thawed base course overlies a frozen subgrade. Even though the pavement surface deflection may be low the tensile strains at the bottom of the asphalt concrete surface may be quite high (hence the potential for fatigue cracking). This was noted in work performed in Alaska by Stubstad and Connor [80]. A single deflection measurement as used in most deflection design procedures does not allow separation of the factors influencing deflection such as thickness of pavement layers, subgrade material strength, or environmental effects. [81]

Today, the Benkelman Beam has been largely replaced by the Falling Weight Deflectometer (FWD) (at least in the U.S.). If an FWD is used, the measured center deflection (D_O) must be converted to an equivalent Benkelman Beam reading. Several agencies have developed correlations between different nondestructive devices. [9, 32] Smith cautions that any correlation made must be developed based on an agency's own test procedures, soil types, environment, pavement sections, pavement layer thicknesses and layer moduli to be valid. [32]

4.2.2 Asphalt Institute Component Analysis

Asphalt Institute's Component Analysis [5], also known as effective thickness procedure, combines the effects of traffic loading, pavement structure, and subgrade conditions to arrive at an overlay design thickness. The procedure does not give guidance in selecting seasonal input. Seasonal input into the design process can be provided in two areas as is discussed below.

The first seasonal input involves the determination of the stiffness properties of the subgrade. Stiffness properties are found by either laboratory or nondestructive testing. If test results are not available the Asphalt Institute gives three classes of soils in which to characterize subgrade soils (shown in Table 2.39). The Asphalt Institute's procedure does not mention seasonal consideration but engineering judgment can be used to select a subgrade condition that might best represent average conditions.

Table 2.39. Subgrade Soil Classification for Asphalt Institute Component Analysis [5]

Subgrade Class	Description	Typical Strength Values		
		Resilient Modulus	CBR	R-Value
Poor	Soils with appreciable amounts of silts and clays, soft and plastic when wet	4.5 ksi	3	6
Medium	Soils such as loam, silty sands, and sand gravels, contains moderate amounts of clay and fine silts, soils retain a moderate degree of firmness under adverse moisture conditions	12.0 ksi	8	20
Good	Soils include clean sands and sand gravels, these soils are not affected by moisture and frost and do not lose strength when wet	25.0 ksi	17	43

The second input involves the determination of an effective thickness of the existing pavement structure. To arrive at an effective thickness, each asphalt, base, or subbase layer is converted to an equivalent thickness of new asphalt. This is accomplished by assigning weighting factors to the separate layers in terms of "new" asphalt concrete. These weighting factors depend upon the general condition of the layer being converted to an equivalent thickness. Engineering judgment can be used to assign a factor at the lower end of the range if base course materials are saturated for periods of the year. The equivalent thickness will be reduced allowing for a thicker overlay.

4.2.3 WASHINGTON STATE DEPARTMENT OF TRANSPORTATION [4]

WSDOT uses an mechanistic-empirical overlay design computer program named EVERPAVE. [4] The program uses a cumulative damage approach based on the failure criteria of horizontal tensile strain at the bottom of asphalt bound layer and vertical compressive strain at the top of subgrade. The seasonal change in pavement materials is provided for by applying seasonal adjustment factors to the base course and subgrade moduli. The temperature dependency of the asphalt concrete allows adjusting the asphalt modulus according to seasonal temperature and Bu-bushait's [11] WSDOT Class B stiffness-temperature relationship. The EVERPAVE program is diagrammed in Figure 2.23.

Previous research by Mahoney, et al. [65] has shown the base and subgrade materials in Washington State tend to display distinct moduli ratios for wet and dry seasons. Eastern Washington tends to display a hot and cold season while western Washington is wet and mild. Moduli ratio values estimated for Western Washington and Eastern Washington are shown in Table 2.19. The moduli ratios are based on FWD deflections obtained over a three-year period (Spring 1985 to Spring 1988). The summer moduli was selected as the reference value (1.0). The seasonal factors reflect more seasonal variation in the base than the subgrade layer. Use of the factors are estimates and require engineering judgment with their application as seasonal variation is site specific.

The unstabilized base course and subgrade moduli can be non-linear or linear as the unbound layer moduli are determined by the stress sensitivity relationships:

$$E_{bs} = K_1 \theta^{K_2} \qquad (20)$$

or

$$E_{sg} = K_3 \theta^{K_4} \text{ or } K_3 \sigma_d^{K_4}$$

where E_{bs} = resilient modulus of base course material, and

E_{sg} = resilient modulus of subgrade material.

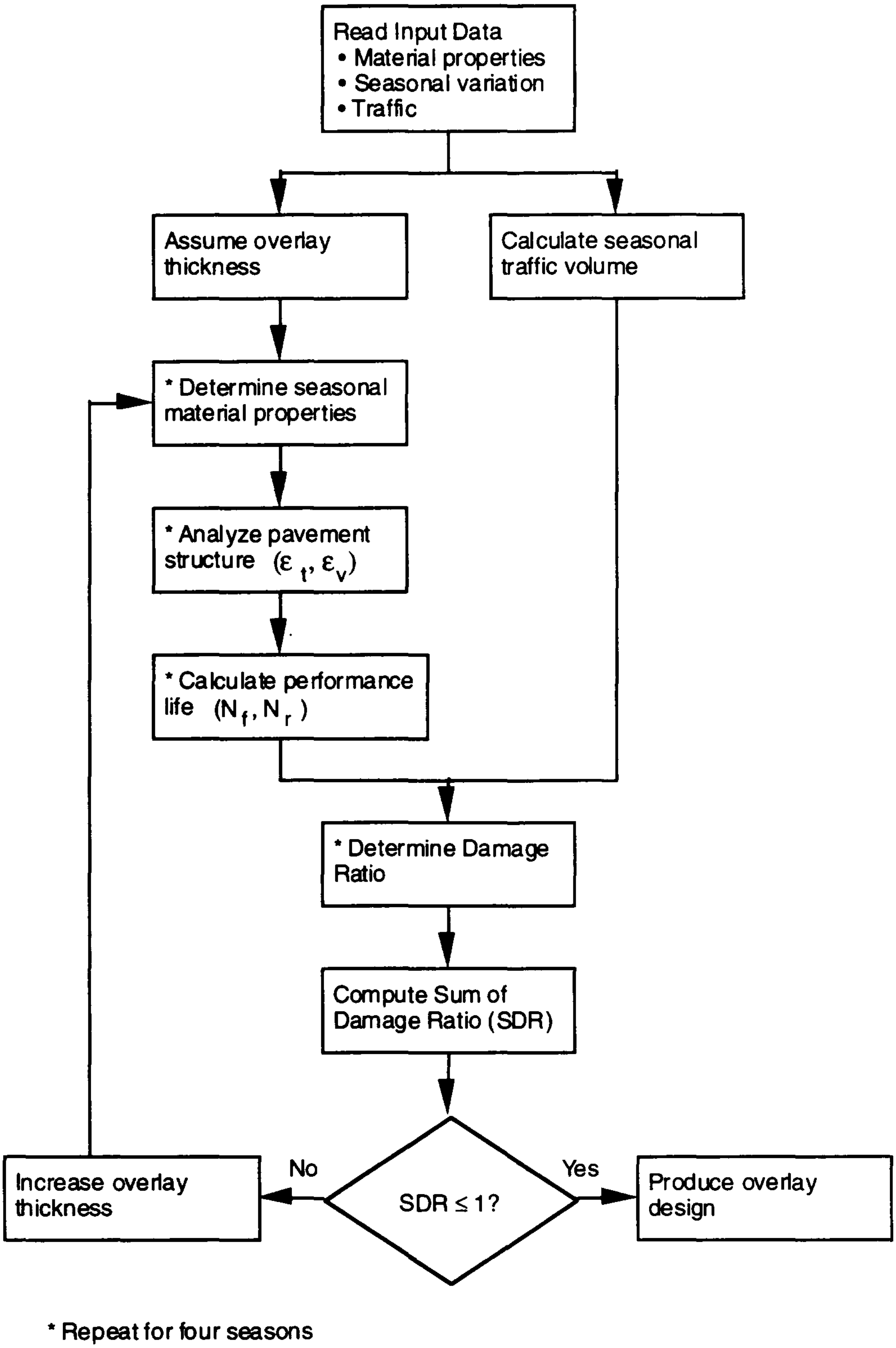


Figure 2.23. WSDOT Overlay Design Flow Chart [9]

The K_1, K_3, and K_2, K_4 values are chosen to characterize the base or subgrade layers based on field (FWD) or laboratory conditions. By applying seasonal adjustment factors to the moduli determined by the preceding relationship, seasonal moduli for distinct periods are determined. The seasonal periods in months is required input for the EVERPAVE program.

4.2.4 REVISED AASHTO OVERLAY DESIGN PROCEDURE

The AASHTO overlay design procedure has been revised and is contained in the 1993 version of the Guide. Seasonal variation for subgrade materials can be handled as discussed in Section 4.1.1.

4.3 FROST DESIGN CONSIDERATIONS

4.3.1 INTRODUCTION

Much of the U.S. has winter temperatures which are low enough to cause ground freezing in pavement structures. One of the assumptions associated with the development of the Asphalt Institute MS-1 [17], for example, uses a mean annual air temperature (MAAT) of 60°F and lower to indicate seasonal layer moduli changes (refer to Tables 2.32 through 2.35). A sample of MAAT's for various U.S. cities is shown in Table 2.40. It is likely that areas above 35° North Latitude need to consider frost effects. Naturally, due to elevation differences, this may vary substantially for USFS roads.

This section will introduce the basic issues associated with frost effects on pavements and some of the design treatments which have been used to deal with such effects.

4.3.2 FROST ACTION PROCESSES

Frost action refers to two separate but related processes: (a) frost heaving resulting mainly from accumulation of moisture (ice lenses) in the soil during the freezing period (note: ice lenses form perpendicular to the direction of heat flow), and (b) thaw weakening of soil when thawing temperatures occur. The conditions necessary for frost heave to occur are

- subfreezing temperatures,
- water, and
- frost susceptible soil.

Remove any of the three conditions above and frost effects will be eliminated or at least minimized. If the three conditions occur uniformly, heaving will be uniform; otherwise, differential heaving will occur resulting in pavement cracking and roughness.

Table 2.40 A Sample of Mean Annual Air Temperatures for Various U.S. Cities

Location	MAAT (°F)
Anchorage, AK	36
Fairbanks, AK	26
Phoenix, AZ	70
San Diego, CA	62
San Francisco, CA	57
Washington, D.C.	56
Miami, FL	75
Atlanta, GA	61
Boise, ID	51
Chicago, IL	50
Boston, MA	50
Minneapolis, MN	45
Santa Fe, NM	49
Las Vegas, NV	64
Oklahoma City, OK	60
Portland, OR	53
Nashville, TN	60
Dallas, TX	66
El Paso, TX	64
Houston, TX	69
Salt Lake City, UT	51
Seattle, WA	52
Spokane, WA	49

4.3.2.1 Frost Heave

Frost heaving of soil is caused by crystallization of ice within the larger soil voids and usually a subsequent extension to form continuous ice lenses, layers, veins or other ice masses. An ice lens grows in thickness in the direction of heat transfer until the water supply is depleted or until freezing conditions at the freezing interface no longer support further crystallization. Ice segregation occurs primarily in soils containing fine particles (i.e., frost susceptible). Clean sands and gravels are non-frost susceptible (NFS). The amount of frost susceptibility is mainly a function of the percentage of fine particles (more on this later). Figure 2.24 illustrates the formation of ice lenses in a frost susceptible soil. Tabor, in 1930 [96], recognized that frost heaving required substantially more water than was naturally available in the soil pores (characterized as "moisture content"). He noted:

> "The average soil seldom contains as much as 50 percent water, but if all the water in such a soil were to freeze in situ, the change in volume could cause an uplift of less than 5 percent of the depth of freezing. The depth of freezing in the colder parts of the United States seldom exceeds 2 or 3 feet; yet a surface heaving of 6 inches is not uncommon and an uplift of a couple of feet has been reported."

Figure 2.24 illustrates the important role capillary water "plays" in frost heaving.

The capillary rise of water can be substantial, up to 20 ft or more. The potential capillary rise can be estimated by the following:

$$h_c = \frac{C}{(e)\,(D_{10})}$$

where h_c = capillary rise (cm),

e = void ratio

D_{10} = soil particle size, 10 percent finer passing (cm), and

C = constant which can range from 0.1 to 0.5 cm^2

Thus, the smaller the soil grain size, the greater the potential for vertical water movement. Silty soils present the greatest problem. A quote by Lobacz, et al. [97] further illustrates the serious nature of capillary rise:

> "A potentially troublesome water supply for ice segregation is present if the highest ground water table at any time of the year is within 5 ft of the proposed subgrade surface or the top of any frost-susceptible base materials used. When the depth to the uppermost water table is in excess of 10 ft throughout the year, ice segregation and frost heave may be expected to be reduced."

Note that Lobacz et al. stated that a water table with a depth greater than 10 ft only <u>reduces</u> the potential for ice lenses.

4.3.2.2 Thawing

Thawing can proceed from the top downward, or from the bottom upward, or both. How this occurs depends mainly on the pavement surface temperature. During a sudden spring thaw, melting will proceed almost entirely from the surface downward. This type of thawing leads to extremely poor drainage conditions. The frozen soil beneath the thawed layer can trap the water released by the melting ice lenses so that lateral and surface drainage are the only paths the water can take.

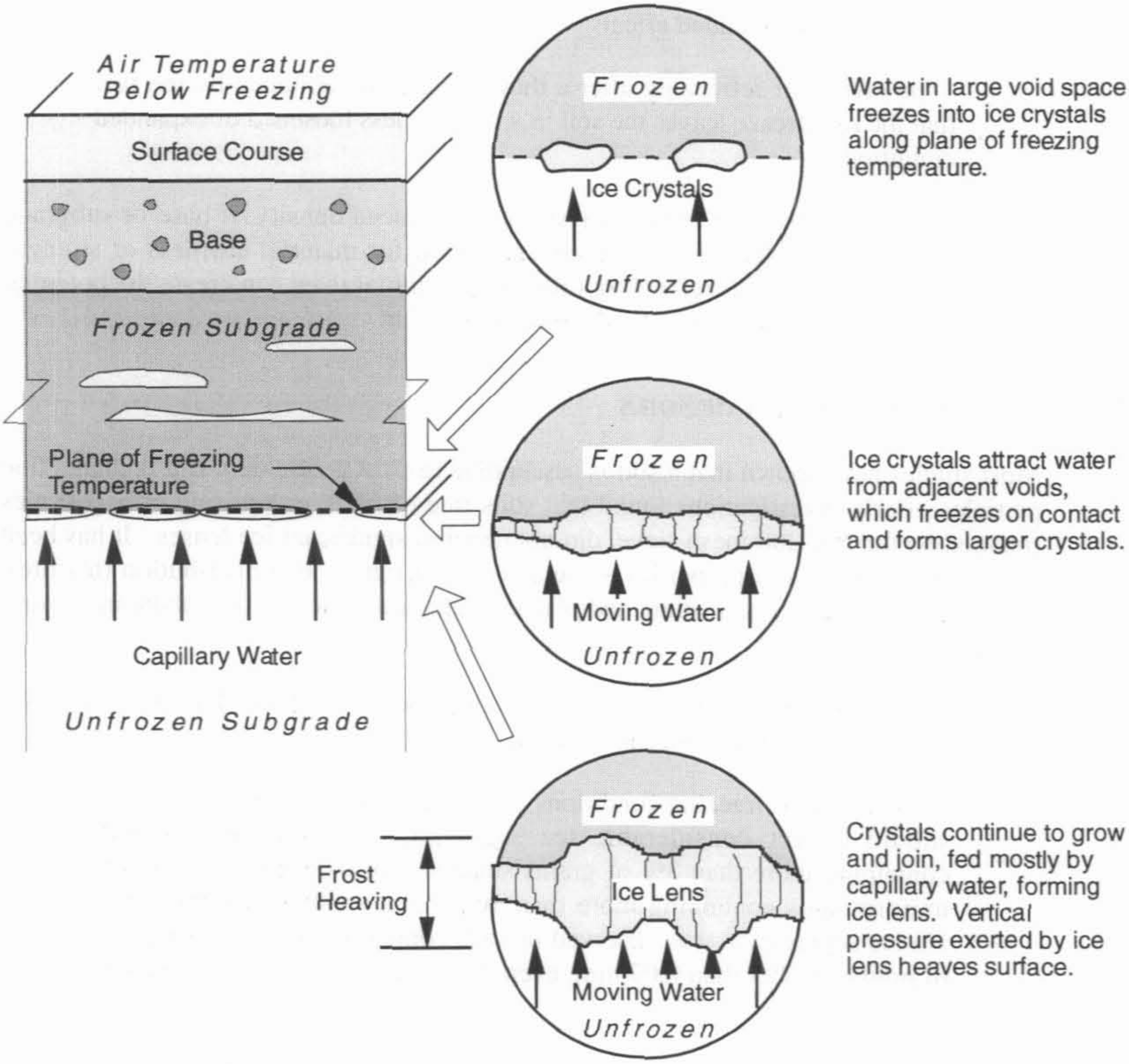


Figure 2.24. Formation of Ice Lenses in a Pavement Structure

The loss of bearing capacity during "spring" thaw periods is one of the most serious problems associated with frost action. The usual pattern of seasonal variation in base and subgrade support includes (usually) a significant increase from "normal" summer/fall values during the time the base and subgrade is frozen. Thawing produces a rapid decrease to levels below the summer/fall values, followed by a gradual recovery over a period of weeks (or months).

Tabor [96] also noted an added effect:

> "The effects of refreezing after a thaw are also accentuated by the fact that the first freeze leaves the soil in a more or less loosened or expanded condition."

This is helpful information in two ways: (1) the reduced density of base or subgrade materials helps to explain the long recovery period for material stiffness or strength following thawing, and (2) refreezing following an initial thaw can create the potential for greater weakening when the "final" thaw does occur.

4.3.3 FROST SUSCEPTIBILITY OF SOILS

Most studies have shown that a soil is susceptible to frost action only if it contains fine particles. Early investigations found that soils free of fines, comprising only particles retained on the No. 200 mesh sieve, did not develop significant ice lenses. It has been observed that other soil properties — such as overall grain size distribution (texture), grain shape, mineral composition, and plasticity characteristics — contribute in varying amounts.

Casagrande in 1932 proposed the following widely known rule-of-thumb criterion for identifying potentially frost susceptible soils:

> "Under natural freezing conditions and with sufficient water supply one should expect considerable ice segregation in non-uniform soils containing more than 3% of grains smaller than 0.02 mm, and in very uniform soils containing more than 10 percent smaller than 0.02 mm. No ice segregation was observed in soils containing less than 1 percent of grains smaller than 0.02 mm, even if the groundwater level is as high as the frost line."

Application of the Casagrande criterion requires a hydrometer test of a soil suspension (in water) to determine the distribution of particles passing the No. 200 sieve and to compute the percentage of particles finer than 0.02 mm.

The Corps of Engineers frost design classification system was developed in the late 1940s to make use of the Casagrande criterion regarding frost susceptibility and to account for the reduced stability of the various types of frost susceptible soils during the thaw-weakened period.

In the current system frost susceptible soils (with 1.5 to 3 percent or more, by weight, finer than 0.02 mm) are classified into one of seven groups, PFS, S1, S2, F1, F2, F3, or F4, for frost design purposes. Soil types are listed in Table 2.41 in approximate order of increasing susceptibility to frost heaving and/or weakening as a result of frost melting. The basis for distinction between the F1 and F2 groups is the F1 material may be expected to show higher bearing capacity than F2 material during thaw, even though both may have experienced equal ice segregation. The F3 and F4 soils, grouped together for reduced strength design, show the greatest weakening during thaw.

Table 2.41. Corps of Engineers Frost Design Soil Classification and USCS Equivalent Grouping (after CRREL Special Report 83-27)

Frost Group	Soil Type	Percentage finer than 0.02 mm by weight	Typical soil types under Unified Soil Classification System
Possibly Frost Susceptible	(a) Gravels Crushed stone Crushed rock	1.5-3	GW, GP
	(b) Sands	3-10	SW, SP
S1	Gravelly soils	3-6	GW, GP, GW-GM, GP-GM
S2	Sandy soils	3-6	SW, SP, SW-SM, SP-SM
F1	Gravelly soils	6 to 10	GW, GP, GW-GM, GP-GM
F2	(a) Gravelly soils	10 to 20	GM, GW-GM, GP-GM
	(b) Sands	6 to 15	SM, SW-SM, SP-SM
F3	(a) Gravelly soils	>20	GM, GC
	(b) Sands, except very fine silty sands	>15	SM, SC
	(c) Clays, Pl > 12	–	CL, CH
F4	(a) All silts	–	ML, MH
	(b) Very fine silty sands	>15	SM
	(c) Clays, Pl < 12	–	CL, CL-ML
	(d) Varved clays and other fine-grained, banded sediments	–	CL, ML, and SM; CL, CH, and ML; CL, CH, ML, and SM

4.3.4 SUMMARY OF AGENCY PRACTICE

4.3.4.1 NCHRP Survey

A 1993 summary on state pavement design practices relative to "frost heave" was prepared by Forsyth [98]. The question posed by Forsyth's survey was: "Does your state's flexible [or rigid] pavement design procedure include consideration of serviceability loss due to frost heave?"

The following selection of responses was noted.

State	Response
Alaska	Control minus No. 200 sieve size material to a depth of 42 in.
Arizona	Judgment
Georgia	AASHTO (72) Regional Factor
Illinois	Top foot of subgrade chemically modified or replaced
Maine	Minimum of 36 in. of pavement and gravel based on degree days
Massachusetts	Increase Structural Number
Michigan	Replace to a depth of 5 ft from pavement surface
Minnesota	Blend frost susceptible soils to frost depth 4 to 6 ft
Nebraska	AASHTO (86)
New Mexico	AASHTO (72) Regional Factor
Ohio	Frost susceptible material removed to a depth of 3 ft from the pavement surface
Utah	Remove and replace frost susceptible material or increase pavement thickness
West Virginia	AASHTO 86
Ontario	Increase base and subbase thickness based on frost susceptibility of subgrade soils

Thus, various states and provinces responding to Forsyth's question have a variety of approaches. Clearly, a commonly used technique is to remove or modify frost susceptible materials to some preset depth from the pavement surface. Many of those depths reported appear to approach the expected depth of freeze, but certainly not all. Another way to view this is that the pavement structure is increased to ensure that frost susceptible materials are at some acceptable depth as measured from the pavement surface. Several states use the process described in the 1993 AASHTO Guide, or one of its earlier versions (the AASHTO Regional Factor (pre-1986 Guide)).

4.3.4.2 AASHTO Guide

The AASHTO Guide (1993) contains a treatment dealing with frost heave in pavement design. The goal is to estimate the differential effects on the road profile and ultimately to estimate the heaves residual effects on the Present Serviceability Index (PSI). Thus, the decrease in PSI with time due to frost effects is "overlayed" onto the loss of PSI due to ESALs.

4.3.4.3 Corps of Engineers Procedure

Since World War II, the Corps of Engineers has developed pavement design procedures (street and airfield) which can be used to develop structural design requirements. The available design procedures for pavements subject to freezing and thawing in the underlying soils are based on two basic concepts (Lobacz et al.):

- Control of surface deformation resulting from frost action.
- Provision for adequate bearing capacity during the most critical climate period.

Based on the above concepts, three separate design approaches can be used:

- Complete Protection Method

  Sufficient thickness of pavement and non-frost susceptible base course is provided to prevent frost penetration into the subgrade.

- Limited Subgrade Frost Penetration Method

  Sufficient thickness of pavement and non-frost susceptible base course is provided to limit subgrade frost penetration to amounts which restrict surface deformation to within acceptable, small limits.

- Reduced Subgrade Strength Method

  The amount of frost heave is neglected and the design is based primarily on the anticipated reduced subgrade strength during the thaw period.

Yoder and Witczak [2.46] noted in reference to the three design approaches mentioned above:

> "...design of highway pavements should be based generally on the reduced subgrade strength design method, with additional thickness (based on local field data and experience) used where necessary to keep pavement heave and cracking within tolerable amounts."

The design period traffic is developed in terms of 18,000 lb single-axle loads. By use of design charts in Lobacz et al. [97] and traffic information in Yoder and Witczak [99] and the National Stone Association [100], the following thicknesses were developed from the COE design charts:

Flexible Pavement—Combined Thickness of Surface Course and Non-Frost Susceptible Base, in.

Traffic [1,2]		COE Subgrade Frost Group[3]		
COE Design Index	Upper Limit of ESAL Range	F1	F2	F3 and F4
DI-1	1,825/yr	9	10	16
DI-2	7,300/yr	10	12	19
DI-3	27,375/yr	12	14	22
DI-4	91,250/yr	13	16	25
DI-5	328,500/yr	14	18	28
[4] DI-6	1,095,000/yr	16	19	30

Notes:
1. Assumes a life expectancy of 20 years.
2. ESAL's can be estimated using AASHTO LEFs.
3. Frost Groups described in Paragraph 4.3.3.
4. Higher Design Indices are available (up to DI-10), with a maximum combined thickness of 42 in. for F3 and F4 Frost Groups.

There are additional requirements on the base course to meet all design requirements (drainage, etc.).

4.3.4.4 Capillary Break

One fundamental way to reduce frost action in a pavement is to stop (or reduce) the available water from forming ice lenses or otherwise saturating the upper layers of the pavement structure. Tabor commented on this in 1930 [96]:

> "The troubles resulting from the formation of segregated ice under pavements can be entirely prevented if, in addition to the usual methods of draining, a thick layer of coarse material is introduced under the pavement extending down to the extreme depth of frost penetration."

This concept has been applied by the Idaho DOT as reported by Mathis [101]. He noted a number of features used in northern Idaho to reduce frost action. The primary element is to use a "rock cap" layer immediately on top of the prepared subgrade to intercept the flow of water. A primary concern was to intercept water entering the pavement section through the surface course(s) as well as lower water sources. Mathis noted:

- The rock cap material is open-graded, with typically 100 percent passing the 3 in. sieve and 0 to 5 percent passing the 0.75 in. sieve. When placed on fine-grained subgrade soils, a geotextile is used as a separator (a filter layer could be used in lieu of a geotextile).

- Apparently, two separate design concepts have been used. One uses a thick rock cap layer with asphalt concrete layers applied directly to the rock cap. The other approach places a dense aggregate base on top of the rock cap. Asphalt concrete is then placed on the dense base to complete the pavement section. Rock cap thicknesses (as reported by Mathis) have ranged from 2.67 ft to 1.0 ft.

- Material properties: Backcalculated material properties (layer moduli) for the rock cap layer range from 25,000 to 60,000 psi. Significantly, the Idaho DOT has not observed significant seasonal change in these moduli. Further, for structural design purposes, the rock cap material is substituted on a 1:1 basis for untreated base.

- Construction: The rock cap material should be 100 percent crushed material for constructibility purposes. Thick layers of such material are inherently unstable, requiring special construction techniques.

- The cost of the rock cap material was reported by Mathis as being about three times less expensive than aggregate base ($2.50/ton vs. $9.00/ton as reported in 1991). Performance data is limited since the earliest rock cap project was built a bit over 10 years ago (1981).

In 1973, Johnson [102] reported on a survey of North American DOT practices with respect to roadway design in seasonal frost areas. He noted that the State of Maryland used a 12 in. "granular cap" over frost susceptible subgrade soils. A CBR of 7 was assigned to such layers. Johnson also noted that Maryland has the option of stabilizing frost susceptible subgrades with cement. The states of Maine and Nebraska were reported as undercutting frost susceptible subgrades with the undercut material being replaced by granular fill.

4.3.4.5 Other Thickness Considerations

A survey conducted during 1985 [103] revealed the following from several "northern" states:

Agency	Use of Frost Protection in Thickness Design
• Alaska DOT	• More than 50 percent but not full
• Maine DOT	• More than 50 percent but not full
• Montana DOT	• Frost protection not included in design
• North Dakota DOT	• Frost protection not included in design
• Oregon DOT	• More than 50 percent but not full
• Washington DOT	• Depth > 50 percent of maximum frost depth expected

Thus, SHAs such as Alaska, Maine, Oregon, and Washington use knowledge about expected frost depths in the design process. Presumably, limiting the depth of frost into the subgrade soils limits, adequately, the potential for frost heave and thaw weakening for most projects/locations.

The above percentages (pavement structural section as a percentage of expected frost depth) are further reinforced by Japanese practice. Kono et al. [104] reported in 1973 that on the island of Hokkaido the pavement structure is set at 70 percent of the expected frost penetration (the pavement materials are non-frost susceptible).

In general, a number of highway agencies increased the total depth of the pavement structure to meet some percentage of the anticipated depth of freeze ranging from 50 to 100 percent.

4.3.4.6 Other Design Considerations

Pavement designs for frost areas should consider past pavement performance in the vicinity of the project in developing the pavement section. Further, the designer should consider items such as:

- The need for a capillary break such as the rock cap layer used by the Idaho DOT.
- The gradation of all materials used in the pavement section which relates to frost susceptibility (recall "high" percent fines passing the No. 200 sieve can make a material (even crushed stone) frost susceptible).
- The anticipated seasonal changes in unstabilized materials (stiffness and/or strength).
- The need for positive subsurface drainage.
- The depth to saturated layers or the water table.
- The anticipated depth of freeze must be considered.
- Removal of highly frost-susceptible materials down to the expected depth of frost.
- Modify high frost-susceptible materials by adding granular material.
- Various combinations of treatments can be considered.

CHAPTER 3

EXAMINATION OF NDT DATA FOR SEASONAL VARIATION IN JAPAN

1. INTRODUCTION

This chapter is a case study of the nondestructive data obtained from a recent Japanese investigation into the seasonal variation in the bearing capacity of pavements. [82] The case study was selected for two reasons: (1) it contained characterization of the seasonal variation of pavement materials on a weekly basis for over a year (which is difficult data to find), and (2) based on the backcalculated data, definite seasonal variation is seen in the layer moduli. Large increases and decreases in subgrade and base course moduli were observed during freeze/thaw conditions. Figure 3.1 illustrates the type of seasonal variation observed in the pavement material moduli during the test period. The reported study provided the deflection data necessary to compare the Japanese results with pavement moduli backcalculated from another backcalculation program.

The test pavement used in the study is located on the campus of the Hokkaido Institute of Technology, Sapporo, Hokkaido, Japan (Sapporo is about the same latitude as Crater lake, Oregon (43° N)). Constructed in 1988, the pavement structure consists of 3.2 inches of asphalt concrete and 7.9 inches of crushed stone granular base placed on a silty sand subgrade. Figure 3.2 illustrates the pavement test section, lists some of the measured material properties, and also shows the location of 11 thermocouples placed in the test pavement to monitor the distribution of temperature and frost penetration (refer to Appendix A for actual FWD and temperature measurements). Special drainage provisions for the test pavement were not provided because the silty sand subgrade is relatively permeable to water. [83]

FWD deflection data was obtained with a Phoenix FWD model PT 5002. Deflection basins were measured with sensor spacings of 0, 11.8, 23.6, 35.4, 47.2, and 78.7 inches. The weekly deflection measurements were taken at exactly the same location from November 1989 to January 1991. The 16 foot by 200 foot test section was not subjected to traffic.

Average weather conditions for Sapporo were encountered for the testing period. [83] Table 3.1 summarizes both the monthly precipitation and average temperatures [84] with the annual mean temperature for 1990 being about 50 °F (similar to several North American Cities — refer to Table 2.40). The Freezing Index for Sapporo, Hokkaido is approximately 600 °C degree days per year as shown in Figure 3.3. Freezing conditions were encountered from December 11, 1989, to March 12, 1990, and began again on December 21, 1990.

