TRC9201

Layer Coefficient / ACHM Stabilized Base

Robert P. Elliott, Muhammad Arif

Final Report

1995

FINAL REPORT

TRC-9201

LAYER COEFFICIENT/ACHM STABILIZED BASE

PRINCIPAL INVESTIGATOR - Robert P. Elliott

GRADUATE ASSISTANT - Muhammad Arif

Conducted by

Mack-Blackwell Transportation Center
Department of Civil Engineering
Engineering Experiment Station
University of Arkansas

in cooperation with

Arkansas State
Highway and Transportation Department

and

U.S. Department of Transportation
Federal Highway Administration

University of Arkansas
Fayetteville, Arkansas

August, 1995

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.

4. Title and Subtitle	5. Report Date
FINAL REPORT, TRC - 9201 LAYER COEFFICIENT/ACHM STABILIZED BASE	AUGUST 1995
	6. Performing Organization Code
7. Author(s) R. P. ELLIOTT AND M. ARIF	8. Performing Organization Report No.
9. Performing Organization Name and Address Mack-Blackwell Transportation Center Department of Civil Engineering University of Arkansas Fayetteville, AR 72701	10. Work Unit No. (TRAIS) 11. Contract or Grant No. TRC 9201
12. Sponsoring Agency Name and Address Arkansas State Highway and Transportation Department Planning and Research Division PO Box 226 Little Rock, AR 72206	13. Type of Report and Period Covered FINAL REPORT 14. Sponsoring Agency Code

15. Supplementary Notes

Study conducted in cooperation with the U. S. Department of Transportation, Federal Highway Administration

16. Abstract

This study was to determine an appropriate layer coefficient for the Asphalt Concrete Hot Mix Base Course (ACHMBC) used by the Arkansas State Highway and Transportation Department. Examination of the specifications for the material showed it to be superior to the asphalt stabilized base used in the AASHO Road Test. The coefficient estimated for the AASHO material was 0.34.

Laboratory tests were conducted on ACHMBC cores from eight projects to determine typical resilient modulus and split tensile strength properties. Coefficients were estimated for each project from the resilient modulus data in the manner prescribed in the 1993 AASHTO Guide. These coefficients ranged from 0.36 to 0.52. Coefficients were also estimated using the AASHTO Guide method for determining SNeff for flexible pavement overlay design. These ranged from 0.34 to 0.40 with an average of 0.37. The split tensile strength data were evaluated using the AAMAS relative fatigue classification. This analysis found the ACHMBC from the projects sampled to be superior to the AASHO Road Test surface and binder mixes which were assigned coefficients of 0.44. The study recommends that a value of 0.36 be assigned as the layer coefficient for AHTD's ACHMBC.

17. Key Words	18. Distribution Statement
Layer Coefficient, Asphalt, Base Course, Flexiible Pavement	No Restrictions

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

ACKNOWLEDGEMENTS/DISCLAIMER

This report is based on the findings of Project TRC-9201, Layer Coefficient/ACHM Stabilized Base.

TRC-9201 is sponsored by the Arkansas State Highway and Transportation Department and the U.S. Department of Transportation, Federal Highway Administration.

SI CONVERSION FACTORS

1 inch = 25.4 mm

1 foot = 0.305 m

1 pcf = 16 kg/m^2

1 psi = 6.9 kN/m^2

1 ksi = 6.9 MN/m^2

1 lb = 4.45 N

TABLE OF CONTENTS

List of Tables

List of Figures

List of Figures (continued)

Chapter 1

INTRODUCTION

1.1 Problem Statement

This study was conducted to select an appropriate value for the structural layer coefficient for Asphalt Concrete Hot Mix base courses (ACHM base) that are prepared under the specifications of the Arkansas State Highway and Transportation Department (AHTD). When the AASHTO approach to pavement design was first adopted, AHTD selected a value of 0.25 for the structural layer coefficient for the ACHM bases. This value may have been appropriate for the material and specifcation used at that time; however, the specifications have since been revised to require higher quality material. Current specifications require a larger maximum size, a crushed particle content, and contain mix design criteria. The purpose of the specification revisions was to increase the structural capacity and performance of the base. However, no change was made in the structural layer coefficient to reflect these improvements.

While the specification changes increased the quality and performance of the base course, they also significantly increased its cost. ACHM base course under current specifications costs about the same as the ACHM binder. The layer coefficient for ACHM binder is 0.44. As a result the use of ACHM base course is not cost effective. Cost effectiveness can be restored if a higher value can be justified for the ACHM base layer coefficient. This study was conducted to determine what that value should be.

1.2 Study Objectives

The primary objective of this study was to establish an appropriate value of structural layer coefficient for the ACHMBC used by AHTD. A secondary objective was to develop typical data on split tensile strength and resilient modulus of ACHMBC and to determine whether split tensile strength can be used to estimate the resilient modulus.

Chapter 2

LAYER COEFFICIENTS AND THE AASHTO GUIDE

2.1 The AASHO Road Test

AHTD follows the "AASHTO Guide for the Design of Pavement Structures, 1986" [2.1]. The guide uses the structural layer coefficient as an empirical index of the relative contribution of the material in a specific pavement layer to the overall performance of the pavement system. This structural layer concept was derived at the AASHO Road Test [2.2, 2.3] from the performance of various test sections. Since the AASHO Road Test serves as the basis of the AASHTO pavement design procedures, a brief review of the Road Test is presented to provide a better understanding of the structural layer concept.

The AASHO Road Test was the third in a planned series of tests (Road Test One-Md and WASHO Road Test being the first two). It was conducted from 1958 to 1960 near Ottawa, Illinois about 80 miles southwest of Chicago. The site was chosen because the soil within the area was considered to be representative of that found in large areas of the country. The climate is typical of that found in the northern United States and much of the earthwork and pavement construction would be used ultimately as a part of Interstate 80.

The test facility consisted of four large loops numbered

3 through 6 and two smaller loops numbered 1 and 2, located as shown in Figure 2.1. Each loop was a segment of four lane divided highway with tangent, parallel roadways connected by turn arounds. The north tangent of each loop was flexible pavement and south tangent was rigid pavement. Within each tangent, many different thickness designs were used. These contained a complete factorial experiment with replication for investigating the effects of varying thickness of ACHM surfacing, crushed stone base, and gravel subbase. Several additional studies were also conducted to evaluate surface treatments, shoulders and two types of stabilized base (cement-treated gravel and bituminous-treated gravel).

No traffic operated over loop 1. All vehicles assigned to any one traffic lane in loops 2 through 6 had the same axle arrangement-axle load combination as described in Figure 2.2. Tire pressures and steering axle loads were representative of normal practice for the time.

The test was conducted over a period of two years with a total of 1,114,000 vehicle passes applied to each loop. All the variables for the pavement studies were concerned with pavement thickness design, load magnitude, and environmental effects. Table 2.1 gives a description of the measured variables.

The AASHO Road Test introduced the concept of serviceability into the thickness design process. During the two years that traffic was on each loop, the riding quality

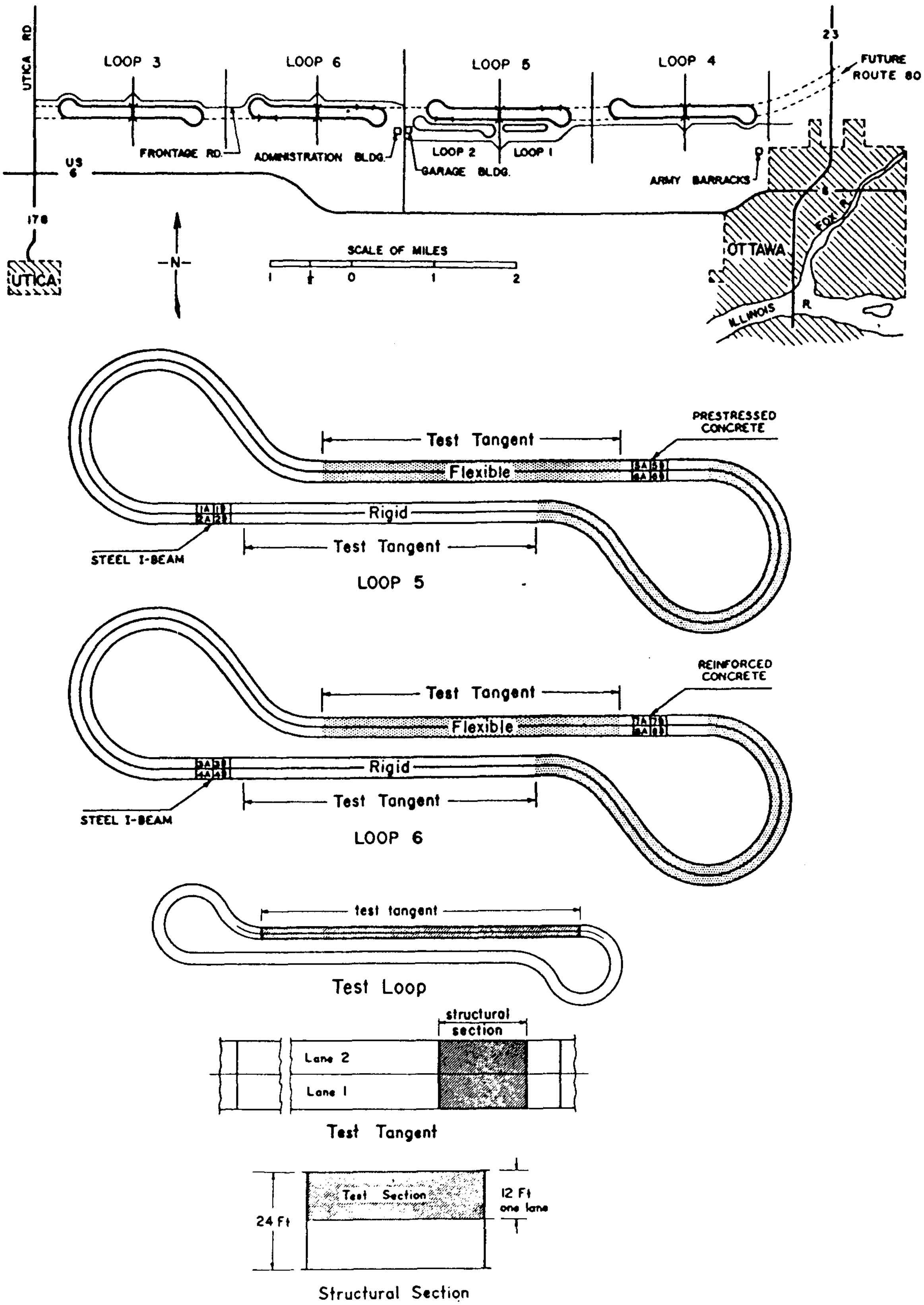
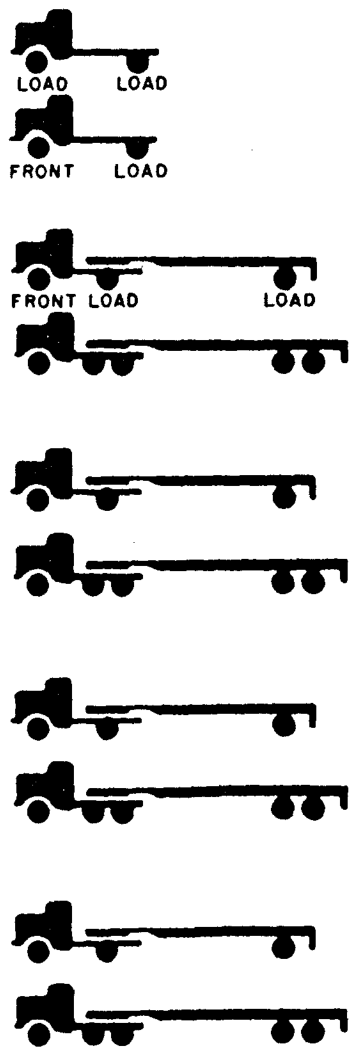


Figure 2.1 Layout of the AASHO Road Test [2.2].



Loop	Lane	Weight in kips: Front Axle	Load Axle	Gross Weight
2	1	2	2	4
2	2	2	6	8
3	1	4	12	28
3	2	6	24	54
4	1	6	18	42
4	2	9	32	73
5	1	6	22.4	51
5	2	9	40	89
6	1	9	30	69
6	2	12	48	108

Figure 2.2 Axle weights and distribution used on various loops of the AASHO Road Test [2.2].

Table 2.1. The AASHO Road Test measured variables.

Factor	Variable Description	Variable	Units
Traffic	No. of axle repetitions	W	No
	Axle weight	L_1	kip
	Axle type	L_2	1 = single
			2 = tandem
Pavement	Surfacing thickness	t_1	inches
	Base thickness	t_2	inches
	Subbase thickness	t_3	inches
Distress	Extent of cracking	C	$ft^2/10^3ft^2$
	Extent of patching	P	$ft^2/10^3ft^2$
	Slope variance (roughness)	SV	
	Rutt depth	RD	inches

and evidence of distress development (cracking and rutting) were measured on each pavement section every other week. These measures were used in an empirically derived equation to estimate the user's opinion of the acceptability of the pavement on a scale of 0 (failed) to 5 (excellent). This value was called the pavement's Present Serviceability Index (PSI).

Three new terms were defined to describe the serviceability and performance of a pavement:

<u>Initial Serviceability (P_0)</u> is the PSI of the newly constructed, untrafficked pavement. The ideal (PSI = 5) is rare. In fact, newly constructed flexible pavements at the Road Test reflected an average P_0 value of 4.2.

<u>Terminal Serviceability (P_t)</u> is the level of PSI at which the pavement is deemed to be no longer acceptable and major maintenance or rehabilitation is needed. The lower limit value of P_t at the Road Test was 1.5. When a pavement's PSI reached this level, the pavement received extensive maintenance and was no longer monitored as a part of the Road Test.

<u>Present Serviceability (P)</u> is the level of PSI at any time during the life of the pavement. Under normal circumstances $P_0 > P > P_t$.

The Present Serviceability Index equation was related to the distress measurements given in Table 2.1 by conducting regression analysis on data generated from panel ratings of in-service pavements throughout the Midwest by groups of highway users. In equation form it is written as:-

$$P=5.03-1.91\log(1+\overline{SV})-0.01\sqrt{\overline{(C+P)}}-1.38\overline{RD^2} \qquad (2.1)$$

where,

P = present serviceability index,

SV = the mean of the slope variance in the two wheel paths,

C+P = the measure of cracking and patching in the pavement

surface,

RD = a measure of rutting in the wheel paths,

A PSI curve was developed for each pavement section by measuring the slope variance, cracking, and rut depth every two weeks throughout the duration of the Road Test. Figure 2.3 shows typical serviceability history of pavements obtained at the AASHO Road Test. With these curves the number of loadings to reduce the serviceability level to a failure level P_t could be determined for each pavement section. These empirical data became the basis for the development of structural design equations for the flexible and rigid pavements.