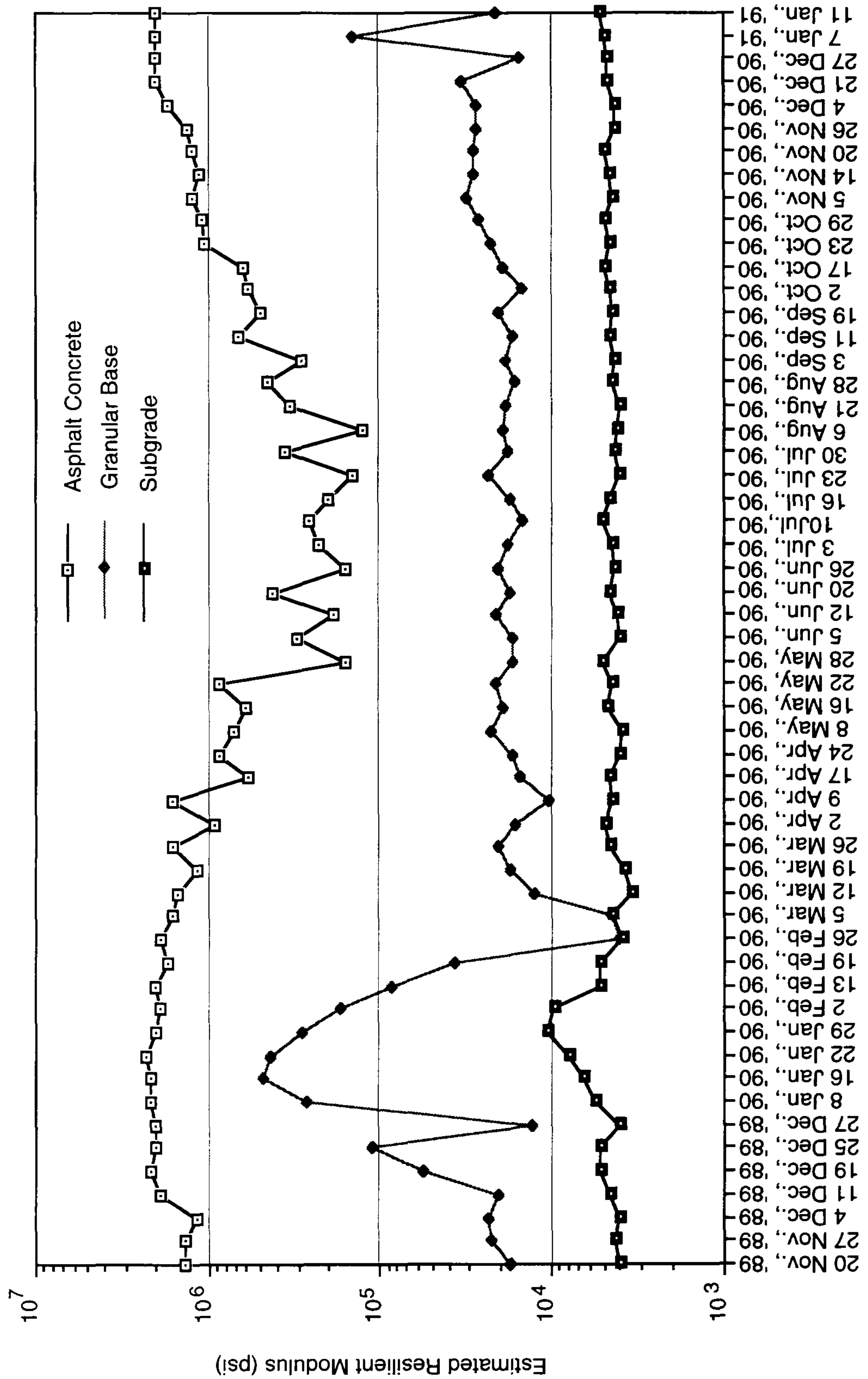


Figure 3.1. Seasonal Variation of Backcalculated Resilient Moduli for Asphalt Concrete, Granular Base, and Subgrade Layers Using the LMBS Backcalculation Program (Depth to Stiff Layer is 114 inches)

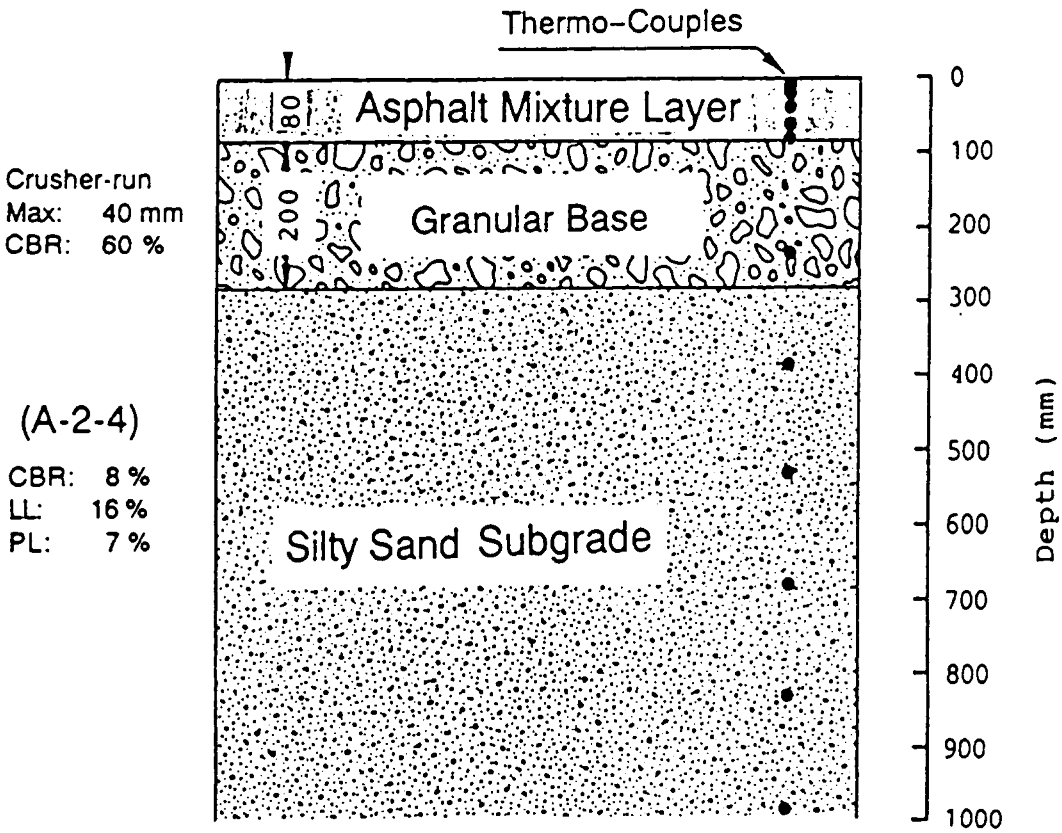


Figure 3.2. Pavement Test Section at the Hokkaido Institute of Technology, Sapporo, Hokkaido, Japan [82]

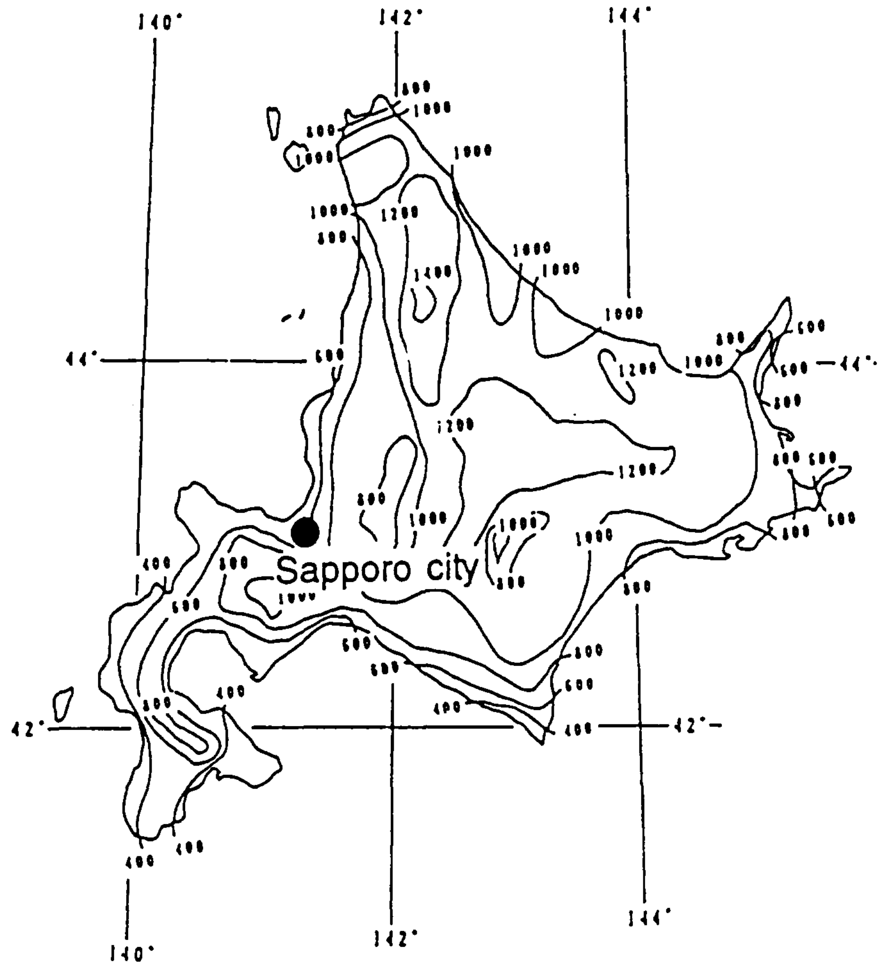


Figure 3.3. Freezing Index (°C-days) in Hokkaido, Japan [82]

Table 3.1 Summary of Monthly Precipitation and Temperatures for Sapporo, Hokkaido, Japan, November 1989 to January 1991 [84]

Year	Month	Precipitation (inch)	Mean Temperature (°F)
1989	Nov	4.09	43.9
	Dec	4.72	30.9
1990	Jan	5.43	23.2
	Feb	2.95	30.7
	Mar	3.07	37.2
	Apr	3.43	46.2
	May	0.98	56.1
	June	1.50	63.9
	July	1.93	69.4
	Aug	5.43	73.6
	Sept	4.88	65.7
	Oct	3.43	55.4
	Nov	2.87	45.5
	Dec	6.22	35.4
1991	Jan	7.17	29.8

The purpose of this chapter is to examine the NDT data provided by the Japanese study. A backcalculation program named LMBS (Layer Moduli Backcalculation System) [85] was used to determine layer moduli. To see if different backcalculation programs cause differences in layer moduli and seasonal variation, the same deflection data was also backcalculated by the EVERCALC Version 3.3 [36] backcalculation program. Backcalculation was performed for conditions with and without a stiff layer. The depth to stiff layer, when used, was varied. The specific assumptions used for both programs are noted. Finally, some seasonal adjustment factors from the results of the two programs with the different assumptions were determined.

2. COMPUTING LAYER MODULI

Several assumptions as reported by Ishitani, et al. [82] were made in backcalculating layer moduli using the LMBS program. These assumptions are as follows:

1. A stiff layer exists at 114 inches (2.9 meters) below the pavement surface. Ishitani et al. [82] chose this depth as it corresponds to the depth of ground water table. The pavement structure is represented by a four layer system.

2. The stiff layer resilient modulus is 14,500 ksi (100 GPa).

3. The asphalt modulus used in the LMBS program was determined by the relationship shown in Figure 3.4. This relationship is based on laboratory dynamic indirect tension tests with a load rate of 30 ms (5.3 Hz.) at various temperatures. Asphalt moduli are selected based on the mean pavement temperature provided by thermocouples placed in the pavement.

Actually two depths to the stiff layer were considered for the LMBS program. Besides the 114 inch (2.9 meter) depth below the surface, additional backcalculated results from a stiff layer depth of 394 inches (10 meters) were obtained from a preliminary paper on the study. [86] The LMBS backcalculated results for both depths are shown in Appendix A, Tables A.1 and A.2.

Several assumptions were also made using EVERCALC to backcalculate layer moduli. The assumptions included for modeling several subgrade conditions are as follows:

1.	Depth to stiff layer = 394 inches (10 meters)	Asphalt concrete modulus selected from the laboratory results of Figure 3.4 using mean pavement temperature.
2.	Depth to stiff layer determined by EVERCALC	Asphalt concrete modulus selected from the laboratory results of Figure 3.4 using mean pavement temperature.
3.	Depth to stiff layer = 114 inches (2.9 meters)	Asphalt concrete modulus backcalculated by EVERCALC
4.	Depth to stiff layer determined by EVERCALC	Asphalt concrete modulus backcalculated by EVERCALC
5.	Depth to stiff layer = 394 inches (10 meters)	Asphalt concrete modulus backcalculated by EVERCALC

The stiff layer used in all cases had an estimated resilient modulus of 1000 ksi. The 1000 ksi modulus was based on previous experience with EVERCALC, and was determined adequate to represent the stiff layer (recall that Ishitani et al. [82] used

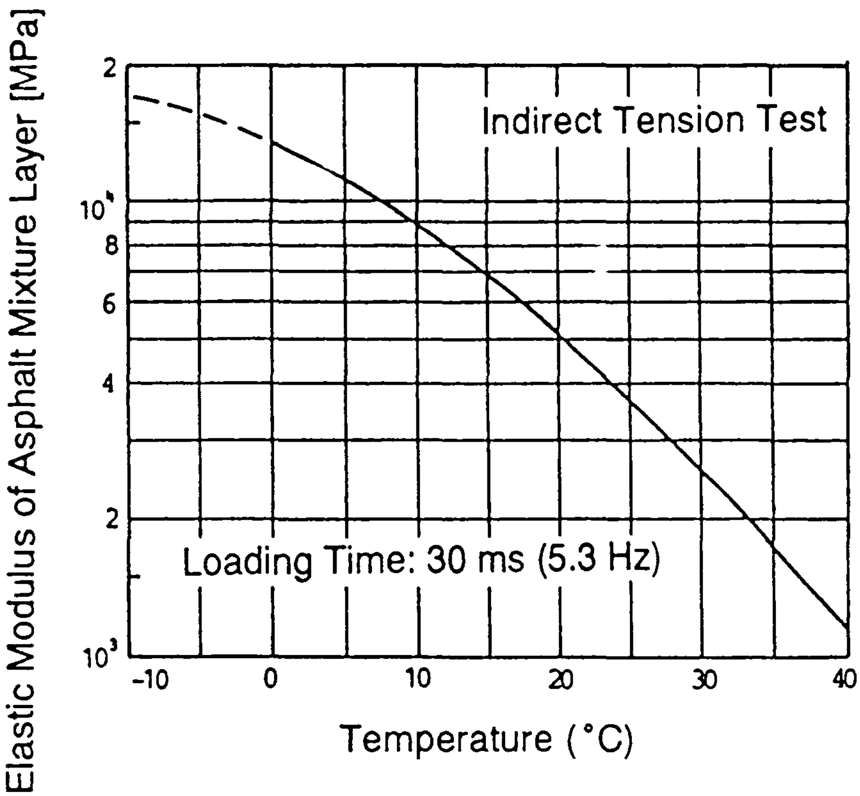


Figure 3.4. Elastic Modulus of Asphalt Concrete Corresponding to FWD Loading Time [82]

14,500 ksi). The deflection data used to backcalculate layer moduli are shown in Appendix A, Tables A.3 and A.4. A unique feature of EVERCALC is that measured deflection basins are shown on the computer screen as backcalculation is being performed. Several of the deflection basins resulting from the deflection data appeared to be defective. It appeared on occasion that the Number 2 or Number 4 sensor from the Phoenix FWD malfunctioned (the deflection was larger than the preceding sensor closer to the load plate). For the EVERCALC runs the deflection sensors with faulty readings were discarded and backcalculation performed with five sensors.

3. OBSERVATIONS FROM BACKCALCULATION

The backcalculated results obtained from EVERCALC and LMBS programs varied. Large differences in estimated moduli were clearly the norm which makes comparison difficult. Two of the cases for EVERCALC gave results which were unrealistic based on the deflection data.

One case which yielded backcalculated moduli which could be considered "reasonable" will be illustrated. This case was where the depth to stiff layer for EVERCALC was set at 394 inches (10 meters) and the asphalt modulus was determined by the program. The results are shown in Appendix A, Table A.5. For comparison the corresponding LMBS run with stiff layer depth at 394 inches (10 meters) is shown. It must be noted the asphalt modulus in the LMBS runs were set according to laboratory test results corresponding to pavement temperature and FWD loading time (Figure 3.4). Also shown is the EVERCALC case where there was no stiff layer and the asphalt modulus was determined by the program.

Figures 3.5, 3.6, 3.7 show the backcalculated asphalt, base, and subgrade moduli for the LMBS and EVERCALC programs with and without stiff layers. Of particular interest is the variability of backcalculated base and asphalt moduli shown in Figure 3.7 (EVERCALC without a stiff layer). Figures 3.5, (LMBS), and Figure 3.6 (EVERCALC with a stiff layer), tend to mimic each other and show less variability. Comparison of the base and asphalt layer moduli for the EVERCALC stiff and no stiff layers are shown in Figures 3.8, and 3.9. The moduli are quite different with more fluctuation occurring in the no stiff layer condition. Such data tend to confirm the importance of using a stiff layer condition for backcalculation of layer moduli.

Figures 3.10, 3.11, and 3.12 show comparisons between the asphalt, base, and subgrade layers for the LMBS and EVERCALC with stiff a layer. Figure 3.10 shows subgrade moduli which are nearly the same for both programs. The base course moduli shown in Figure 3.11 show some similarity, but overall the LMBS results tend to be lower. Asphalt moduli (Figure 3.12) show the greatest variability. The LMBS asphalt moduli determined form the laboratory results show less variability than the asphalt moduli determined by EVERCALC, as one would expect.

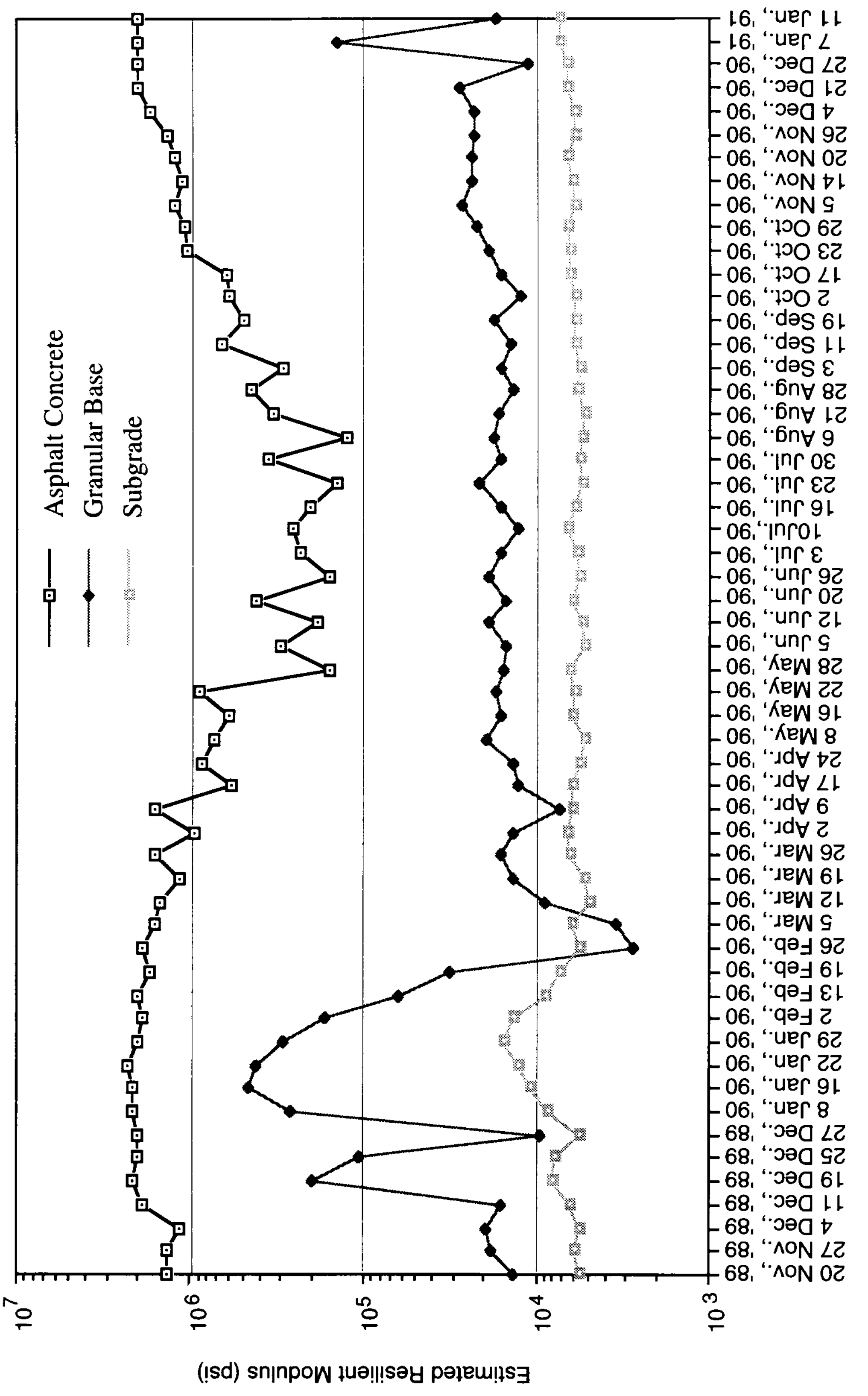


Figure 3.5 Seasonal Variation of Backcalculated Resilient Moduli for Asphalt Concrete, Granular Base, and Subgrade Layers Using the LMBS Backcalculation Program (Depth to Stiff Layer is 394 inches)

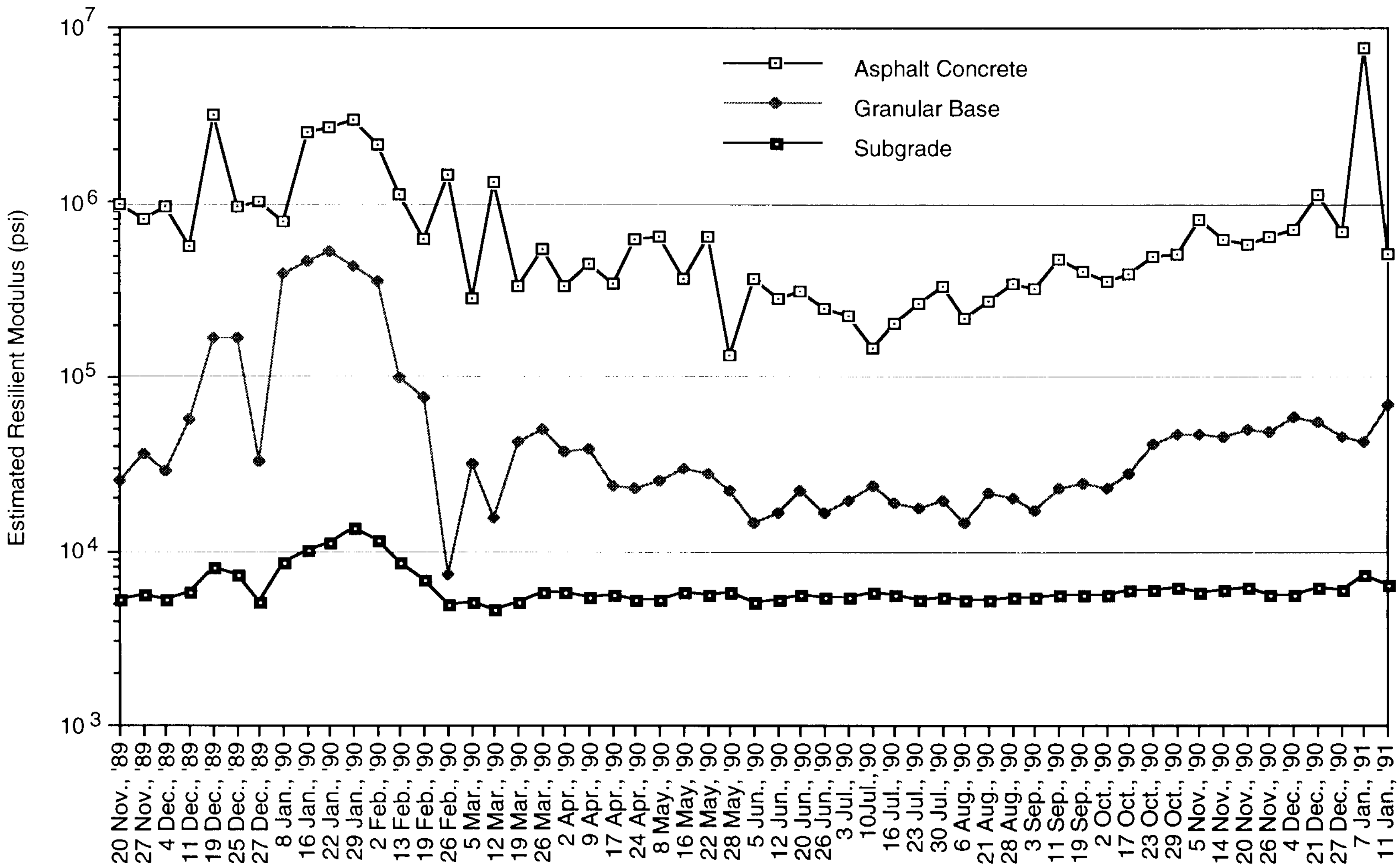


Figure 3.6 Seasonal Variation of Backcalculated Resilient Moduli for Asphalt Concrete, Granular Base, and Subgrade Layers Using the EVERCALC Backcalculation Program (Depth to Stiff Layer is 394 inches)

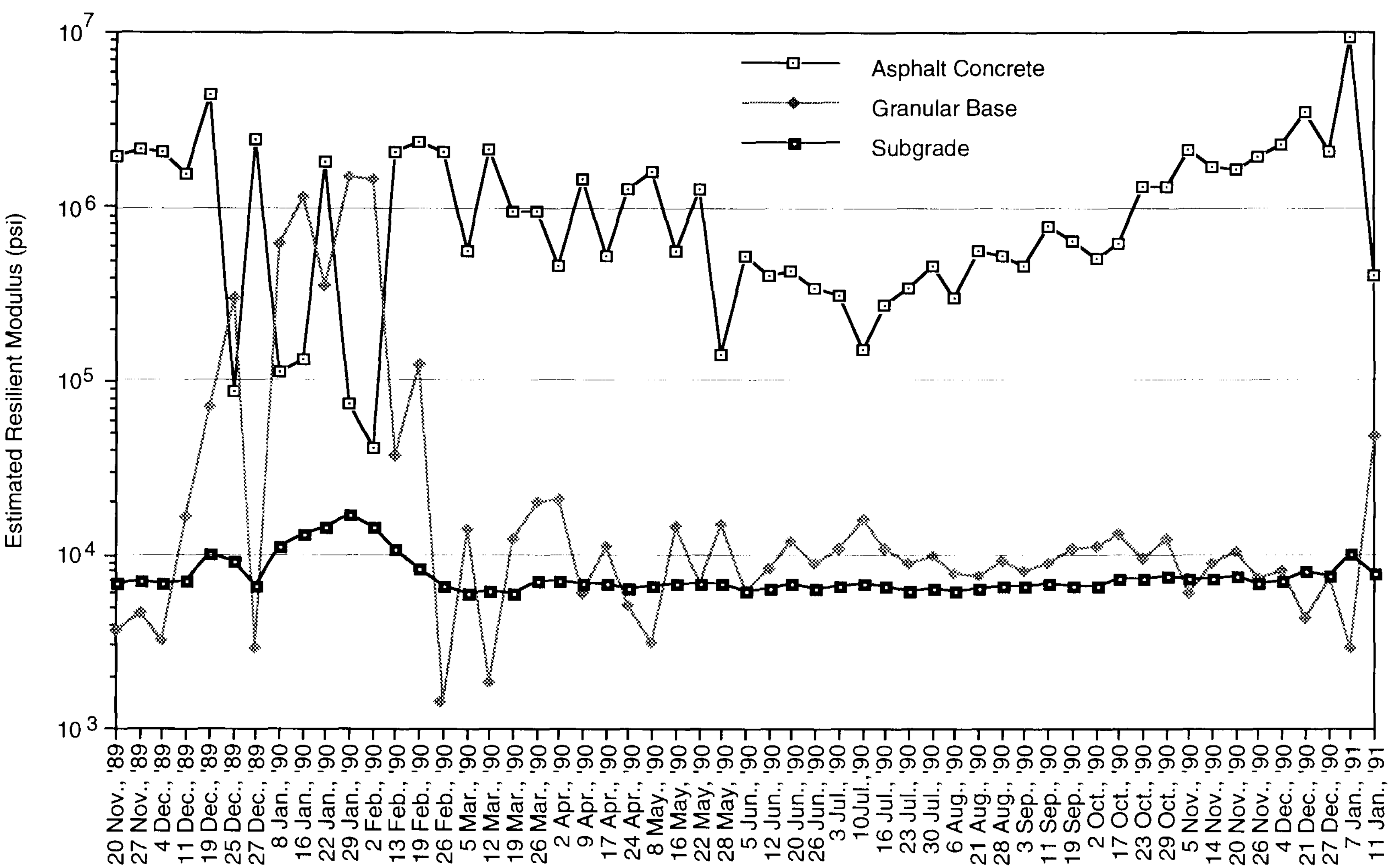


Figure 3.7 Seasonal Variation of Backcalculated Resilient Moduli for Asphalt Concrete, Granular Base, and Subgrade Layers Using the EVERCALC Backcalculation Program (Stiff Layer not Used)