The basic serviceability-performance equation derived by performing regression analysis is as follows:-

$$\left(\frac{W}{\rho}\right)^{\beta}=\frac{(P_0-P)}{(P_0-P_t)} \qquad (2.2)$$

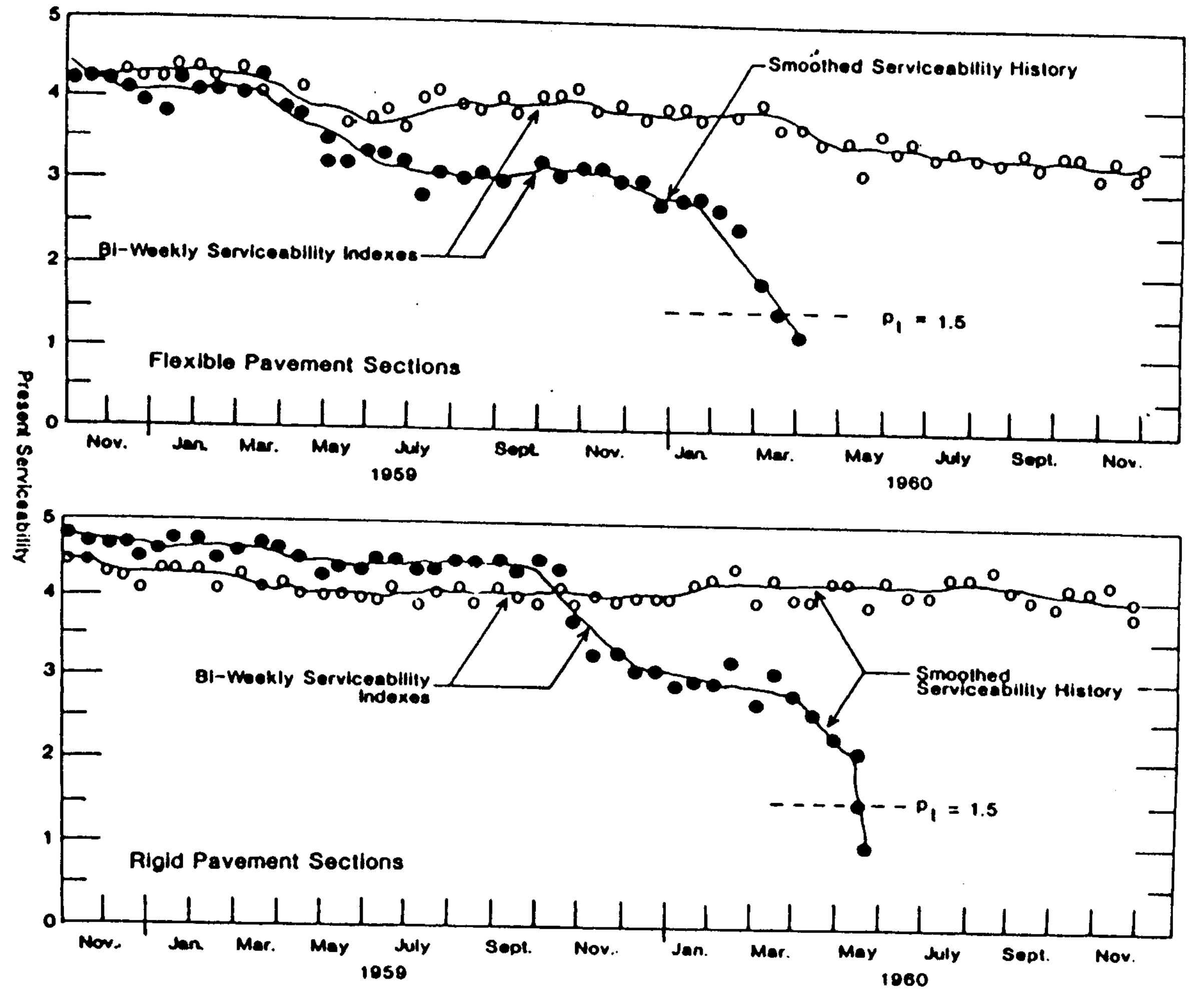


Figure 2.3. Typical serviceability history of pavements obtained at the AASHO Road Test. [2.3]

or

$$\log_{10}(W) = \log_{10}(\rho) + \frac{\log_{10}(P_0-P)/(P_0-P_t)}{\beta} \qquad (2.3)$$

where,

- W = number of axle repetitions that will reduce the serviceability from P_0 to P_t,
- ρ = the number of axle repetitions at terminal serviceability (P_t=1.5),
- β = shape factor.

The ρ and β terms were considered to be functions of loading magnitude, axle configuration, and pavement design (structure and/or thickness).

On each section of the Road Test, the present serviceability (P) and the traffic (W) were determined at two week intervals through the life of the section. The unknown parameters β and ρ in equation 2.3, were determined by regression analysis using data for serviceabilities of 3.5, 3.0, 2.5, 2.0, and 1.5. These five points provided the data for the regression used to obtain estimates of β and ρ.

Having obtained the two parameters (β and ρ) for each section, it was assumed that these parameters were functions of each section's pavement design (i.e. thickness and/or structure) and the applied traffic loading (i.e. axle weights and axle types). With this assumption, the following functional relationships were assigned to β and ρ.

$$\rho = A_0 (D_1+1)^{A1} . (L_1+L_2)^{A2} . L_2^{A3} \qquad (2.4)$$

$$\beta = 0.4+B_0 . (D+1)^{B1} . (L_1+L_2)^{B2} . L_2^{B3} \qquad (2.5)$$

where L_1 was the axle load in kips, L_2 was 1 if the major load axle were a single axle or 2 if that axle were a tandem axle, and D was a parameter called the thickness index that represented the pavement's thickness design.

In these equations L_1, L_2 and the thickness design were known for each section. The eight unknown constants $A_{0\text{-}3}$ and $B_{0\text{-}3}$ were obtained by regression analysis. A variant regression was conducted in which the thickness index, D, was given by:

$$D = a_1t_1 + a_2t_2 + a_3t_3$$

The coefficients (a_1, a_2, and a_3 were permitted to vary so that the three layers of the pavement structure (surface, base and subbase) might each enter into the thickness index (D) with a different weight per unit thickness. This linear combination of the layer thickness in the AASHO model has become better known as the Structural number (SN) and the coefficients $a_{1\text{-}3}$ known as the layer coefficients. The layer coefficients were determined individually for five of the loops of the Road Test and were found to vary widely:

a_1 (asphalt concrete surface course) = 0.33 to 0.83

a_2 (crushed stone base course) = 0.11 to 0.25

a_3 (gravel sand subbase course) = 0.09 to 0.11

The average values of the coefficients over all the loops were found to be 0.44, 0.14 and 0.11 for a_1, a_2, and a_3 respectively. Table 2.2, which is reproduced from the AASHO Road Test report [2.3] shows the average layer coefficients from the individual loops.

Note that Table 2.2 does not contain a value for bituminous stabilized base material. The reason for this is that only non-stabilized granular base (crushed stone) was used in the main experiment of the Road Test. A limited number of sections that incorporated bituminous and cement stabilized base were constructed outside the main experiment of the Road Test as a special base experiment. These sections were subsequently evaluated by the AASHO Committee on Design and used to estimate appropriate values for stabilized bases. The estimated coefficients for stabilized base were published in the 1972 AASHO Interim Guide [2.4].

Table 2.3 is reproduced from the 1972 Guide. Note that the footnote to this table states that the stabilized base layer coefficients were established with less precision than were the coefficients for the asphalt surface course, granular base course, and granular subbase.

Table 2.2 AASHO Road Test Analysis Showing Layer Coefficients by Loop. [2.3]

ANALYSIS OF VARIANCE FOR LOG ρ ESTIMATES[1] WITHIN LOOPS, WEIGHTED APPLICATIONS

Item	Loop 2		Loop 3		Loop 4		Loop 5		Loop 6	
No. of test sections	44		60		60		60		60	
No. of replicate sections	8		6		6		6		6	
Effects[2]:										
Lane mean difference		$\underline{13.25}$		$\underline{0.32}$		$\underline{0.14}$		0.04		$\underline{1.55}$
D_1 (surface) linear:										
Lanes combined	$\underline{16.58}$		$\underline{6.89}$		$\underline{6.94}$		$\underline{7.87}$		$\underline{3.83}$	
Lane interaction		0.00		0.00		0.00		0.00		0.01
D_2 (base) linear:										
Lanes combined	$\underline{11.04}$		$\underline{7.78}$		$\underline{6.16}$		$\underline{6.11}$		$\underline{4.04}$	
Lane interaction		0.14		0.07		0.03		0.01		0.00
D_3 (subbase) linear:										
Lanes combined	0.62		$\underline{6.94}$		7.51		$\underline{7.20}$		$\underline{7.07}$	
Lane interaction		0.08		0.08		0.01		0.01		0.00
D_1, D_2, D_3 non-linear:										
Lanes combined	$\underline{0.90}$		0.13		0.05		0.09		0.04	
Lane interaction		$\underline{0.45}$		0.04		0.01		0.01		0.09
D_1, D_2, D_3 interactions:										
Lanes combined	0.10		0.07		0.09		0.03		0.11	
Lane interaction		0.03		0.01		0.01		0.02		0.01
Replicate differences:										
Lanes combined	0.27		0.01		0.01		0.02		0.05	
Lane interaction		0.13		0.01		0.01		0.01		0.05
Within loop regression coefficient:										
For D_1	0.83		0.44		0.44		0.47		0.33	
D_2	0.25		0.16		0.14		0.14		0.11	
D_3	0.09		0.11		0.11		0.11		0.11	
Within lane	1	2	1	2	1	2	1	2	1	2
Coefficient for log $(D+1)$	8.39	9.07	7.47	6.52	9.27	9.10	10.30	10.14	10.09	10.41
Percent of variation explained by regression	71	88	81	84	87	93	91	93	85	77
Mean square for unexplained variation	0.32	0.13	0.11	0.07	0.06	0.03	0.04	0.03	0.05	0.09

[1] Data from which this table arose are the estimates log $\hat{\rho}$ as described in Appendix G.
[2] Mean squares for effects; underlined values considered to be significant relative to replicate differences pooled with interaction effects.

Table 2.3 Layer Coefficients Appearing in the 1972 AASHO Interim Guide. [2.4]

Structural Layer Coefficients Proposed by AASHO Committee on Design, October 12, 1961

Pavement Component	Coefficient[3]
Surface Course	
Roadmix (low stability)	0.20
Plantmix (high stability)	0.44*
Sand Asphalt	0.40
Base Course	
Sandy Gravel	0.07[2]
Crushed Stone	0.14*
Cement-Treated (no soil-cement)	
Compressive strength @ 7 days	
650 psi or more[1]	0.23[2]
400 psi to 650 psi	0.20
400 psi or less	0.15
Bituminous-Treated	
Coarse-Graded	0.34[2]
Sand Asphalt	0.30
Lime-Treated	0.15-0.30
Subbase Course	
Sandy Gravel	0.11*
Sand or Sandy-Clay	0.05-0.10

* Established from AASHO Road Test Data

[1] Compressive strength at 7 days.

[2] This value has been estimated from AASHO Road Test data, but not to the accuracy of those factors marked with an asterisk.

[3] It is expected that each state will study these coefficients and make such changes as experience indicates necessary.

2.2 History of the Guide

At the conclusion of the Road Test, the empirical regression models were used to develop pavement design procedures that were presented in 1961 as the AASHO Interim Guide for the design of rigid and flexible pavements. This guide incorporated design factors that were not present in the original AASHO flexible pavement model. These were:

1. Soil support scale.
2. Axle load equivalency factor.
3. Regional climatic factor.
4. Estimated layer coefficients for asphalt and cement stabilized bases.

In 1962, the AASHO Committee on Design issued these Interim Guides to the States to be used for a one-year trial period. The purpose of this trial period was to allow the States to review the design procedures and to check their validity in actual practice. After the trial period, and subsequent receipt of comments from the States, the AASHO Committee on Design did not consider it necessary at that time to revise the Guides or the instructions. They were retained in their interim status.

While the Guide was under development, AASHO initiated a research program within the National Cooperative Highway Research Program [2.7,2.8] to developed guidelines for satellite studies of pavement performance. It was anticipated that these would provide data needed to extend AASHO Road Test results and strengthen the weaker areas of the Guides. However, relatively few such satellite studies were initiated by the States. Because the possibility of acquiring data from a truly nationwide satellite study in the near future appeared to be remote, the NCHRP Advisory Panel C1-11, on recommendation from AASHO, formulated a research project. Conceived as a practical alternative to the satellite study, this project was supposed to evaluate the various techniques used and the results obtained by the individual States after applying the Guides to pavement structure design. The results of this project was published in the form of the " AASHO Interim guide for design of pavement structures, (1972)" [2.4] and the NCHRP report 128 [2.9].

The NCHRP Report 128 [2.9] used the layered elastic theory to develop a method for selecting structural layer coefficients. The limiting pavement response criteria used were:

1. Surface deflection
2. Tensile strain in the asphaltic concrete
3. Vertical compressive strain in the subgrade

This guide was again revised and published in 1981 with incorporation of some modifications to the rigid pavement sections [2.10].

In 1983, further evaluations of the Guide were undertaken. From this evaluation it was concluded that although the Guide was still serving its main objectives, some improvements could be made to incorporate advances in pavement design and analysis technology that had been made since 1972. Thus, in 1984-85 the Subcommittee on Pavement Design and a team of consultants revised the existing guide under NCHRP Project 20-7/24 and issued the version entitled " AASHTO Guide for the Design of Pavement Structures, (1986)" [2.1].

The 1986 Guide [2.1] retained the modified AASHO Road Test performance prediction equations as the basic models for use in pavement design. Major flexible pavement design procedure changes have been made in several areas, including:

1. Incorporation of a design reliability factor (based on a shift in the design traffic) to allow the designer to use the concept of risk analysis for various classes of highways.
2. Replacement of the Soil Support number with the resilient modulus (AASHTO test method T274) to provide a rational testing procedure for evaluating subgrade properties.
3. Use of the resilient modulus test for assigning layer coefficients to both stabilized and unstabilized materials.
4. Provision of guidance for the construction of subsurface drainage systems and modifications to the design equations to take advantage of improvements in performance that results from good

drainage.

5. Replacement of the subjective regional factor with a "rational" approach to the adjustment of designs to account environmental considerations such as moisture and temperature climate considerations, including thaw-weakening and other seasonal variations in material properties.

The 1986 Guide also included recommendations and guidelines for conducting economic analysis of alternative designs and a summary of the latest concepts concerning the development and use of mechanistic-empirical design procedures.

The guide was revised again in 1993. For new flexible pavement design the only changes were minor editorial revisions and corrections. The major changes were in the design of overlays for pavement rehabilitation.

2.3 Structural Layer Coefficient Relationships

Although the concept of layer coefficient is still central to the AASHTO flexible pavement design procedures, the current AASHTO Guide [2.1] relies more heavily on the determination of material properties for the estimation of appropriate layer coefficient values. A major step in this direction was the incorporation of the resilient modulus as the subgrade soil property for design. This replaced the soil support scale for flexible and rigid pavements. The Guide also uses the resilient modulus as the main material characterization property for the determination of layer coefficients for surface course, base course, and subbase.

The Guide [2.1] recommends that the modulus (E_{AC}) for hot mix asphalt concrete be determined from the resilient modulus test at 68°F as determined from the diametral (or indirect tensile) test ASTM D4123. The Guide however does not state which value (instantaneous and total resilient modulus) is to be used. Although ASTM D4123 is the test recommended for obtaining values to be used when entering the AASHTO charts to determine layer coefficients, this test was not used to characterize the stiffness of asphaltic concrete by Van Til, et al [2.9] when they established the modulus-layer coefficient relationship. The hot mix asphalt concrete stiffness were originally based on dynamic modulus data, as reported by Kallas and Riley [2.11]. The dynamic modulus is measured by compression tests while the resilient modulus is normally measured by indirect tensile tests. These two tests do not produce the same values.

In the development of the AASHTO methodology for selecting asphalt concrete layer coefficients, the value for a dense-graded ACHM surface and binder was set at 0.44 (determined at the Road Test) for a modulus of 450 ksi. This modulus value was considered to represent the average dynamic modulus at the average pavement temperature recorded during the Road Test (67.5°F) [2.12]. To obtain the relationship between modulus and layer coefficient, calculations of surface deflection, asphaltic concrete tensile strain, and vertical compressive strain on the subgrade were made for different levels of surface, base and subgrade stiffness and for varying surface and base thickness by Van Til, et al [2.9]. The relationship adopted for bituminous-treated base course material is shown in Figure 2-4. This figure is taken from the 1993 AASHTO Guide.

The Guide [2.1] also gives a relationship of layer coefficients with other material tests developed by different agencies. It should be noted, however, that the correlations between the resilient modulus and other material properties available in the AASHTO Guide and other references are generally poor. The main reason for these poor correlations is that each test measures a specific material property, and the different properties do not necessarily relate to one another. For example, Marshall stability and flow values are parameters related to the resistance of the asphalt concrete material to deformation under certain temperature and loading conditions, while the diametral resilient modulus (ASTM D4123) is a measure of the elastic stiffness of the material under different temperature and loading conditions.

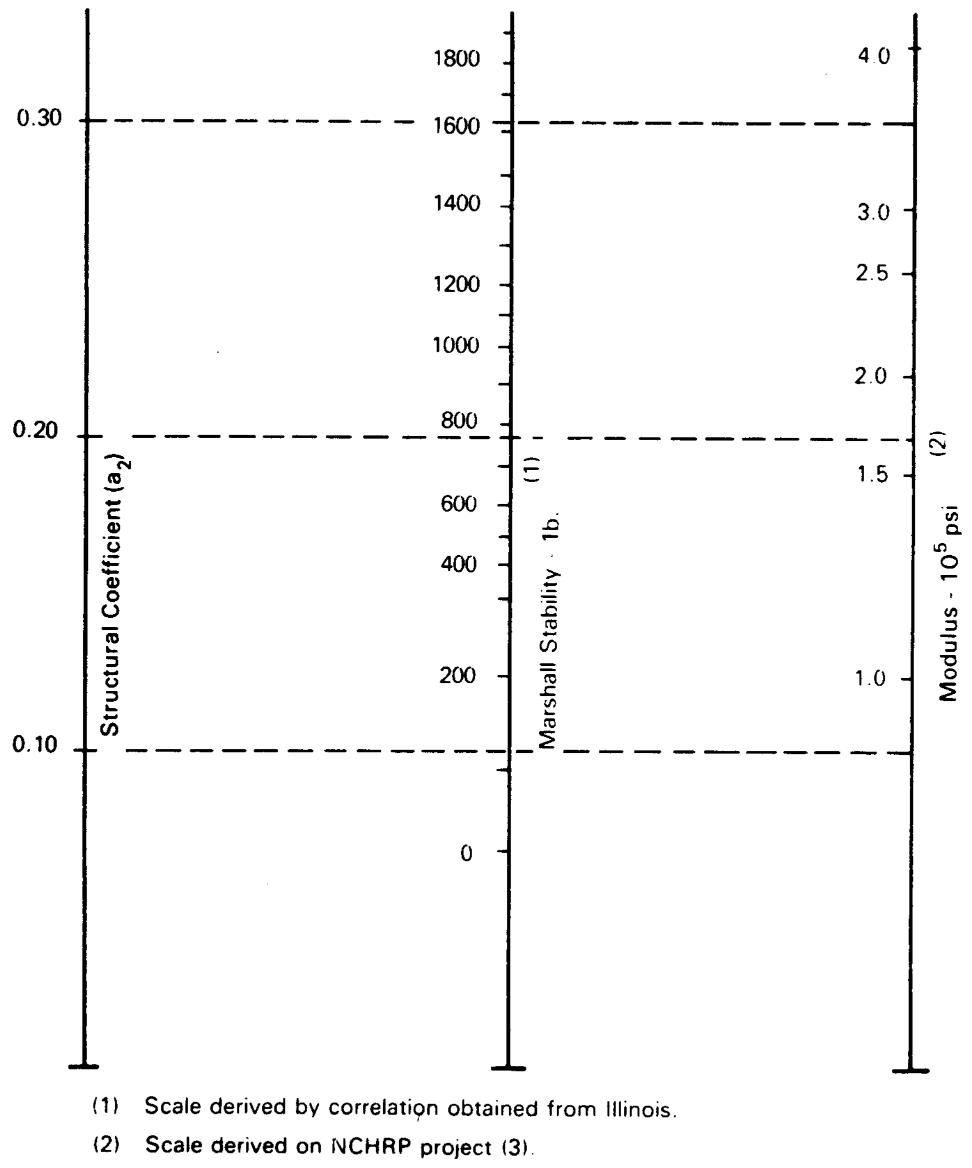


Figure 2.4 Layer Coefficient/Modulus Relationship from 1993 AASHTO Guide.

Chapter 3

STRUCTURAL LAYER COEFFICIENT RESEARCH

3.1 Introduction

As described in Chapter 2, the layer coefficients derived at the AASHO road test were simply regression coefficients from an empirical model with no physical meaning. As such, no effort was made at that time to correlate them with any engineering material property. In the years since the AASHO Road Test researchers and agencies have used mechanistic (analytical), empirical (satellite road tests), and deflection techniques in attempts to establish coefficients for other materials and other configurations of materials. One of the first attempts was reported by Shook and Finn [3.1] at the 1st International Conference on the Structural Design of Asphalt Pavements. Shook and Finn stated: " It is believed that the coefficients a_1,a_2, and a_3 are functions of the strength of various layers involved. At present time (1962), however, no entirely satisfactory techniques are available for defining or measuring these strength factors."

Some researchers also worked on establishing thickness equivalencies among different pavement materials, a concept similar to the layer coefficients. The only difference is that the term "thickness equivalency" is related to either asphalt concrete surface or crushed stone base, which is assigned a

Table 3.1 1962 Interim AASHO Coefficients *.

PAVEMENT COMPONENT	COEFFICIENT a_1	a_2	a_3	THICKNESS EQUIVALENT TO 1 INCH PLANT MIX SURFACE, inches
Surface course:				
Road mix (low stability)	0.20			2.2
Plant mix (high stability)	0.44*			1
Sand asphalt	0.40			1.1
Base course:				
Sandy gravel		0.07[b]		6.3
Crushed stone		0.14*		3.1
Cement treated (no soil-cement)				
650 psi or more*		0.23[b]		1.9
400-650 psi		0.20		2.2
400 psi or less		0.15		2.9
Bituminous treated				
Coarse graded		0.34[b]		1.3
Sand asphalt		0.30		1.5
Lime treated		0.15-0.30		2.9-1.5
Subbase:				
Sandy gravel			0.11*	4
Sand or sandy clay			0.05-0.10	8.8-4.4

[a] Compressive strength at 7 days

[b] This value has been estimated from AASHO Road Test data, but not to the accuracy of those factors marked with an asterisk.

* AASHO Interim Guide for Design of Pavement Structures-1972

rating value of 1. Thickness equivalents in inches of asphalt concrete surfacing (plant mix surface course) and based on the original AASHO coefficients are shown in Table 3.1. The development and use of equivalents suggests that they are constant and independent of load, pavement layer thickness, and subgrade support.

3.2 Deflection Approach

The surface deflection caused by a wheel moving over a flexible pavement is directly related to the deformation properties of the various layers which constitute the pavement and its foundation. If weakening of the materials by cracking or shearing occurs then the measured deflection will increase. Any factor tending to stiffen any of the materials will be expected to result in a decrease of deflection. This stiffness approach regards deflection as an indicator of pavement condition. Using this approach, elastic layer theory has been applied to predict pavement condition. This requires that load application, deflection and material property data be available.

Monismith et al. [3.2] conducted research relative to base course layer equivalency at the University of California at Berkeley. They emphasized that no one layer equivalency could be assigned to a specific material. The material equivalency was reported to be dependent upon a number of factors. Some of the factors reported by Monismith were:

1. Wheel load and contact pressure
2. Stiffness of the particular material
3. Layer thickness
4. Stiffness of the other materials

Three prototype pavements were constructed at the University of California in order to investigate the base course effectiveness and therefore its material equivalency. Each pavement consisted of an asphalt concrete surface course, and asphalt treated aggregated base course, and a natural subgrade. Laboratory tests were conducted on the individual layers of the test pavements. Elastic theory (both Bousinesq and Burmister) was used to compute deflections using the laboratory test properties. These computed deflections were compared with the actual field deflections measured from repeated or dynamic plate load tests. Some of the important findings of this research were:

1. For tests at the surface of the two layer pavements containing the asphalt emulsion and liquid asphalt treated materials, the Bousniesq stress distribution was adequate in determining deflection during curing of the base material.
2. For the two layer pavement containing the black base material, layered elastic theory was found to be more suitable to measure the response to loading.
3. Pavement deflections were predicted within

reasonable accuracy from laboratory tests and the application of theory.

Vaswani [3.3] determined thickness equivalency values for paving materials commonly used in Virginia. Benkelman Beam deflection data, subgrade support value based on CBR and 'Resilience Value', and a stepwise regression analysis were used to determine the thickness equivalency values shown in Table 3.2. In 1969, Vaswani [3.4] proposed other coefficients also shown in Table 3.2. His main objective was to provide a pavement design method for Virginia based on the AASHO Road Test results in terms of thickness equivalencies of the materials, soil support values and traffic. For the 1969 study, the Dynaflect was used to measure deflections instead of the Benkelman Beam.

A comparison of the coefficients given in Table 3.2 shows that the "cement treated stone base" had a lower value in the 1969 listing than in 1968. The same trend is noted for the "cement stabilized subbase".

In 1970 Vaswani [3.5] carried out a new study of thickness equivalency values for cement treated aggregate subbase based on:

1. Soil support value, defined as the product of the CBR and its "resiliency".
2. Traffic, in terms of 18 kip ESAL, and
3. Deflections.

The final values from these studies are given in Table 3.3.

Table 3.2. Vaswani-Virginia Thickness Equivalencies. [3.3,3.4]

Material	Thickness Equivalencies	
	Ref.3.15	Ref.3.16
Asphalt Concrete	1.00	1.00
Stone Base	0.35	0.35
Cement-treated Stone base	1.10	1.00
Asphalt treated Stone base	0.75	---
Select Material subbase	0.00	0.00
Cement-stabilized subgrade	0.50	0.40

Table 3.3. Additonal Thickness Equivalences for Virginia Materials . [3.5]

Serial No.	Location	Material	Values: Primary and Interstate Roads	Values: Secondary and Subdivision Roads
1	Surface	Asphaltic concrete	1.0	1.0
3	Base	Asphaltic concrete	1.0	1.0
		Cement-treated aggregate over dense-graded aggregate base or soil cement or soil lime and under asphaltic concrete mat.	1.0	1.0
		Dense-graded aggregate, crushed or uncrushed	0.35	0.60
		Select material I (Va. specifications) directly under asphlatic concrete material and over a subbase of a good quality.	0.35	--
		Select material cement-treated	--	0.80
3	Subbase	Select material I, II and III (Va. specifications)		
		In Piedmont area	0.00	0.00
		In valley and ridge area and coastal plain	0.20	0.50
		Soil cement	0.40	0.60
		Soil lime	0.40	0.55
		Select material cement treated	0.40	0.80
		Cement treated aggregate directly over subgrade	0.60	--

The table shows that thickness equivalency values vary with the type of roads (from primary to secondary) indicating that the structural layer coefficients vary with the traffic level.

In 1971, Vaswani [3.6] introduced the concept of 'Spreadibility' which is the average deflection in percent of the maximum deflection. Relating this concept to the thickness index ($a_1h_1 + a_2h_2 + a_3h_3 = SN$) and based on Boussinesq's and Terzaghi's analysis he established thickness equivalency values for the pavement materials under study. These values agree closely with those previously determined. Some of the values are listed in Table 3.4.

Chu et al.[3.7] conducted field and laboratory investigations to develop a tentative procedure for subgrade evaluation. They found good correlations between the back calculated AASHO soil support values and triaxial strength parameters. They also developed coefficients for various paving materials (Table 3.5).

The "Granular Equivalent" (G.E) concept is utilized in the Minnesota DOT flexible pavement design procedure [3.8]. Minnesota established a comprehensive flexible pavement surface deflection historical data base (primarily Benkelman Beam). Lukanen [3.8] stated:

> "The current design system which was adopted as a design standard in 1974 combines the AASHO Road Test relationship of the peak spring Benkelman Beam deflection versus the Standard Axle Load applications with the Investigation 183 relationship of Benkelman beam deflection versus subgrade R-value and G.E. The combination of these two relationships allows the design

Table 3.4. Thickness Equivalencies Developed by Vaswani. [3.6]

Material	Thickness Equivalence
Asphalt concrete	1.0
Cement-treated aggregate in base	1.0
Untreated aggregate	0.35
Cement-treated subgrade	0.44
Lime-treated subgrade	0.44

Table 3.5 Suggested Coefficients South Carolina. [3.7]

Pavement Component Remarks	Coefficient of Strength	Relative
Asphlat concrete	0.44*	0.27 for old asphlat concrete underlying new bituminous surfacing
Bituminous Surface Treatement	0.30	0.25 for old surface treatment underlying new bituninous surfacing
Sand Asphalt Base	0.25*	
Bituminous Stabilized soil	0.20	0.07 for bituminous stabilized soil in inferior condition
Granular Base or Subbase	0.07*	
Crushed Stone Base	0.14*	

* Values based on information given in reference [3.35]

G.E to be determined."

Typical G.E. values for selected Minnesota materials are shown in Table 3.6.[3.8]. Note that the G.E. factors are "constant".

A comprehensive study of "full-depth asphalt pavements" was conducted by Minnesota [3.9]. Twenty-six sections ranging in thickness from 5 to 17.5 inches were intensely monitored for several years using Benkelman Beam deflection procedures. Pavement serviceability trends were established in terms of roughness, rut depth, and surface condition. The study results indicated the following relationship between G.E. and "full depth" asphalt concrete thickness.

$$G.E = 5.68T - 28.9$$

where,

G.E = Granular Equivalent Thickness (inches), and

T = Thickness of full depth asphalt construction (inches).

Note that the G.E factor is not constant (Figure 3.1). It increases for increased total pavement thickness. Some interesting conclusions from the Minnesota full depth study were:

1. As the pavement thickness increases beyond twelve to thirteen inches, there is little decrease in measured deflections.
2. Full-depth deflection behavior is primarily influenced by the temperature of the asphalt concrete and to a lesser extent by seasonal

Table 3.6. Minnesota DOT G.E. Factors [3.8]

Material	GE Factor
Bituminous surface (2361 or 2341)	2.25
Bituminous surface, binder or base (2331)	2.00
Class 5 or Class 6 Base	1.00
Class 3 or Class 4 Sub-base	0.75

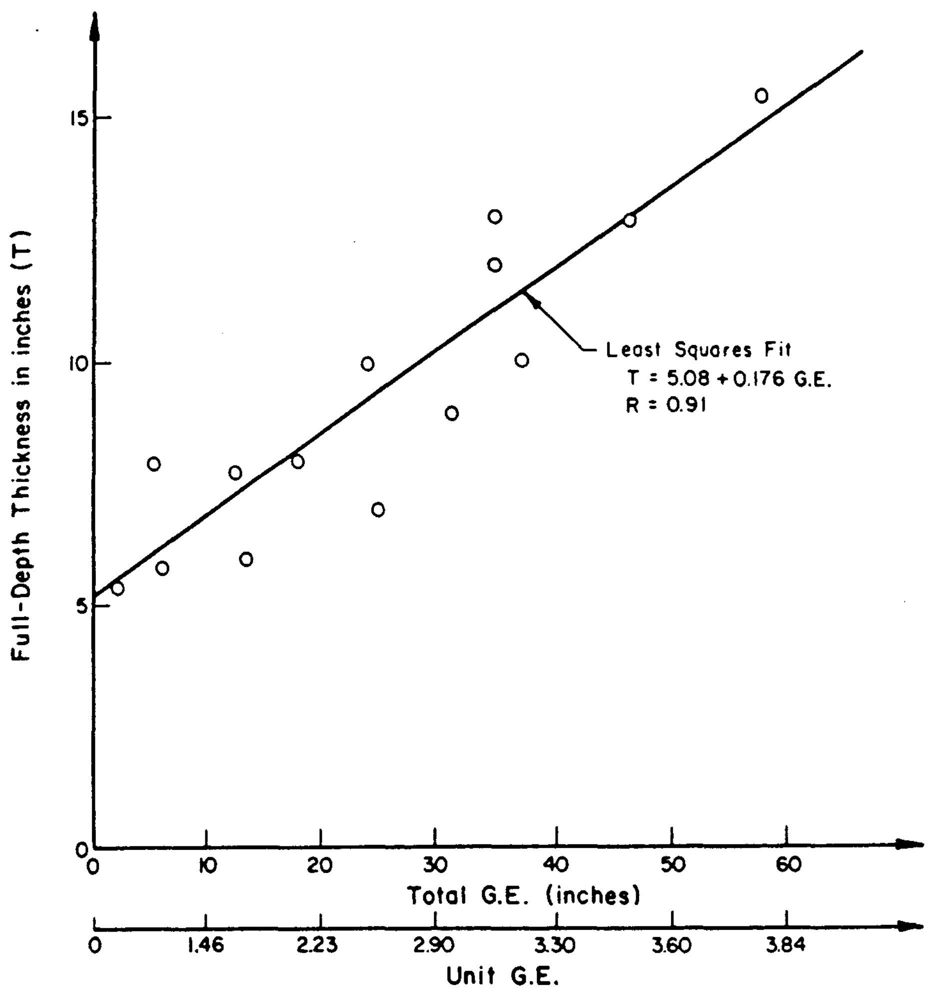


Figure 3.1. Minnesota Granular Equivalency Relations for Full Depth Asphalt Sections.[3.9]

effects.

3.3 Analytical Approach

The analytical approaches used to establish layer coefficients are based on mechanistic pavement models. The basis of these models are the mechanics of materials and elastic theory. The elastic layer theory models use inputs, such as wheel loads to predict the deflection of the pavement and stresses and strains in the layers. In the use of such models either laboratory testing data or observed performance data are used to provide a relationship between loadings and failure. The relationship is then used to predict distress in pavements.