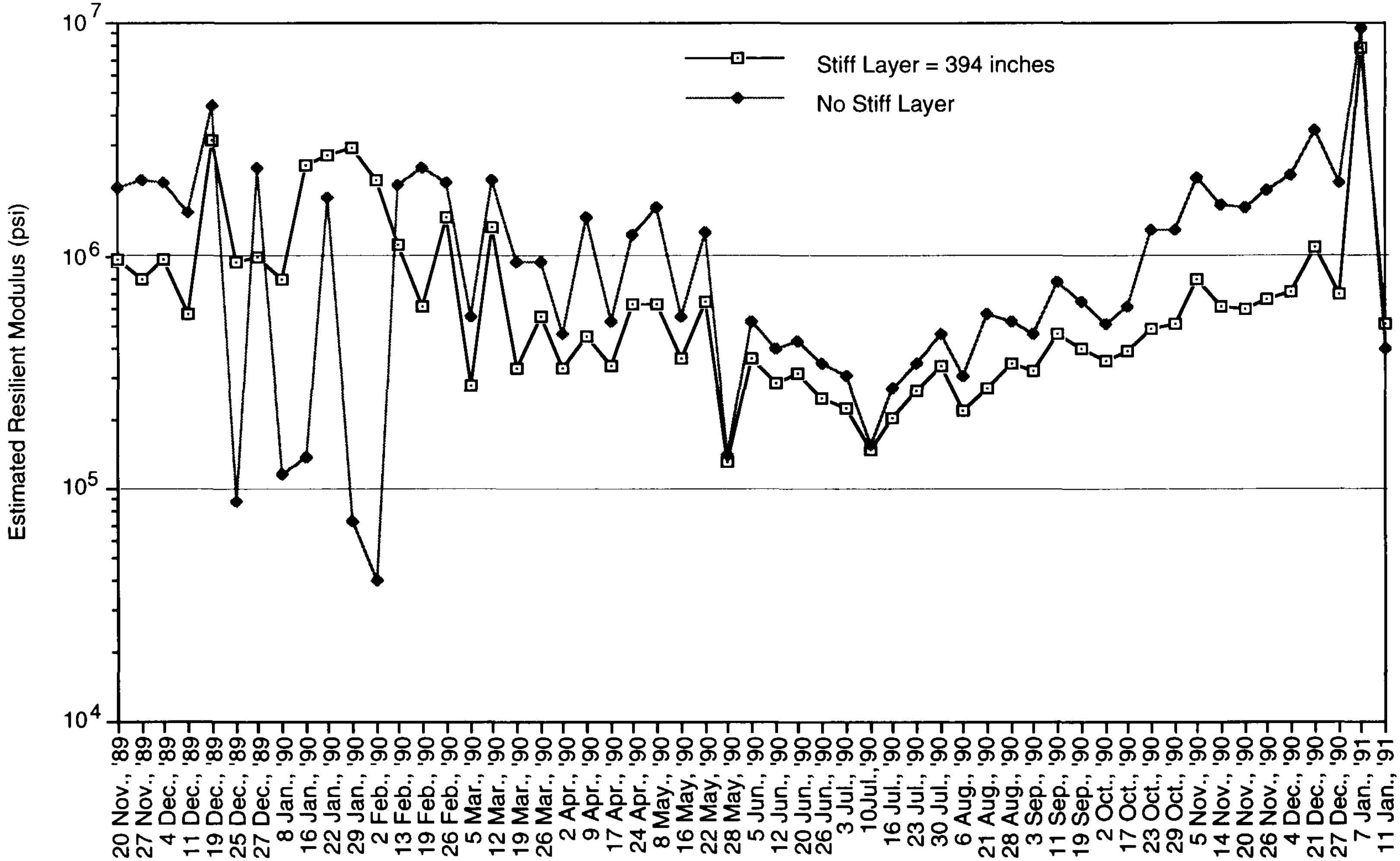


Figure 3.8 Comparison of Backcalculated Asphalt Concrete Resilient Moduli Using the EVERCALC Backcalculation Program With and Without a Stiff Layer

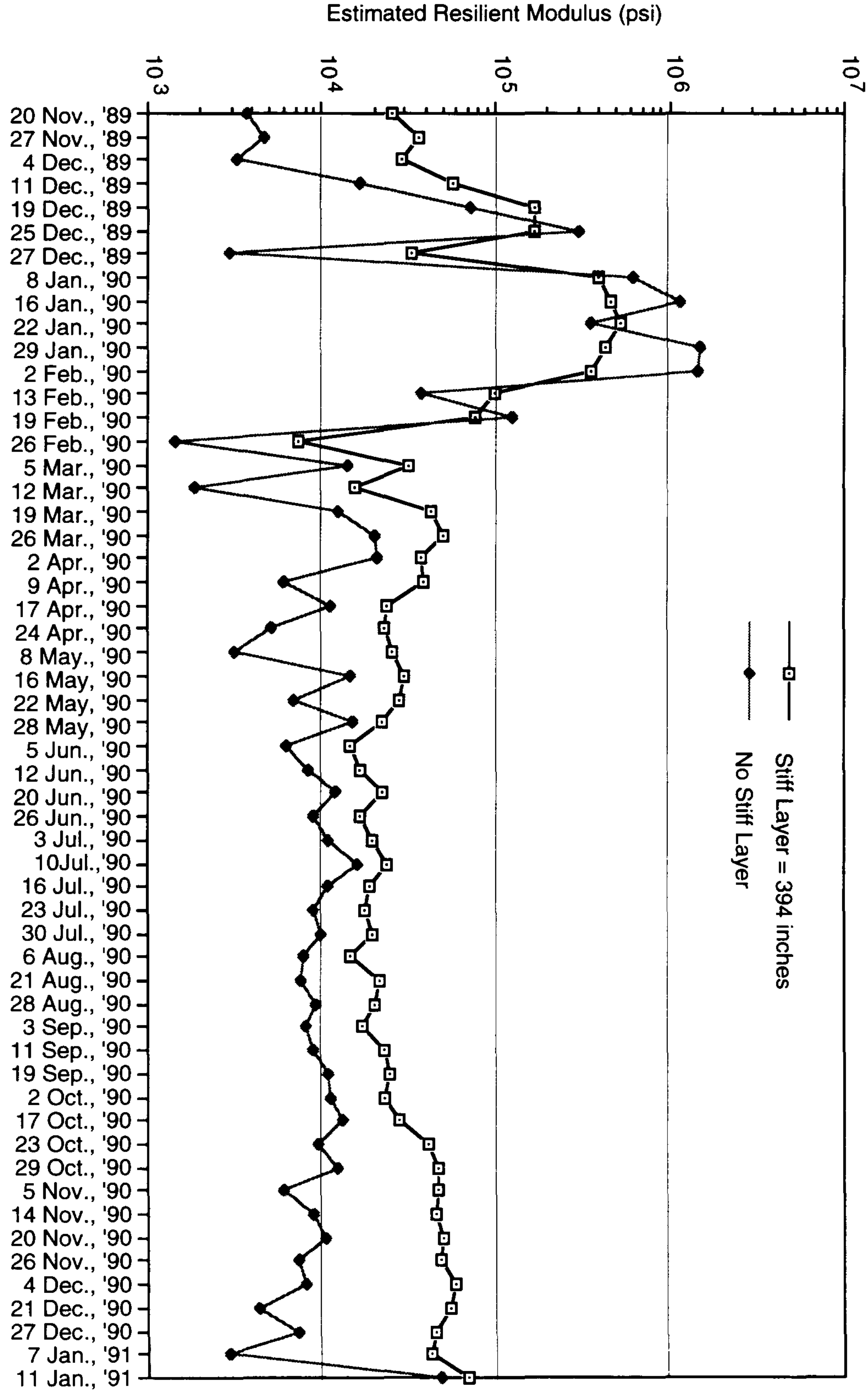


Figure 3.9 Comparison of Backcalculated Granular Base Resilient Moduli Using the EVERCALC Backcalculation Program With and Without a Stiff Layer

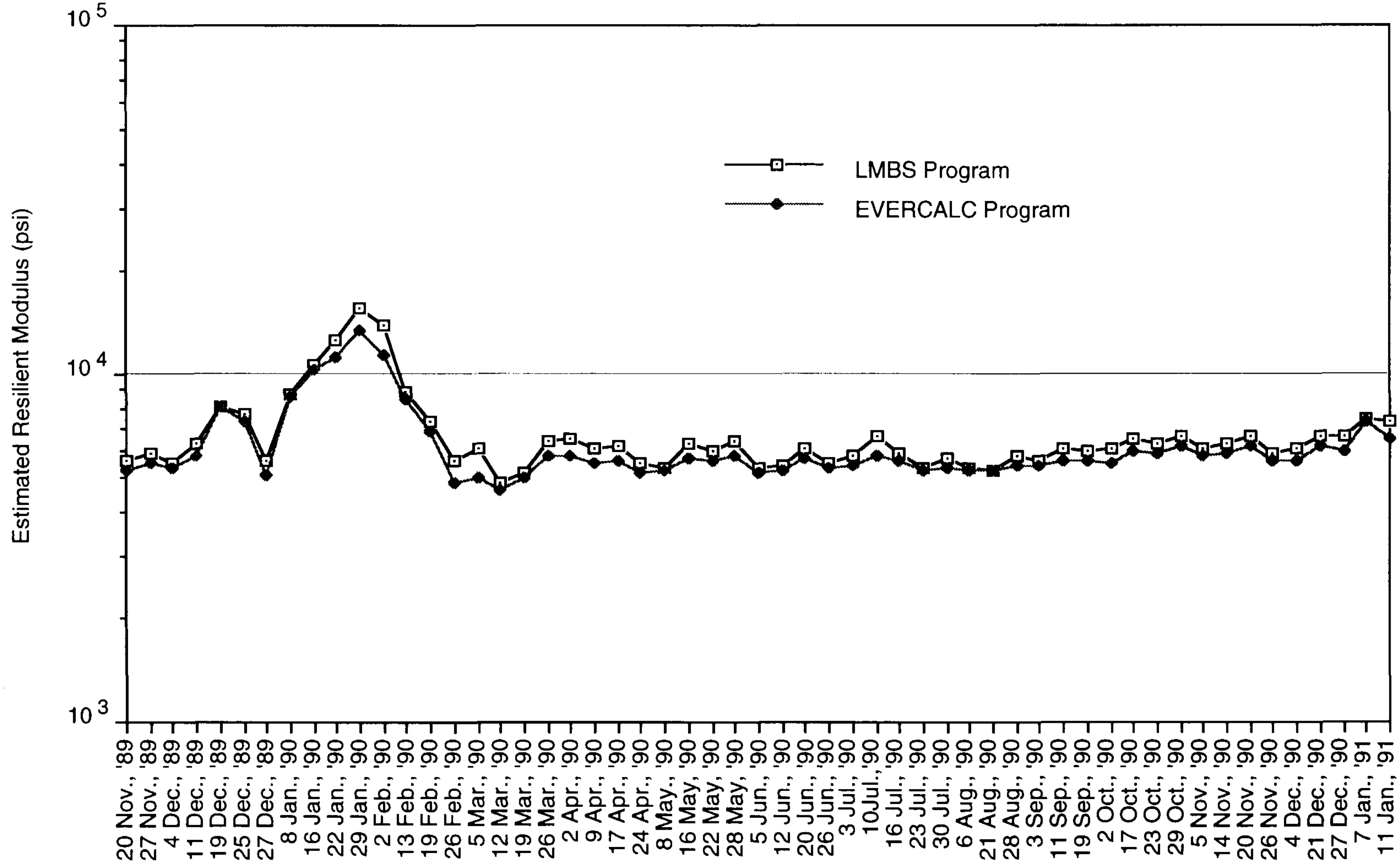


Figure 3.10 Comparison of Backcalculated Subgrade Resilient Moduli Using the LMBS and EVERCALC Backcalculation Programs With a Stiff Layer at 394 inches

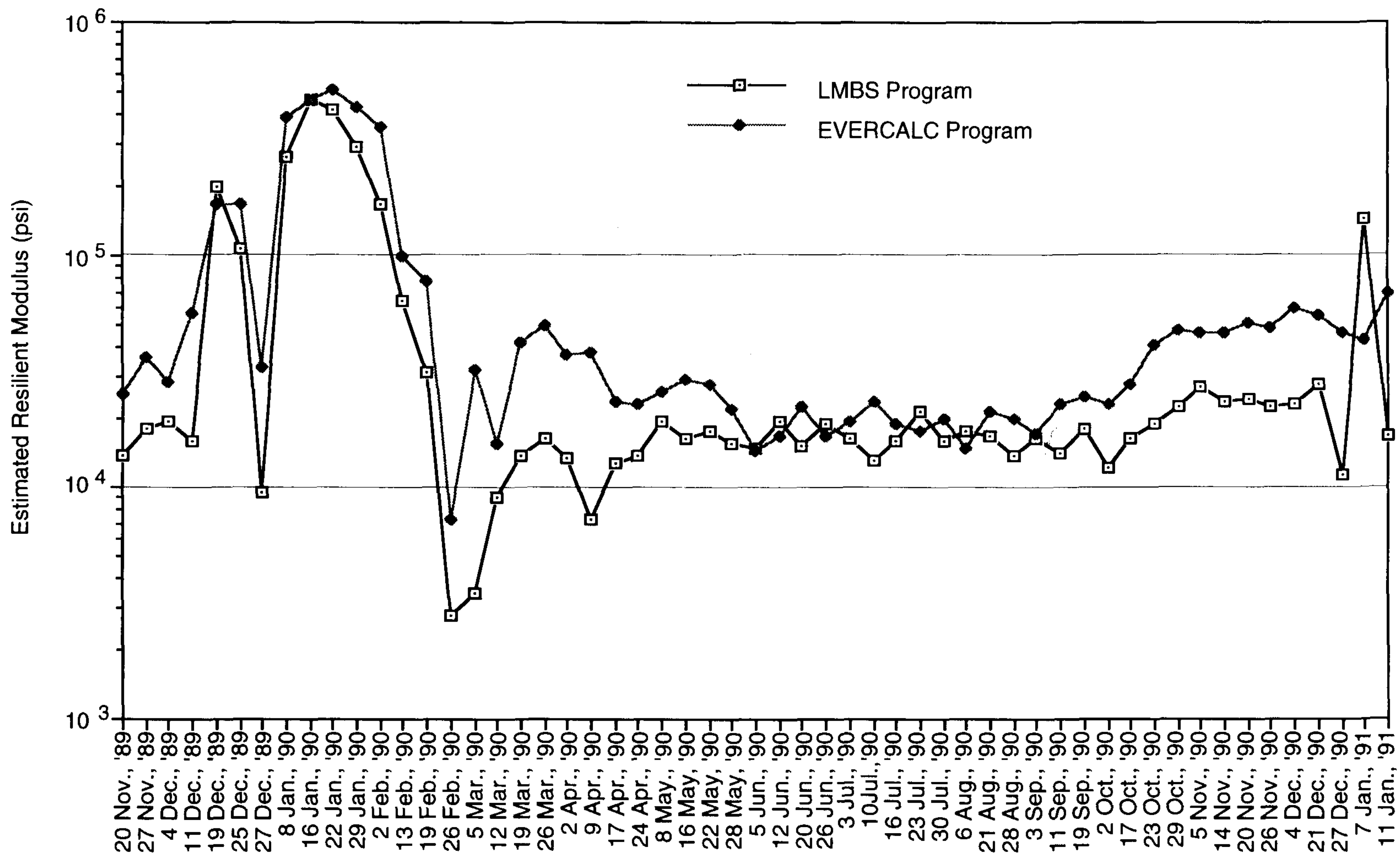


Figure 3.11 Comparison of Backcalculated Granular Base Resilient Moduli Using the LMBS and EVERCALC Backcalculation Programs With a Stiff Layer at 394 inches

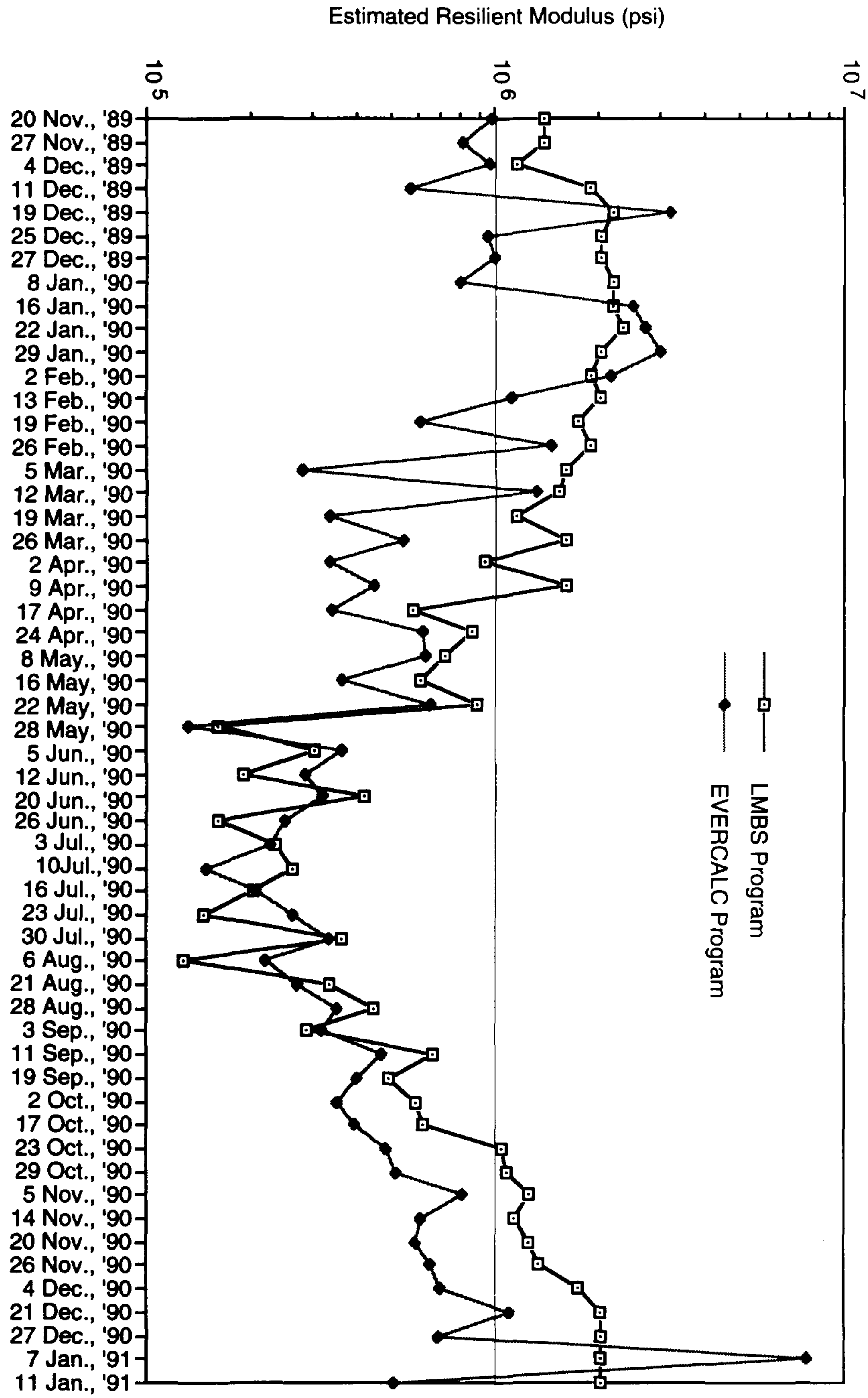


Figure 3.12 Comparison of Backcalculated Asphalt Concrete Resilient Moduli Using the LMBS and EVERCALC Backcalculation Programs With a Stiff Layer at 394 inches

EVERCALC provides Root Mean Square (RMS) values as output for pavement moduli. Normally, RMS values of less than 1.5 to 2.0 percent are desired for convergence of the backcalculation routine. RMS values given for both the stiff layer and no stiff layer results vary from 1.1 to 10.0. The RMS values for the no stiff layer condition are slightly lower which, at first glance, would indicate the no stiff layer solution is better. This was not the case according to Figure 3.7 which showed large, unrealistic fluctuations of the base and asphalt moduli.

Another case to consider is the EVERCALC results where the stiff layer depth was set to 114 inches (2.9 meters) and the asphalt modulus was determined by the program. Results are shown Appendix A, Table A.6. Also shown in this table are EVERCALC results without a stiff layer and the LMBS results with a stiff layer at 114 inches (2.9 meters). Both EVERCALC cases showed unrealistic layer moduli. The base course for the stiff layer condition was consistently estimated in the 100,000's psi, while the no stiff layer condition estimated moduli which fluctuated from 1,400 to 16,250 psi. Values changed drastically from week to week. RMS values ranged from 6.4 to 31.8 for the stiff layer case. Corresponding figures for the stiff and no stiff layer cases are shown in Figures A.1 , A.2, and A.3 of Appendix A for the results shown Table A.6.

Results for EVERCALC where the <u>depth</u> to the stiff layer and asphalt modulus were determined by the <u>program</u> gave base moduli that fluctuated or were over estimated. Appendix A, Tables A.7 and A.8 show these backcalculated results. Figures A.9 and A.10 correspond to Table A.8 and Figures A.4, A.5, A.6, A.7, and A.8 correspond to Table A.7.

A final comparison can be made where the asphalt modulus for EVERCALC was selected from laboratory tests corresponding to mean pavement temperature (Figure 3.4). Where the stiff layer depth was set at 394 inches (10 meters) the EVERCALC results were higher than the LMBS results but were still reasonable values (20,000 - 35,000 psi for base, 5,000 to 6,000 psi for subgrade). The run without a stiff layer gave base and subgrade moduli which were lower. Table A.9 of Appendix A shows the results. Also shown are Figures A.11, A.12, A.13, and A.14 that show moduli comparisons for base, asphalt, and subgrade. The results where the stiff layer depth was determined by EVERCALC is shown in Appendix A, Table A.10. Base moduli were overestimated in this run. Figures A.15, A.16, A.17, and A.18 also show the moduli results.

One observation which can be made based on these results is the large variability in backcalculated moduli. The assumptions made play an important part in arriving at moduli that are reasonable. The use of a stiff layer in some cases improved the consistency of the layer moduli but overall the moduli were over or under estimated. Why the EVERCALC produced questionable moduli is unknown. Possibly, the high RMS values shown in all the runs indicate possible problems with the deflection data as is indicated by the problems with the Number 2 and 4 sensors. Another source of discrepancy between some of the results is that the LBMS program had the restriction of using a laboratory derived asphalt concrete modulus.

4. SEASONAL FACTORS FOR JAPAN STUDY

To identify the seasonal variation, the moduli for the base and subgrade materials were separated by season. Breaks were made approximately at the calendrical season (spring, summer, fall, and winter). The rainfall at Sapporo was well-distributed throughout the year except June and July (which were lower). Moduli ratios were determined for each date by dividing the base and subgrade values by the summertime average moduli. The ratios for a particular season were then averaged.

The determination of moduli ratios was done by two methods. The first was by using all the values in each season. The second was by selectively looking at all the values and then discarding those that did not fit the trend of values within the season. Many times this included discarding the high and low values which resulted in more uniform moduli. Moduli ratios were then developed based on the summertime average.

Winter months were handled slightly different. For the Japan test site, the winter months mostly included frozen ground. Moduli exceeding 100,000 psi for base course materials were observed. In the same season reduced moduli were observed due to thawing conditions. The moduli ratios for the winter months reflects the range in the thawed condition moduli.

The cases for which seasonal factors (moduli ratios) were developed for base and subgrade materials include:

1. LMBS program - depth to stiff layer = 394 inches (10 meters) — Asphalt concrete modulus selected from Figure 3.4 using mean pavement temperature.

2. LMBS program - depth to stiff layer = 114 inches (2.9 meters) — Asphalt concrete modulus selected from Figure 3.4 using mean pavement temperature.

3. EVERCALC program - depth to stiff layer = 394 inches (10 meters) — Asphalt concrete modulus selected from Figure 3.4 using mean pavement temperature.

4. EVERCALC program - depth to stiff layer = 394 inches (10 meters) — Asphalt concrete modulus determined by EVERCALC.

5. EVERCALC program - depth to stiff layer determined by EVERCALC — Asphalt concrete modulus determined by EVERCALC.

Appendix A includes Tables A11 to A26 which summarizes the moduli ratios for all the base and subgrade seasonal moduli. For all cases computed by the LMBS program, the asphalt modulus used was that determined from the laboratory relationship.

EVERCALC runs, where the laboratory relationship was used, are indicated on the tables. Also indicated on each table is the number of the summary table from which the layer moduli were obtained.

The seasonal factors developed for the five cases are shown in Tables 3.2 to 3.6. The LMBS runs in Tables 3.2 and 3.3 show less seasonal variation than the EVERCALC runs shown in Tables 3.4, 3.5, and 3.6. The EVERCALC runs in Tables 3.5, and 3.6 gave seasonal factors for the base material which appear high. Base course factors ranged form 1.35 to 3.3 over that of summer. Wintertime factors reflecting periods of thaw gave values greater than one.

Although it is difficult to compare results of the two programs due to the variability of backcalculated results, some similarities in trends are observed. One trend in backcalculated cases is that the base course backcalculated moduli seem to vary more than the subgrade (a trend repeatedly noted in Chapter 2).

A second trend for the base course is that during periods of thawing, substantial reductions in moduli resulted. For the LMBS study, moduli ratios ranged from 17 to 72 percent relative to summer moduli (Tables 3.2, 3.3). Moduli down to 2,800 psi were observed. EVERCALC cases (Tables 3.4, 3.5) showed moduli ratios of 22 to 85 percent of the summer moduli. Subgrade moduli ratio changes for both the EVERCALC and LMBS studies were also noted. For the LMBS cases (Tables 3.2, 3.3), the subgrade moduli ratios ranged from 80 to 97 percent relative to summer moduli. Percentages of about 84 to 97 percent were observed with EVERCALC (Tables 3.4, 3.5).

On the other hand, during periods of freeze, base moduli backcalculated by LMBS and EVERCALC were substantially increased. Moduli of approximately 475 ksi for the LMBS program were observed with EVERCALC showing frozen moduli near 525 ksi (Table A.5).

Table 3.2 Base Course and Subgrade Moduli Ratios Using LMBS Program (Ratios Determined From Averaged or Selected Moduli From Each Season—Stiff Layer Depth is 114.2 inches)

Stiff Layer Depth 114.2	Base				Subgrade			
	Summer	Fall	Winter	Spring	Summer	Fall	Winter	Spring
(Average) Ratio	1.00	1.20	0.22 - 0.71	0.96	1.00	1.05	0.80 - 0.91	1.00
Modulus (psi)	18619	24045	4047 - 13271	17949	4396	4502	3481 - 3989	4417
(Selected) Ratio	1.00	1.30	0.22 - 0.72	0.98	1.00	1.05	0.80 - 0.91	1.02
Modulus (psi)	18553	24180	4047 - 13271	18214	4365	4581	3481 - 3989	4469

Notes: 1. Laboratory asphalt modulus values corresponding to FWD loading time and temperature used for backcalculation

2. Winter factors reflect ratios for thawing conditions

Table 3.3 Base Course and Subgrade Moduli Ratios Using LMBS Program (Ratios Determined From Averaged or Selected Moduli From Each Season—Stiff Layer Depth is 404.7 inches)

Stiff Layer Depth 404.7 inch	Base				Subgrade			
	Summer	Fall	Winter	Spring	Summer	Fall	Winter	Spring
(Average) Ratio	1.00	1.2	0.17 - 0.57	0.91	1.00	1.07	0.84 - 0.97	1.03
Modulus (psi)	16502	19806	2770 - 9456	14992	5785	6187	4888 - 5613	5941
(Selected) Ratio	1.00	1.21	0.17 - 0.58	0. 91	1.00	1.08	0.85 - 0.97	1.04
Modulus (psi)	16356	19835	2770 - 9456	14917	5749	6248	4888 - 5613	5952

Notes: 1. Laboratory asphalt modulus values corresponding to FWD loading time and temperature used for backcalculation

2. Winter factors reflect ratios for thawing conditions

Table 3.4 Base Course and Subgrade Moduli Ratios Using EVERCALC Program (Ratios Determined From Averaged or Selected Moduli From Each Season—Stiff Layer Depth is 404.7 inches)

Stiff Layer Depth 404.7 inch	Base				Subgrade			
	Summer	Fall	Winter	Spring	Summer	Fall	Winter	Spring
(Average) Ratio	1.00	1.44	0.22 - 0.84	1.09	1.00	1.06	0.84 - 0.94	1.00
Modulus (psi)	19581	28159	4404 - 16369	21402	5480	5790	4626 - 5159	5506
(Selected) Ratio	1.00	1.47	0.23 - 0.85	1.13	1.00	1.05	0.84 - 0.94	1.00
Modulus (psi)	19180	28252	4404 - 16369	21762	5486	5748	4626 - 5159	5450

Notes: 1. Laboratory asphalt modulus values corresponding to FWD loading time and temperature used for backcalculation

2. Winter factors reflect ratios for thawing conditions

Table 3.5 Base Course and Subgrade Moduli Ratios Using EVERCALC Program (Ratios Determined From Averaged or Selected Moduli From Each Season—Stiff Layer Depth is 404.7 inches)

Stiff Layer Depth 404.7 inch	Base				Subgrade			
	Summer	Fall	Winter	Spring	Summer	Fall	Winter	Spring
(Average) Ratio	1.00	2.1	0.37 - 1.69	1.46	1.00	1.09	0.84 - 0.97	1.02
Modulus (psi)	19763	41528	7273 - 33377	28761	5478	5788	4614 - 5134	5529
(Selected) Ratio	1.00	2.02	0.37 - 1.69	1.35	1.00	1.05	0.85 - 0.97	1.01
Modulus (psi)	19774	40032	7273 - 33377	26728	5483	5743	4614 - 5134	5542

Notes: 1. Asphalt modulus determined by EVERCALC used for backcalculation

2. Winter factors reflect ratios for thawing conditions

Table 3.6 Base Course and Subgrade Moduli Ratios Using EVERCALC Program (Ratios Determined From Averaged or Selected Moduli From Each Season—Stiff Layer Depth is Determined by EVERCALC)

Layer Depth EVERCALC Sets	Base				Subgrade			
	Summer	Fall	Winter	Spring	Summer	Fall	Winter	Spring
(Average) Ratio	1.00	2.96	2.29 - 4.09	1.91	1.00	0.93	0.57 - 0.70	0.92
Modulus (psi)	18263	54116	41868 - 74767	34970	5789	5396	3315 - 4039	5306
(Selected) Ratio	1.00	3.28	2.35 - 4.19	2.21	1.00	0.94	0.57 - 0.70	0.94
Modulus (psi)	17826	58417	41868 - 74767	39351	5794	5441	3315 - 4039	5440

Notes:

1. Asphalt modulus determined by EVERCALC used for backcalculation

2. Winter factors reflect ratios for thawing conditions

CHAPTER 4

EXAMINATION OF NDT DATA FOR WASHINGTON STATE DOT TEST SITES

1. INTRODUCTION

WSDOT has made available to this study the FWD deflection data for 16 sites monitored seasonally from 1985 to 1988. The FWD data provided by WSDOT was used in previous studies [19, 87] and is reexamined as additional seasonal FWD testing has been performed. The following sections provide discussion in the development of seasonal adjustment factors for the eastern and western regions of Washington State. Given the range of climate and soil conditions in Washington State, seasonal adjustment factors developed for the state should reflect those developed for other geographical regions; however, the WSDOT pavement sites are located on a variety of state owned routes (Interstate to low volume) which may or may not be typical of paved U.S. Forest Service roads.

2. TEST SITE DESCRIPTIONS

The 16 test sites monitored in this study are shown in Figure 4.1. Selection of the sites as summarized by Lee [19] was based on the uniformity of conditions at each test site and the variety of WSDOT pavements. Lee defined uniformity as uniform pavement structures and subgrade soils of each test section. Variety means various climates, traffic volumes, thickness of base and asphalt layers, distress conditions, and age were represented. Table 4.1 lists descriptions of each test site. Each test site was 1000 feet long with 21 deflection stations located at 50 feet intervals.

The sites are divided into eastern and western regions, based on the climatic division of Washington state. The eastern sites experience both hot and cold seasons, and precipitation is relatively low. Seasonally, winters are generally characterized by freezing temperatures causing frozen ground. The spring season encompasses the spring thaw which usually occurs during late February or early March depending upon location. Hot summers and cool fall seasons are represented by dryer conditions.

Western Washington typically experiences warm/dry summers, and cool/damp fall seasons. Moderate to heavy precipitation with mild temperatures occurs during the winter and spring seasons.