Thompson [3.10] used linear elastic theory to evaluate soil-lime and granular base behavior. The objective was to provide information for establishing appropriate soil-lime coefficients for use in the Illinois Department of Transportation design procedure which is nearly identical to the AASHTO procedure. Coefficient determinations were based on equivalent surface deflection and subgrade vertical stress. For a given soil-lime mixture strength, the thickness requirement was found to increase for weaker subgrades (Figure 3.2). For a given subgrade strength, the required base thickness was found to vary inversely with the mixture strength. Figure 3.2 shows that the coefficient a_2 (base coefficient) is influenced by soil-lime mixture strength and

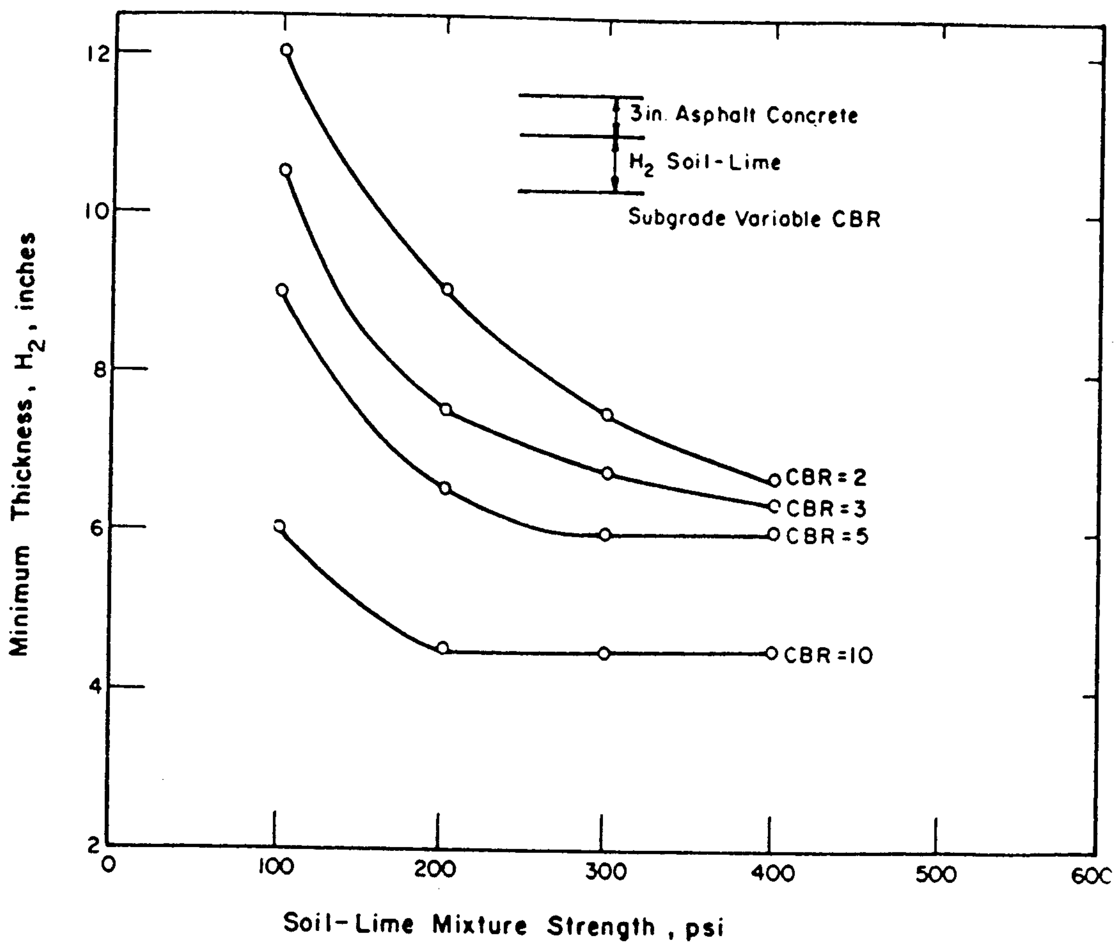


Figure 3.2. Soil-Lime Thickness-Mixture Strength Relationships. [3.10]

subgrade strength. The coefficient defined by the research was not constant. Its value ranged from 0.12 to 0.26. Based on this work, the Illinois DOT adopted coefficients for soil-lime mixtures of 0.11 for base and 0.12 for subbase. (Note that these values and the use of soil-lime as a base or subbase have since been abandoned.)

In 1978, Thompson [3.11] reported using a stress dependent finite element pavement model (ILLI-PAVE) developed to determine the "thickness equivalency ratios" (TER) for granular, bituminous stabilized base course, and stabilized bases. Factors considered were: thickness of asphalt concrete layer, asphalt concrete modulus, stabilized layer thickness and modulus, and subgrade resilient modulus. TER were determined by comparing the base course material thickness required to provide equivalent pavement response. Response parameters considered were: maximum subgrade deviator stress, maximum subgrade compressive strain, and maximum subgrade normal stress.

Figures 3.3, 3.4 and 3.5 show the TER variation between granular and stabilized layers for various subgrade moduli. Figure 3.6 shows the TER variation with respect to the subgrade modulus, with the other factors remaining constant.

In a study for the Maryland State Highway Administration, Rada and Witcak [3.12] conducted a comprehensive laboratory study and theoretical analysis to establish material layer coefficients for unbound granular materials. The six granular

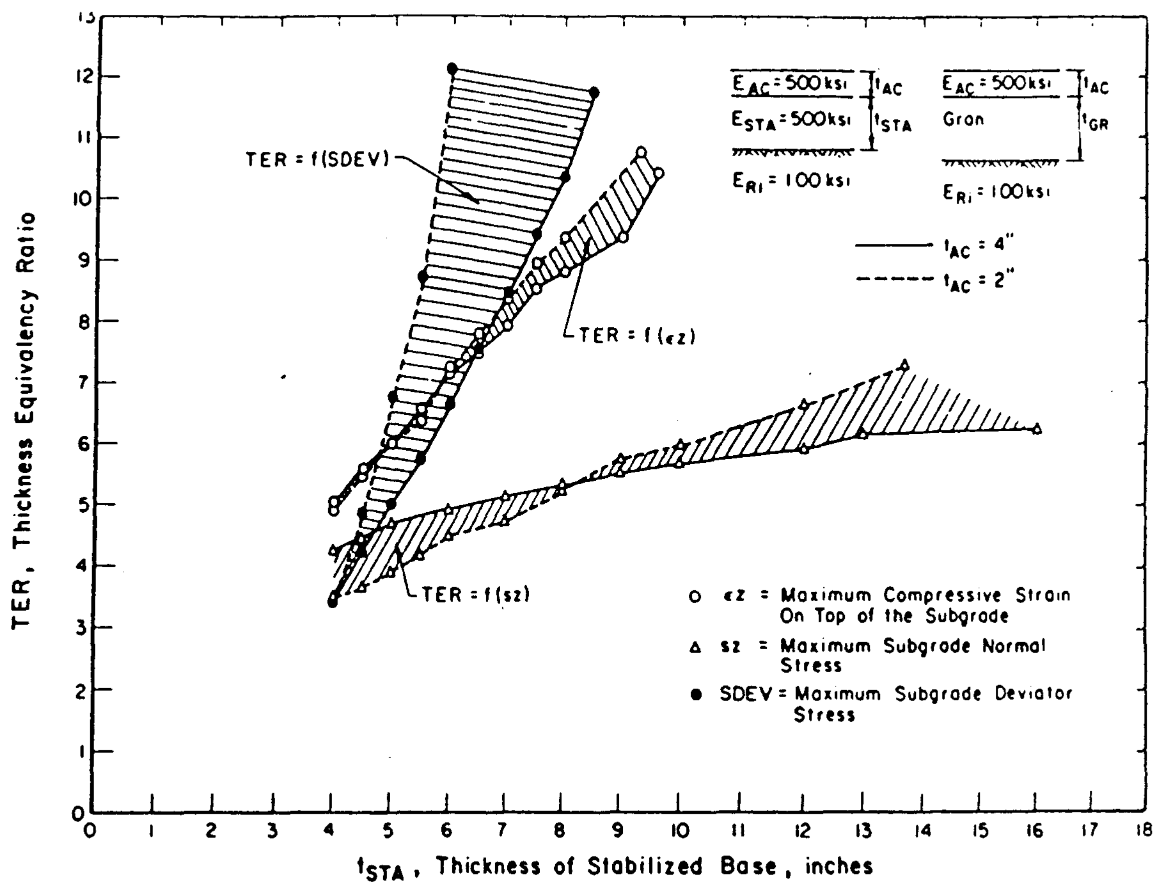


Figure 3.3 Thickness Equivalency Ratio-Base Thickness Relations for Stabilized Bases (E_{Ri} = 1 ksi). [3.11]

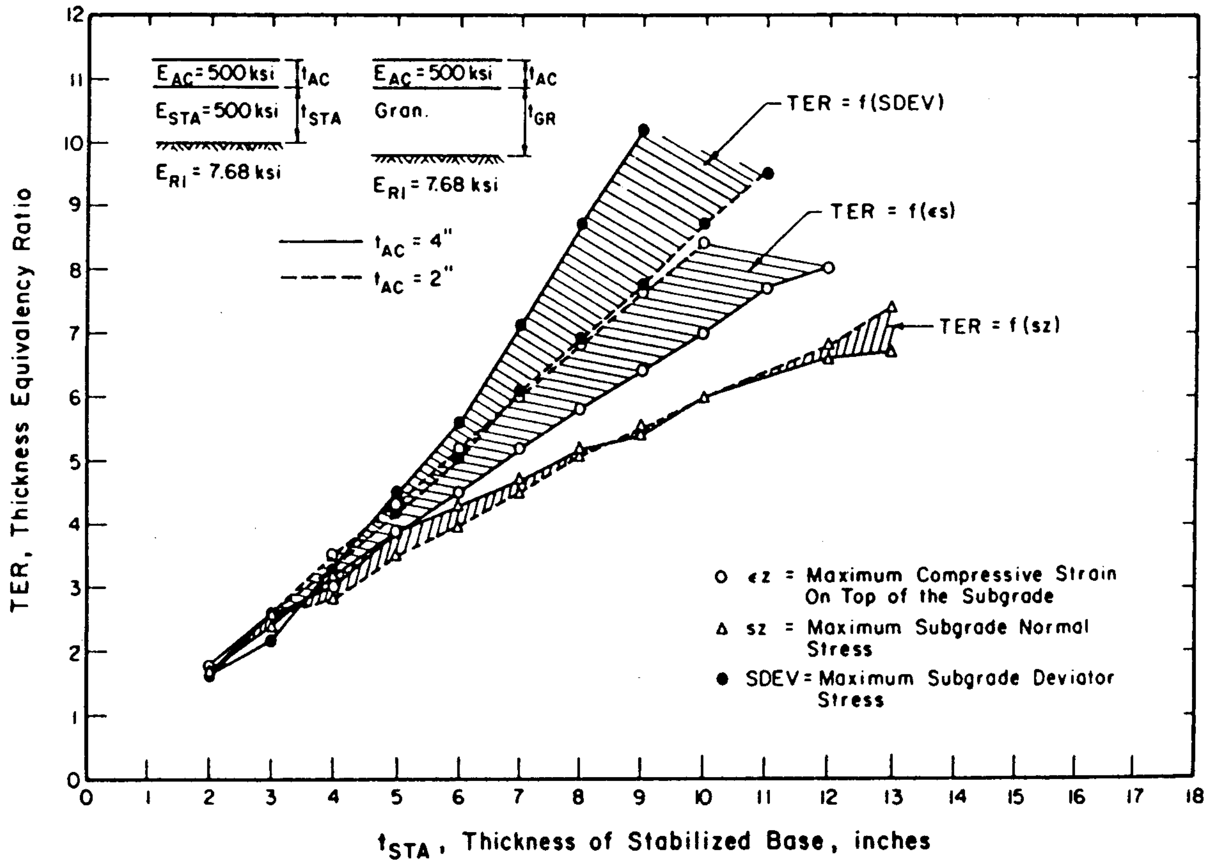


Figure 3.4 Thickness Equivalency Ratio-Base Thickness Relations for Stabilized Bases (E_{Ri} =7.68 ksi). [3.11]

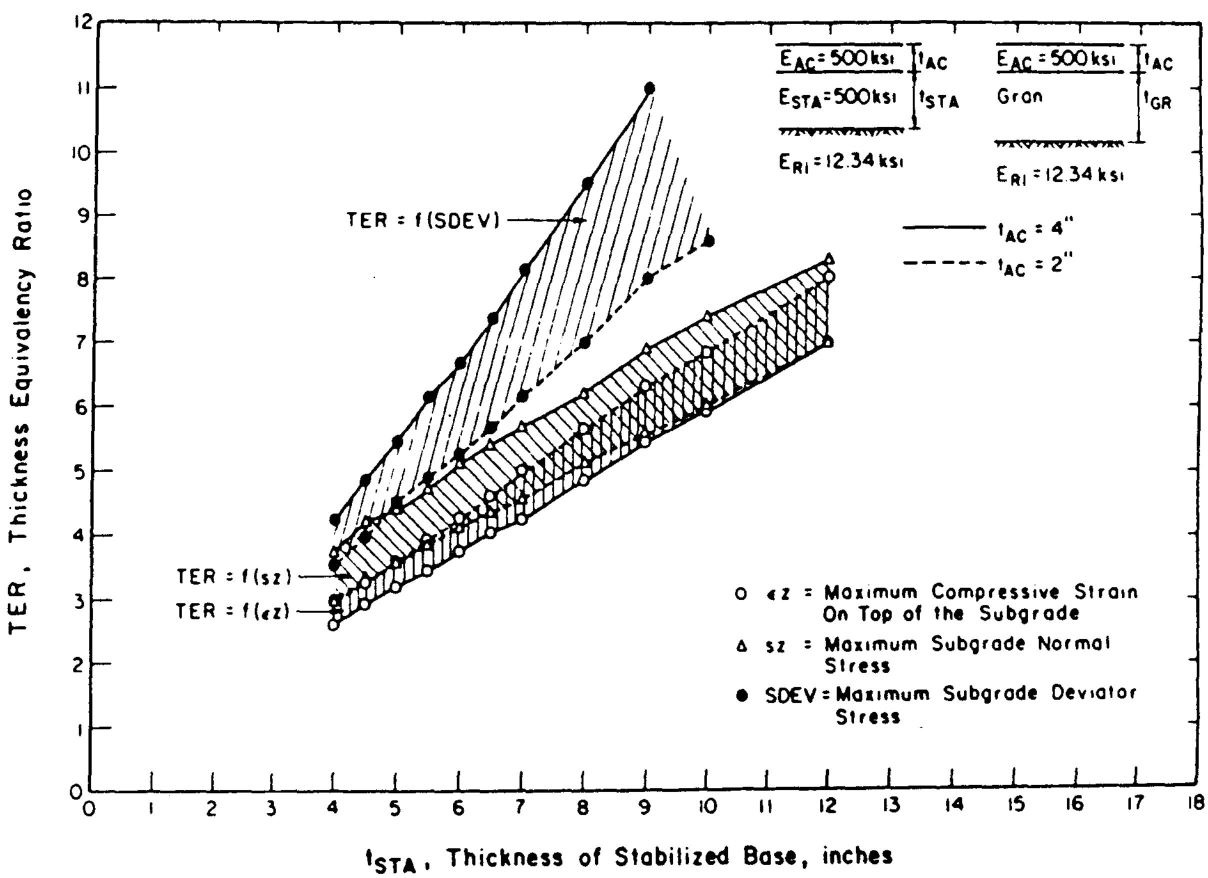


Figure 3.5 Thickness Equivalency Ratio-Base Thickness Relations for Stabilized Bases (E_{Ri} = 12.34 ksi). [3.11]

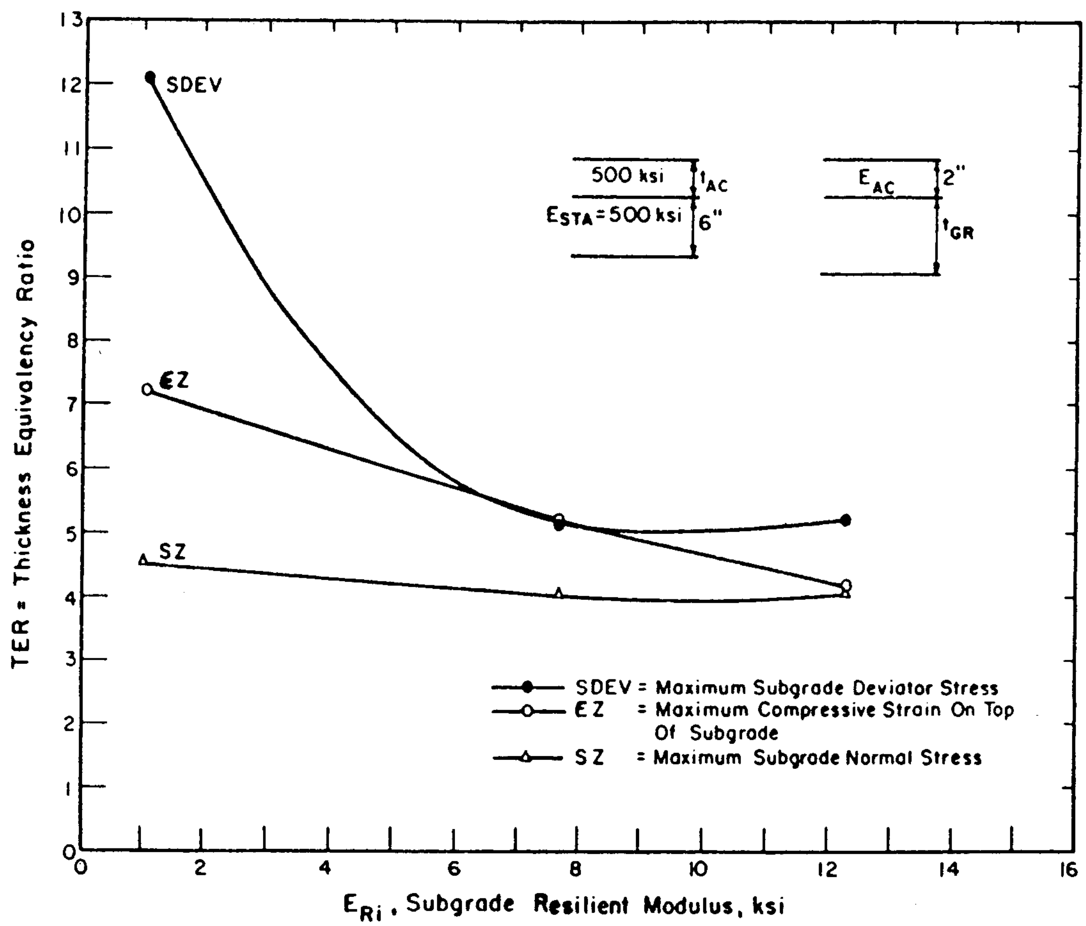


Figure 3.6 Subgrade Modulus Effects on Thickness Equivalency Ratios. [3.11]

materials ranged from dense graded aggregates to bank-run gravel. The level of saturation and compaction were varied for the materials. The resilient modulus-stress state (M_r - Θ) relations developed from the lab program were utilized in a stress dependent linear elastic pavement analysis program. From the structural analysis, a "resilient modulus" value was back calculated. The resilient modulus value was used in conjunction with Figure 2.8 from the AASHTO Guide to establish an a_2 value.

The Maryland study results indicated that material type/source, subgrade strength, compacted density, saturation, and asphalt concrete thickness were factors that were, "significant from a design standpoint". Table 3.7 is a summary of the typical layer coefficients developed in the Maryland study. It is apparent that the coefficients vary considerably for a given material and for material type/source.

Kalankamary [3.14] reported the development of a unique method for determining the layer coefficient of flexible pavement materials. The material properties influencing the layer coefficient by his method were resilient modulus, poisson's ratio, and fatigue susceptibility. The elastic properties data were developed from laboratory tests, and the fatigue from literature study. Intended for the use by the Mississipi State Highway Department (MSHD), layer coefficients of several materials indigenous to that State (asphalt concrete, soil-cement, and soil-lime) were developed in this

Table 3.7. Typical Layer Coefficients Developed in Maryland Study.

Material	Asphalt Thickness (in)	Subgrade CBR	Dry (S_r < 60 percent) SCE*	Dry (S_r < 60 percent) MCE**	Wet (S_r > 85 percent) SCE*	Wet (S_r > 85 percent) MCE**
Base						
DGA	< 5	3	0.124	0.150	0.092	0.117
		5	0.130	0.157	0.098	0.125
		10	0.144	0.170	0.113	0.139
		25	0.171	0.197	0.145	0.171
	> 5	3	0.100	0.126	0.065	0.090
		5	0.104	0.130	0.069	0.095
		10	0.110	0.137	0.076	0.102
		25	0.139	0.165	0.108	0.134
Crusher Run	< 5	3	0.096	0.140	0.091	0.129
		5	0.103	0.147	0.100	0.138
		10	0.120	0.164	0.124	0.163
		25	0.155	0.199	0.174	0.212
	> 5	3	0.066	0.110	0.048	0.087
		5	0.071	0.115	0.055	0.093
		10	0.079	0.123	0.067	0.105
		25	0.115	0.159	0.116	0.155
Slag	< 5	3	0.137	0.187	0.050	0.101
		5	0.141	0.192	0.054	0.105
		10	0.154	0.204	0.067	0.118
		25	0.178	0.229	0.091	0.142
	> 5	3	0.115	0.166	0.028	0.080
		5	0.119	0.170	0.032	0.083
		10	0.124	0.175	0.037	0.089
		25	0.149	0.200	0.062	0.114
Subbase						
Sand/gravel	< 5	3	0.060	0.100	0.024	0.042
		5	0.071	0.117	0.035	0.060
		10	0.100	0.145	0.054	0.102
	> 5	3	0.060	0.100	0.024	0.042
		5	0.068	0.113	0.029	0.054
		10	0.082	0.128	0.033	0.078
		25	0.103	0.148	0.064	0.110

* Standard Compactive Effort

** Modified Compactive Effort

study.

First an analytical model for predicting the life of flexible pavements was developed and then, using this model, the researcher established "equivalency" between pavement materials as well as layer coefficients. Fatigue cracking was the criterion employed in deriving the structural layer coefficient. The researcher developed a probabilistic fatigue model. The primary steps involved in developing the model were:

1. Solving for primary structural response;
2. Predicting fatigue life from structural response using empirical relationship;
3. Predicting cumulative fatigue damage using Miner's hypothesis.

The model was developed with traffic, material properties, and environmental effects as stochastic variables. The resulting equation was amenable to direct solution for design of flexible pavements. The structural thickness resulting from the suggested probabilistic fatigue design was somewhat larger than that of the 1972 AASHO Interim Guide design.

Layer coefficient calculations were based on the premise that it is possible to establish a "thickness equivalency" between layers. First, it was established that the surface mixture of the AASHO Road Test and that used by MSHD were identical. In view of their nearly identical properties, the MSHD surface mixture was assigned a layer coefficient value of

0.44. Equivalence between surface mixture and base mixture was established on the basis of equal fatigue lives. The layer coefficient of soil-cement base was computed by comparing its fatigue life with that of asphalt base. The fact that the soil-lime helps to alleviate fatigue cracking in the asphalt base layer led to the layer coefficient determination of the former.

The layer coefficient values derived using the probabilistic fatigue design method were compared to those proposed in the 1972 AASHO Interim Guide [Table 3.8]. It was concluded that the satisfactory agreement between the two sets of values attested to the validity of the proposed method for layer equivalency determination including the probabilistic fatigue design method of pavement design.

3.4 Performance Approaches

Performance, or empirical models, have been used to establish layer coefficients. Such models typically relate observed field performance to design variables. Many of these have been developed from the AASHO Road Test data.

Typical of this are studies performed by and for The Asphalt Institute. One study reported by Shook and Finn [3.1] demonstrated that the thickness equivalency ratio varies with AC thickness as shown in Figure 3.7. In another study for TAI, Skok and Finn [3.16] concluded that:

1 inch of AC = 2 inches of good crushed stone

Table 3.8. Comparision of layer coefficients. [3.14]

No.	Material/layer	Layer Coefficient	
		Recommended	AASHO
1.	Plant-mix asphalt surface with AC-20	0.44	0.44
2.	Plant-mix asphalt base with AC-40	0.38	0.34
3.	Soil-cement base (7-day compressive strenght no less than 600 psi)	0.24	0.20-0.23
4.	Soil-lime subbase (CBR no less than 20)	0.24	0.15-0.30

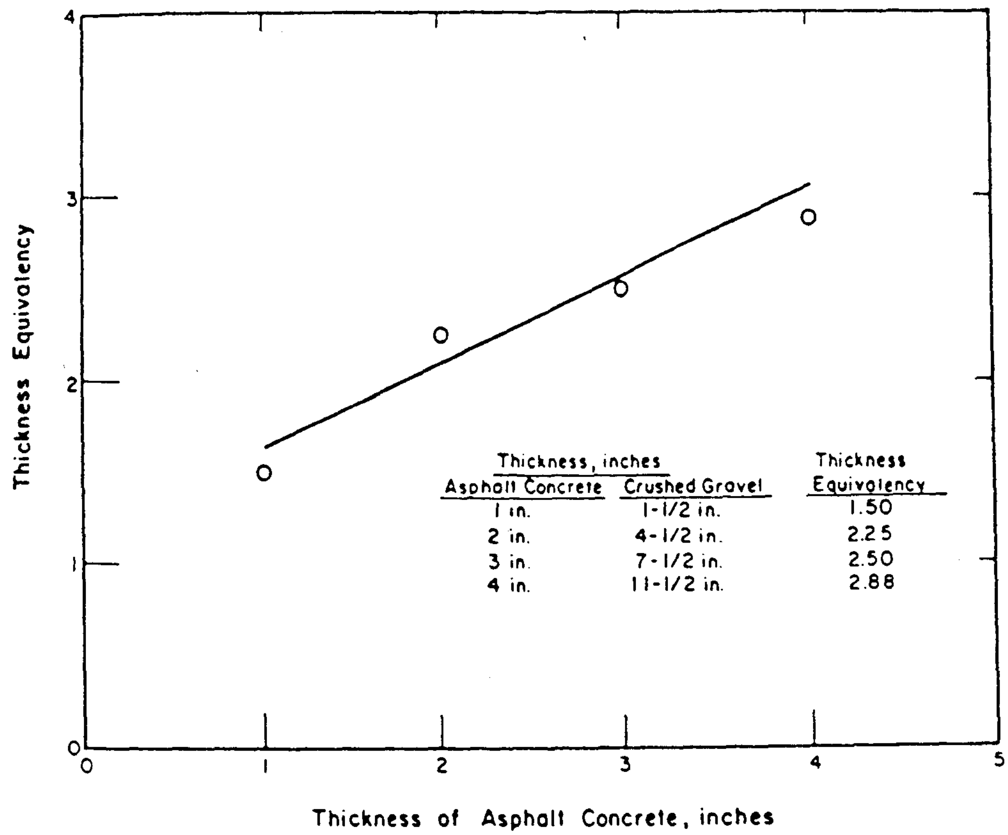


Figure 3.7. Thickness Equivalency-Thickness Relations for Asphalt Concrete Sections. [3.1]

1 inch of AC = 2.67 inches of granular subbase

These equivalents were used in the Asphalt Institute's MS-1 Manual [3.17] utilized during the late 1960's and the 1970's. (The recently revised MS-1 Manual [3.18] does not utilize the "thickness equivalency" concept.)

In Canada, Kamel et.al, [3.19] analyzed Brampton Road Test results relative to secondary road applications. Structural responses (elastic layer theory-stresses, strains, deflections) were correlated with performance for different traffic and environmental conditions. The relations in Figure 3.8 are based on sections that have a constant thickness of asphalt concrete surface (3-1/2 inches) and an RCI (Riding Comfort Index) loss equal to 4. The initial RCI is assumed to be 85.

The upper part of Figure 3.8 shows the relation between vertical stress level on the subgrade and accumulated equivalent 18-kip single-axle loads for four pavement types. The lower part of the figure shows the relation between vertical stress level and equivalent base thickness (i.e., actual base thickness plus transformed subbase) for each pavement type.

Subbase thicknesses were converted into equivalent base thickness using the equivalencies suggested by Phang [3.20]. One inch of subbase equals 0.57 inches of granular base or 0.57 inches of bituminous stabilized base or 0.23 inches of asphalt concrete base. The Brampton Road Test data indicated

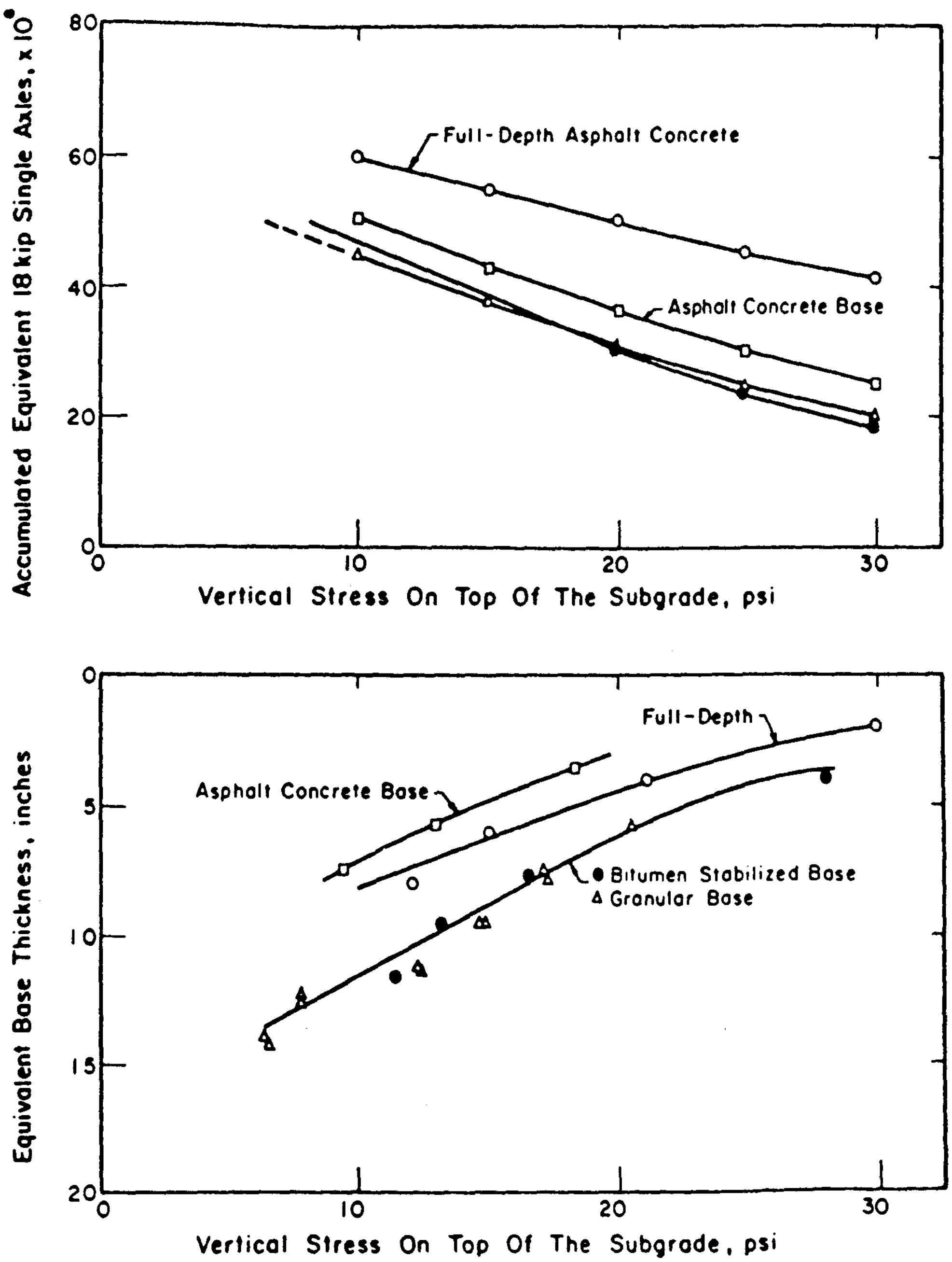


Figure 3.8. Brampton Test Road Materials Relationships. [3.19]

that after 0.5 million load applications the pavements reached a terminal RCI of 4.5. Entering Figure 3.8 with this number of applications, the subgrade stress levels corresponding to 4.0 units of RCI loss can be determined as well as the equivalent base thickness for each pavement type. The following base thicknesses were determined:

13.5 inches of granular base,

12.5 inches of bituminous stabilized base,

26.7 inches of asphalt concrete base (with subbase),

4.0 inches of full depth asphalt.

Thickness equivalencies based on these results are given in Table 3.9.

Jung and Phang [3.21] further considered the Brampton Road Test data using elastic layer theory in an attempt to develop a more rational method of pavement design for Ontario. A relationship between an equivalent granular base thickness and the moduli and thickness of the layers was established by using a derivation of Newmark's formula for vertical deflections in the center of a circular load area in an elastic half space and Odemark's transformation. This relationship is:

$$H_e = h_1 \sqrt[3]{\frac{E_1}{E_{2g}}} + h_2 \sqrt[3]{\frac{E_2}{E_{2g}}} + h_3 \sqrt[3]{\frac{E_3}{E_{2g}}}$$

where

H_e = equivalent granular base thickness,

Table 3.9. Brampton Road Test Layer Equavalencies. [3.19]

Type of Material	Equivalencies of Granular base (in)
1 inch of granular base (crushed gravel- or cruhsed rock)	1.0
1 inch of sand subbase	0.6
1 inch of bituminous-stabilized base	1.1
1 inch of asphalt concrete base (with subbase)	2.0
1 inch of asphlat concrete base (without-subbase, i.e., full depth)	3.4

h_1, h_2, h_3 = thickness of the bitumen layer, base and subbase, respectively,

E_1, E_2, E_3 = modulus of the bitumen layer, base and subbase, respectively, and

E_{2g} = modulus of granular base.

In the Ontario design approach, the values of the moduli E_1, E_2, and E_3 are constant for a given traffic level. Therefore the only variation in stiffness considered is the modulus of the subgrade E_m (Figure 3.9). The above equation can be written

$$H_e = C_1 h_1 + C_2 h_2 + C_3 h_3$$

as:

where;

C_1, C_2, and C_3 are coefficients which express the effect of each layer in resisting the load P to generate a vertical subgrade deflection W_s. These constants can thus be regarded as structural layer coefficients. The following coefficients were selected in accordance with Ontario experience (3.19). $C_1 = 2$; $C_2 = 1$; and $C_3 = 2/3$.

or

$$H_e = 2h_1 + h_2 + \frac{2}{3} h_3$$

Which indicated the following equivalencies:

1" AC = 2" Granular base = 3" Granular subbase.