Monthly average temperatures for eastern and western Washington are shown on Figure 4.2. The annual precipitation and number of frost free days for Washington state are shown in Figures 4.3 and 4.4. The temperature and precipitation data as

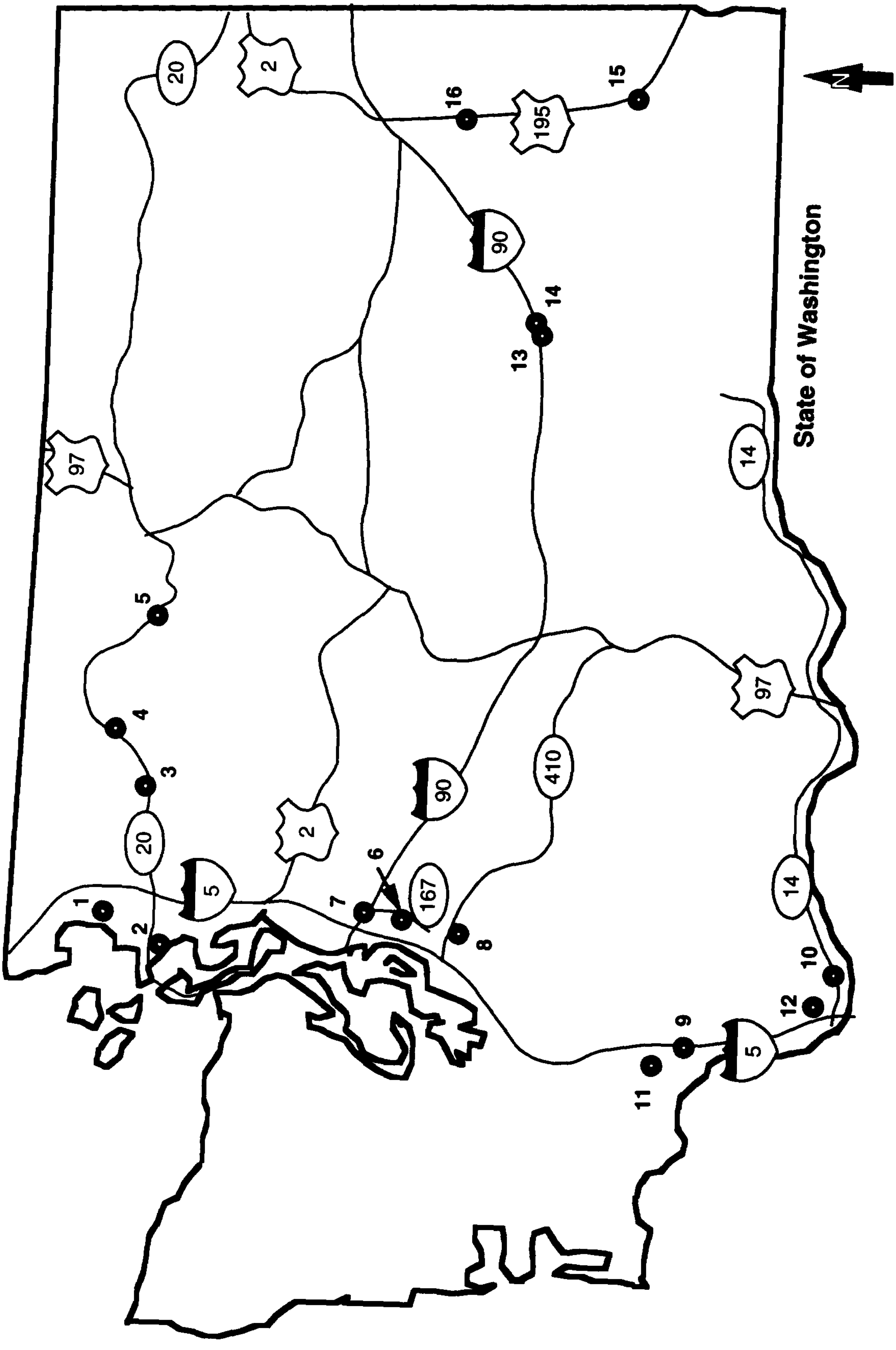


Figure 4.1. Location of Test Sections [87]

Table 4.1. Washington State Test Site Descriptions (after [19, 87])

Test Site			Year Original Construction (overlay year)	ACP Thickness (inch)	Base Thickness (inch)	Annual Mean Temp (°F)	Annual Mean Precip (in.)	Traffic in 1984 (10^3 ESAL)
No.	Route	Milepost						
1	SR 11	20.85	72	5.2	28.8	49.5	35.3	79.1
2	SR 20	53.50	73	4.9	4.8	50.5	25.6	74.7
3	SR 20	77.50	68(85)	10.9	6.6	50.3	46.4	24.0
4	SR 20	108.20	78	3.5	9.0	50.7	69.4	19.2
5	SR 20	140.80	72	3.4	6.6	-	-	-
6	SR 167	17.80	68(80)	11.2	-	51.5	39.2	45.7
7	SR 202	30.12	78	13.0	-	50.1	46.7	-
8	SR 410	9.60	68	7.3	3.6	51.2	40.8	58.2
9	SR 5	35.80	73	16.4	-	51.1	46.1	911.9
10	SR 14	18.15	73	9.0	3.6	51.9	41.1	34.2
11	SR 411	18.05	79	6.8	21.0	50.7	46.7	5.3
12	SR 500	3.20	79	6.3	8.4	51.9	41.1	40.7
13	SR 90	208.65	73	9.6	8.4	48.3	11.3	223.2
14	SR 90	208.85	73	9.6	8.4	48.3	11.3	223.2
15	SR 195	7.24	70(80)	6.2	11.4	47.1	21.3	66.6
16	SR 195	63.80	76	8.5	12.0	46.8	17.4	104.0

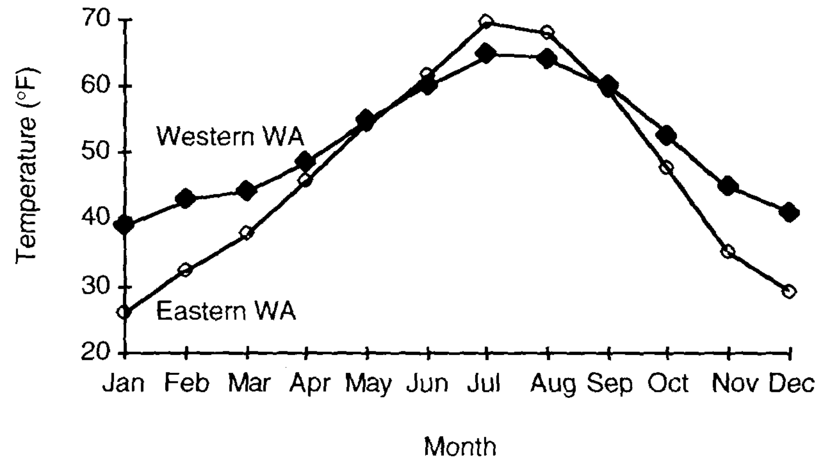


Figure 4.2. Monthly Average Temperature for Washington State Represented by Spokane (Eastern Washington) and Seattle (Western Washington) [88]

summarized by Lee [19] from National Oceanic and Atmospheric Administration (NOAA) records [89] are provided in Tables B.1 and B.2 in Appendix B. The data was obtained from the closest weather station to each site.

3. DATA COLLECTION

Deflection data was collected from March 1985 to March 1988 by the WSDOT, using a Dynatest model 8000 FWD. Two drops were made at each station at load levels of approximately 6000, 9000, 12000, and 15000 pounds. Sensor spacings for the FWD were at 0, 8, 12, 24, 36, and 48 inches. Pavement temperatures were recorded by drilling a hole and filling it with water, inserting a thermometer, and allowing the water to stabilize to the asphalt temperature. Table 4.2 shows the testing schedule that was maintained over the three year test program. Attempts were made to monitor pavement sections during the spring thaw in eastern Washington and during wet periods in western Washington. Due to difficulties in identifying the exact time when the pavement structure was thawing, it was not possible to coordinate FWD data collection with these critical periods.

Laboratory testing of the test site materials was performed by WSDOT. Results of laboratory resilient modulus tests for base course and subgrade materials are provided Table B.3 in Appendix B. Table 4.3 shows the results of asphalt concrete cores used to provide asphalt concrete depths for backcalculation of layer moduli.

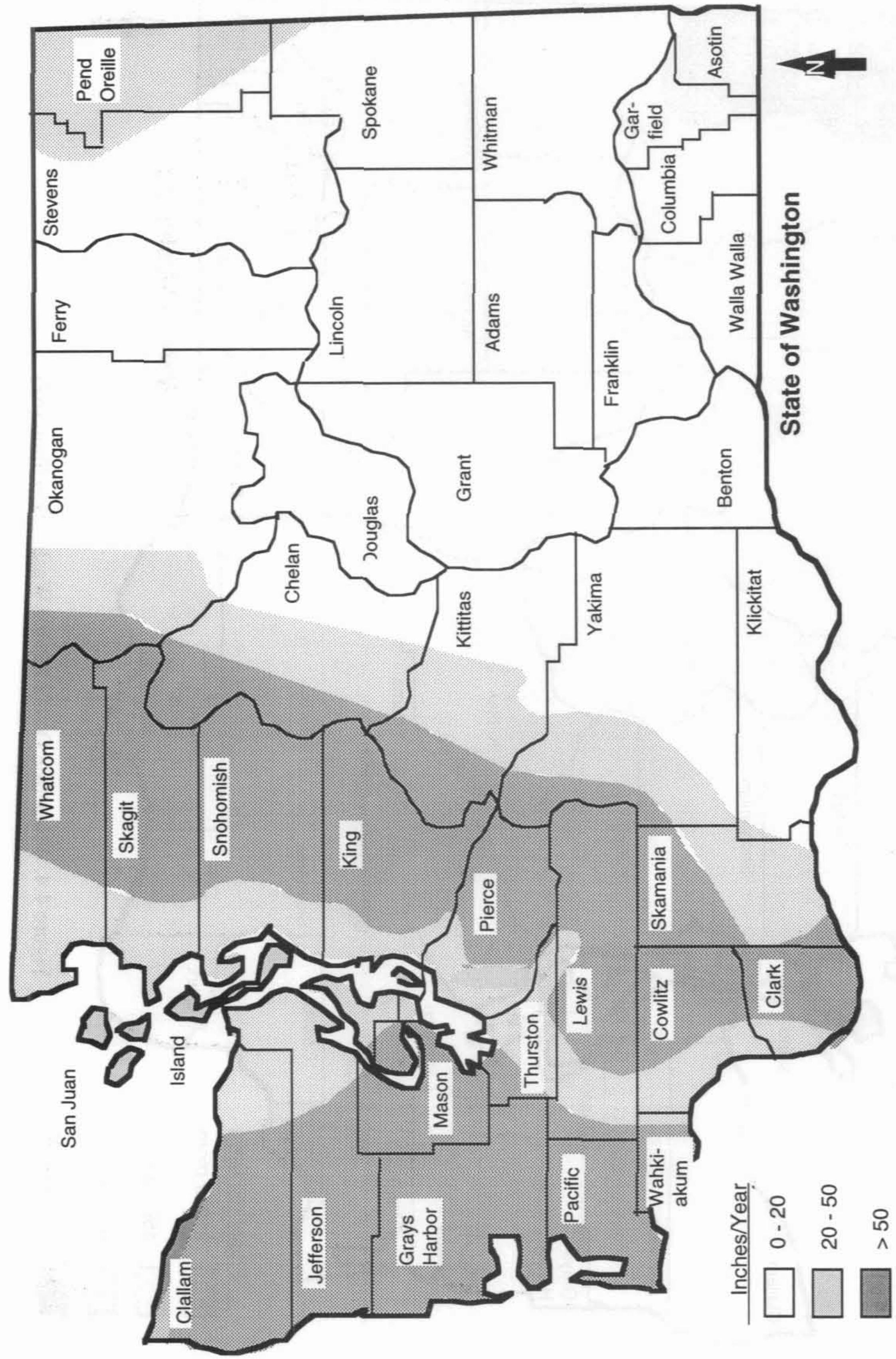


Figure 4.3. Annual Rainfall for Washington State [87]

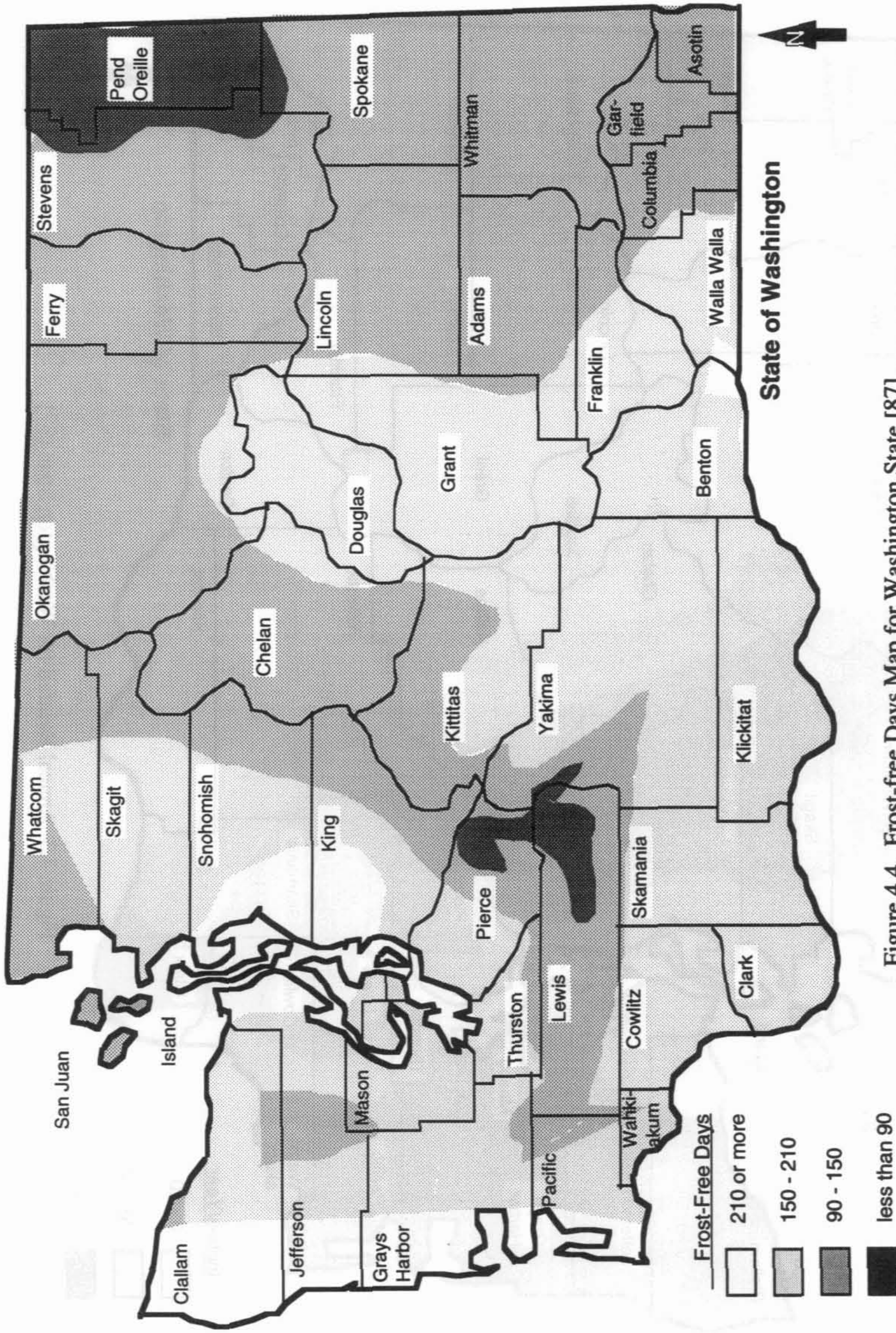


Figure 4.4. Frost-free Days Map for Washington State [87]

Table 4.2 Falling Weight Deflectometer Measurement Dates for Washington Test Sites

Test	1985			1986			1987				1988	
Site	SPR	SUM	FAL	SPR	SUM	FAL	WIN	SPR	SUM	FAL	WIN	SPR
1	05-21	08-08			07-16	10-09	01-27	06-01	08-17	11-12	02-01	
2	05-21	08-08			07-15	10-09	01-27	06-03		11-13	02-01	04-26
3	05-21	08-27			07-15	10-08	01-26	06-03		11-12	02-01	
6	05-21	07-23			07-17	10-08					01-28	
7	05-29	08-21			07-17							
8	06-06	08-19				10-21	01-27	06-21	08-12			
9	05-15	07-24		04-28	07-24	10-14	01-30	05-12	07-23			
10	05-13	08-06		04-24	07-23	10-13	01-29	05-11	07-23			03-31
11	05-14	08-22		04-29	07-23	10-13	01-29	05-12	07-23			03-30
12	05-13	08-06		04-28	07-23	10-13	01-29	05-11	07-23			03-31
14	04-10	08-30		04-02	07-14	10-06		03-19	07-15			03-09
15	04-16	07-17	10-09	04-26	07-10	10-01		03-18	07-16			03-08
16	04-18	07-17	10-09	03-27	07-10	10-01		03-17	07-14			03-88

Table 4.3. Asphalt Concrete Core Thicknessses for Washington Test Sites [19]

Test Site	Station No.	Core Thickness (inch)	Station No.	Core Thickness (inch)	Station No.	Core Thickness (inch)
1	0 + 50	5.46	5 + 50	4.98	9 + 50	5.20
2	0 + 50	4.97	5 + 50	5.03	9 + 50	4.72
3	0 + 50	10.88	5 + 50	11.50	9 + 50	10.38
4	0 + 50	3.50	5 + 50	3.34	9 + 50	3.61
5	0 + 50	3.75	5 + 50	3.50	9 + 50	2.75
6	0 + 50	11.35	5 + 50	10.72	9 + 50	11.34
7	0 + 50	12.53	5 + 50	12.94	9 + 50	13.60
8	0 + 50	7.00	5 + 50	7.50	9 + 50	7.50
9	0 + 50	17.84	5 + 50	15.38	9 + 50	15.91
10	0 + 50	8.84	5 + 50	9.09	9 + 50	9.21
11	0 + 50	6.63	5 + 50	6.83	9 + 50	7.00
12	0 + 50	6.25	5 + 50	6.06	9 + 50	6.53
13	0 + 50	9.91	5 + 50	9.56	9 + 50	10.16
14	0 + 50	9.85	5 + 50	9.91	9 + 50	10.03
15	0 + 50	6.25	5 + 50	6.69	9 + 50	5.63
16	0 + 50	8.69	5 + 50	8.06	9 + 50	8.84

4. BACKCALCULATION OF LAYER MODULI

The WSDOT backcalculation program EVERCALC Version 3.3 [36] was used to estimate the test site layer moduli. EVERCALC Version 3.3 includes the Rohle and Scullion [39] equation to estimate a depth to a stiff layer as determined from FWD deflection data. Stiff layer conditions are sometimes caused by bedrock, stress sensitive materials, the presence of a water table, or saturated soil conditions. [38, 90]

The backcalculation of the FWD deflection data was performed by Wang. [91] The results are provided in Tables B.4 to B.42 in Appendix B. Only 13 of the 16 original test sites are included, as EVERCALC was unable to converge on reasonable solutions for Test Sites 4, 5, and 13.

Wang [91], in his analysis considered three conditions when modeling the test sites for various subgrade conditions. The three conditions included backcalculation with:

- No stiff layer,
- A stiff layer with an elastic modulus of 50 ksi, and
- A stiff layer with an elastic modulus of 1,000 ksi.

Wang [91] found that the various stiff assumptions resulted in backcalculated moduli that appeared more reasonable and had lower root mean square (RMS) values.

Sites 1, 11, and 15 were analyzed with base course layers. The remaining sites had base layers that were thin compared to the asphalt concrete surface. When base layers were encountered that were approximately the same thickness as the asphalt concrete or thinner, the base material was included with the subgrade. Past experience with EVERCALC has shown that the base course thickness should be about 1.5 times (or more) greater than the asphalt concrete surface course in order to achieve reasonable estimates of base moduli. [92]

5. CALCULATION OF SEASONAL MODULI RATIOS

In order to develop seasonal adjustment factors, a "base" season with which to compare moduli had to be selected. This base season provides the "standard" moduli to which the moduli occurring during other seasons of the year are compared. For the Washington state test sites the summer or dry season modulus was chosen. The moisture in the unbound layer is typically expected to be the lowest in the summer months, and prevailing temperatures do not affect base/subgrade performance (Note: asphalt concrete temperature does affect the "stress state" in the base and subgrade).

Inspection of the backcalculated results in Tables B.4 to B.42, Appendix B, shows that at a specific site the summer subgrade or base modulus for different years was seldom constant. For example the backcalculated results for Test Site 1, Station 5+50, with the stiff layer condition of 50 ksi shows that the 1985 summer subgrade modulus was 15.5 ksi in 1985, 10.6 ksi in 1986, and 11.5 ksi in 1987 (Table 4.4). With a range of 10.6 to 15.5 ksi the choice of the summer modulus would effect the moduli ratios found by comparing summer to the other seasons.

Table 4.4. Backcalculated Subgrade Moduli for Test Site 1, Station 5 + 50

Test Site	Station		1985		1986		1987			1988	
			Spr	Sum	Sum	Fal	Win	Spr	Sum	Fal	Win
1	5 +50	Modulus (ksi)	8.30	15.50	10.60	12.30	8.70	9.60	11.50	11.70	8.50

Note: Highlighted moduli are those used to determine summer select or average moduli.

Generally selection of the stiff layer condition as indicated on Tables B.4 to B.42 was based on the lowest RMS value. This was not always the case as sometimes the low RMS value gave unreasonable results.

Engineering judgment was required to choose a summertime modulus for a given site that seemed reasonable. For Site 1, Station 5+50 the summer subgrade modulus was selected as 11.2 ksi, the average of 10.6 and 11.5 ksi. Typically, if moduli of about the same magnitude were observed and seemed reasonable the average of the moduli was determined. Backcalculated moduli which seemed unreasonably high or low were not used.

Of course, choosing a summertime modulus in this fashion allows room for prejudice in deciding what modulus seems "reasonable." To consider this possibility summertime moduli were also determined by averaging all the summertime moduli to arrive at an average summertime modulus. On occasion a backcalculated modulus seemed "unrealistic" for a base or subgrade and was not used in the average.

It should be noted that the climatic seasons (spring, summer, fall, winter) used to develop moduli ratios were identified by calendar dates indicated in Table 4.5. In situ moisture conditions in the unbound materials were not monitored to help determine season distinctions. Previous work by Lary et al. [93] showed that monitoring precipitation and temperature did not provide a viable means for predicting changes in pavement stiffness. Lee [19] also attempted to correlate seasonal moduli with monthly precipitation data but could not find a relationship. This is not to say seasonal factors such as precipitation are not important but only that straight forward correlations between precipitation and moduli are difficult. Seasonal effects such as the growth of ice lenses during the winter freeze appear to cause much of the moduli reduction observed in the northern states.

Moduli for the spring, fall, and winter seasons monitored over the three year period were compared to the select or average summer modulus at Stations 0 + 50, 5 + 50, and 9 + 50. Moduli ratios of each season compared to summer were computed. For some sites up to four moduli ratios for the spring season were determined for each station. The subgrade moduli ratios determined for the 13 Washington State DOT sites are shown in Table B.43, Appendix B (based on selective summer moduli).

Table 4.5. Calendar Dates to Indicate Seasonal Periods

Season	Calendar Dates
Spring	March 20 to June 19
Summer	June 20 to September 19
Fall	September 20 to December 19
Winter	December 20 to March 19

Base course moduli ratios were computed by the same procedure used for subgrade layers. The complete base course moduli ratios using select summertime moduli (dry season) are shown in Table B.44, Appendix B. Moduli ratios determined by using average summer moduli for subgrades and base course layers are provided in Tables B.45 and B.46, Appendix B.

The next step was to summarize the results obtained from the subgrade and base course moduli ratios for individual sites. Table 4.6 provides the average of subgrade moduli ratios for each site and season at Stations 0+50, 5+50, and 9+50 for western Washington. Moduli ratios were developed from select summer moduli. In Table 4.6, the lower ratio represents the average of subgrade moduli for a particular season that are lower than the summer modulus and the higher ratios reflects the average of moduli greater than the summer modulus. The higher ratios are provided to show the range of moduli ratios observed.

Table 4.6. Summary of Subgrade Moduli Ratios for Western Washington Test Sites (Using Select Summertime Modulus)

Test Site	Spring	Summer	Fall	Winter
1	0.87	1.00	0.97/1.06	0.77/1.14
2	0.82/1.05	1.00	0.80	0.78
3	0.83/1.18	1.00	0.95/1.05	0.94/1.18
6	0.99/1.11	1.00	0.88/1.01	0.77
7	1.13	1.00		
8	0.91/1.28	1.00	0.80/1.11	0.81/1.13
9	0.90/1.12	1.00	0.94	0.89/1.00
10	0.86/1.22	1.00	0.96/1.32	1.13
11	0.82/1.5	1.00	0.84	0.79
12	0.91/1.10	1.00	0.98/1.10	0.93/1.03
Average	0.87/1.19	1.00	0.90/1.16	0.84/1.13

Note: 1. Moduli ratios greater than 1 reflect ratios where spring, fall or winter moduli were greater than the select summer modulus.

For design purposes it is the reduction of stiffness (or strength) from the dry season that is of greatest concern. Roadway design is not based on the "best" performance of materials. For this reason the lower moduli ratios will be used to determine seasonal factors which will be discussed in later sections. Table 4.7 shows a summary of moduli ratios for eastern Washington subgrade sites determined from the select summer modulus.

In a similar way summary tables were made for subgrade moduli ratios determined by using an average summer modulus. Tables 4.8 and 4.9 are the summary of subgrade moduli ratios for western and eastern Washington sites. These tables were developed from the subgrade moduli ratios contained in Table B.45 of Appendix B.

Of particular note are Tables 4.10 and 4.11 which show the comparisons of western and eastern Washington subgrade moduli ratios based on selective and average summertime moduli. The final, average subgrade moduli ratios for all of the eastern or western Washington sites indicate very little difference whether the summer modulus was calculated using select or average summer moduli.

Base course moduli ratios were summarized similarly to those of subgrade. Tables 4.12 and 4.13 summarize western and eastern Washington moduli ratios determined by using a select summer moduli. Tables 4.14 and 4.15 are the results by using an average summer modulus. As with subgrade moduli ratios, the method of choosing a summer modulus had very little impact on the average of base course moduli ratios for the sites.

Table 4.7. Summary of Subgrade Moduli Ratios for Eastern Washington Test Sites (Using Select Summertime Modulus)

Test Site	Spring	Summer	Fall	Winter
14	0.88/1.18	1.00	0.79/1.16	
15	0.94/1.06	1.00	1.35	
16	1.60	1.00	1.70	
Average	0.91/1.28	1.00	0.79/1.40	

Note: 1. Moduli ratios greater than 1 reflect ratios where spring, fall or winter moduli were greater than the select summer modulus.

2. No data was collected for the winter season.

Table 4.8. Summary of Subgrade Moduli Ratios for Western Washington Test Sites (Using Average Summertime Modulus)

Test Site	Spring	Summer	Fall	Winter
1	0.80	1.00	0.94/1.02	0.70/1.06
2	0.82/1.05	1.00	0.80	0.78
3	0.82/1.10	1.00	0.95/1.07	0.94/1.23
6	0.99/1.11	1.00	0.88/1.01	0.77
7	1.13	1.00		
8	0.87/1.06	1.00	1.17	1.31
9	1.17/0.94	1.00	1.13	0.94/1.21
10	0.88/1.24	1.00	1.25	1.19
11	0.92/1.7	1.00	0.95	0.81/1.06
12	0.90/1.08	1.00	1.07	0.94 /1.02
Average	0.88/1.18	1.00	0.90/1.10	0.84/1.15

Note: 1. Moduli ratios greater than 1 reflect ratios where spring, fall or winter moduli were greater than the average summer modulus.

Table 4.9. Summary of Subgrade Moduli Ratios for Eastern Washington Test Sites (Using Average Summertime Modulus)

Test Site	Spring	Summer	Fall	Winter
14	0.89/1.16	1.00	0.83/1.16	
15	0.94/1.07	1.00	1.38	
16	1.66	1.00	1.79	
Average	0.92/1.30	1.00	0.83/1.44	

Note: 1. No data was collected for the winter season.

Table 4.10. Comparison of Subgrade Moduli Ratios Determined by Select and Average Summertime Modulus for Western Washington

Subgrade	Spring	Summer	Fall	Winter
Select	0.87/1.19	1.00	0.90/1.16	0.84/1.13
Average	0.88/1.18	1.00	0.90/1.10	0.84/1.15

Table 4.11. Comparison of Subgrade Moduli Ratios Determined by Select and Average Summertime Modulus for Eastern Washington

Base	Spring	Summer	Fall	Winter
Select	0.91/1.28	1.00	0.79/1.40	
Average	0.92/1.30	1.00	0.83/1.44	

Note: 1. No data was collected for the winter season.

Table 4.12. Summary of Base Moduli Ratios for Western Washington Test Sites (Using Select Summertime Modulus)

Test Site	Spring	Summer	Fall	Winter
1	0.88/1.07	1.00	0.94/1.04	0.76/1.04
11	0.82/1.12	1.00	0.91/1.06	0.72
Average	0.85/1.10	1.00	0.93/1.05	0.74/1.04

Table 4.13. Summary of Base Moduli Ratios for Eastern Washington Test Sites (Using Select Summertime Modulus)

Test Site	Spring	Summer	Fall	Winter
15	0.66/1.37	1.00	0.66/1.31	
Average	0.66/1.37	1.00	0.66/1.31	

Note: 1. No data was collected for the winter season.

Table 4.14. Summary of Base Moduli Ratios for Western Washington Test Sites (Using Average Summertime Modulus)

Test Site	Spring	Summer	Fall	Winter
1	0.92/1.05	1.00	0.98/1.07	0.80/1.04
11	0.81/1.09	1.00	0.91	0.69
Average	0.87/1.07	1.00	0.95/1.07	0.75/1.12

Table 4.15. Summary of Base Moduli Ratios for Eastern Washington Test Sites (Using Average Summertime Modulus)

Test Site	Spring	Summer	Fall	Winter
15	0.64/1.28	1.00	0.60/1.14	
Average	0.64/1.28	1.00	0.60/1.14	

Note: 1. No data was collected for the winter season.

6. DESIGN SEASONAL ADJUSTMENT FACTORS FOR WASHINGTON STATE DOT TEST SITES

This section will provide the appropriate "design" seasonal adjustment factors based on the seasonal moduli of the 13 test sites. Seasonal adjustment factors for a specific region (eastern or western Washington) were determined by averaging the moduli ratios of all sites for a particular season. As an example consider Table 4.6. The average of the spring subgrade moduli ratios for all western Washington sites (determined by using the select summertime moduli) is 0.87. For fall and winter the average ratios are 0.90 and 0.84 respectively. Table 4.7 includes the average moduli ratios for eastern Washington subgrade determined by using select summer moduli. Tables 4.8 and 4.9 provide average subgrade moduli ratios for western and eastern Washington ratios determined by using average summertime moduli.

Tables 4.12, 4.13, 4.14, and 4.15 provide the average moduli ratios determined by using select or average summertime moduli for the base course layer.

The seasonal factors listed in Table 4.16 are a summary of the average moduli ratios for each region and season. The seasonal factors are identified by seasonal period (spring,

Table 4.16. Design Moduli Ratios for Western and Eastern Washington Base Course and Subgrade Materials

Region		Seasonal Period: Spring	Summer	Fall	Winter
Western Washington	Climate:	Cool/Wet	Warm/Dry	Cool/Damp	Cool/Wet
	Months:	March April May	June July August September	October November	December January February
	Base	0.85	1.00	0.90	0.75
	Subgrade	0.85	1.00	0.90	0.85
Eastern Washington	Climate:	Thaw	Hot/Dry	Cool/Dry	Freeze
	Months:	February March April May	June July August September	October November December	January
	Base	0.65	1.00	0.90	1.10
	Subgrade	0.90	1.00	0.90	1.10

summer, fall, and winter) and eastern or western Washington regions. Within each season is listed the corresponding climate and months that are typical for that period. The seasonal factors in Table 4.16 are reflective of what a designer should consider for long-term design. (Note: These moduli ratios do not account for stress sensitive moduli relationships. This will be discussed in some detail later in the chapter.)

The seasonal factors in Table 4.16 reflect a variety of subgrade materials represented at the test sites. During the sampling of subgrade materials, improved subgrade materials were usually encountered in contrast to the native materials indicated by site selection. [19, 87] Often times WSDOT removes inferior subgrade materials and substitutes with improved borrow materials. Many of the test sites were built on fills with the natural subgrade several feet below the surface.

The moduli ratios for western Washington were chosen from summary Tables 4.6 and 4.12 which are based on the select summer modulus. The months included for each season are based on the temperature and precipitation data (Table B.1 and B.2 in Appendix B) that is characteristic for a particular period. For example for the summer months of June, July, August, and September, the mean air temperature is approximately 60°F and precipitation is low. The summer seasonal factor for base and subgrade is 1.0.

The fall months of October and November show increased precipitation and reduced temperatures. A 0.90 seasonal factor is used for the fall months for both the base and subgrade layers. The winter months of December, January, and February show increased precipitation and reduced temperatures of that over fall. The subgrade modulus is reduced by a factor of 0.85 and base 0.75. The spring season includes March, April, and May. Precipitation is reduced from winter period and temperatures increase. Seasonal factors are 0.85 for both base and subgrade materials.

Seasonal factors for eastern Washington subgrade and bases were determined similarly to those for western Washington. The primary difference is that the months included in the seasonal periods are different.

The major climatic change which affects layer moduli in eastern Washington is the freeze/thaw process. Generally this process occurs during a specific period each year. Lary et al. [57] has identified the thawing process as a two week period that generally occurs the last week of February through the first week in March. The duration for this process occurs at a minimum of two weeks. [57]

The spring season for the eastern Washington seasonal factors in Table 4.16 is identified by thawing conditions and includes the months of February, March, April and May. Basing the thaw period on a four month period accounts for variability in predicting the exact time of spring thaw and the resulting high moisture conditions that may occur in base and subgrade layers. The seasonal factors used for the spring seasonal period is 0.65 for the base course and 0.90 for the subgrade. These factors were selected from Tables 4.7 and 4.13.

The period where freezing conditions can be expected is typically the month of January. This time period is based on WSDOT experience as verified by the freezing monthly mean air temperature indicated for the month of January in Table B.2 (Appendix B). Table 4.16 specifies the winter season as freezing conditions and the month of January.

Inspection of Tables 4.7 and 4.13 show that no data was collected for the winter season for eastern Washington sites. Since frozen ground indicates increased base course or subgrade moduli a seasonal factor increased over that of summer was selected. The factor selected for the winter season (January) is 1.1.

The summer period of Table 4.16 is comprised of the months of June, July, August, and September. The months of June through September are specified as hot/dry months. The cool/dry months mean temperature (Table B.2, Appendix B) have a temperature range of about 45 to 50°F. Mean temperatures for hot/dry months ranges from about 50 to 70°F. Rainfall for the summer season is minimal throughout the entire period. The seasonal factor for summer months is 1.0.

The fall months (November and December) show a slight increase in rainfall and a decline in temperature (Tables B.1 and B.2, Appendix B). A seasonal factor of 0.90 for base course was selected for the eastern Washington fall season. The reason why the design ratio is 0.90 rather than the 0.66 ratio for fall base course (as shown on summary Table 4.13) is that the 0.66 average value was determined from only fall 1985 moduli. The fall 1985 moduli for Stations 0 + 50, 5 + 50, and 9 + 50 mostly indicated factors greater than one. The 0.90 ratio was selected primarily due to correspond to increased moisture that may result from increased precipitation as represented by weather data (Tables B.1 and B.2).

The ratio for eastern Washington subgrade was determined similarly. The eastern Washington fall subgrade seasonal factor was also selected at 0.90. The low number of eastern Washington test sites made selection of a reasonable seasonal factor difficult for both base course and subgrade materials.

In the original condition survey done for the 16 test sites [87], information was given as to the extent of pavement cracking. Table 4.17 is a summary of observed surface cracking and also a list of the individual subgrade moduli ratios determined for eastern and western Washington sites using select summertime values. The moduli ratios did not seem to correlate with the amount of cracking. For instance, western Washington Site 1, which had surface cracking did not have lower spring moduli values than Site 11 with no severe cracking. Winter values were about the same. A comparison of this nature is quite limited as each site experiences different geographic or micro climatic effects.

Table 4.17. Comparison of Condition Survey and Associated Moduli Ratios for Individual Test Sites (Using Ratios Determined from Select Summertime Modulus)

Test Site			Observed Surface Distress	Subgrade Modular Ratios			
No.	Route	Milepost		Spring	Summer	Fall	Winter
1	SR 11	20.85	Extensive cracking	0.87	1.00	0.97	0.77
2	SR 20	53.50	Minor wheel path cracking	0.83	1.00	0.80	0.78
3	SR 20	77.50	Extensive fatigue cracking	0.83	1.00	0.95	0.94
6	SR 167	17.80	No visible distress	0.99	1.00	0.88	0.77
7	SR 202	30.12	Extensive and severe fatigue cracking	-	-	-	-
8	SR 410	9.60	Extensive and severe fatigue cracking	0.78	1.00	0.80	0.60
9	SR 5	35.80	No cracking but rutting evident	0.90	1.00	0.96	0.89
10	SR 14	18.15	Light longitudinal cracking throughout test site	0.86	1.00	0.96	-
11	SR 411	18.05	No cracking present	0.82	1.00	0.84	0.79
12	SR 500	3.20	No cracking present	0.91	1.00	0.98	0.93
14	SR 90	208.85	Numerous transverse cracks, numerous longitudinal cracks	0.89	1.00	0.83	-
15	SR 195	7.24	Recent overlay	0.95	1.00	-	-
16	SR 195	63.80	Construction joint crack along entire center strip	-	-	-	-

Note: Test Sites No. 14, 15, and 16 are located in Eastern Washington. The remainder are in Western Washington.

7. STRESS SENSITIVITY ANALYSIS

The seasonal variations seen in pavement layer moduli are often associated with changing material properties; this depends on the material. For example, asphalt concrete moduli change occur with temperature. For granular or fine grain soils the change is most often associated with increased stiffness due to freezing and reduced stiffness with increased moisture levels.

While some of the moduli changes can be explained by seasonal variation there is another issue which needs to be explored. This issue is the modulus change caused by the stress sensitive nature of granular and fine grained materials found in base course and subgrade layers. Since these layers are stress sensitive, stress changes in a pavement structure can have a large impact on the resulting resilient modulus.

For a given load much of the stress distributed to unbound layers is a function of the asphalt concrete modulus. To illustrate this, consider a three layer pavement (asphalt concrete, granular base, and subgrade layers). In the summertime when prevailing temperatures are the highest, the asphalt concrete moduli will be the lowest. Stresses in the base course and subgrade layers will be the highest. In contrast, wintertime stresses in base course and subgrade layers will be the lowest due to the relatively high asphalt concrete modulus.

The result of the preceding example is that during the summer higher bulk stress develops in the base layer. For a granular stress sensitive material the higher bulk stress yields a higher resilient modulus. Conversely, in the winter, a lower bulk stress yields a lower resilient modulus (all other factors being equal). This relationship was observed during many of the testing dates for the WSDOT test sites. Summer resilient modulus may be higher but is this an environmental effect or simply a phenomena that can be explained by stress sensitivity? The subsequent sections explore this question.

8. STRESS SENSITIVITY RELATIONSHIPS FOR WASHINGTON STATE DOT TEST SITES

The pavement sections analyzed in this chapter are the 13 Washington State test sites previously described. Stress sensitivity relationships were developed for both laboratory samples and from the FWD data used to backcalculate layer moduli. Descriptions of each method follows.

8.1 LABORATORY STRESS SENSITIVITY RELATIONSHIPS

Although selection of the test sites was based on sites with a variety of subgrade soils, during sampling it was found the subgrades were predominantly coarse grained materials. The reason for this was that many of the sites were built on fill or improved subgrade materials. Often times the original, native subgrade was several feet below the surface and sampling native subgrade soil proved impractical. To represent the site for elastic analysis, subgrade material samples were taken directly below the base layer. Base course materials were more easily identifiable.

The samples of base course and subgrade materials were collected at or near Station 5 + 50 (recall: each test section was 1000 feet in length). The disturbed samples were remolded and recompacted as close as possible to field measured densities and moisture conditions. Triaxial tests were performed and, by correlating modulus with bulk or deviator stress, stress sensitivity relationships were developed.

Laboratory stress sensitivity equations for the test site are provided in Table B3, Appendix B. As indicated in the table most of the test site subgrades were "granular" materials. The samples were compacted and tested three times with most samples showing comparable results. The most variable is the June/July 1988 testing period. Specific modulus values are shown for bulk stress levels of 25 psi (base course) and deviator stress of 10 psi (subgrade) are shown.

8.2 BACKCALCULATED STRESS SENSITIVITY RELATIONSHIPS

Stress sensitivity relationships were also developed from the FWD data for each site for Stations 0 + 50, 5 + 50, and 9 + 50 (essentially the beginning, middle and end of each test section). At each station two FWD drops each were made at approximate load levels of 6,000, 9,000, 12,000, and 15,000 pounds. From each load the deflection basins as measured by the FWD data were used to backcalculate layer moduli using the EVERCALC Version 3.3 computer program [36]. Stations 0 + 50, 5 + 50, and 9 + 50 were chosen primarily because asphalt concrete cores (Table 4.3) were taken at these stations to verify asphalt thicknesses. Accurate layer thicknesses are needed for the backcalculation process. The stiff layer modulus was fixed at 1000 ksi.

The next step was to determine bulk stress or deviator stress levels occurring in base course and subgrade layers. The ELSYM5 elastic layer program [94] was used to compute bulk or deviator stress levels for each load level for each test date. Bulk or deviator stress levels were determined at the top of subgrade and top of base course layers. Site 15 had a base thickness of intermediate thickness (11 inches) and bulk stress was also computed at mid-depth. For Sites 1 and 11 mid-depth bulk stresses were not computed due to calculated tensile stresses occurring in these thick base layers (28.8 and 21 inches).

Stress sensitivity relationships for each season were then regressed from the backcalculated moduli and bulk stress or deviator stress values. Results for several of the Washington State test sites are shown in Tables C.1 to C.5 of Appendix C. Review of the results show a varying range of results as indicated by the coefficient of determination (R^2) values.

8.3 COMPARISON OF LABORATORY AND BACKCALCULATION EQUATION STRESS SENSITIVITY RELATIONSHIPS

Table 4.18 has been prepared for selected sites to compare laboratory and "field" stress sensitivity computed relationships. Regression equations chosen from Tables C.1 to C.15 for comparison were selected to correspond to the station (5 + 50) and date (summer 1985) when field samples were taken. The sites showing the closest comparison were Sites 1 and 15. Site 1 showed a good comparison for the base course material only. Site 15 compared well for both the base course and subgrade materials. It is important to note that of all the WSDOT test sites, Site 15 was the closest to being a "classic" three layer pavement structure. For this site, 6.9 inches of asphalt concrete was placed on 11.4 inches of base which rested on native eastern Washington Palouse silt.

Subgrade regression and laboratory stress sensitivity equations generally showed little similarity. The effect of the improved fills over the subgrade material likely had an influence making sites slightly different than three layer structures. The deflection data from the FWD represents the complete pavement system including improved fill and native subgrade.

Overall the comparison of the laboratory and backcalculated stress sensitivity relationships is poor but for some understandable reasons.

9. SEASONAL VARIATION AND STRESS SENSITIVITY COMPARISON

To determine the effect of stress sensitivity on the seasonal variation of moduli, five WSDOT test sites were considered. The sites were chosen to provide a variety of pavement sections with asphalt concrete thicknesses ranging from 3.6 to 17.9 inches as indicated in Table 4.19. Three of the sites had substantial base layers.

Comparison of seasonal variation in backcalculated layer moduli to stress sensitive moduli computed from laboratory relationships was done in the following manner. For the sites at Stations 0 + 50, 5 + 50, and 9 + 50, bulk or deviator stress levels were determined for the unbound layers of the pavement structure at the locations indicated in Table 4.20. Bulk or deviator stresses were determined by using the backcalculated moduli and a 9000 pound load as input to the ELSYM5 computer program.

The laboratory relationships for specific sites were next evaluated using the ELSYM5 computed bulk or deviator stress levels for each test date and station (0 +50, 5 + 50, and 9 +50). Use of the laboratory relationships requires the assumption that the K_2 coefficient remains constant. A constant K_2 coefficient is indicated in the discussion in Chapter 2, Section 2.1.2.1, and also by Lary et al. [93] Comparing the moduli

Table 4.18. Comparison of Field (Backcalculated) and Laboratory Determined Stress Sensitivity Coefficients

		Regression Equation Determined			Laboratory Determined		
Site	Material	K_1 (K_3)	K_2 (K_4)	R^2	K_1 (K_3)	K_2 (K_4)	R^2
1	Base	8397	0.371	0.97	7844	0.375	0.87
					8154	0.362	0.83
					9847	0.320	0.87
	Subgrade	17478	-1.250	0.94	5278	0.299	0.83
					7629	0.418	0.90
					8977	0.373	0.88
2	Subgrade	11633	0.047	0.28	5278	0.531	0.60
					6693	0.511	0.88
					5172	0.595	0.86
3	Subgrade	33667[1]	-0.456	0.96	6220	0.476	0.95
					10837	0.366	0.88
					9306	0.403	0.90
6	Subgrade	34890	-0.2666	0.75	-	-	-
7	Subgrade	75527	-0.396	0.88	-	-	-
8	Subgrade	7965	-0.2618	0.95	(3194)	(0.358)	0.93
					3102	0.293	0.64
					(4511)	(0.21)	0.54
9	Subgrade	42786[1]	-0.168	0.45	-	-	-
10	Subgrade	10852	0.021	0.02	13901	0.260	0.88
					16742	0.234	0.74
					17956	0.198	0.68
11	Base	11703	0.314	0.84	4768	0.436	0.90
					5423	0.385	0.89
					6013	0.398	0.87
12	Subgrade	9895	0.008	0.00	7074	0.270	0.90
					9428	0.242	0.84
					9916	0.337	0.85
15	Base	6732	0.401	0.84	6012	0.449	0.88
					9947	0.312	0.83
					10011	0.344	0.87
	Subgrade	21493	-0.234	0.79	(18049)	(-0.291)	0.56
					(20160)	(-0.247)	0.67
					(16019)	(-0.301)	0.89
16	Subgrade	7290	-0.225	0.94	(4960)	(+0.079)	0.30
					(13079)	(-0.184)	0.69
					(7466)	(-0.091)	0.39

Note: 1. Negative stress sensitivity relationship expressed with bulk stress

Table 4.19. Summary of Layer Thicknesses for Test Sites Used in Stress Sensitivity Analysis

Site	Asphalt ConcreteThickness inch	Base Thickness inch
1	5 - 5.5	28.8
3	10.4 - 11.5	None
9	15.4 - 17.9	None
11	6.6 - 7.0	21.0
15	3.6 - 6.9	11.4

Table 4.20. Summary of Locations for Bulk or Deviator Stress for WSDOT Test Sites Used in Stress Sensitivity Analysis

Site	Bulk Stress	Deviator Stress
1	Top Base Top Subgrade	- -
3	Top Subgrade	-
9	Top Subgrade	-
11	Top Base	-
15	Top Base Mid Base -	- - Top Subgrade

determined from the laboratory relationship evaluated at each stress level with backcalculated moduli is an imperfect comparison, but for this study it allows a comparison in trends of the moduli.

Comparisons of backcalculated and laboratory computed subgrade and base resilient moduli are shown in Tables 4.21, 4.22, 4.23, and 4.24 for Site 1, Stations 0 + 50 and 5 + 50. Site 1, Station 9 + 50 base and subgrade and the remaining sites listed in Table 4.19 are included in Tables C.16 to C.35, Appendix C. The backcalculated moduli listed in Tables 4.21 and 4.22 are from the EVERCALC runs. The laboratory moduli

Table 4.21. Comparison of Backcalculated and Laboratory Measured Base Course Moduli for Site 1 (SR 11, MP 20.85, Station 0+50)

Date	Mean Asphalt Temp. °F	Backcalculated[1] Modulus Asphalt (ksi)	Backcalculated[1] Modulus Base (ksi)	Backcalculated[1] Difference in Base Modulus (ksi)	Backcalculated[1] Difference in Base Modulus (%)	Laboratory Measured[2] Bulk Stress Top Base (psi)	Laboratory Measured[2] Modulus Base (ksi)	Laboratory Measured[2] Difference in Base Modulus (ksi)	Laboratory Measured[2] Difference in Base Modulus (%)
21-May-85	66	673.1	28.0			24.5	26.6		
				-2.1	-7.5			-0.3	-1.2
8-Aug-85	85	691.2	25.9			23.7	26.3		
				-1.9	-7.3			-1.1	-4.2
16-Jul-86	50	822.1	24.0			21.0	25.2		
				0.3	1.3			-0.4	-1.5
9-Oct-86	46	908.4	24.3			20.1	24.8		
				0.6	2.5			-1.7	-6.7
27-Jan-87	26	1348.1	24.9			16.5	23.1		
				0.4	1.6			5.0	21.5
1-Jun-87	89	434.1	25.3			28.6	28.1		
				0.2	0.8			-1.1	-3.9
17-Aug-87	92	565.5	25.5			25.6	27.0		
				0.0	0.0			-1.7	-6.2
12-Nov-87	61	832.4	25.5			21.4	25.3		
				0.5	2.0			-2.5	-10.0
1-Feb-88	36	1578.5	26.0			15.8	22.8		

Notes: 1. Stiff Layer Resilient Modulus = 50 ksi, Asphalt Thickness = 5.5 inch, Base Thickness = 28.8 inch
2. Laboratory Modulus = $8615\ \theta^{.352}$

Table 4.22. Comparison of Backcalculated and Laboratory Measured Base Course Moduli for Site 1 (SR 11, MP 20.85, Station 5+50)

Date	Mean Asphalt Temp. F	Backcalculated[1]				Laboratory Measured[2]			
		Modulus		Difference in Base Modulus		Bulk Stress Top Base	Modulus	Difference in Base Modulus	
		Asphalt (ksi)	Base (ksi)	(ksi)	(%)	(psi)	Base (ksi)	(ksi)	(%)
21-May-85	66.0	719.2	22.0			24.8	26.7		
				1.2	5.5			0.2	0.7
8-Aug-85	85.0	755.9	23.2			25.3	26.9		
				-4.5	-19.4			-1.5	-5.6
16-Jul-86	50.0	850.7	18.7			21.5	25.4		
				3.1	16.6			-0.9	-3.7
9-Oct-86	46.0	1233.1	21.8			19.3	24.4		
				-7.7	-35.3			-2.9	-12.1
27-Jan-87	26.0	1632.1	14.1			13.4	21.5		
				6.5	46.1			6.6	30.6
1-Jun-87	89.0	501.3	20.6			28.6	28.1		
				2.8	13.6			-0.1	-0.5
17-Aug-87	92.0	588.5	23.4			28.2	28.0		
				0.8	3.4			-2.1	-7.4
12-Nov-87	61.0	968.7	24.2			22.7	25.9		
				-5.5	-22.7			-3.0	-11.6
1-Feb-88	36.0	1490.9	18.7			16.0	22.9		

Notes: 1. Stiff Layer Resilient Modulus = 50 ksi, Asphalt thickness = 5.0 inch, Base Thickness = 28.8 inch
2. Laboratory Modulus = 8615 $\theta^{.352}$

Table 4.23. Comparison of Backcalculated and Laboratory Measured Subgrade Moduli for Site 1 (SR 11, MP 20.85, Station 0 + 50)

Date	Mean Asphalt Temp. °F	Backcalculated[1] Modulus Asphalt (ksi)	Backcalculated[1] Modulus Subgrade (ksi)	Backcalculated[1] Difference in Subgrade Modulus (ksi)	Backcalculated[1] Difference in Subgrade Modulus (%)	Laboratory Measured[2] Bulk Stress Top Subgrade (psi)	Laboratory Measured[2] Modulus Subgrade (ksi)	Laboratory Measured[2] Difference in Subgrade Modulus (ksi)	Laboratory Measured[2] Difference in Subgrade Modulus (%)
21-May-85	66	673.10	13.0			2.1	11.1		
				5.3	40.8			1.0	8.6
8-Aug-85	85	691.20	18.3			2.5	12.0		
				-3.2	-17.5			-0.4	-3.1
16-Jul-86	50	822.10	15.1			2.3	11.6		
				-0.3	-2.0			-0.1	-1.2
9-Oct-86	46	908.40	14.8			2.3	11.5		
				-2.9	-19.6			-2.1	-18.1
27-Jan-87	26	1348.10	11.9			1.4	9.4		
				1.8	15.1			2.3	24.6
1-Jun-87	89	434.10	13.7			2.4	11.7		
				1.3	9.5			0.0	0.0
17-Aug-87	92	565.50	15.0			2.4	11.7		
				-0.5	-3.3			-0.3	-2.6
12-Nov-87	61	832.40	14.5			2.2	11.4		
				2.7	18.6			-0.2	-1.5
1-Feb-88	36	1578.50	17.2			2.2	11.3		

Notes: 1. Stiff Layer Resilient Modulus = 50 ksi, Asphalt Thickness = 5.5 inch, Base Thickness = 28.8 inch
2. Laboratory Modulus = $8303\ \theta^{.395}$

Table 4.24. Comparison of Backcalculated and Laboratory Measured Subgrade Moduli for Site 1 (SR 11, MP 20.85, Station 5 + 50)

Date	Mean Asphalt Temp. °F	Backcalculated[1] Modulus Asphalt (ksi)	Backcalculated[1] Modulus Subgrade (ksi)	Backcalculated[1] Difference in Subgrade Modulus (ksi)	Backcalculated[1] Difference in Subgrade Modulus (%)	Laboratory Measured[2] Bulk Stress Top Subgrade (psi)	Laboratory Measured[2] Modulus Subgrade (ksi)	Laboratory Measured[2] Difference in Subgrade Modulus (ksi)	Laboratory Measured[2] Difference in Subgrade Modulus (%)
21-May-85	66	719.2	8.3			2.1	11.1		
				7.2	86.7			1.1	9.5
8-Aug-85	85	755.9	15.5			2.6	12.1		
				-4.9	-31.6			-0.5	-3.9
16-Jul-86	50	850.7	10.6			2.3	11.6		
				1.7	16.0			-0.2	-1.7
9-Oct-86	46	1233.1	12.3			2.2	11.4		
				-3.6	-29.3			-0.4	-3.7
27-Jan-87	26	1632.1	8.7			2.0	11.0		
				0.9	10.3			0.6	5.2
1-Jun-87	89	501.3	9.6			2.3	11.6		
				1.9	19.8			0.1	0.7
17-Aug-87	92	588.5	11.5			2.4	11.7		
				0.2	1.7			-0.3	-2.4
12-Nov-87	61	968.7	11.7			2.2	11.4		
				-3.2	-27.4			-0.5	-4.6
1-Feb-88	36	1490.9	8.5			2.0	10.9		

Notes: 1. **Stiff Layer Resilient Modulus = 50 ksi ACP Thickness = 5.0 inch, Base Thickness = 28.8 inch**
2. **Laboratory Modulus = 8303 $\theta^{.395}$**

were determined by using the appropriate laboratory determined stress sensitivity equation and ELSYM5 computed bulk or deviator stress values (the ELSYM5 program used backcalculated moduli). The difference in base course or subgrade modulus from season to season are shown for both backcalculated and laboratory conditions.

Figures 4.5, 4.6, and 4.7 show graphically the percentage change in moduli for base course materials at Station 5 + 50 for Sites 1, 11, and 15. Percent changes for both backcalculated and laboratory determined resilient moduli are indicated. A modest amount of agreement in these trends are observed.

Specifically, Site 11 (Figure 4.6) shows the closest similarity in trends of moduli changes from season to season. Some agreement is observed in Site 1 (Figure 4.5), from January 1987 to February 1988. The poorest agreement occurs at Site 15.

Percentage increases or decreases in subgrade backcalculated and laboratory determined moduli for Sites 1, 3, 9, and 15 Station 5 + 50 are shown in Figures 4.8, 4.9, 4.10, and 4.11. Site 15 (Figure 4.11) shows the strongest agreement between the backcalculated and laboratory moduli. Sites 1, 3, and 9 show more of a random nature.

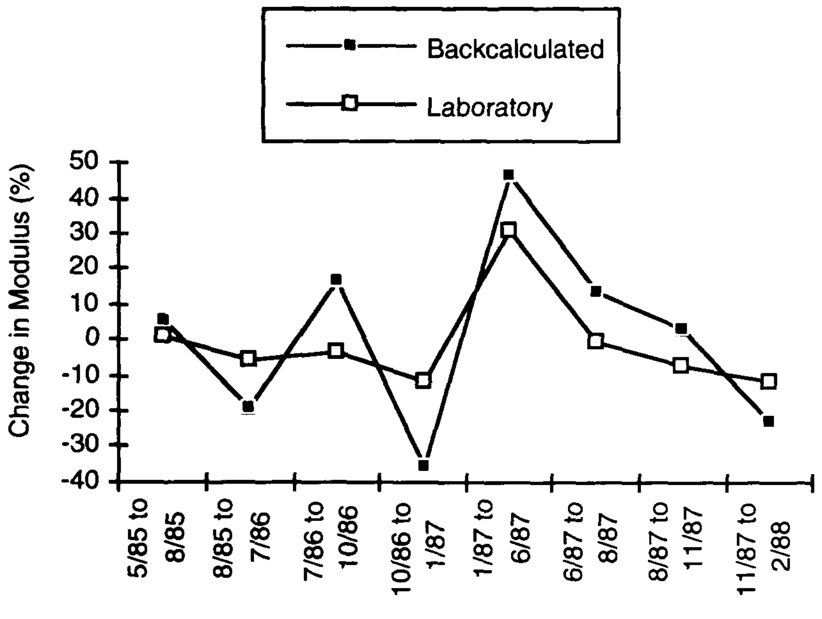


Figure 4.5. Relative Change in Site 1 Base Course Moduli at Station 5+50 (SR11, MP 20.85)

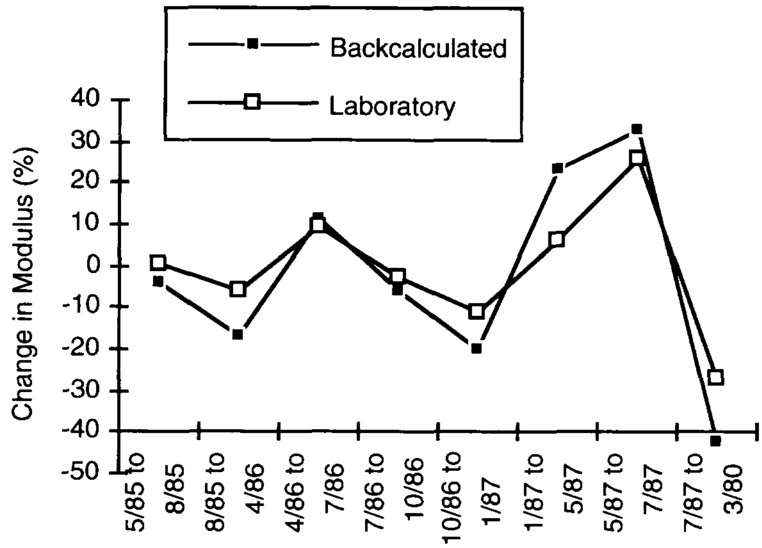


Figure 4.6. Relative Change in Site 11 Base Course Moduli at Station 5+50 (SR11, MP 18.05)

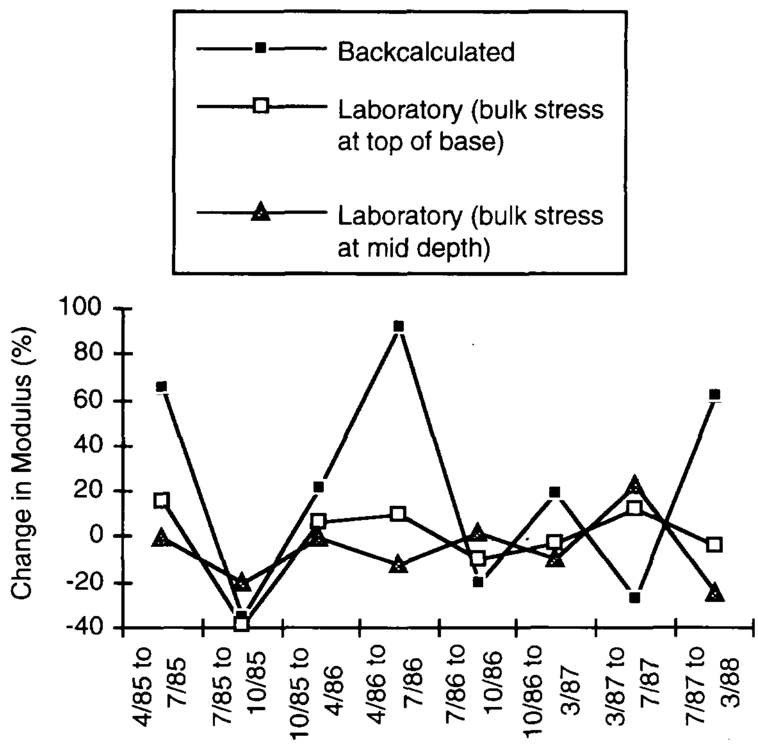


Figure 4.7. Relative Change in Site 15 Base Course Moduli at Station 5+50 (SR195, MP 7.24)

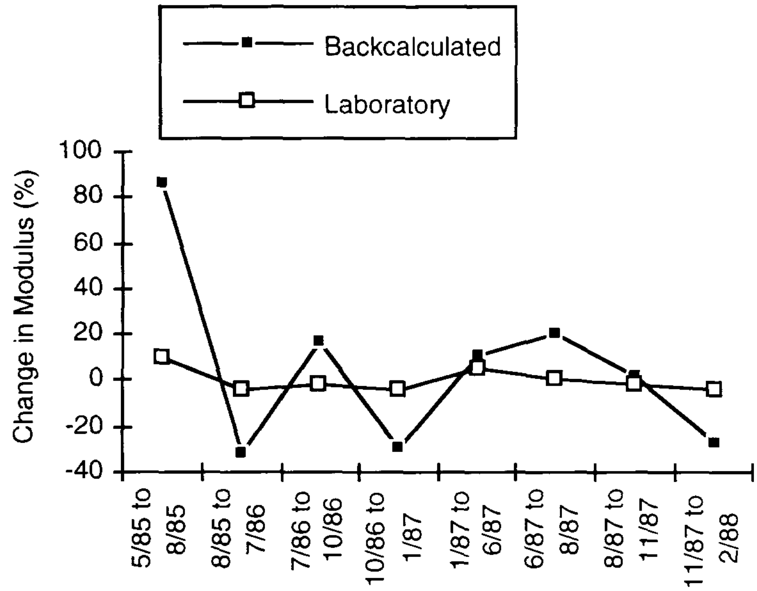


Figure 4.8. Relative Change in Site 1 Subgrade Moduli at Station 5+50 (SR11, MP 20.85)