TYPICAL SUBGRADE MODULI IN ONTARIO

GRAN. TYPE MATERIALS SUITABLE AS GRAN. BORROW	SANDY SILT AND CLAY LOAM TILL			LACUSTRINE CLAYS	VARVED AND LEDA CLAYS
	SILT < 40 V.F. Sa and Si. < 45	SILT 40 - 50 V.F. Sa. and Si. 45.60	SILT > 50 V.F. Sa. and Si. > 60		
psi 11,000	psi 5,000 TO 7,000	psi 4,000 TO 6,000	psi 3,000 TO 5,000	psi 3,500 TO 6,000	psi 2,000 TO 4,500

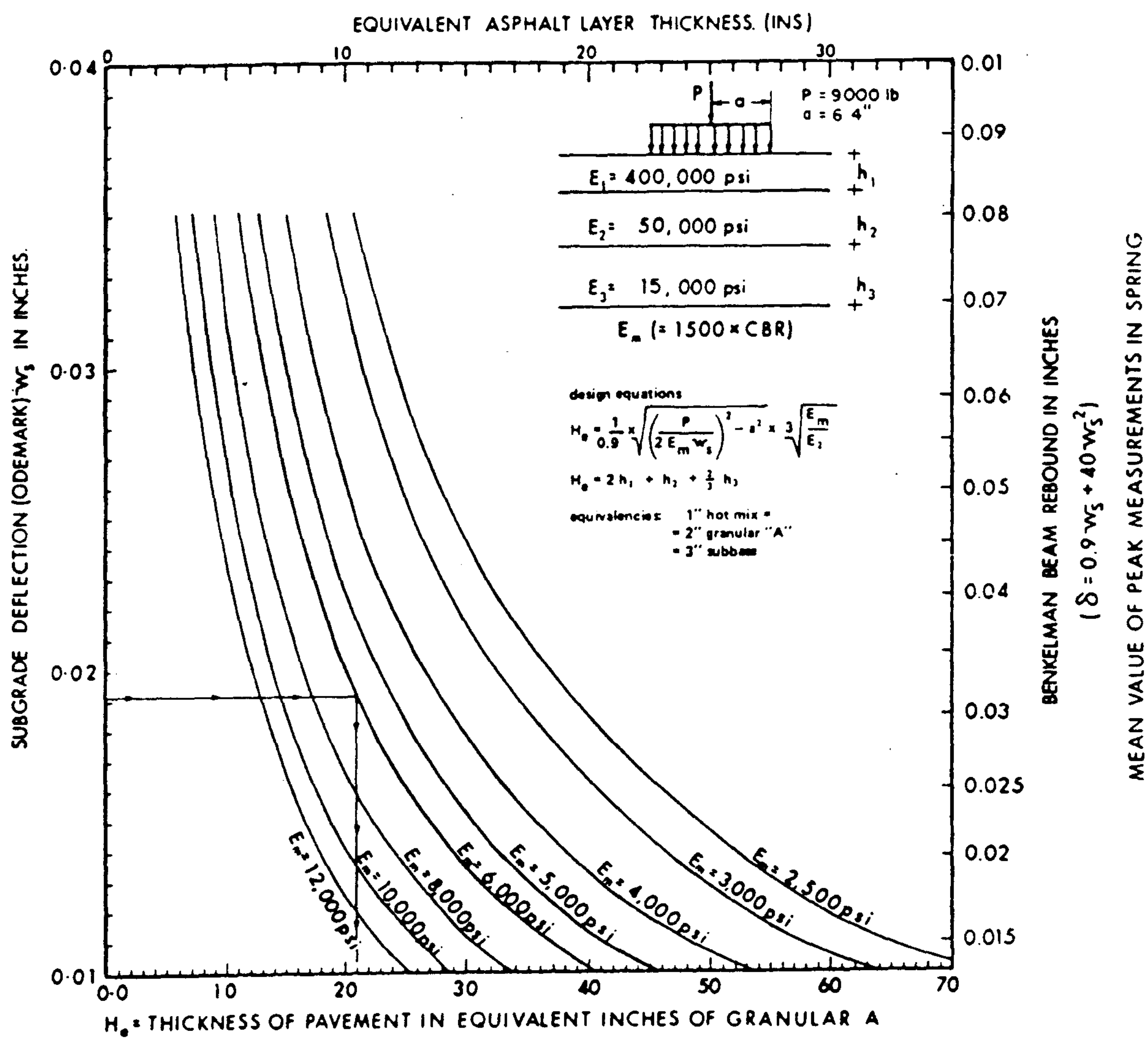


Figure 3.9. Ontario Flexible Pavement Design Chart. [3.21]

Therefore the Ontario design is only affected by the subgrade modulus and the traffic level as shown in Figure 3.9.

Takeshita [3.22] used the SN concept to analyze an existing four-lane highway in Japan. The subgrade conditions were the same for all lanes, but the traffic conditions were quite different. After one year of service, the lanes with the heaviest traffic were conspicuously cracked while lanes in the other direction did not show such damage. The AASHO calculated structural number was 6.3 with an overall thickness of 35.4 inches. Takeshita developed a design concept relating SN, equivalent wheel load, and the subgrade CBR. Takeshita's recommended coefficient relationship is shown in Figure 3.10. The lower moduli values correspond to crushed stone, gravel and sand in descending order. An increase in modulus effects an increase in the structural layer coefficient.

The Pennsylvania State University Test Track was utilized in an extensive study to establish structural coefficients for stabilized base materials. The base course materials were:

1. Aggregate bituminous base course
2. Aggregate cement base course
3. Aggregate lime-pozzolan base course
4. Bituminous concrete base course.

The subbase material was standard crushed limestone.

One of the first findings of the Penn-Test Track [3.23]

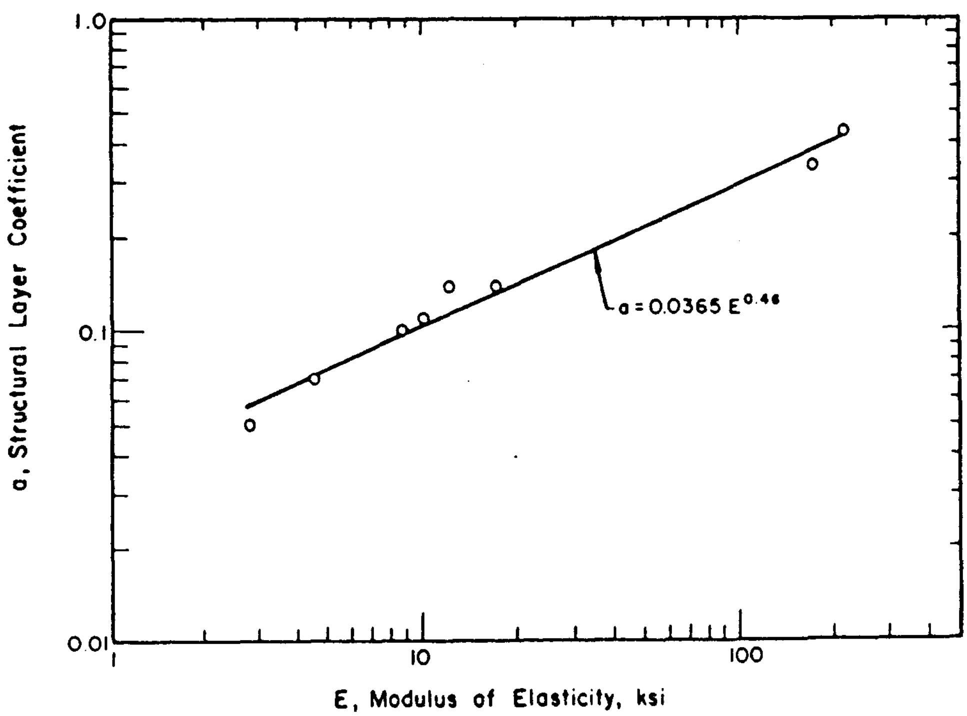


Figure 3.10 Coefficient-Modulus Relations Proposed by Takeshtia. [3.22]

was that the structural layer coefficient changes with layer thickness. Figure 3.11 shows the variability of the overall structural layer coefficient of the surface and base with respect to total thickness of surface and base.

In 1977, Wang and Larson [3.24] evaluated bituminous concrete base in the Test Track. Performance data together with response, limiting strain and limiting deflection criteria were used. The effect of layer thickness on the structural coefficients was determined as shown in Figure 3.12. The a_2 values are for bituminous-concrete base and the a_3 values are for crushed limestone subbase. Figure 3.12 also shows that the structural coefficient depends on the thickness of the asphalt concrete surface layer.

In 1979, Wang and Larson [3.25] evaluated the structural coefficients for asphalt stabilized and cement stabilized base course materials. They used two different methods for analysis namely:

1. The AASHO performance analysis approach,
2. The limiting criteria approach.

The AASHO performance analysis was based on the field performance of 11 bituminous concrete pavements and three cement aggregate pavements. The limiting criteria approach was based on maximum tensile strain at the bottom of the base course, maximum pavement compressive strain at the top of the subgrade, and maximum pavement surface deflection. The field performance data collected were rutting, cracking, and present

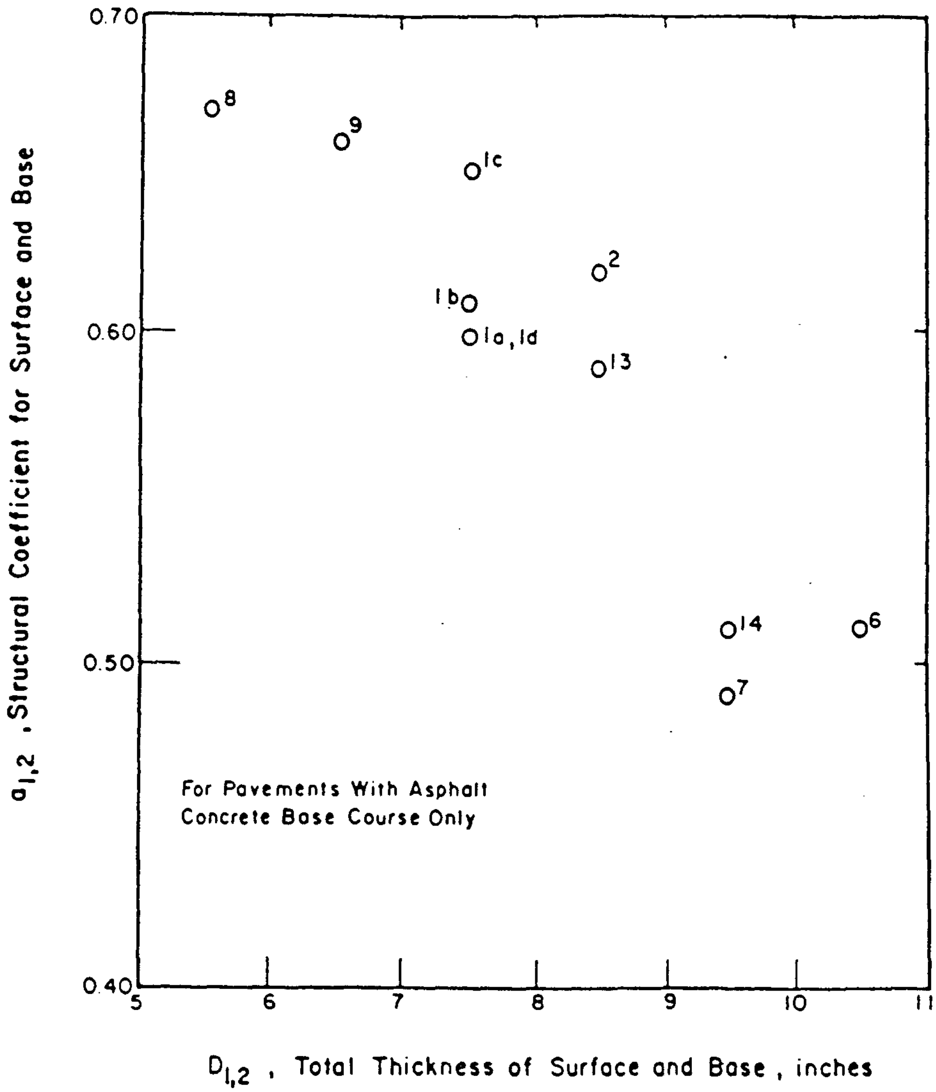


Figure 3.11 Structural Coefficient (a_1, a_2) -Total thickness relationship for asphalt concrete sections. [3.23]

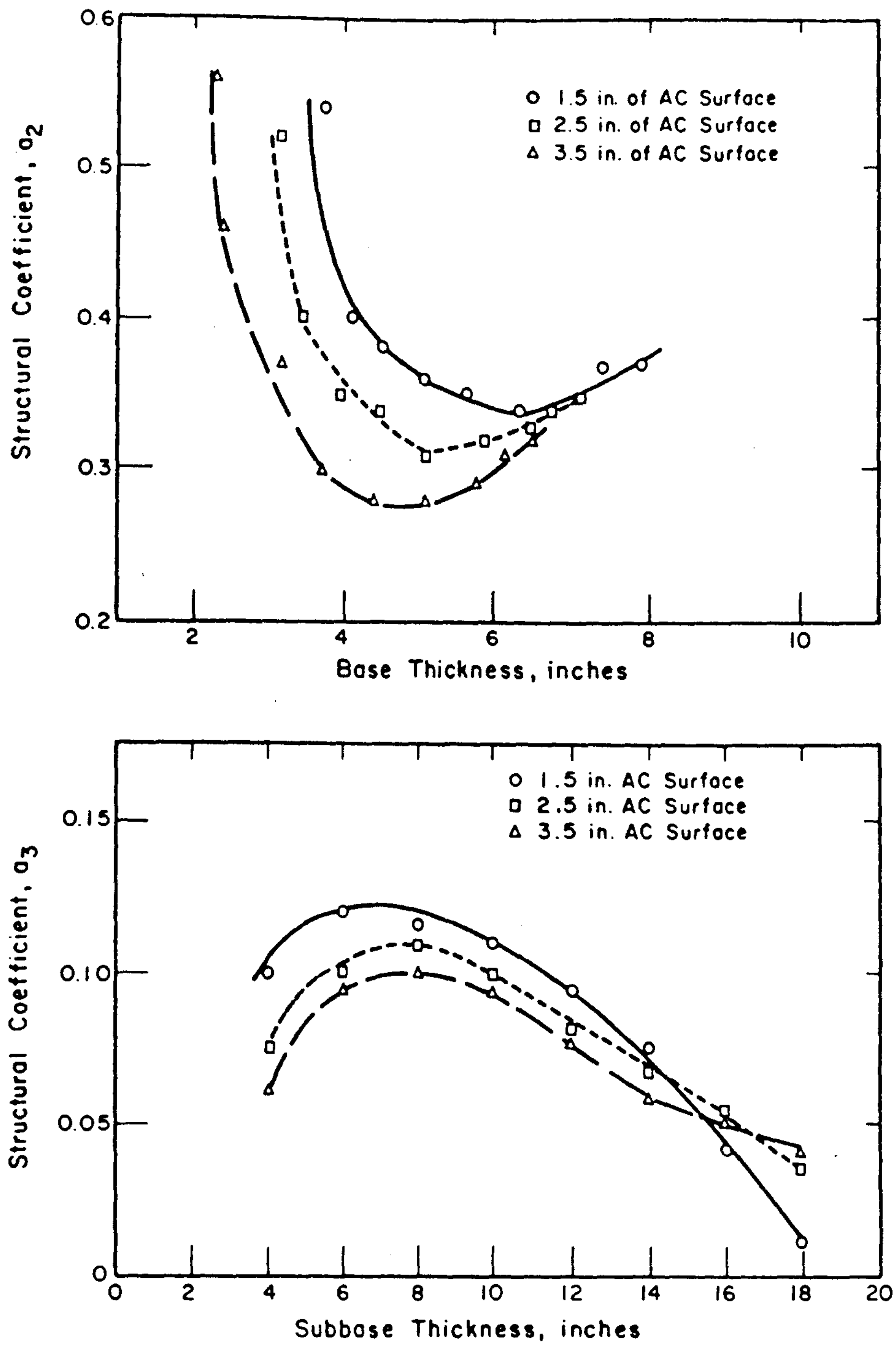


Figure 3.12. Layer Thickness Effects on Structural Coefficients (a_2-asphalt concrete; a_3-crushed stone). [3.24]

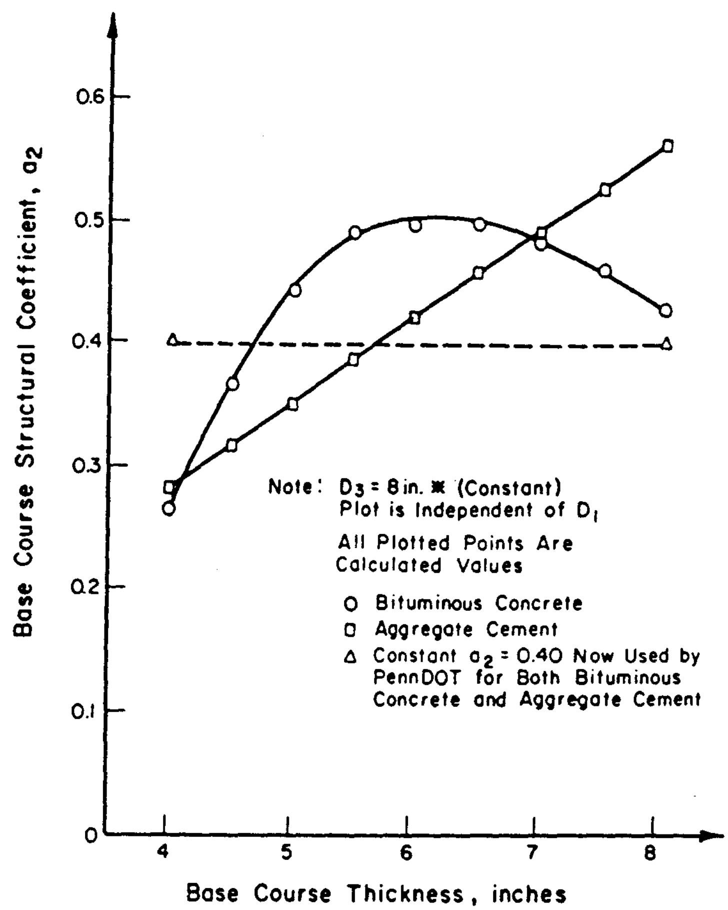
serviceability index. Limiting criteria were developed by using the BISAR computer program and the rutting and cracking data for the test pavements.

Results of the evaluation show good agreement between the two methods of analysis. The structural coefficients of base-course materials were found to vary with many factors, such as thickness and stiffness of each pavement layer, structural coefficients of other pavement layers, and pavement life. It was concluded that it is very difficult to assign a constant value to the structural coefficient of a base course material.

Figure 3.13 shows the structural coefficients of base course materials determined by the performance approach. Figure 3.14 contains the structural coefficients obtained by limiting criteria approach for bituminous concrete and aggregate cement base with limestone subbase. Figure 3.15, gives the comparison of values the coefficients for bituminous concrete base obtained by the two methods.

3.5 The Rehabilitated AASHO Road Test Site

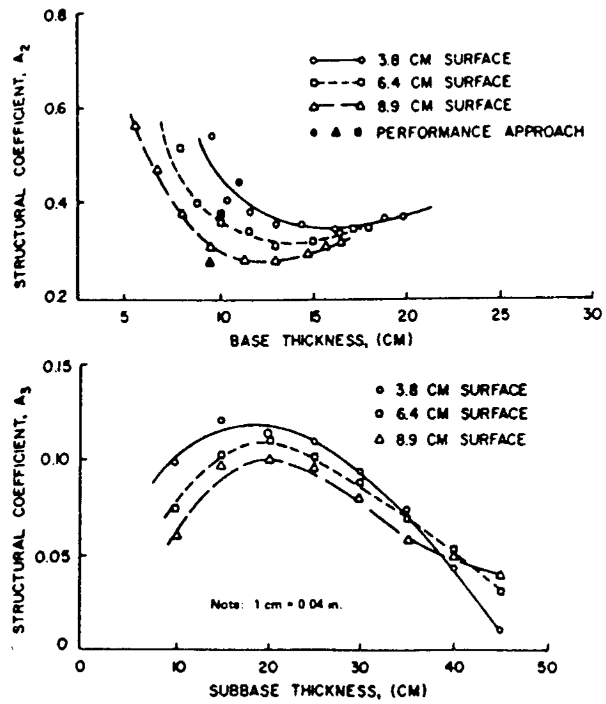
The conclusion of the AASHO Road Test in 1962 not only marked the end of the single most important pavement study, it also marked the beginning of one of history's longer duration pavement test projects. This project was at the site of the AASHO Road Test and started when the pavements were rehabilitated and incorporated into Interstate 80 by the Illinois Department of Transportation. During the process of



Base Course Coefficient-Base Course Thickness Relations
for Bituminous Concrete and Aggregate Cement

Figure 3.13. Structural coefficients of base-course materials determined by performance approach.[3.25]

Structural coefficients of bituminous concrete base and limestone subbase.



Structural coefficients of aggregate cement base and limestone subbase.

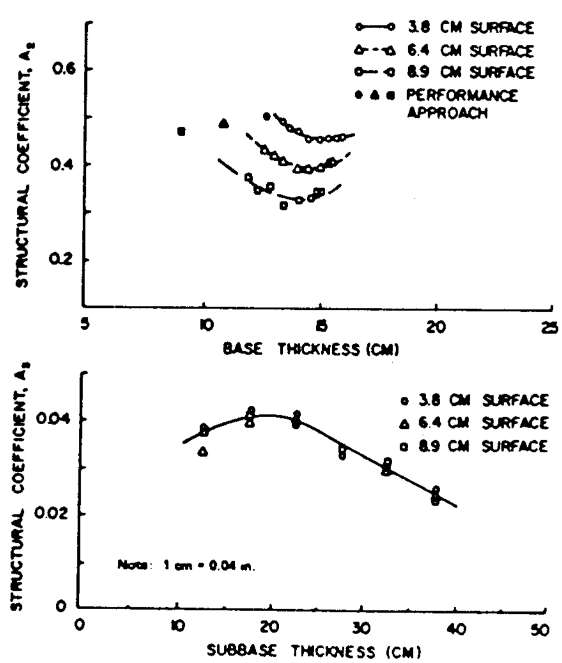


Figure 3.14 Structural coefficients for base and subbase courses for different materials. [3.25]

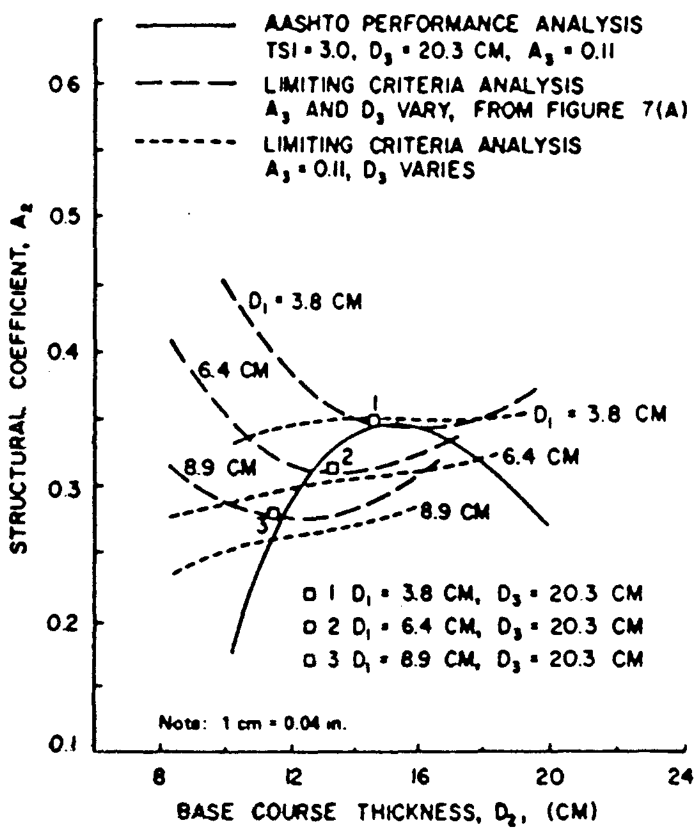


Figure 3.15. Comparision of Structural coefficients for bituminous concrete. [3.25]

rehabilitation, new pavements were also constructed. These pavements were duplicates of the original pavements (same thickness, materials etc,).

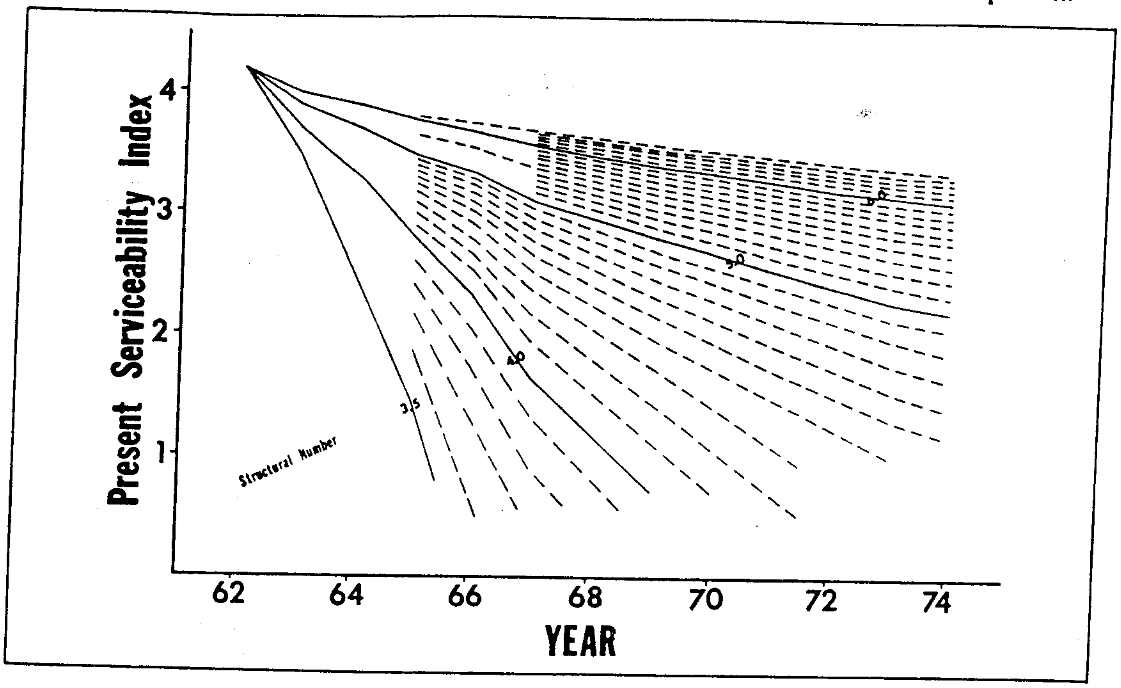
The behavior of all the pavements was monitored under the normal traffic of Interstate 80 until 1974 (for 12 years) when these pavements were resurfaced. During the monitoring process, complete performance data similar to that of the original AASHO Road Test were collected. These included:

1. Traffic volumes,
2. Vehicles Weights,
3. Amount of cracking,
4. Road smoothness,
5. Depth of rutting,
6. Areas of patching.

The collected performance data from the flexible pavements were analyzed by Elliott [3.26]. The analysis of the performance data from the asphalt stabilized base sections indicated that serviceability (PSI) of the rehabilitated sections did not significantly change after 12 years of service. In fact the performance of the deep strength or full depth asphalt pavements was much better than that predicted by the AASHO serviceability-performance equation.

Figure 3.16 has a comparison of the actual and predicted performances of asphalt stabilized base sections. From analysis of the actual performance structural layer coefficients were calculated for the deep strength asphalt

Asphalt pavement performance as predicted from the AASHO Road Test equation.



Performance of asphalt stabilized base sections.

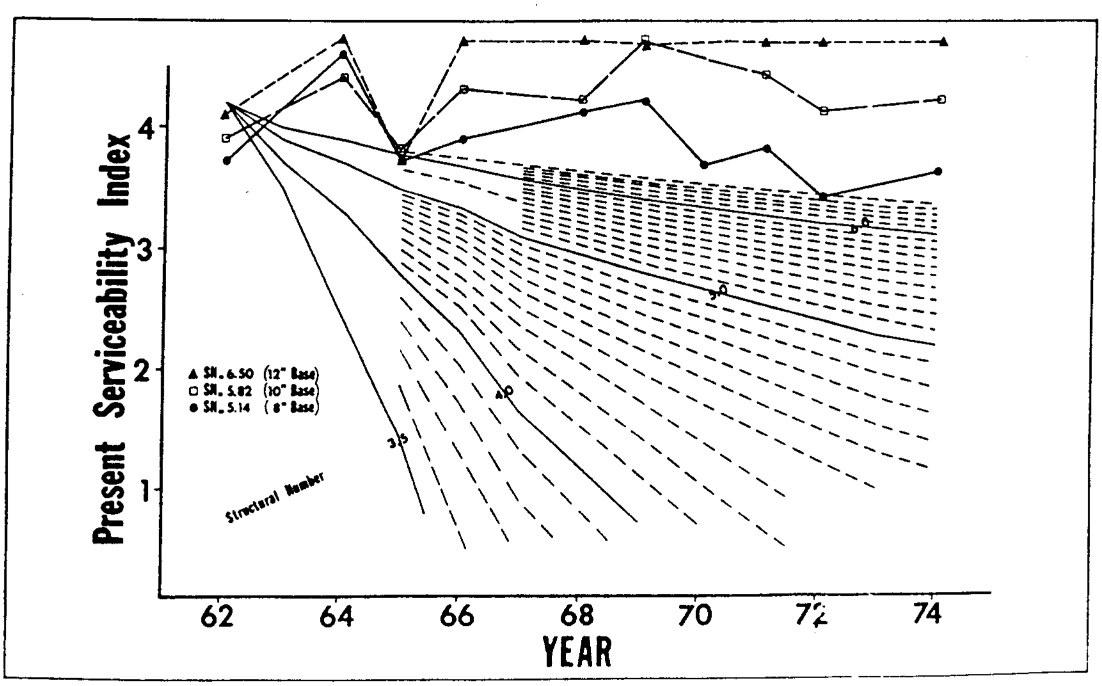


Figure 3.16. Comparision of the actual to the predicted performances of asphalt stabilized base sections. [3.26]

pavements. These coefficients are as follows:

Asphalt concrete surface course	0.57
Bituminous stabilized base course	0.44

These layer coefficients were determined for pavements with a total asphalt thickness (surface plus base) of at least 12.5 inches. The study indicated that the layer coefficients for total asphalt thicknesses of 8 inches or less would be in the range originally set by AASHO (0.44 and 0.35).

To implement these results, a transition between 8 and 12.5 inches was suggested. The coefficients then recommended in the AASHTO Guide would be used for asphalt thicknesses of 8 inches and less. The higher coefficients would be used for thicknesses of 12.5 inches and greater. Between these thicknesses, a straight line transition in coefficient values would be used as shown in Figure 3.17.

3.6 Critiques of the Structural Number/Layer Equivalency Concepts

Since the completion of the AASHO Road Test researchers have attempted to establish structural layer coefficients and thickness equivalencies relevant to specific designs in different geographic areas and for conditions different than those which existed at the AASHO Road Test site. Some of these studies were summarized above. Most of the approaches (e.g. empirical, mechanistic, etc) adopted to establish these coefficients or equivalencies received criticism for being

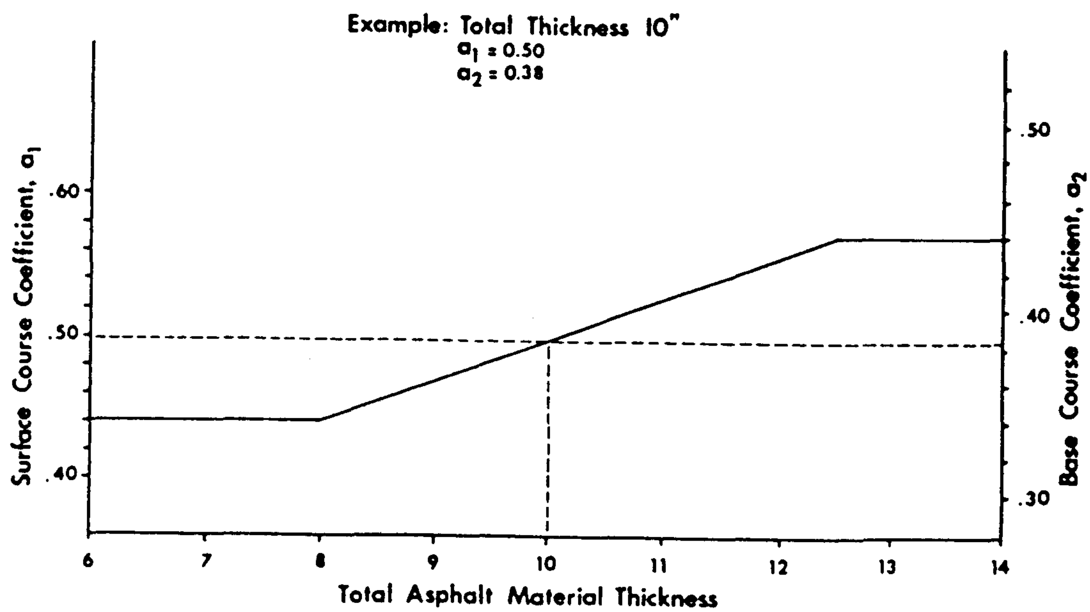


Figure 3.17 Asphalt Concrete Coefficient-Thickness Relationships.[3.26]

incompatible with the basic concept of layer coefficients and structural number. The following is a brief summary of some of the criticisms.