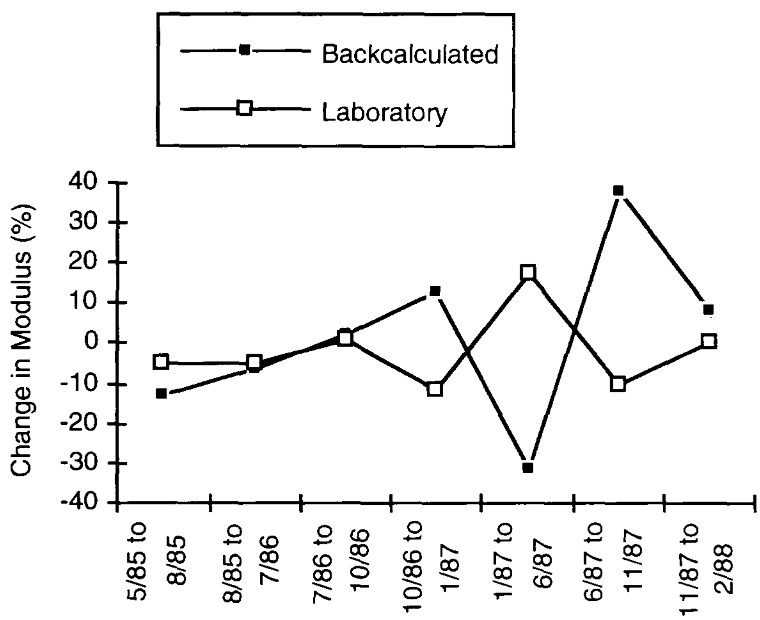


Figure 4.9. Relative Change in Site 3 Subgrade Moduli at Station 5+50 (SR20, MP 77.50)

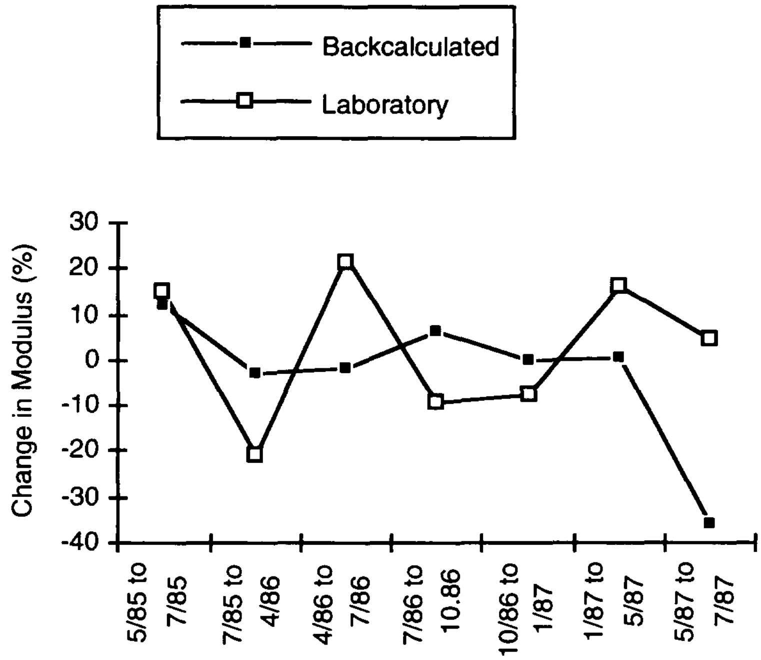


Figure 4.10. Relative Change in Site 9 Subgrade Moduli at Station 5+50 (SR5, MP 35.80)

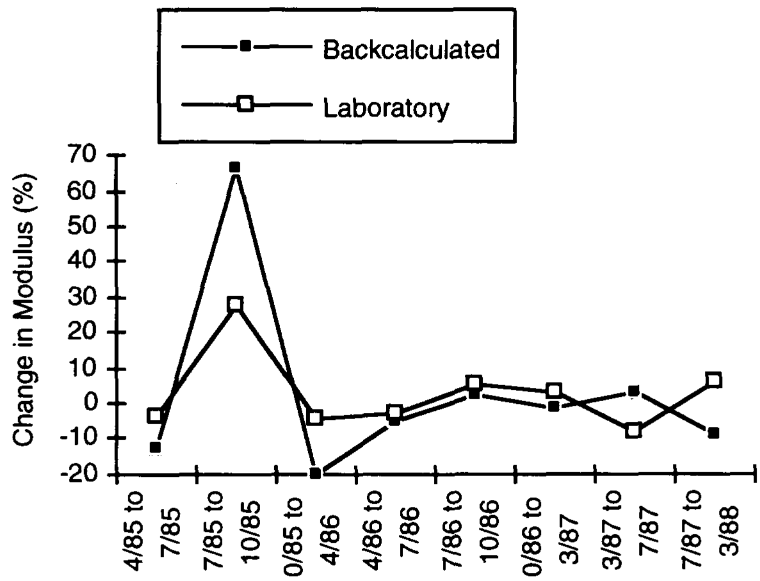


Figure 4.11. Relative Change in Site 15 Subgrade Moduli at Station 5+50 (SR195, MP 7.24)

As a final analysis of stress sensitivity, consideration was given to comparing moduli determined by backcalculation and field regression equations. This analysis will be used to determine seasonal factors determined by evaluating the field regression equations at the same bulk stress level (recall: the backcalculated moduli are evaluated at different stress levels). The field regression stress sensitivity equations as discussed in Section 2.2 were developed from the WSDOT test site data backcalculated using a stiff layer resilient modulus of 1,000 ksi. The results for Test Site 1 are used in this analysis.

The regression equation relationships were used similarly to the laboratory stress sensitivity relationships in determining the seasonal variation of layer moduli. The only exception was that for a given season the bulk stress that was selected to evaluate the equation was that of a selected summer season. Selection of one summertime bulk stress level evaluates each of the regression equations (which are different for a given season) at the same bulk stress level. Tables 4.25, 4.26, and 4.27 show the comparison of backcalculated and regression equation determined moduli for Site 1 Station 0 + 50, 5 + 50, and 9 + 50. Indicated on each table is the summer season from which bulk stress was determined. The percent differences in moduli are compared to this summer test date.

Moduli ratios for the regression equations determined moduli were developed according to the method described earlier in this chapter. Table 4.28 shows the moduli ratios for Site 1 Stations 0 + 50, 5 + 50, and 9 + 50 based on a select summer modulus. For comparison the laboratory relationship for the Site 1 base course was used to compute layer moduli. Results for Stations 0 + 50, 5 + 50, and 9 + 50 are shown in Tables 4.29, 4.30, and 4.31. Moduli ratios for the laboratory determined moduli are shown in Table 4.32. The select summer modulus for the same season as was used for the regression equation ratios and is indicated on the table. Table 4.33 show moduli ratios for the backcalculated moduli where a 1000 ksi stiff layer was used. The select summer moduli as chosen as was discussed above.

Table 4.34 shows the average ratios of all stations for individual season determined by the regression equation, laboratory relationship, and backcalculated moduli. Some agreement is seen in the factors determined by different means.

Figures 4.12, 4.13, and 4.14 show the relationship of backcalculated to laboratory determined resilient moduli for the Test Site 1 base course. Backcalculation was performed with a stiff layer of 1,000 ksi. The figures show poor agreement.

Table 4.25. Comparison of Backcalculated and Regression Equation Measured Base Course *Moduli for Site 1 (SR 11 MP 20.85 Station 0+50)*

Date[1]	Mean Asphalt Temp. °F	Backcalculated[2] Modulus Asphalt (ksi)	Backcalculated[2] Modulus Base (ksi)	Backcalculated[2] Difference in Base Modulus (ksi)	Backcalculated[2] Difference in Base Modulus (%)[3]	Regression Equation Measured Bulk Stress Top Base psi	Regression Equation Measured Modulus Base (ksi)	Regression Equation Measured Difference in Base Modulus (ksi)	Regression Equation Measured Difference in Base Modulus (%)[3]
21-May-85	66	495.8	34.0	-0.7	-2.0	31.1	34.4	-0.5	-1.5
8-Aug-85	85	468.5	34.7	0.0	0.0	31.1	34.9	0.0	0.0
16-Jul-86	50	616.1	30.1	-4.6	-13.3	31.1	32.3	-2.6	-7.5
9-Oct-86	46	690.3	30.0	-4.7	-13.5	31.1	32.3	-2.6	-7.4
27-Jan-87	26	970.1	32.4	-2.3	-6.6	31.1	35.6	0.7	2.1
1-Jun-87	89	320.6	30.6	-4.1	-11.8	31.1	28.3	-6.6	-18.9
17-Aug-87	92	416.4	31.3	-3.4	-9.8	31.1	31.2	-3.7	-10.6
12-Nov-87	61	652.3	30.6	-4.1	-11.8	31.1	32.7	-2.1	-6.1
1-Feb-88	36	1264.9	32.2	-2.5	-7.2	31.1	37.5	2.6	7.5

Notes: 1. Shaded Area Denotes Selected Summer Season

2. Stiff Layer Resilient Modulus = 1000 ksi, Asphalt Thickness = 5.5 inch, Base Thickness = 28.8 inch

3. Percent Difference Based From Select Summer Season

Table 4.26. Comparison of Backcalculated and Regression Equation Measured Base Course Moduli for Site 1 (SR 11 MP 20.85 Station 5+50)

Date[1]	Mean Asphalt Temp. °F	Backcalculated[2] Modulus Asphalt (ksi)	Backcalculated[2] Modulus Base (ksi)	Difference in Base Modulus (ksi)	Difference in Base Modulus (%)[3]	Regression Equation Measured Bulk Stress Top Base (psi)	Modulus Base (ksi)	Difference in BasPe Modulus (ksi)	Difference in BasPe Modulus (%)[3]
21-May-85	66	596.0	24.6	-6.0	-19.6	33.5	25.1	-5.7	-18.6
8-Aug-85	85	503.3	30.6	0.0	0.0	33.5	30.9	0.0	0.0
16-Jul-86	50	658.7	22.8	-7.8	-25.5	33.5	25.3	-5.5	-17.9
9-Oct-86	46	897.1	26.8	-3.8	-12.4	33.5	29.3	-1.5	-5.0
27-Jan-87	26	1463.4	16.1	-14.5	-47.4	33.5	22.2	-8.6	-28.0
1-Jun-87	89	387.8	23.8	-6.8	-22.2	33.5	23.5	-7.4	-23.9
17-Aug-87	92	422.6	28.3	-2.3	-7.5	33.5	27.5	-3.4	-10.9
12-Nov-87	61	716.4	29.5	-1.1	-3.6	33.5	31.4	0.5	1.6
1-Feb-88	36	1205.0	22.4	-8.2	-26.8	33.5	26.6	-4.3	-13.8

Notes: 1. Shaded Area Denotes Selected Summer Season

2. Stiff Layer Resilient Modulus = 1000 ksi, Asphalt thickness = 5.0 inch, Base Thickness = 28.8 inch

3. Difference Based From Select Summer Season

Table 4.27. Comparison of Backcalculated and Regression Equation Measured Base Course Moduli for Site 1 (SR 11 MP 20.85 Station 9+50)

Date[1]	Mean Asphalt Temp. °F	Backcalculated[2] Modulus Asphalt (ksi)	Backcalculated[2] Modulus Base (ksi)	Backcalculated[2] Difference in Base Modulus (ksi)	Backcalculated[2] Difference in Base Modulus (%)[3]	Regression Equation Measured Bulk Stress Top Base (psi)	Regression Equation Measured Modulus Base (ksi)	Regression Equation Measured Difference in Base Modulus (ksi)	Regression Equation Measured Difference in Base Modulus (%)[3]
21-May-85	66	731.1	18.9	-3.5	-15.6	26.1	19.1	-2.5	-11.6
8-Aug-85	85	573.6	22.4	0.0	0.0	26.1	21.6	0.0	0.0
16-Jul-86	50	903.7	17.6	-4.8	-21.4	26.1	19.1	-2.5	-11.4
9-Oct-86	46	1002.0	21.4	-1.0	-4.5	26.1	22.8	1.2	5.8
27-Jan-87	26	1148.3	15.0	-7.4	-33.0	26.1	18.8	-2.8	-12.9
1-Jun-87	89	396.8	17.4	-5.0	-22.3	26.1	16.9	-4.7	-21.7
17-Aug-87	92	447.1	23.1	0.7	3.1	26.1	22.1	0.5	2.5
12-Nov-87	61	854.3	24.1	1.7	7.6	26.1	24.1	2.5	11.6
1-Feb-88	36	1253.2	16.4	-6.0	-26.8	26.1	19.9	-1.7	-8.0

Notes: 1. Shaded Areas Denotes Select Summer Season

2. Stiff Layer Resilient Modulus = 1000 ksi , Asphalt Thickness = 5.2 inch, Base Thickness = 28.8 inch

3. Difference Based From Select Summer Season

Table 4.28. Seasonal Base Moduli and Moduli Ratios for WSDOT Test Site 1 (SR 11, MP 20.85)
(Ratios Determined With the Indicated Select Summertime Modulus With Regression Equation Relationship)

Test Site	Station		1985			1986				1987				1988		Select Summer Modulus (ksi)
			SPR	SUM	FAL	WIN	SPR	SUM	FAL	WIN	SPR	SUM	FAL	WIN	SPR	
1	0 +50	Modulus (ksi)	34.30	34.90				32.30	32.30	35.60	28.30	31.20	32.70	37.50		34.90
		Ratio	0.98	1.00				0.93	0.93	1.02	0.81	0.89	0.94	1.07		
1	5 + 50	Modulus (ksi)	25.10	30.90				25.30	29.30	22.20	23.50	27.50	31.40	26.60		30.90
		Ratio	0.81	1.00				0.82	0.95	0.72	0.76	0.89	1.02	0.86		
1	9 + 50	Modulus (ksi)	19.10	21.60				19.10	22.80	18.80	16.90	22.10	24.10	19.90		21.60
		Ratio	0.88	1.00				0.88	1.06	0.87	0.78	1.02	1.12	0.92		

Table 4.29. Comparison of Backcalculated and Laboratory Measured Base Course Moduli for Site 1 (SR 11 MP 20.85 Station 0+50)

Date	Mean Asphalt Temp. °F	Backcalculated[1] Modulus Asphalt (ksi)	Backcalculated[1] Modulus Base (ksi)	Backcalculated[1] Difference in Base Modulus (ksi)	Backcalculated[1] Difference in Base Modulus (%)	Laboratory Measured[2] Bulk Stress Top Base (psi)	Laboratory Measured[2] Modulus Base (ksi)	Laboratory Measured[2] Difference in Base Modulus (ksi)	Laboratory Measured[2] Difference in Base Modulus (%)
21-May-85	66	495.8	34.0			30.1	28.6		
				0.7	2.1			0.3	1.2
8-Aug-85	85	468.5	34.7			31.1	28.9		
				-4.6	-13.3			-2.0	-6.8
16-Jul-86	50	616.1	30.1			25.5	27.0		
				-0.1	-0.3			-0.3	-1.2
9-Oct-86	46	690.3	30.0			24.6	26.6		
				2.4	8.0			-1.3	-4.7
27-Jan-87	26	970.1	32.4			21.5	25.4		
				-1.8	-5.6			4.7	18.6
1-Jun-87	89	320.6	30.6			34.8	30.1		
				0.7	2.3			-1.0	-3.5
17-Aug-87	92	416.4	31.3			31.5	29.0		
				-0.7	-2.2			-2.1	-7.1
12-Nov-87	61	652.3	30.6			25.5	27.0		
				1.6	5.2			-2.6	-9.7
1-Feb-88	36	1264.9	32.2			19.1	24.4		

Notes: 1. Stiff Layer Resilient Modulus = 1000 ksi, Asphalt Thickness = 5.5 inch, Base Thickness = 28.8 inch

2. Laboratory Modulus = $8615\ \theta^{.352}$

Table 4.30. Comparison of Backcalculated and Laboratory Measured Base Course Moduli for Site 1 (SR 11 MP 20.85 Station 5+50)

Date	Mean Asphalt Temp. °F	Backcalculated[1] Modulus Asphalt (ksi)	Backcalculated[1] Modulus Base (ksi)	Backcalculated[1] Difference in Base Modulus (ksi)	Backcalculated[1] Difference in Base Modulus (%)	Laboratory Measured[2] Bulk Stress Top Base (psi)	Laboratory Measured[2] Modulus Base (ksi)	Laboratory Measured[2] Difference in Base Modulus (ksi)	Laboratory Measured[2] Difference in Base Modulus (%)
21-May-85	66	596.0	24.6			28.2	27.9		
				6.0	24.4			1.7	6.2
8-Aug-85	85	503.3	30.6			33.5	29.7		
				-7.8	-25.5			-2.5	-8.5
16-Jul-86	50	658.7	22.8			26.1	27.2		
				4.0	17.5			-0.7	-2.4
9-Oct-86	46	897.1	26.8			24.3	26.5		
				-10.7	-39.9			-4.2	-15.9
27-Jan-87	26	1463.4	16.1			14.9	22.3		
				7.7	47.8			7.4	33.4
1-Jun-87	89	387.8	23.8			33.7	29.8		
				5.7	23.9			0.4	1.3
17-Aug-87	92	422.6	29.5			35.0	30.1		
				-7.1	-24.1			-2.2	-7.3
12-Nov-87	61	716.4	22.4			28.2	27.9		
				-3.7	-16.5			-3.5	-12.7
1-Feb-88	36	1205.0	18.7			19.2	24.4		

Notes: 1. Stiff Layer Resilient Modulus = 1000 ksi, Asphalt thickness = 5.0 inch, Base Thickness = 28.8 inch

2. Laboratory Modulus = $8615\ \theta^{.352}$

Table 4.31. Comparison of Backcalculated and Laboratory Measured Base Course Moduli for Site 1 (SR 11 MP 20.85 Station 9+50)

Date	Mean Asphalt Temp. °F	Backcalculated[1]				Laboratory Measured[2]			
		Modulus		Difference in Base Modulus		Bulk Stress Top Base	Modulus Base	Difference in Base Modulus	
		Asphalt (ksi)	Base (ksi)	(ksi)	(%)	(psi)	(ksi)	(ksi)	(%)
21-May-85	66	731.1	18.9			21.5	25.4		
				3.5	18.5			1.8	7.1
8-Aug-85	85	573.6	22.4			26.1	27.2		
				-4.8	-21.4			-3.0	-11.0
16-Jul-86	50	903.7	17.6			18.7	24.2		
				3.8	21.6			0.3	1.3
9-Oct-86	46	1002.0	21.4			19.4	24.5		
				-6.4	-29.9			-2.2	-8.8
27-Jan-87	26	1148.3	15.0			14.9	22.3		
				2.4	16.0			5.3	23.9
1-Jun-87	89	396.8	17.4			27.5	27.7		
				5.7	32.8			0.7	2.4
17-Aug-87	92	447.1	23.1			29.4	28.4		
				1.0	4.3			-2.7	-9.5
12-Nov-87	61	854.3	24.1			22.1	25.7		
				-7.7	-32.0			-3.3	-12.8
1-Feb-88	36	1253.2	16.4			15.0	22.4		

Notes: 1. Stiff Layer Resilient Modulus = 1000 ksi, Asphalt Thickness = 5.2 inch, Base Thickness = 28.8 inch

2. Laboratory Modulus = $8615\ \theta^{.352}$

Table 4.32. Seasonal Base Moduli and Moduli Ratios for WSDOT Test Site 1 (SR 11, MP 20.85)
(Ratios Determined Using Laboratory Determined Moduli with the Indicated Select Summertime Modulus)

Test Site	Station		1985			1986				1987				1988		Select Summer Modulus (ksi)
			SPR	SUM	FAL	WIN	SPR	SUM	FAL	WIN	SPR	SUM	FAL	WIN	SPR	
1	0 +50	Modulus (ksi)	28.60	28.90				27.00	26.60	25.40	30.10	29.00	27.00	24.40		28.90
		Ratio	0.99	1.00				0.93	0.92	0.88	1.04	1.00	0.93	0.84		
1	5 + 50	Modulus (ksi)	27.90	29.70				27.20	26.50	22.30	29.80	30.10	27.90	24.40		29.70
		Ratio	0.94	1.00				0.92	0.89	0.75	1.00	1.01	0.94	0.82		
1	9 + 50	Modulus (ksi)	25.40	27.20				24.20	24.50	22.30	27.70	28.40	25.70	22.40		27.20
		Ratio	0.93	1.00				0.89	0.90	0.82	1.02	1.04	0.94	0.82		

Table 4.33. Seasonal Base Moduli and Moduli Ratios for WSDOT Test Site 1 (SR 11, MP 20.85)
(Ratios Determined Using Backcalculated Moduli with the Indicated Select Summertime Modulus)

Test Site	Station		1985			1986				1987				1988		Select Summer Modulus (ksi)
			SPR	SUM	FAL	WIN	SPR	SUM	FAL	WIN	SPR	SUM	FAL	WIN	SPR	
1	0 +50	Modulus (ksi)	34.00	34.70				30.10	30.00	32.40	30.60	31.30	30.60	32.20		34.70
		Ratio	0.98	1.00				0.87	0.86	0.93	0.88	0.90	0.88	0.93		
1	5 + 50	Modulus (ksi)	18.90	22.40				17.60	21.40	15.00	17.40	23.10	24.10	16.40		22.40
		Ratio	0.84	1.00				0.79	0.96	0.67	0.78	1.03	1.08	0.73		
1	9 + 50	Modulus (ksi)	24.60	30.60				22.80	26.80	16.10	23.80	29.50	22.40	18.70		30.60
		Ratio	0.80	1.00				0.75	0.88	0.53	0.78	0.96	0.73	0.61		

Table 4.34. Summary of Base Course Moduli Ratios Determined by Regression Equation, Laboratory Relationship, and Backcalculated Moduli (SR 11, MP 20.85 - Base Course)

Calculation Method	Spring	Summer	Fall	Winter
Regression Equation (Evaluated at the Same Stress Level)	0.84	1.00	0.94/1.07	0.84/1.04
Laboratory Relationship (Evaluated at the Variable Stress Levels)	0.95/1.02	1.00	0.92	0.82
Backcalculated (Evaluated at Variable Stress Levels)	0.84	1.00	0.90/1.08	0.73

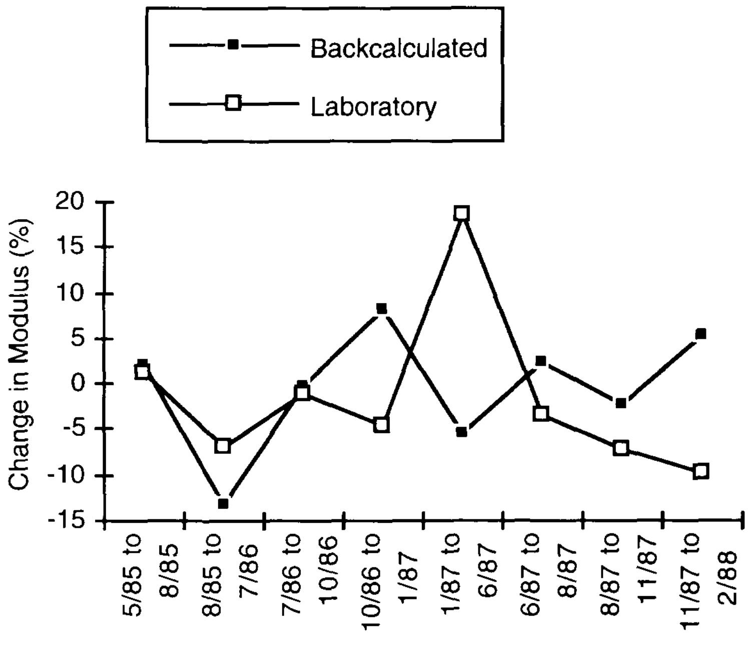


Figure 4.12. Relative Change in Site 1 Base Course Moduli at Station 0+50 (SR11, MP 20.85, stiff layer resilient modulus = 1000 ksi)

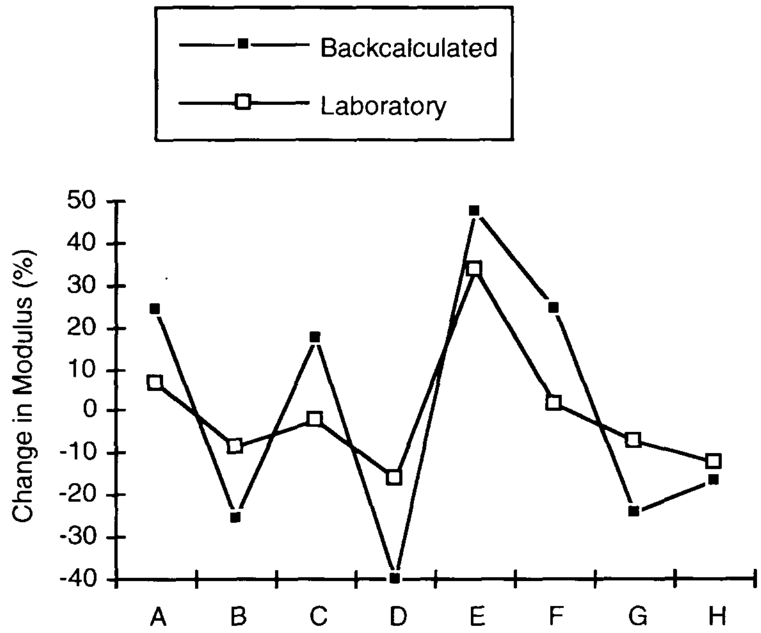


Figure 4.13. Relative Change in Site 1 Base Course Moduli at Station 5+50 (SR11, MP 20.85, stiff layer resilient modulus = 1000 ksi)

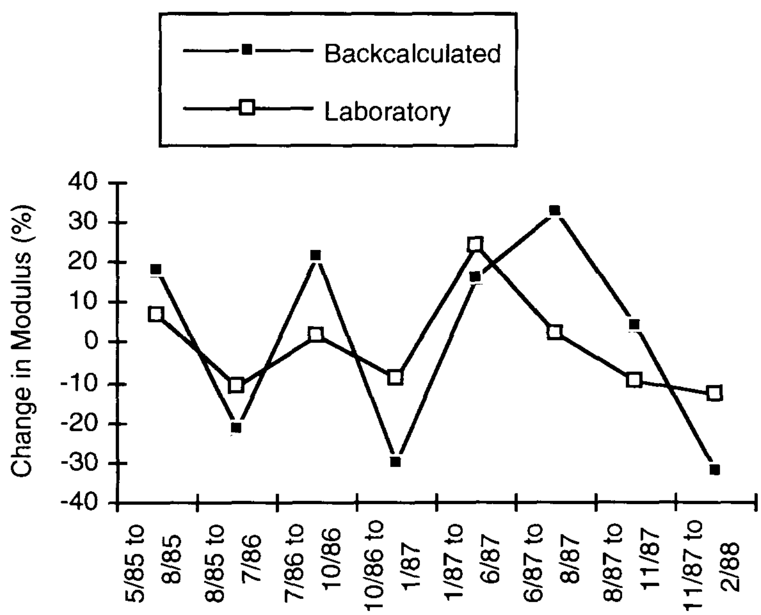


Figure 4.14. Relative Change in Site 1 Base Course Moduli at Station 9+50 (SR11, MP 20.85, stiff layer resilient modulus = 1000 ksi)

10. IMPLICATIONS FOR DESIGN

The implication of the preceding analysis is that seasonal variation observed in base and subgrade layers may be more closely related to stress sensitivity than has previously been given credit. If this is the case, then the seasonal adjustment factors as discussed in this chapter (and specifically Table 4.16) can be overly conservative if the unbound layers are characterized by stress sensitive relationships. Designs done without the use of stress sensitivity relationships should be able to take advantage of the moduli ratios as developed in this chapter without modification.

CHAPTER 5

EXAMINATION OF NDT DATA FOR U.S. FOREST SERVICE TEST SITES

1. INTRODUCTION

The USDA- FS contracted with Pavement Services, Inc., to obtain FWD pavement deflection data on 15 test sites in the Olympia National Forest. Of these test sites, five were aggregate surfaced and ten asphalt surfaced. The FWD field testing occurred from November 1989 to November 1990 with 10 separate test periods. Additionally, Benkelman Beam testing was conducted from April 1990 to June 1991 with 11 separate test periods.

2. TEST SITE DESCRIPTIONS

The 15 test sites monitored in the Olympic National Forest are described in Tables 5.1 and 5.2. In general, the aggregate surfaced sections ranged from 10 to 22 inches of aggregate over subgrades ranging from gravel to silts and clays. The asphalt surfaced sections had asphalt thicknesses ranging from 2.1 to 5.8 inches over subgrades ranging from gravels to sands and silts. The elevation of the test sites ranged from a low of about 300 feet to a high of about 1,000 feet. Clearly, the range of conditions (layer thicknesses and materials) was large.

The test sites were located on the west and north sides of the Olympic Peninsula in an area of coastal marine weather with rainfall exceeding 80 inches per year. Only brief periods of subfreezing temperatures occurred and ground freezing was, at most, less than 6 inches. Thawing effects were deleted from the data analysis. Temperatures are moderated by the marine influence and typically range from winter lows between 20°F and 30°F to summertime highs between 70°F and 80°F. During the test period, rainfall ranged from 110 to 130 inches depending on location. This rainfall occurred during five months of "heavy" rain (5 to 6 inches per week), three months of "moderate" rain at 1.5 inches per week, and 4 months of little or no rainfall.

3. DATA COLLECTION

FWD deflection data were collected on 10 separate visits during the period from November 1989 to November 1990. Further, Benkelman Beam surveys were done on 11 separate visits during the period from April 1990 to June 1991.

At each site, five "spots" were tested, "A" through "E," each being 15 feet apart. At asphalt surface sites, the spot locations were painted on the pavement surface; at the

Table 5.1. Olympic National Forest Test Site Descriptions

Site No.	Test Site Road No. — MP	Surface Type[1]	Asphalt Thickness (inch)	Aggregate Thickness (inch)	Subgrade USCS	Rigid Layer @[2] (inch)	Elevation (feet)	Nearest NOAA Station[3]
1	22-37.30	AGG		10"	ML	32"	650'	Aberdeen (20NNE)
2	22-27.05	AGG		22"	GM	87"	750'	
3	22-12.10	AC	2.1"	16"	GM	519"	580'	
4	22-10.05	AC	2.1"	19"	SM	118"	780'	
5	22-8.05	AC	4.8"	29"	GM	377"	520'	
6	22-4.40	AC	4.7"	12"	GM	600"	320'	
7	2220-0.40	AGG		15"	ML	157"	520'	
8	2220-1.60	AGG		18"	CL	53"	560'	
9	21-0.30	AC	5.8"	12"	ML	70"	510'	
10	21-4.00	AC	5.3"	25"	SM	600"	580'	
11	30-0.30	AC	3.0"	11"	GW-GM	68"	640'	Sappho (8E)
12	30-3.65	AC	3.5"	12"	ML	145"	850'	
13	29-36.90	AC	3.8"	26"	ML	196"	960'	
14	2065-0.50	AGG		14"	GM	21"	800'	
15	2918-2.15	AC	3.8"	19"	SM	262"	1060'	

Notes:
1. Surface type
   AGG = Aggregate surface
   AC = Asphalt concrete
2. Rigid layer depth as calculated by EVERCALC
3. NOAA Station elevations:
   Aberdeen (20NNE) — 435'
   Sappho (8E) — 760'

Table 5.2. Subgrade Materials Properties—Olympic National Forest Test Sites

Site No.	Test Site Road No.—MP	USCS	% Passing			Plastic Index	Maximum Density (pcf)	Optimum Moisture (%)	Soaked CBR @		
			1.0"	No. 4	No. 200				90%	95%	100%
1	22-37.30	ML	94	83	65	NP	79.6	38	6.4	10.8	15.3
2	22-27.05	GM	86	54	24	NP	118	14.2	10.3	19.3	28.3
3	22-12.10	GM	92	50	16	NP	105.9	22	2.8	11.1	19.5
4	22-10.05	SM	93	72	26	NP					
5	22-8.05	GM	84	47	20	NP	86.3	28.8	5.8	14.3	22.8
6	22-4.40	GM	76	38	13	NP					
7	2220-0.40	ML	90	83	61	NP	71.5	43	3.9	4.4	5
8	2220-1.60	CL	98	74	66	13	99.3	23.9	4.7	6.8	8.8
9	21-0.30	ML	99	96	62	NP					
10	21-4.00	SM	100	96	44	NP	106.9	17.1	9.8	16.4	23
11	30-0.30	GW-GM	78	29	6.3	NP					
12	30-3.65	ML	99	90	54	NP	99.2	23.9	6.3	10.1	13.8
13	29-36.90	ML	98	81	64	2					
14	2065-0.50	GM	90	58	30	NP	96	25.8	6.9	8.1	9.4
15	2918-2.15	SM	96	88	29	NP	99.5	22.8	12.5	16.4	20.2

aggregate surface sites, offset stakes were used. FWD testing was conducted at each of the 75 spots (15 test sites x 5 spots each) at roughly one month intervals. The FWD was provided by Pavement Services, Inc., and was a PaveTech/KUAB device. This FWD utilized LVDT type seismometers for deflection measurement. At each test spot on each test date, seven deflection basins were measured: two at a load of 12 kips, two at about 8.5 kips, two about 6 kips, and an initial seating drop. Deflections were measured at radial distances of 0, 12, 18, 24, 36, and 60 inches at the asphalt surfaced sites, and 0, 12, 18, 24, and 36 inches at the aggregate surfaced sites. The test spots were generally located between the wheel tracks since this was the most feasible for the trailer mounted FWD.

For deflection basin analysis, the two drops at each load were averaged and a linear interpolation made to calculate deflections at the 9,000 lb target load. The 0-, 12-, 18-, 24-, and 36-inch sensors were generally used for backcalculation with selected sensors manually deleted during analysis if an obvious "kink" occurred at that sensor location in the deflection basin.

Three AC cores were obtained from each pavement site and dimetral resilient modulus tests were conducted at each of three temperatures. The resulting modulus versus temperature correlations were used with contractor measured site temperatures as input to the EVERCALC analysis.

4. SEASONAL SURFACE DEFLECTIONS

As discussed in Chapter 2, pavement surface deflections collected on the Olympic test sites can be used to estimate seasonal deflection ratios (recall Tables 2.20 and 2.21). The source data are shown in Tables 5.3 and 5.4 where Table 5.3 is FWD Sensor 1 (adjusted to a 9,000 lb load and a 70°F) and Table 5.4 is for actual Benkelman Beam data (also corrected to a 9,000 lb load (1/2 of a standard single axle load) and 70°F)).

Table 5.5 shows seasonal deflection ratios for each test site visit based on the deflections from the FWD Sensor 1. The ratios are based on an "average summer" deflection as noted in the table. Table 5.6 was prepared in the same manner as Table 5.5 except the source data was from a Benkelman Beam.

A comparison of the deflection ratios shown in Tables 5.5 and 5.6 for the same test sections and comparable time periods shows similar results with a few exceptions.

Various plots of seasonal deflection factors are shown in Volume II, Appendix D (Figures D.1-D.8).

Table 5.3. Benkelman Beam Equivalent Deflections @ FWD Sensor No. 1 (mils)—Olympic National Forest Test Sites

Site No.	Test Site Road No. — MP	Test Date									
		11/28-29/89	1/3-4/90	2/22-23/90	3/6-7/90	3/29-30/90	5/3-4/90	6/7-8/90	8/29-30/90	10/2-3/90	11/5-6/90
1	22-37.30	86	100	110	109	85	100	100	102	103	95
2	22-27.05	16	15	15	16	16	15	16	15	15	16
3	22-12.10	19	20	ICE/SNOW	20	19	17	18	17	17	22
4	22-10.05	23	24	ICE/SNOW	24	22	22	21	19	19	24
5	22-8.05	12	12	23	12	11	12	12	12	12	13
6	22-4.40	4	4	6	4	4	4	4	6	5	5
7	2220-0.40	33	37	42	34	40	41	35	29	30	35
8	2220-1.60	26	38	ICE/SNOW	40	27	38	40	15	19	32
9	21-0.30	7	7	8	8	7	7	7	8	8	7
10	21-4.00	9	9	11	8	8	7	9	9	9	9
11	30-0.30	23	24	32	25	20	20	20	19	20	26
12	30-3.65	29	30	42	36	30	28	28	25	26	31
13	29-36.90	20	23	28	27	23	21	22	20	20	25
14	2065-0.50	31	33	24	35	34	37	34	31	29	31
15	2918-2.15	15	17	13	19	14	14	14	13	13	16

Deflections corrected to 9,000 lb load and 70°F

Table 5.4. Benkelman Beam Deflections (mils)—Olympic National Forest Test Sites

Site No.	Test Site Road No.—MP	Test Date										
		4/17-18/90	5/31-6/1/90	6/21-22/90	9/19-20/90	11/28-29/90	1/23-24/91	2/27-28/91	3/19-20/91	4/24-25/91	5/15-16/91	6/25-26/91
3	22-12.10	25	25	24	22	25	31	31	28	24	19	22
4	22-10.05	29	29	28	25	29	33	32	31	29	22	25
5	22-8.05	14	14	14	14	13	20	16	16	14	14	12
6	22-4.40	5	6	9	4	4	7	7	7	8	4	7
9	21-0.30	13	10	11	9	10	13	14	13	9	7	15
10	21-4.00	12	12	14	9	11	14	13	13	9	8	13
11	30-0.30	25	20	23	24	29	35	33	34	24	23	23
12	30-3.65	33	31	31	27	36	44	42	44	39	30	34
13	29-36.90	27	26	26	22	31	33	35	29	39	30	30
15	2918-2.15	20	18	16	14	17	20	24	16	14	18	19

Deflections corrected to 9,000 lb load and 70°F.

Table 5.5. Seasonal Deflection Factors Based on FWD Sensor No. 1 (mils)—Olympic National Forest Test Sites

Site No.	Test Site Road No.—MP	Test Date									
		11/28-29/89	1/3-4/90	2/22-23/90	3/6-7/90	3/29-30/90	5/3-4/90	6/7-8/90	8/29-30/90	10/2-3/90	11/5-6/90
1	22-37.30	0.84	0.98	1.07	1.06	0.83	0.98	0.98	1.00*	1.00*	0.93
2	22-27.05	1.07	1.00	1.02	1.07	1.07	1.00	1.07	1.00*	1.00*	1.07
3	22-12.10	1.12	1.18	ICE/SNOW	1.18	1.12	1.00	1.06	1.00*	1.00*	1.29
4	22-10.05	1.21	1.26	ICE/SNOW	1.26	1.16	1.16	1.11	1.00*	1.00*	1.26
5	22-8.05	1.00	1.00	1.92	1.00	0.92	1.00	1.00	1.00*	1.00*	1.08
6	22-4.40	0.73	0.73	1.09	0.73	0.73	0.73	0.73	1.09*	0.91*	0.91
7	2220-0.40	1.12	1.25	1.42	1.15	1.36	1.39	1.19	0.98*	1.02*	1.19
8	2220-1.60	1.53	2.24	ICE/SNOW	2.35	1.59	2.24	2.35	0.88*	1.12*	1.88
9	21-0.30	0.88	0.88	1.00	1.00	0.88	0.88	0.88	1.00*	1.00*	0.88
10	21-4.00	1.00	1.00	1.22	0.89	0.89	0.78	1.00	1.00*	1.00*	1.00
11	30-0.30	1.18	1.23	1.64	1.28	1.03	1.03	1.03	0.97*	1.03*	1.33
12	30-3.65	1.14	1.18	1.65	1.41	1.18	1.10	1.10	0.98*	1.02*	1.22
13	29-36.90	1.00	1.15	1.40	1.35	1.15	1.05	1.10	1.00*	1.00*	1.25
14	2065-0.50	1.03	1.10	0.80	1.17	1.13	1.23	1.13	1.03*	0.97*	1.03
15	2918-2.15	1.15	1.31	1.00	1.46	1.08	1.08	1.08	1.00*	1.00*	1.23

*Indicates deflection test values averaged for seasonal factor = 1.00.

Table 5.6. Seasonal Deflection Factors Based on Benkelman Beam Testing—Olympic National Forest Test Sites

Site No.	Test Site Road No.—MP	Test Date										
		4/17-18/90	5/31-6/190	6/21-22/90	9/19-20/90	11/28-29/90	1/23-24/91	2/27-28/91	3/19-20/91	4/24-25/91	5/15-16/91	6/25-26/91
3	22-12.10	1.15	1.15	1.10*	1.01*	1.15	1.43	1.43	1.29	1.10	0.87*	1.01*
4	22-10.05	1.16	1.16	1.12*	1.00*	1.16	1.32	1.28	1.24	1.16	0.88*	1.00*
5	22-8.05	1.00	1.00	1.00*	1.00*	0.93	1.43	1.14	1.14	1.00*	1.00*	0.86
6	22-4.40	0.83	1.00	1.50*	0.67*	0.67	1.17	1.17	1.17	1.33	0.67*	1.17*
9	21-0.30	1.44	1.11	1.22*	1.00*	1.11	1.44	1.56	1.44	1.00	0.78*	1.67
10	21-4.00	1.09	1.09	1.27*	0.82*	1.00	1.27	1.18	1.18	0.82	0.73*	1.18*
11	30-0.30	1.08	0.86	0.99*	1.03*	1.25	1.51	1.42	1.46	1.03	0.99*	0.99*
12	30-3.65	1.08	1.02	1.02*	0.89*	1.18	1.44	1.38	1.44	1.28	0.98*	1.11*
13	29-36.90	1.00	0.96	0.96*	0.81*	1.15	1.22	1.30	1.07	1.44	1.11*	1.11*
15	2918-2.15	1.19	1.07	0.96*	0.84*	1.01	1.19	1.43	0.96	0.84	1.07*	1.13*

*Indicates deflection test values used for seasonal factor basis = 1.00.

4.1 TRENDS—AGGREGATE SURFACED SITES

For the aggregate surfaced test sites, the maximum deflection ratios were obtained from Table 5.5. The following results:

Site No.	Aggregate Thickness (inches)	Subgrade Classification	Estimated Depth to Stiff Layer (inches)	Maximum Deflection Ratio (and Time of Occurrence)
1	10	ML	32	1.07 (2/90)
2	22	GM	87	1.07 (11/89, 3/90, 6/90, 11/90)
7	15	ML	157	1.42 (2/90)
8	18	CL	53	2.35 (3/90, 6/90)
14	14	GM	21	1.23 (5/90)

There are not clear "causes" as to why a specific site has a higher or lower deflection ratio; however, an additional examination of the data shows:

Site No.	Date of Maximum Deflection Ratio	Maximum Deflection (mils)
1	2/90	110
2	11/89, 3/90, 6/90, 11/90	16, 16, 16, 16
7	2/90	42
8	3/90, 6/90	40, 40
14	5/90	37

Clearly, the test section with the thinnest aggregate surface (Site No. 1) has an extremely high deflection—in fact, throughout the year. This suggests that the ML subgrade soil along with a shallow depth to stiff layer suggests a drainage problem. The thickest aggregate section (Site No. 2) on a GM subgrade soil has a low deflection essentially throughout the year. The three remaining test sites have little difference in aggregate thickness and the subgrade soils "suggest" a trend in deflection ratio.

4.2 SUMMARY OF DEFLECTION RATIOS—AGGREGATE SURFACED SITES

Overall, there exists some consistency when comparing deflection ratios obtained during similar time periods (1990 and 1991). Given this and the prior analysis, it appears that a deflection ratio of 1.5 might be adequate (say to use as the seasonal adjustment factor in The Asphalt Institute's MS-17). The critical period varies but generally occurs during February and March.

4.3 TRENDS—ASPHALT SURFACED SITES

For the asphalt surfaced test sites, the maximum deflection ratios were obtained from Tables 5.5 and 5.6. The following results:

Site No.	Asphalt Thickness (inches)	Aggregate Thickness (inches)	Subgrade Classification	Estimated Depth to Stiff Layer (inches)	Maximum Deflection Ratio (and Time of Occurrence)			
					FWD		BB	
3	2.1	16	GM	519	1.29	(11/90)	1.43	(1/91, 2/91)
4	2.1	19	SM	118	1.26	(1/90, 11/90 3/90)	1.32	(1/91)
5	4.8	29	GM	377	1.92	(2/90)	1.43	(1/91)
6	4.7	12	GM	600	1.09	(2/90, 8/90)	1.33	(4/91)
9	5.8	12	ML	70	1.00	(several)	1.56	(2/91)
10	5.3	25	SM	600	1.22	(2/90)	1.27	(6/90, 1/91)
11	3.0	11	GW-GM	68	1.64	(2/90)	1.51	(1/91)
12	3.5	12	ML	145	1.65	(2/90)	1.44	(1/91, 3/91)
13	3.8	26	ML	196	1.40	(2/90)	1.44	(4/91)
15	3.8	19	SM	262	1.46	(3/90)	1.43	(2/91)

Data from both the FWD and the Benkelman Beam are shown and there is general agreement between the deflection ratios with a couple of notable exceptions (Site Nos. 5 and 9). As was the case for the aggregate surfaced sections, there is no clear trend with the exception that higher asphalt thicknesses generally result in lower deflection ratios. A listing of the maximum measured deflections will assist in this examination.

Site No.	Maximum Deflection (mils) FWD	BB
3	22	31
4	24	33
5	23	20
6	6	8
9	8	14

Site No.	Maximum Deflection (mils) FWD	BB
10	11	14
11	32	35
12	42	44
13	28	39
15	19	24

The test sections with thicker asphalt surfaces exhibit lower measured maximum surface deflections. Further, the three ML subgrades seem to contribute to higher deflections. These two observations are as one might expect. The depths to stiff layers show no clear trends.

4.4 SUMMARY OF DEFLECTION RATIOS—ASPHALT SURFACED SITES

Overall, there exists some consistency when comparing maximum deflections, deflection ratios, and critical time periods (recall that the FWD and Benkelman Beam deflection data were collected in partially overlapping time periods). Generally, the critical deflection period, as expected, occurred during the months of January, February, or March. As was the case of the aggregate surfaced sites, a deflection ratio of 1.5 appears to be adequate.

4.5 DEFLECTION ADJUSTMENTS BY TIME OF YEAR

The deflections shown in Tables 5.3 and 5.4 further suggest how they can be adjusted depending on when such data are collected. This breakdown is approximate and likely varies a bit from year to year.

Time Period when Deflection Data Obtained	Deflection Data Adjustment Factor
January-March	1.00
April-June	1.25
July-September	1.50
October-December	1.25

5. BACKCALCULATION OF LAYER MODULI

5.1 INTRODUCTION

The backcalculation program (EVERCALC) as described in Chapter 4 was used to estimate the test site layer moduli.

The backcalculated moduli are summarized in Tables 5.7 and 5.8, and include both the aggregate and asphalt surface sites. The discussion of these results will be in separate subsections that follow.

5.2 MODULI FOR AGGREGATE SURFACED SITES

The backcalculated RMS values are generally poor ranging from 2 to 12 percent with some exceeding 20 percent. The moduli shown in Tables 5.7 and 5.8 are average results based on the five test spots at each site (Table 5.7) and lowest RMS value at a specific spot (Table 5.8). Best fit spots at three different sites had RMS's of about 1 to 3 percent over the year of deflection testing.

Table 5.7. EVERCALC Determined Pavement Layer Moduli (ksi) Using Lab Determined Asphalt Values—Average Results from 5 Test Spots at Each Site (Olympic National Forest)

Site No.	Test Site Road No.—MP	Pavement Layer	Test Date									
			11/28-29/89	1/3-4/90	2/22-23/90	3/6-7/90	3/29-30/90	5/3-4/90	6/7-8/90	8/29-30/90	10/2-3/90	11/5-6/90
1	22-37.30	Aggregate	48.4	39.1	35.2	37.2	50.8	36.2	37.5	39.1	38	39.8
		Subgrade	1.1	1.1	1.1	1.1	1.2	1.1	1.1	1	1.1	1.1
		RMS	10.0	9.7	10.7	11.4	7.4	8.5	7.4	9.5	11.1	10.2
2	22-27.05	Aggregate	68	63.5	57.1	60.8	53.8	54.1	54	69.2	69.9	58.5
		Subgrade	10.2	13.5	16.2	12.4	17.8	15.5	15.4	9.5	9.5	10.9
		RMS	7.9	12.6	11.1	10.5	12.7	12.8	12.4	4.2	4.8	7.5
3	22-12.10	Asphalt	718.1	953.7	ICE/SNOW	889.4	590.7	432.5	543.3	397.8	579.8	940.5
		Aggregate	49.7	43.3		43.7	44.4	45.9	41.9	51.6	53.5	43.6
		Subgrade	18.1	20.4		20.9	21.5	20.8	21.5	16.9	17.4	18.3
		RMS	2.8	4.6		3.2	4.8	5.1	5.2	3.1	2	2.1
4	22-10.05	Asphalt	445.7	698.2	ICE/SNOW	638.3	509.9	389.5	426.1	311.2	445.7	730.3
		Aggregate	31.3	28.1		28.1	28.7	30.7	30.8	41.5	41.7	30.6
		Subgrade	11.4	13.6		13.9	14	13.4	13.2	10.6	11.3	12.1
		RMS	7.5	9.1		9.7	12.8	11.3	11.3	6.6	4.7	6.5
5	22-8.05	Asphalt	1408.1	1765.5	3316.3	1542.9	556.2	224	655.1	624.4	882.9	2059.4
		Aggregate	38.2	36.3	2.9	36.2	45.7	51.7	42.7	46.9	43.8	26.8
		Subgrade	9.5	11.4	37.6	11.7	10.4	9.8	10.2	8.6	9.6	12
		RMS	7.7	6.9	7.2	7.7	12.7	14.2	11.4	9.8	9	4.4
6	22-4.40	Asphalt	566.2	910.8	1554.5	808.7	232.3	66.7	312.6	206.3	331.7	910.8
		Aggregate	284.6	121.7	123.3	158	199	481.7	155	335.5	322.9	269.8
		Subgrade	59.8	105.5	116.2	113.3	111.7	99.8	111.2	44.9	48.2	58.9
		RMS	5	14.3	4.9	18.6	16.7	20.5	17.7	4.3	3.8	2
7	2220-0.40	Aggregate	54.2	45.5	37.8	49.9	40.7	41.6	55.3	67.6	60.1	53.2
		Subgrade	5	4.9	5.1	5.2	5.1	4.8	4.6	5.5	5.4	4.7
		RMS	6.9	8.7	10.1	9.8	8.4	6.9	8.5	4.1	4.9	6.6
8	2220-1.60	Aggregate	57.2	30.7	ICE/SNOW	35.5	45.6	31.7	31.6	99.7	74.3	41.6
		Subgrade	2.9	3.6		3.2	4.1	3.6	3.2	4.2	4.2	3.2
		RMS	12	12.4		13.1	7.6	10	13.5	2.8	4.6	8.9
9	21-0.30	Asphalt	440.6	825.1	698.2	718.6	294.4	129.3	209.7	154.5	232.5	526.6
		Aggregate	165.4	109	107.4	128.1	116.4	110.2	140.7	196.5	139.4	128
		Subgrade	10.5	16.2	13.3	15.2	17.8	24.1	17.9	11.6	12	12.2
		RMS	4	4.4	3.5	5.1	6.6	15.3	9.2	4.3	3.3	2.2
10	21-4.00	Asphalt	1101	1504	2074.9	1233.8	1134.2	623.1	607.9	511.2	822.1	1362.2
		Aggregate	79.9	52.6	27.3	65.6	47.9	71.1	70	74.8	66.1	61.8
		Subgrade	13.4	19.6	21.3	20.2	21.2	19.1	18.6	14.8	14.7	15.4
		RMS	4	2.3	2.8	2.1	5.6	5.8	5.7	4.1	4.6	2.4
11	30-0.30	Asphalt	800.9	894.5	1361.4	1099.5	587.8	574.9	572.8	423.4	556.2	1157.7
		Aggregate	30.4	19.1	22.7	18.9	26.3	26.3	26.8	40.1	38.2	31
		Subgrade	10.7	14.2	10.7	13.8	14	14	13.8	10	10.4	11
		RMS	4.2	4.9	2.6	8.4	8.2	7.9	10.7	3.8	3.2	2.3
12	30-3.65	Asphalt	924.8	1022.1	1691.4	1433.6	777.9	649.3	766	531.6	669.5	1433.6
		Aggregate	17.2	12.9	6	8.1	13.9	16.9	16.5	25.2	23.4	15.7
		Subgrade	8.9	10.4	12.2	11.8	10.5	10.3	10.4	9.1	9.7	11
		RMS	1.3	3	5.5	2.6	2.4	1.7	3.1	1.8	1.9	3.9
13	29-36.90	Asphalt	874.1	1065.7	1238.3	1273.9	904.3	745.9	798.3	643.7	763	1273.9
		Aggregate	20.8	16.1	12.8	13.2	15.1	15.7	17.4	21.5	20.9	15.8
		Subgrade	11.1	14	13.6	14.2	13.4	13.2	12.1	9.5	10.2	12.3
		RMS	1.5	3.8	5.4	4.3	2.7	2.5	5.1	3.7	2.5	3.6
14	2065-0.50	Aggregate	56.7	41	62.3	46.5	39	36.7	40.5	55.8	54.5	50.8
		Subgrade	1.8	2.3	1.6	1.7	1.7	1.7	2	1.7	1.7	1.9
		RMS	21.1	17.8	9.1	20.8	11.8	14.2	26.2	19.6	20.8	21.1
15	2918-2.15	Asphalt	794.3	1127.2	1258.9	1479.1	376.7	613.8	657.7	436.5	642.7	1264.7
		Aggregate	38.1	25.3	45.1	20.6	34.5	28.3	35.2	50.6	47.1	36.3
		Subgrade	17.5	22.4	25.1	20.7	19.2	22.9	20.3	16.3	17.3	18.9
		RMS	4.8	4.8	4	3.9	9	8.4	4.4	7.5	5.9	3.5

Table 5.8. EVERCALC Determined Pavement Layer Moduli (ksi) Using Lab Determined Asphalt Values—Best RMS Spot at Designated Sites (Olympic National Forest)

Site No.	Test Site Road No.—MP	Pavement Layer	Test Date 11/28-29/89	1/3-4/90	2/22-23/90	3/6-7/90	3/29-30/90	5/3-4/90	6/7-8/90	8/29-30/90	10/2-3/90	11/5-6/90
1	22-37.30 SPOT "C"	Aggregate	76.4	80.7	57.4	64.3	102.5	68.1	73.8	74.8	72.2	70.7
		Subgrade	1	0.8	1	0.9	1	0.9	0.9	0.8	1	1
		RMS	9.8	2.1	1.9	2.9	2	2.7	1.6	3.7	3.8	3.1
2	22-27.05 SPOT "A"	Aggregate	69.2	74	62.2	65	64	60.2	57.9	72.7	81.1	66.2
		Subgrade	5.1	4.7	6.2	5.4	6	5.8	6.1	4.8	4.9	5.4
		RMS	1.9	2.7	1.9	3.9	1.2	1.7	1	1.2	2.6	2.3
3	22-12.10 SPOT "D"	Asphalt	718.1	953.7	ICE/SNOW	889.4	590.7	432.5	543.2	398.8	579.8	940.5
		Aggregate	47.3	41.2		41.5	41.4	47.7	42	50	51.6	43.4
		Subgrade	16.3	19.4		19.2	20.2	18.4	19.7	15.5	15.9	16.8
		RMS	3.1	4.5		2.3	3.9	3.1	5.1	3.9	1	0.5
7	2220-0.40 SPOT "A"	Aggregate	64.3	60.3	63.8	74.3	61.1	59.1	100.6	113.7	77.1	91.4
		Subgrade	5.3	4.5	5.1	5.4	5.5	4.5	4.5	5.8	5.4	4.6
		RMS	1.9	2.5	4.3	3.5	1.6	1.6	4.1	1.9	2.3	0.8
8	2220-1.60 SPOT "A"	Aggregate	127.6	56.4	ICE/SNOW	88.2	74.9	56.9	60.1	143.3	109.8	71.3
		Subgrade	3.1	4.3		3.9	5	4.6	4.3	5.1	5.1	4
		RMS	7.8	2.5		10.1	1.6	1.4	5	5.1	5.1	4
9	21-0.30 SPOT "E"	Asphalt	440.6	825.1	698.2	718.6	294.4	129.3	209.7	154.5	232.5	526.6
		Aggregate	181.7	115	122.6	135.2	138.2	160.8	149.9	171.7	145.3	136
		Subgrade	8.9	12.4	10.8	12.6	13.8	15.6	13.9	9.9	9.7	9.3
		RMS	2.1	3.1	1	4.1	3	3.9	8.3	6.1	2.6	1.3
11	30-0.30 SPOT "D"	Asphalt	800.9	894.5	1361.4	1099.5	587.8	574.9	572.8	423.4	556.2	1157.7
		Aggregate	25.1	16.8	17	17.6	23.5	24.4	22.7	32.9	31.2	24.3
		Subgrade	10	12.8	10.9	12.5	12.4	12.6	12.5	9.4	9.7	10.3
		RMS	2.9	1.3	3.7	1.9	4.6	4.9	8.6	3.3	1.7	2.7
12	30-3.65 SPOT "D"	Asphalt	924.8	1022.1	1691.4	1433.6	777.9	649.3	766	531.6	669.5	1433.6
		Aggregate	14.2	11.8	5.4	7	12.6	16.5	15.9	22	19.2	13.4
		Subgrade	8.1	9.4	11.6	11.2	9.6	9.4	9.7	8.4	9	10
		RMS	1.1	2.4	4.4	1.6	1.9	1.3	3.2	2.3	1.8	2.6
13	29-36.90 SPOT "C"	Asphalt	874.1	1065.7	1238.3	1273.9	904.3	745.9	798.3	643.7	763	1273.9
		Aggregate	15.4	11.9	11.9	10.7	13.8	14.2	15.5	18	16.5	11.7
		Subgrade	10.2	13.4	12.7	12.8	12.5	11.9	10.9	9.2	10	12.1
		RMS	1.2	3.3	2.6	2.2	2.7	2.4	5.9	4.2	2.3	3.4
15	2918-2.15 SPOT "A"	Asphalt	794.3	1127.2	1258.9	1479.1	376.7	613.8		436.5	642.7	1264.7
		Aggregate	40.7	26.2	63.3	21.2	45.4	30		50.4	46.5	34.7
		Subgrade	12.7	16	18.8	14.5	11.4	14.9		12.7	13.3	14.6
		RMS	3.2	2.3	1.6	4.6	1	6.8		5.4	4.2	2

Significant differences exist between the backcalculated moduli between the average site values and the best fit spot values. A likely cause, at least in part, is the variable thickness of the aggregate layer at each site. The thickness used in the analysis was based on judgment, the coring done to install tensiometers, and DCP testing. For the most part, these test sites are older, mainline roads which probably have received variable types and amounts of aggregate over the years. Further, substantial compaction should have occurred under traffic.

An example of the potential thickness variability can be illustrated with Site No. 1 (FR 22-37.30). This site was the only one which could be penetrated with the DCP. For three DCP tests within the site, the estimated aggregate thicknesses were 5.4, 15.0, and 6.5 inches. Additionally, when the tensiometer was installed, the aggregate thickness was estimated at 12 inches. Given these various estimates, an aggregate layer thickness of 10 inches was used in the backcalculation analysis. A difference of ± 5 inches in the aggregate layer can make a significant difference in the resulting layer moduli.

Overall, an examination of the aggregate moduli show modest increases during the summer and fall with the possible exception of Site No. 1. A comparison of August 1990 data (average values) to other periods show:

Site No.	August 1990 Modulus (ksi)	Moduli Range—Other Periods (ksi)
1	39	35-51
2	69	54-70
7	68	38-60
8	100	31-74
14	56	37-62

The above data were obtained from the averaged results (Table 5.7). Thus, the peak values ranged from about 40 to 100 ksi (averaged spots). This peak range is more like 75 to 140 ksi based on the best spot per site values.

The subgrade moduli generally indicate only modest seasonal variation. Again, a comparison of the August 1990 data (average values) to other time periods:

Site No.	August 1990 Modulus (ksi)	Moduli Range—Other Periods (ksi)
1	1.0	1.1
2	9.5	9.5-17.8
7	5.5	4.6-5.4
8	4.2	2.9-4.2
14	1.7	1.6-2.3

The above suggest that a summer period (August in this case) might not exhibit the highest moduli. For all of the aggregate surfaced sites, recall that the RMS values were generally high.

Various plots of layer moduli for the aggregate surfaced sites are shown in Volume II, Appendix D, Figures D.9-D.17.

5.3 MODULI FOR ASPHALT SURFACED SITES

The results shown in Tables 5.7 and 5.8 were based on using laboratory determined resilient moduli for the asphalt surfacing. This resulted in generally lower RMS values which presumably provide more realistic layer moduli for each test site.

The thicker asphalt sites have some of the highest RMS values. This may, in part, be due to the large thermal gradients within the asphalt layer. This might explain the greater variability of the asphalt moduli at these sites as well. Additionally, the dimetral resilient modulus test can produce large moduli errors at higher test temperatures. This is of extra significance since all cores were initially tested at 100°F.

The aggregate base moduli determined at the "thin" asphalt surfaced sites (Site Nos. 3, 4, and 11) ranged from about 19 to 54 ksi based on the average of the values from the five test spots at each site. This is summarized below:

Site No.	Thin/ Thick	Aggregate Base Moduli Range (ksi)	Subgrade Moduli Range (ksi)
3	Thin (2.1")	41.9-53.5	17.4-21.5
4	Thin (2.1")	28.1-41.7	10.6-14.0
5	Thick (4.8")	2.9-51.7	8.6-37.6
6	Thick (4.7")	121.7-481.7	44.9-116.2
9	Thick (5.8")	107.4-196.5	10.5-24.1
10	Thick (5.3")	27.3-79.9	13.4-21.3
11	Thin (3.0")	18.9-40.1	10.0-14.2
12	Thick (3.5")	6.0-25.2	8.9-12.2
13	Thick (3.8")	12.8-21.5	9.5-14.2
15	Thick (3.8")	20.6-50.6	16.3-25.1

The subgrade moduli at the "thin" sites ranged from 10 to 22 ksi as shown above (again, based on "averaged" spots).

The "thick" asphalt surfaced sites (Site Nos. 5, 6, 9, 10, 12, 13, 15) generally exhibited erratic aggregate and subgrade moduli. Further, the moduli developed for Site No. 6 make little sense unless this section is sited on top of a highly consolidated glacial gravel outwash.

At least two sites indicate possible ground freezing effects during the February 1990 testing. Specifically, Site Nos. 5 and 12 show significant reductions in the aggregate base moduli from the preceding months. One explanation for these reductions is due to thaw-weakened bases following a freezing period. The data suggest thawed base moduli as low as 3 to 6 ksi. At Site No. 15, the aggregate base increased from 25 ksi (January 1990) to 45 ksi (February 1990) then decreased to 21 ksi (March 1990); this possibly being due to a "lightly" frozen base followed by thawing (note that the subgrade moduli did not change significantly).

In general, the moduli results show that a minimum asphalt modulus coincides with a maximum in aggregate base modulus and a minimum of subgrade modulus. Given the potential stress sensitivity of the base and subgrade materials, this is as one might expect.

The monthly RMS values at the sites with the smallest temperature variations are generally in the range of 1 to 5 percent (averaged spots). The "best fit" RMS values are about one percent lower. In general, the asphalt surfaced sites exhibit lower RMS values than the aggregate surfaced sites and the difference between the average site RMS and the "best" RMS is also less. This indicates more uniformity in the pavement structures. To support this view, the three asphalt cores obtained at each site showed less than 0.25 inch variation from the average value.

Various plots of layer moduli for asphalt surfaced sites are shown in Volume II, Appendix D, Figures D.18-D.34.

6. CALCULATION OF SEASONAL MODULI RATIOS

6.1 INTRODUCTION

The moduli ratios for both the aggregate and asphalt surfaced test sites are shown in Table 5.9 (averaged spots) and Table 5.10 (best spot). In both tables, a "base" set of moduli had to be selected and these are indicated for each test site in Tables 5.9 and 5.10.

6.2 SEASONAL MODULI RATIOS—AGGREGATE SURFACED SITES

The aggregate surface layers, as expected, exhibit minimum ratios in late winter-early spring with maximum ratios during late summer-early fall. The minimum ratios are about 40 to 80 percent of the average summer values.

The subgrade ratios vary significantly less than the aggregate ratios with values ranging from 80 to 100 percent of average summer values throughout the year.

Table 5.9. Seasonal Moduli Ratios Determined from Average Deflections of 5 Test Spots at Each Site (Olympic National Forest)

Site No.	Test Site Road No.—MP	Pavement Layer	Test Date									
			11/28-29/89	1/3-4/90	2/22-23/90	3/6-7/90	3/29-30/90	5/3-4/90	6/7-8/90	8/29-30/90	10/2-3/90	11/5-6/90
1	22-37.30	Aggregate	1.26	1.01	0.92	0.96	1.32	0.94	0.97	1.01*	0.99*	1.03
		Subgrade	0.97	0.98	1.01	0.98	1.06	0.98*	1.02*	0.90	0.99	1.02
2	22-27.05	Aggregate	0.98	0.91	0.82	0.87	0.77	0.78	0.78	1.00*	1.00*	0.84
		Subgrade	0.66	0.87	1.05	0.80	1.15	1.00*	1.00*	0.62	0.62	0.70
3	22-12.10	Aggregate	0.94	0.82	ICE/SNOW	0.83	0.84	0.87	0.80	0.98*	1.02*	0.83
		Subgrade	0.85	0.96		0.99	1.01*	0.98*	1.01*	0.79	0.82	0.86
4	22-10.05	Aggregate	0.75	0.68	ICE/SNOW	0.68	0.69	0.74	0.74	1.00*	1.00*	0.74
		Subgrade	0.84	1.01		1.03	1.03*	0.99*	0.97*	0.79	0.83	0.90
5	22-8.05	Aggregate	0.84	0.80	0.06	0.80	1.01	1.14	0.94	1.03*	0.97*	0.59
		Subgrade	0.94	1.13	3.70	1.16	1.03*	0.97*	1.01*	0.85	0.94	1.18
6	22-4.40	Aggregate	0.86	0.37	0.37	0.48	0.60	1.46	0.47	1.02*	0.98*	0.82
		Subgrade	0.56	0.98	1.08	1.05	1.04*	0.93*	1.03*	0.42	0.45	0.55
7	2220-0.40	Aggregate	0.85	0.71	0.59	0.78	0.64	0.65	0.87	1.06*	0.94*	0.83
		Subgrade	0.91	0.90	0.93	0.96	0.93	0.88	0.85	1.01*	0.99*	0.87
8	2220-1.60	Aggregate	0.66	0.35	ICE/SNOW	0.41	0.52	0.36	0.36	1.15*	0.85*	0.48
		Subgrade	0.69	0.85		0.76	0.96	0.86	0.75	1.00*	1.00*	0.76
9	21-0.30	Aggregate	0.98	0.65	0.64	0.76	0.69	0.66	0.84	1.17*	0.83*	0.76
		Subgrade	0.53	0.81	0.67	0.76	0.89*	1.21*	0.90*	0.58	0.60	0.61
10	21-4.00	Aggregate	1.13	0.75	0.39	0.93	0.68	1.01	0.99	1.06*	0.94*	0.88
		Subgrade	0.68	1.00	1.09	1.03	1.08*	0.97*	0.95*	0.76	0.75	0.78
11	30-0.30	Aggregate	0.78	0.49	0.58	0.48	0.67	0.67	0.68	1.02*	0.98*	0.79
		Subgrade	0.76	1.02	0.77	0.99	1.01*	1.00*	0.99*	0.72	0.75	0.79
12	30-3.65	Aggregate	0.71	0.53	0.24	0.34	0.57	0.69	0.68	1.04*	0.96*	0.65
		Subgrade	0.86	1.00	1.17	1.14	1.01*	0.99*	1.00*	0.87	0.93	1.06
13	29-36.90	Aggregate	0.98	0.76	0.61	0.63	0.71	0.74	0.82	1.01*	0.99*	0.75
		Subgrade	0.86	1.08	1.06	1.10	1.04*	1.03*	0.93*	0.73	0.79	0.96
14	2065-0.50	Aggregate	1.03	0.74	1.13	0.84	0.71	0.67	0.73	1.01*	0.99*	0.92
		Subgrade	1.08	1.35	0.96	1.01	1.00	0.98	1.16	0.99*	1.01*	1.12
15	2918-2.15	Aggregate	0.78	0.52	0.92	0.42	0.71	0.58	0.72	1.04*	0.96*	0.74
		Subgrade	0.84	1.08	1.21	0.99	0.92*	1.10*	0.97*	0.78	0.83	0.91

*Indicates backcalculated moduli test values averaged for seasonal moduli ratio = 1.00.

Note: Moduli ratios show combined effects of stress sensitivity and environmental factors. Ratios are appropriate for use in design if stress sensitive moduli relationships are not used. If stress sensitive relationships are used, use of these ratios may overestimate seasonal effects.

Table 5.10. Seasonal Moduli Ratios Determined from Best RMS Spot at Each Designated Site (Olympic National Forest)

Site No.	Test Site Road No.—MP	Pavement Layer	Test Date									
			11/28-29/89	1/3-4/90	2/22-23/90	3/6-7/90	3/29-30/90	5/3-4/90	6/7-8/90	8/29-30/90	10/2-3/90	11/5-6/90
1	22-37.30	Aggregate	1.04	1.10	0.78	0.87	1.39	0.93	1.00	1.02*	0.98*	0.96
	SPOT "C"	Subgrade	0.97	0.83	0.98	0.88	0.95	0.91	0.92	0.83	0.99*	1.01*
2	22-27.05	Aggregate	0.90	0.96	0.81	0.85	0.83	0.78	0.75	0.95*	1.05*	0.86
	SPOT "A"	Subgrade	0.85	0.79	1.04	0.90	1.00	0.98*	1.02*	0.80	0.81	0.90
3	22-12.10	Aggregate	0.93	0.81	ICE/SNOW	0.82	0.82	0.94	0.83	0.98*	1.02*	0.85
	SPOT "D"	Subgrade	0.84	1.00		0.99	1.04*	0.95*	1.01*	0.80	0.82	0.86
7	2220-0.40	Aggregate	0.60	0.56	0.60	0.69	0.57	0.55	0.94*	1.06*	0.72	0.85
	SPOT "A"	Subgrade	0.95	0.80	0.90	0.97	0.97	0.81	0.81	1.04*	0.96*	0.82
8	2220-1.60	Aggregate	1.01	0.45	ICE/SNOW	0.70	0.59	0.45	0.48	1.13*	0.87*	0.56
	SPOT "A"	Subgrade	0.62	0.84		0.77	0.99	0.90	0.85	0.99*	1.01*	0.78
9	21-0.30	Aggregate	1.15	0.73	0.77	0.85	0.87	1.01	0.95	1.08*	0.92*	0.86
	SPOT "E"	Subgrade	0.62	0.86	0.75	0.87	0.96*	1.08*	0.96*	0.69	0.67	0.65
11	30-0.30	Aggregate	0.78	0.52	0.53	0.55	0.73	0.76	0.71	1.03*	0.97*	0.76
	SPOT "D"	Subgrade	0.80	1.02	0.87	1.00	0.99*	1.01*	1.00*	0.76	0.78	0.82
12	30-3.65	Aggregate	0.69	0.57	0.26	0.34	0.61	0.80	0.77	1.07*	0.93*	0.65
	SPOT "D"	Subgrade	0.85	0.99	1.21	1.18	1.01*	0.98*	1.01*	0.88	0.94	1.05
13	29-36.90	Aggregate	0.89	0.69	0.69	0.62	0.80	0.82	0.90	1.04*	0.96*	0.68
	SPOT "C"	Subgrade	0.86	1.14	1.08	1.09	1.06*	1.01*	0.93*	0.78	0.85	1.03
15	2918-2.15	Aggregate	0.84	0.54	1.31	0.44	0.94	0.62		1.04*	0.96*	0.72
	SPOT "A"	Subgrade	0.97	1.21	1.43	1.10	0.87*	1.13*		0.97	1.01	1.11

*Indicates backcalculated moduli test values averaged for seasonal moduli ratio = 1.00.

Note: Moduli ratios show combined effects of stress sensitivity and environmental factors. Ratios are appropriate for use in design if stress sensitive moduli relationships are not used. If stress sensitive relationships are used, use of these ratios may overestimate seasonal effects.

The seasonal variation in the aggregate surfacing moduli ratios show a slight trend with precipitation. The precipitation data for the closest weather stations are shown in Table 5.11.

The approximate lower minimum ratio is 0.5 for the aggregate surfacing, as the data summary shown below suggests. For the subgrade, a value of about 0.7 is reasonable.

Site	Aggregate Surface Moduli Ratios (ksi)		Subgrade Moduli Ratios (ksi)	
No.	Lowest	Highest	Lowest	Highest
1	0.92	1.32	0.90	1.02
2	0.77	1.00	0.62	1.15
7	0.59	1.06	0.85	1.01
8	0.35	1.15	0.69	1.00
14	0.67	1.13	0.96	1.35

Moduli ratio plots for the aggregate surfaced sites are shown in Volume II, Appendix D, Figures D.35-D.39.

6.3 SEASONAL MODULI RATIOS—ASPHALT SURFACED SITES

The minimum aggregate base moduli ratios tend to occur during late winter-early spring with the maximum ratios during late summer-early fall, as observed for the aggregate surfaced sites.

The minimum subgrade moduli ratios occur during the late summer-early fall with maximums during the winter-early spring. This trend can be attributed to the stress sensitive nature of these materials. The higher stresses occur during the summer-fall period due to high asphalt temperatures (hence lower asphalt moduli).

The approximate lower minimum modulus ratio for aggregate base is about 0.5 with no significant difference between "thin" and "thick" asphalt surfaced sections. The lower bound for the subgrade soils appears to be a modulus ratio of about 0.7. The summary below is provided to show the origin of this conclusion.

Table 5.11. Accumulated Precipitation Between November 1, 1989, and November 5, 1990
NOAA Stations Aberdeen 20NNE and Sappho 8E

Date	Day No.	Aberdeen 20NNE	Sappho 8E
10/31/89	0		
11/7/89	7	4.47	4.38
11/14/89	14	14.28	14.84
11/21/89	21	16.87	15.94
11/28/89	28	20.32	18.22
12/5/89	35	27.67	24.76
12/12/89	42	30.55	26.37
12/19/89	49	30.67	26.37
12/26/89	56	31.25	26.37
1/2/90	63	32.57	28.42
1/9/90	70	44.93	31.56
1/16/90	77	46.74	37.54
1/23/90	84	49.59	39.26
1/30/90	91	58.96	45.34
2/6/90	98	66.37	52.72
2/13/90	105	75.30	60.72
2/20/90	112	78.00	61.48
2/27/90	119	79.02	62.56
3/6/90	126	81.37	63.58
3/13/90	133	85.96	67.89
3/20/90	140	88.27	69.70
3/27/90	147	88.51	69.86
4/3/90	154	88.51	69.86
4/10/90	161	88.54	69.86
4/17/90	168	89.95	71.03
4/24/90	175	92.73	72.96
5/1/90	182	92.73	74.89
5/8/90	189	95.95	76.21
5/15/90	196	96.46	77.01
5/22/90	203	98.52	78.32
5/29/90	210	99.61	78.80
6/5/90	217	104.21	82.12
6/12/90	224	106.64	85.67
6/19/90	231	106.81	85.78
6/26/90	238	106.81	85.78
7/3/90	245	108.01	86.27
7/10/90	252	109.84	86.89
7/17/90	259	109.84	86.89
7/24/90	266	109.84	86.89
7/31/90	273	109.84	86.89
8/7/90	280	109.84	86.89
8/14/90	287	109.84	86.90
8/21/90	294	110.57	87.10
8/28/90	301	110.94	87.36
9/4/90	308	110.94	89.45
9/11/90	315	110.94	89.45
9/18/90	322	111.02	89.82
9/25/90	329	111.02	89.82
10/2/90	336	111.38	90.01
10/9/90	343	115.07	95.28
10/16/90	350	119.26	98.52
10/23/90	357	123.60	100.52
10/30/90	363	127.86	105.53
11/6/90	370	129.31	107.05

Site	Aggregate Surface Moduli Ratios (ksi)		Subgrade Moduli Ratios (ksi)	
No.	Lowest	Highest	Lowest	Highest
3	0.80	1.02	0.79	1.01
4	0.68	1.00	0.79	1.03
5	0.06	1.14	0.85	3.70
6	0.37	1.46	0.42	1.08
9	0.64	1.17	0.53	1.21
10	0.39	1.13	0.68	1.09
11	0.48	1.02	0.72	1.02
12	0.24	1.04	0.86	1.17
13	0.61	1.01	0.73	1.10
15	0.42	1.04	0.78	1.21

Moduli ratio plots for the asphalt surfaced sites are shown in Volume II, Appendix D, Figures D.40-D.50.

7. DESIGN SEASONAL ADJUSTMENT FACTORS FOR THE OLYMPIC NATIONAL FOREST TEST SITES

This section will provide estimates of "design" seasonal adjustment factors based on seasonal moduli from the 15 test sites. The aggregate and asphalt surfaced sections will be discussed separately.

7.1 AGGREGATE SURFACED SECTIONS

The range of moduli ratios for the aggregate surfacing and subgrades were briefly discussed in Subsection 6.2. There exists a modest trend between precipitation and the variation in aggregate surfacing moduli ratios. Thus, seasonal moduli ratio magnitudes and durations are at least in part a function of the climate. The annual rainfall of 110 to 130 inches occurred with five months of "heavy" rainfall (4 to 6 inches/week), four months of "medium" rainfall (1.0 to 2.5 inches/week), two months of low rainfall (0.25 to 0.75 inch/week), and, in essence, one month with no rainfall. These "rainfall" months are:

Month	"Classification"
October-February	High
March-June	Medium
July-August	Low
September	None

The precipitation data shown in Table 5.11 are for two stations: Aberdeen, which is located about 20 miles north-northeast of Aberdeen, Washington, and Sappho, which is located between Forks, Washington, and Lake Crescent.

Tables 5.12 and 5.13 were prepared to examine possible trends in moduli ratios and rainfall for the five aggregate surfaced test sites and the closest weather station. In each table the cumulative rainfall preceding the deflection testing was determined (total between test dates). This value was divided by the number of days between site visits to provide an approximate rainfall per day value. The moduli ratios in Table 5.12 (Aberdeen Weather Station) are provided at two levels: an average for the four sites and an average for the two lowest moduli ratio sites. Based on such information and judgment, the following moduli ratios result:

Month	Aggregate Surface Moduli Ratios	Subgrade Moduli Ratios
January	0.6	0.9
February	0.5	0.8
March	0.5	0.8
April	0.5	0.8
May	0.6	0.8
June	0.7	0.8
July	0.9	0.9
August	1.0	1.0
September	1.0	1.0
October	0.8	0.9
November	0.8	0.9
December	0.6	0.9

Clearly, the subgrade moduli ratios are at least partially a function of stress sensitive materials. Overall, if a design procedure accounts for stress sensitive materials, then such moduli ratios as those above are overly conservative for adjusting unstabilized layer moduli.

7.2 ASPHALT SURFACED SECTIONS

Tables 5.14 and 5.15 were prepared similarly to Tables 5.12 and 5.13 except for asphalt surfaced test sites. The moduli trends by month were based on this information and judgment. The following moduli ratios result:

Month	Aggregate Surface Moduli Ratios	Subgrade Moduli Ratios
January	0.5	0.9
February	0.5	0.9
March	0.5	1.0
April	0.6	1.0
May	0.7	1.0
June	0.8	0.9
July	0.9	0.8
August	1.0	0.8
September	0.9	0.8
October	0.7	0.8
November	0.7	0.8
December	0.6	0.9

Table 5.12. Seasonal Moduli Ratios and Rainfall—Aggregate Surfaced Sites—Olympic National Forest—Aberdeen Weather Station

Deflection Test Date(s)	Total Rainfall Between Test Dates[1] (inches)	Approximate Rainfall (inches/day)	Averaged Aggregate Base Moduli Ratios,[2, 3] (Test Sites 1, 2, 7, 8)	Averaged Subgrade Moduli Ratios[2, 3] (Test Sites 1, 2, 7, 8)
Nov. 28-29, 89	20	0.7	0.94 0.76	0.80 0.68
Jan. 3-4, 90	12	0.4	0.74 0.53	0.90 0.86
Feb. 22-23, 90	45	0.9	0.78 0.70	1.00 0.93
Mar. 6-7, 90	3	0.3	0.76 0.60	0.88 0.78
Mar. 29-30, 90	7	0.3	0.81 0.58	1.02 0.94
May 3-4, 90	4	0.1	0.68 0.50	0.93 0.87
June 7-8, 90	11	0.3	0.74 0.57	0.91 0.80
Aug. 29-30, 90	7	0.1	1.06 1.00	0.88 0.76
Oct. 2-3, 90	0.4	~ 0	0.94 0.90	0.90 0.80
Nov. 5-6, 90	18	0.5	0.80 0.66	0.84 0.73

Notes: 1. Example: Between November 29, 89, and January 3, 90, total rainfall was 12 inches.

2. Source for moduli ratios is Table 5.9

3. Averages: Top average for all four sites
Bottom average for two lowest moduli ratio sites

Table 5.13. Seasonal Moduli Ratios and Rainfall — Aggregate Surfaced Sites — Olympic National Forest — Sappho Weather Station

Deflection Test Date(s)	Total Rainfall Between Test Dates[1] (inches)	Approximate Rainfall (inches/day)	Aggregate Base Moduli Ratio,[2, 3] (Test Site 14)	Subgrade Moduli Ratios[2] (Test Site 14)
Nov. 28-29, 89	18	0.7	1.03	1.08
Jan. 3-4, 90	10	0.3	0.74	1.35
Feb. 22-23, 90	33	0.7	1.13	0.96
Mar. 6-7, 90	2	0.2	0.84	1.01
Mar. 29-30, 90	6	0.3	0.71	1.00
May 3-4, 90	5	0.2	0.67	0.98
June 7-8, 90	7	0.2	0.73	1.16
Aug. 29-30, 90	5	0.1	1.01	0.99
Oct. 2-3, 90	3	0.1	0.99	1.01
Nov. 5-6, 90	17	0.5	0.92	1.12

Notes: 1. Example: Between November 29, 89, and January 3, 90, total rainfall was 10 inches.

2. Source for moduli ratios is Table 5.9.

Table 5.14. Seasonal Moduli Ratios and Rainfall—Asphalt Surfaced Sites — Olympic National Forest—Aberdeen Weather Station

Deflection Test Date(s)	Total Rainfall Between Test Dates[1] (inches)	Approximate Rainfall (inches/day)	Averaged Aggregate Base Moduli Ratios,[2, 3] (Test Sites 3, 4, 5, 6, 9, 10)	Averaged Subgrade Moduli Ratios[2, 3] (Test Sites 3, 4, 5, 6, 9, 10)
Nov. 28-29, 89	20	0.7	0.92 0.82	0.73 0.59
Jan. 3-4, 90	12	0.4	0.68 0.57	0.98 0.92
Feb. 22-23, 90	45	0.9	0.36 0.22	0.95
Mar. 6-7, 90	3	0.3	0.75 0.64	1.00 0.93
Mar. 29-30, 90	7	0.3	0.75 0.66	1.01 0.98
May 3-4, 90	4	0.1	0.98 0.76	1.01 0.96
June 7-8, 90	11	0.3	0.80 0.67	0.98 0.94
Aug. 29-30, 90	7	0.1	1.04 1.00	0.70 0.59
Oct. 2-3, 90	0.4	~ 0	0.96 0.92	0.73 0.60
Nov. 5-6, 90	18	0.5	0.77 0.70	0.81 0.65

Notes: 1. Example: Between November 29, 89, and January 3, 90, total rainfall was 12 inches.

2. Source for moduli ratios is Table 5.9

3. Averages: Top value: average for six sites
Bottom value: average for three lowest moduli ratio sites

Table 5.15. Seasonal Moduli Ratios and Rainfall — Asphalt Surfaced Sites — Olympic National Forest — Sappho Weather Station

Deflection Test Date(s)	Total Rainfall Between Test Dates[1] (inches)	Approximate Rainfall (inches/day)	Averaged Aggregate Base Moduli Ratios,[2, 3] (Test Sites 11, 12, 13, 15)	Averaged Subgrade Moduli Ratios[2, 3] (Test Sites 11, 12, 13, 15)
Nov. 28-29, 89	18	0.7	0.81 0.74	0.83 0.80
Jan. 3-4, 90	10	0.3	0.58 0.50	1.04 1.01
Feb. 22-23, 90	33	0.7	0.59 0.41	1.05 0.92
Mar. 6-7, 90	2	0.2	0.47 0.38	1.06 0.99
Mar. 29-30, 90	6	0.3	0.67 0.62	1.00 0.96
May 3-4, 90	5	0.2	0.67 0.62	1.03 1.00
June 7-8, 90	7	0.2	0.72 0.68	0.97 0.95
Aug. 29-30, 90	5	0.1	1.03 1.02	0.78 0.72
Oct. 2-3, 90	3	0.1	0.97 0.96	0.82 0.77
Nov. 5-6, 90	17	0.5	0.73 0.70	0.93 0.83

Notes: 1. Example: Between November 29, 1989 and January 3, 1990, total rainfall was 10 inches.

2. Source for moduli ratios is Table 5.9

3. Averages: Top average for all four sites
Bottom average for two lowest moduli ratio sites

CHAPTER 6

SUMMARY AND CONCLUSIONS

1. SUMMARY

The basic objectives of this study were to examine seasonal adjustment factors for deflections and layer moduli and to provide guidance for selecting seasonal adjustment factors that provide for more realistic pavement design.

Tables 6.1, 6.2, and 6.3 were prepared from results of the literature review (Chapter 2) and the backcalculation analyses (Chapters 3, 4, and 5) to indicate seasonal moduli trends for unstabilized base courses (Table 6.1), aggregate surfacings (Table 6.2), and subgrades (Table 6.3). Based on both laboratory and backcalculated moduli, unstabilized base courses had a wide range of moduli ratios (winter or spring modulus divided by the summer modulus). Overall, a moduli ratio of about 0.7 reflects much of the results in Table 6.1. The backcalculated results for the Hokkaido, Japan, test section indicated moduli ratios of as low as 0.22 - 0.24; however, these ratios are applicable to only a two week period. Further, the laboratory results of Rada and Witczak [23] and Hicks and Monismith [22] do not support that low of a moduli ratio; however, undoubtedly such low values can and do occur in the field. The results for the subgrade moduli ratios show less variation with low range typical values of about 0.85 to 0.90.

A composite set of moduli ratios for both aggregate base and subgrade soils were selected from the results (Chapters 2-5). These values do not necessarily represent the most conservative but should be representative of flexible pavements located in areas with modest annual freezing and thawing (a Freezing Index of say less than 700°F-days) or a wet climate. These moduli ratios are:

	Moduli Ratios	
Month	Aggregate Base	Subgrade
January	0.6	0.9
February	0.6	0.8
March	0.6	0.8
April	0.6	0.8
May	0.7	0.9
June	0.8	0.9
July	0.9	0.9
August	1.0	1.0
September	1.0	1.0
October	0.8	0.8
November	0.7	0.8
December	0.6	0.9

These results are similar to those used by WSDOT for Eastern Washington (Table 4.16), an area with cold winters but modest annual precipitation.

With regard to seasonal deflection ratios, the data shown for FWD data (converted to Benkelman Beam) obtained in the Willamette National Forest, reveals wet season to dry season ratios of 1.05 to 1.69. Similarly calculated deflection ratios from the Olympic National Forest (Chapter 5) are about 1.5 for both aggregate and asphalt surfaced pavements. Deflection ratios from one road in the Kootenai National Forest (FS Road No. 92) revealed values ranging from 3 to 8 during the spring thaw period. This further reinforces the view that seasonal changes in deflection data are very site specific. On the other hand, if no prior, site specific deflection data is available, a deflection ratio of about 1.5 could be used for sites with limited freeze-thaw effects (note: many of the sites characterized in Chapters 4 and 5 have subgrades classified as coarse-grained; the sites described in Chapter 5 have little or no winter freezing). A higher deflection ratio is likely required for areas with severe winter freezing and thaw (more like the ratio of 2.5 used by RTAC in Canada).

The results shown in Chapter 4 suggest that the seasonal variation of the layer moduli, can, in part, be attributed to changing stress state in the base and subgrade layers (due mostly to the changing asphalt concrete layer moduli). Based on about one-half of the WSDOT test site and station combinations, about 50 percent of the seasonal moduli changes for the base course and about 15 to 30 percent for the subgrade can be attributed to these changing stress conditions. For the remaining test site and station combinations, there were little or no relation between seasonal backcalculated and laboratory moduli changes due to stress sensitivity considerations. These comparisons suggest that the stress sensitivity of the base and subgrade moduli should be considered in future efforts to obtain seasonal moduli adjustment factors (or, more specifically, moduli ratios).

The data in Chapter 4 and Appendix C also show that backcalculated and laboratory layer moduli can be in approximate agreement. This is an issue which is being and will continue to receive national attention.

A summary of past and current practices used to address seasonal freeze and thaw issues is provided in Chapter 2. Volume III of this study illustrates the use of seasonal adjustment factors in various pavement design procedures.

Table 6.1. Moduli Ratios for Unstabilized Aggregate Base Courses

Condition	Source Reference No.	Moduli Ratio
• Ratio of Laboratory Wet to Dry Moduli		
• Dense Graded Limestone Aggregate	23	0.64
• Crushed Rock (Slag)	23	0.45
• Sand Aggregate Blend	23	0.91
• Bank Run Gravel	23	0.62
• Crushed Aggregate	22	0.83
		Avg = 0.69
• North Carolina — Backcalculated Moduli (ELMOD results): (Spring ÷ Summer)	61	
• US 64 (two sections)		0.70
		0.55
• I 40 (two sections)		0.84
		1.09
• US 19 (two sections)		0.86
		0.80
		Avg = 0.81
• Washington State — Backcalculated Moduli (Spring or Winter ÷ Summer)	57	
• SR2, Sunnyslope		0.63
• SR2, MP 159.6		0.68
• SR172, MP 2.0		0.67
		Avg = 0.66
• Washington State — Backcalculated Moduli —	This Report	
• Western Washington		
• TS No. 1 (Winter ÷ Summer)		0.76
• TS No. 11 (Winter ÷ Summer)		0.72
		Avg = 0.74
• Eastern Washington		
• TS No. 15 (Spring ÷ Summer)		0.66
• Hokkaido, Japan — Backcalculated Moduli (LMBS results)	82	
• December 27, 1989 (unfrozen)		0.72
• January 8, 1990 (frozen)		14.71
M		M
• February 19, 1990 (frozen)		2.02
• February 26, 1990 (unfrozen)		0.22
• March 5, 1990 (unfrozen)		0.24
• March 12, 1990 (unfrozen)		0.68
• March 19, 1990 (unfrozen)		0.94
• March 26, 1990 (unfrozen)		0.90
• Olympic National Forest — Aggregate Base Course (Asphalt Surfaced Sites)	This Report	
• January		0.5
• February		0.5
• March		0.5
• April		0.6
• May		0.7
• June		0.8
• July		0.9
• August		1.0
• September		0.9
• October		0.7
• November		0.7
• December		0.6

2. CONCLUSIONS

The following conclusions are appropriate:

- Seasonal moduli and deflection factors are generally site specific.
- Decreases in base course moduli occur over the course of a few days in thawing environments.
- Development of seasonal factors requires measurement of layer moduli over several seasons.
- Seasonal variation observed in base course layers was greater than that of the subgrade, especially in freezing and thawing environments.
- The choice of stiffness for a stiff layer affects the backcalculated results and can provide backcalculated moduli that appear more reasonable and have lower convergence errors (root mean square (RMS) values).

Table 6.2. Moduli Ratios for Unstabilized Aggregate Surface Courses

Condition	Source Reference No.	Moduli Ratio
• Olympic National Forest — Aggregate Surfaced Sites Backcalculated Moduli	This Report	
• January		0.6
• February		0.5
• March		0.5
• April		0.5
• May		0.6
• June		0.7
• July		0.9
• August		1.0
• September		1.0
• October		0.8
• November		0.8
• December		0.6
(Generalized conclusions from Olympia NF and analysis)		

Table 6.3. Moduli Ratios for Unstabilized Subgrades

Condition	Source Reference No.	Moduli Ratio
• North Carolina — Backcalculated Moduli (ELMOD results): (Spring ÷ Summer)	61	
• US 64 (two sections)		0.93
		0.88
• I 40 (two sections)		1.08
		1.57
• US 19 (two sections)		0.83
		1.17
		Avg = 1.08
• Washington State — Backcalculated Moduli (Spring or Winter ÷ Summer)	57	
• SR2, Sunnyslope		0.98
• SR2, MP 159.6		1.17
• SR172, MP 2.0		0.86
		Avg = 1.00
• Hokkaido, Japan — Backcalculated Moduli (LMBS results)	82	
• December 27, 1989 (unfrozen)		0.91
• January 8, 1990 (frozen)		1.27
M		M
• March 5, 1990 (frozen)		1.02
• March 12, 1990 (unfrozen)		0.80
• March 19, 1990 (unfrozen)		(no results)
• March 26, 1990 (unfrozen)		1.06

Table 6.3. Moduli Ratios for Unstabilized Subgrades (Continued)

Condition	Source Reference No.	Moduli Ratio
• Washington State — Backcalculated Moduli —	This Report	
• Western Washington		
• TS No. 1 (Winter ÷ Summer)		0.77
• TS No. 2 (Winter ÷ Summer)		0.78
• TS No. 3 (Spring ÷ Summer)		0.83
• TS No. 6 (Winter ÷ Summer)		0.77
• TS No. 8 (Winter ÷ Summer)		0.81
• TS No. 9 (Winter ÷ Summer)		0.89
• TS No. 10 (Spring ÷ Summer)		0.86
• TS No. 11 (Winter ÷ Summer)		0.79
• TS No. 12 (Spring ÷ Summer)		0.91
		Avg = 0.82
• Eastern Washington		
• TS No. 14 (Fall ÷ Summer)		0.79
• TS No. 15 (Spring ÷ Summer)		0.94
• TS No. 16 (Spring ÷ Summer)		1.60
		Avg = 0.86 (w/o TS No. 16)
• Olympic National Forest — Backcalculated Moduli — Asphalt Surfaced Sites	This Report	
• January		0.9
• February		0.9
• March		1.0
• April		1.0
• May		1.0
• June		0.9
• July		0.8
• August		0.8
• September		0.8
• October		0.8
• November		0.8
• December		0.9
• Olympic National Forest — Backcalculated Moduli — Aggregate Surfaced Sites	This Report	
• January		0.9
• February		0.8
• March		0.8
• April		0.8
• May		0.8
• June		0.8
• July		0.9
• August		1.0
• September		1.0
• October		0.9
• November		0.9
• December		0.9

CHAPTER 7

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GPO 784-089/31011