3.6.1 Criticisms of the Structural Number

Darter and Devos (1977) [3.27] stated:

> "These results indicate that there is an approximate correlation between the structural coefficient of the base and its resilient modulus. The correlation was found to also depend upon the thickness of the base course... It should not be concluded, however, that this correlation is absolute in any sense and probably varies with climate, subgrade support, and other factors. The results should be further verified using fatigue, subgrade strain, and other factors... The proposed approach is tentative and should be subjected to further verification in the field using actual project conditions. The approach has several limitations. The most significant is in assuming that the "structural coefficient" of a given material is only dependent upon its resilient modulus and base thickness, and does not depend upon other pavement factors such as surfacing type and thickness, subbase type and thickness, subgrade type and support characteristics."

Gomez and Thompson [3.28] presented an evaluation of the concept of layer coefficient and thickness equivalency ratios. They reported:

> " It has been demonstrated in several studies that the structural layer coefficients vary with respect to the following factors:
>
> 1. Layer thickness
> 2. Material type
> 3. Material quality
> 4. Layer location (base, subbase)
> 5. Traffic level
> 6. Limiting criterion (stress, strain, deflection,etc.)
>
> "It is apparent that "layer coefficients" are not constants. It would be very difficult to develop a "sliding scale" for layer coefficients which would appropriately consider the many important influencing

factors."

Coree and White [3.29] did an evaluation of the patterns of performance data contained in the AASHO Road Test flexible pavement raw data. They also examined the mathematical formulation of the performance and design equations and did a probabilistic analysis of the Road Test results, treating the layer coefficient as a distributed random variable instead of a uniquely determined number. They concluded that:

> "Within the AASHO model, the layer coefficients are shown to be secondary regression coefficients with no direct physical significance. To attribute to them a significance as indicators of strength is spurious. Instead, the layer coefficients are indicators to resistance to serviceability loss."

Ioannides [3.30] also pointed out some flaws in the structural number concept. He stated:

> "Deriving from its statistical/empirical nature is the fact that the structural number concept ignores the effect of the interactions between the various layers of the pavement system. Instead, it considers that a given layer behaves (or contributes to the structural capacity of the system) in exactly the same manner, independent of the pavement layer sequence it finds itself in... The major weakness of the structural number concept is that emphasis is placed exclusively on pavement materials, rather than on the behavior of the pavement as a system of interacting components. This limitation is also inherent in the conventional classification of all pavements as 'flexible' or 'rigid', primarily on account of the material of the surface layer. Further, the structural number concept ignores the influence on pavement system behavior of two very important factors, namely subgrade support and geometry of the applied load. In real in situ pavement systems exhibiting nonlinear or stress-dependent behavior, the concept also ignores the effect of load level. Thus, this statistical/empirical concept may be expected to serve its intended purpose as a design tool adequately only as long as these factors are similar to those prevailing at the AASHO Road Test, which provided the original data

> from which the structural number concept was developed...Whenever possible, the fundamental cause-and-effect relations innvolved in the phenomena observed empirically should be interpreted in the light of mathematical formulations of basic laws of engineering mechanics, rather than heuristic rules of thumb that are valid only in a statistical sense. Efforts aimed at replacing statistical/empirical constructs (e.g. SN, ESAL, and Miner's fatigue concepts) by more mechanistic procedures should therefore be intensified, and attempts to define statistical/empirical parameters (e.g.,layer coefficients, PSI, and load equivalency factors) using mechanistic theoretical tools should be abandoned."

3.6.2 Criticisms on the Layer Equivalency Concept

Monismith, et al. (1968) [3.31] presented numerous layered elastic analyses which emphasized that not one single layer equivalency can be assigned to a specific material in the structural pavement section. For this reason, they made the following recommendations:

> " When using the equivalency value which can be assigned to a particular material, since the equivalency depends on such factors as the intensity of wheel load and contact pressure, thickness of other material layer considered, subgrade characteristic, and the characteristics of the other materials of pavement section. In addition, when establishing the equivalencies for asphalt-treated materials, cognizance must be taken of their response to loading, variable climatic conditions (e.g. temperature), and, in the case of aggregates treated with asphalt emulsions and liquid asphalts, the effects of curing."

The sensitivity of layer equivalencies for different materials to the failure criterion selected was pointed out by Coffman, et al. (1968) [3.32], as one of a host factors that would enter a theoretical determination of these numbers. The following is a brief quotation from their paper:

> "On this basis, equivalence can be defined in the

terms of this study by the following generalized equation:

$$\text{Equiv.} = f(L, V, T, A, H, E, C, N)$$

where:

L = the loadings,
V = their velocities,
T = the times of their applications,
A = the contact areas,
H = the layer thickness,
E = the material,
C = the climate or environment, and
N = the number of applications to failure,

"This is an interesting equation. With the inclusion of the failure term to the concept of average equivalence, as anticipated in this study, it is clear that exactly those considerations that would be expected in a rational design formula are collected in the equation. With the exception of N, it is also clear that the tools and techniques necessary to such an approach are available and indeed have been used in this study, to some approximation. This points to the pressing need for research that will fill the blanks represented in the failure term. When these blanks are completely filled the need for equivalences will presumably have vanished."

In developing layer thickness equivalencies for various materials, Vaswani (1968; 1969) [3.3,3.4] employed Benkelman Beam or Dynaflect deflections rather than layered elastic theory, but still recognized that the same "independent variables" enter the evaluation of these "constants", as the determination of the measured deflections themselves. These include, "the thickness of the overlying layer; the thickness equivalency of the overlying layer; the ratio of the strength of the overlying or underlying layer; and the strength of the layer itself... including the soil resiliency and the environmental conditions effecting it; and traffic" [3.3].

Nicholas [3.33] criticized Vaswani's approach. He pointed out the fallacy of a constant "thickness equivalency" concept. Nicholas argued that: "Vaswani's design procedure is based on 'thickness equivalencies' that are determined once and for all from deflections measured by the Dynaflect, a machine that measures a pavement's response to relatively light, pulsating loads... [Vaswani] shows unique equivalency values for broad classes of material regardless of the material's thickness, quality, or position in the structure..." Interestingly, Vaswani [3.3] in his closure agreed with practically all points raised by Nicholas [3.33].

In a following paper, Vaswani [3.5] sought to obtain "optimum thickness equivalency values" (as opposed to unique constants), by considering the "location of the materials in the structure," and evaluating qualitatively " the effect of thickness and modulus of strength of a given layer with respect to the thickness and modulus of strength of the layers in the pavement system." Thus, thickness equivalencies for untreated base and for certain other materials were found to be lower for heavy-duty roads (primary and Interstate) than for light duty roads. Vaswani [3.5] explained this by suggesting that "the thickness equivalency value of the material decreases as the thickness of the cover increases." Even a broad and qualitative conclusion such as this, however was rebutted in the discussion by Foster (1970), who presented his own results indicating "an increase in thickness equivalency with a increase in depth of cover."

Ioannides made the following comments in relation to the above mentioned findings of the two researchers [3.34]. He stated:

> "This should not be interpreted as a weakness in the approach of either of these two investigators. Neither one needs to be "wrong". It is the equivalency concept that is at the root of the problem. Reasonable equivalencies cannot be defined even if the effect of cover is accounted for, simply because a host of other factors still remain unaccounted. It can be postulated that the disagreement between Vaswani and Foster hinges on differences in subgrade support and geometry of applied loads pertaining to the cases they considered, and in their interpretation approaches to the AASHO Road Test results. A vigorous and fundamental concept would not lend itself to such discrepancies."

The above mentioned criticism clearly brings to light the flaws and limitations of the structural number and layer equivalency concepts responsible in inadequately defining the structural capacity of a pavement structure. But in the absence of any other parameter that can more closely represent the structural capacity of various layers in a pavement system these concepts seem to be the only acceptable choices in the frame work of AASHTO's method of pavement design.

Chapter 4

LABORATORY TESTING

4.1 Base Core Sampling Scheme

AHTD provided 4-inch diameter core samples of ACHM base courses from three different highway projects. Each core was cut transversely using a diamond coated saw blade into two or three specimens, each 2.5 inches thick. Thus two or three Marshall size specimen (4"× 2.5") samples were recovered from each core. Table 4.1 gives a description of the cores provided and the Marshall size specimens recovered.

To estimate the structural layer coefficient of the ACHM base course in accordance with the recommendations of the "AASHTO Guide for the Design of Pavement Structures (1986)", each specimen was tested for Resilient Modulus (ASTM 4123) at 68°F . Split tensile strength tests were also conducted after the resilient modulus tests. The results of split tensile strength tests were used to investigate the relationship between maximum split tensile strength and resilient modulus properties, and to classify the fatigue characteristics of the base course according to the AAMAS relationship [2.12].

4.2 Diametral Resilient Modulus test

Resilient Modulus (M_R) is defined as the ratio of the repeated stress to the corresponding resilient (recoverable)

Table 4.1 Summary of Core sampling plan

Highway or Job site #	No of ACHMBC Cores obtained from AHTD	Average Height of Cores (inches)	No of Marshall size specimen recovered
Job# 10940	9	9	25
Highway 67	8	9	16
Highway 10	8	6.5	21

strain. In this study, the resilient modulus was measured using the diametral test in which the specimen is placed on its side and load is applied on the sides across the diameter. Deformation is measured along the diameter perpendicular to the loaded diameter. This method of test is based on the theory of elasticity and, as such, assumes that the specimen behaves as an elastic solid. Although it is recognized that an asphaltic material is not elastic, the diametral test is generally used and considered acceptable because at short durations of loading the asphalt response is essentially elastic.

The Retsina Mark V device used in this project applies a light pulsating load across the vertical diameter of the Marshall size (4"× 2.5") specimen. This pulsating load causes a corresponding deformation across the horizontal diameter of the specimen. This deformation was measured using two variable differential transducers (LVDT), that lie on the opposite sides of the horizontal diameter of the specimen. A dynamic load of 75 ± 10 lbs was applied for 0.1 second duration with a 3 second rest period between the loads. This magnitude of load was chosen according to the recommended range (i.e. 10 to 50 % of indirect tensile strength) of ASTM 4123-82. The magnitude of the load was controlled by adjusting the regulator for the compressed air. The dynamic vertical load and its corresponding horizontal deformation were recorded from a digital readout device.

The samples were tested at a temperature of 68 ± 2°F as recommended by the AASHTO guide (1986) for the estimation of structural layer coefficients. For temperature control the Retsina device was fitted inside a wooden box and was placed in a temperature controlled room with the temperature lowered below 68°F. The heating inside the box was provided by a heater controlled by a thermostat. A thermocouple attached to a Marshall size (4"× 2.5") of asphalt concrete mix specimen was used to check the temperature of the samples being tested.

As discussed above, this test and the equation used to calculate resilient modulus assume linear elastic behavior. In view of the test temperature and with the brief load pulse used, this assumption is considered to be reasonable. Figure 4.1 shows the stress distribution in the specimen when load is applied, assuming elastic behavior and a plane stress condition. The resilient modulus was calculated for each sample by using the following equation.

$$M_R = \frac{P[\nu+0.27]}{tD} \tag{4.1}$$

where,

M_R = resilient modulus (psi),

P = peak load (lbs),

ν = poission ratio (0.35 assumed),

t = specimen thickness,

D = horizontal deformation across the specimen,

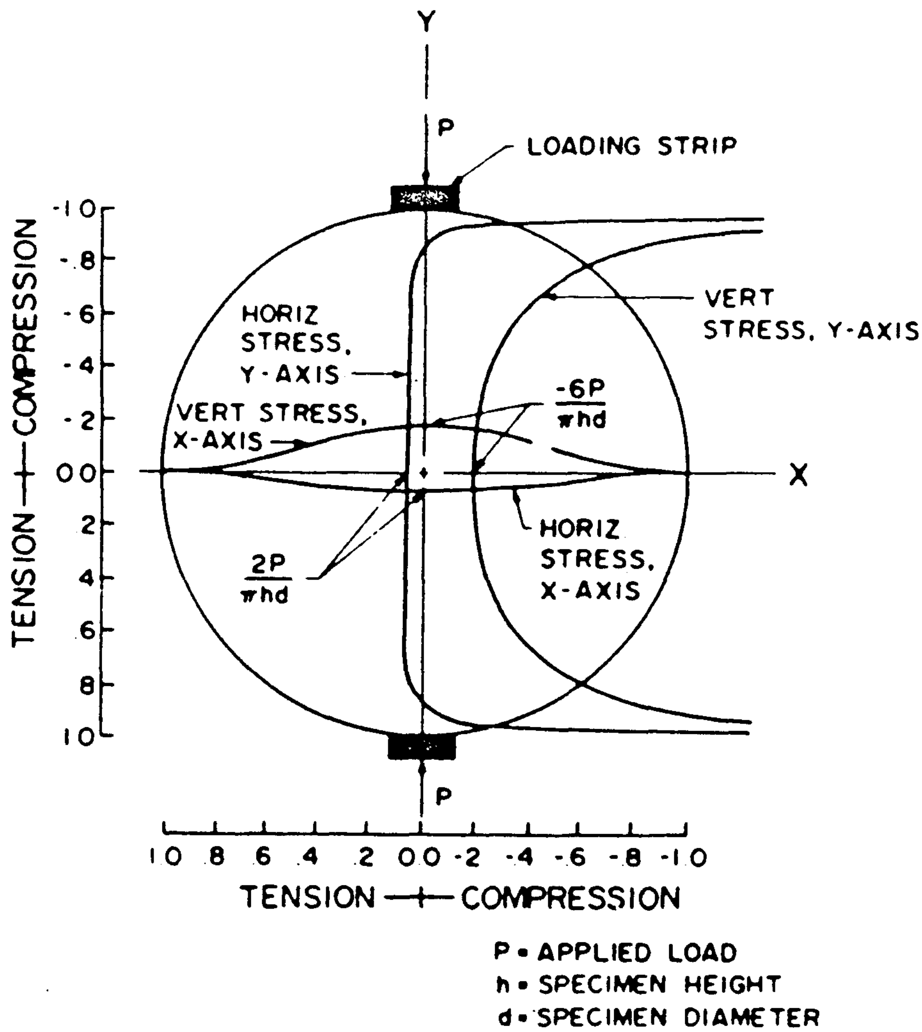


Figure 4.1 Stress distribution along the principle axes of specimen during diametral resilient modulus test.

Six readings were taken on each sample with the resilient modulus device. Three readings were taken across the same points. Then the sample was rotated 90 degrees and three more readings were taken across its diameter. Tables 1 through 3 given in the appendix present a summary of the resilient modulus test results conducted on three ACHMBC mixes from Job# 10940, Highway 10 and Highway 67 respectively.

4.3 Split Tensile Testing

4.3.1 Split Tensile/Fatigue Relationships

Tensile strength is the maximum tensile stress a specimen can withstand before failure. It is necessary to perform destructive testing to obtain this property. Some researchers have suggested that tensile strength be used as a design requirement for thermal or shrinkage cracking in the pavement, as a criterion for quality control and for a rough estimate of fatigue life [4.1]. Marias [4.2] indicated that there is a strong correlation between the indirect tensile strength and the service life of laboratory mixes investigated. Since repeated load laboratory tests are time consuming and expensive, some researchers have attempted to correlate the split tensile strength with the fatigue life of the sample for different stress levels.

Maupin and Freeman [4.3] showed that split or indirect tensile strength of an asphalt mix specimen can be used as a reasonable predictor of either the constant stress or the constant strain fatigue properties of a given mix. The general form of the equation for the fatigue life of an asphalt concrete in the constant stress mode is:

$$\log N = \log k + n \times \log\left(\frac{1}{e}\right) \qquad (4.2)$$

where,

N = fatigue life in terms of load repetitions,

k and n = fatigue constants determined by testing,

e = strain induced in the specimen by each load,

Maupin and Freeman developed the following relationship for predicting the fatigue constants:

$$n = 0.0374 \times ST - 0.744 \qquad (4.3)$$

$$\log k = 7.92 - 0.122 \times ST \qquad (4.4)$$

where,

ST = the Split tensile strength in psi,

With these relationships, the fatigue life prediction equation based on split tensile strength becomes:

$$\log N = 7.92 - 0.122 \times ST + (0.0374 \times ST - 0.744)\log\left(\frac{1}{e}\right) \qquad (4.5)$$

Maupin [4.4] also investigated the use of indirect tensile stiffness in predicting fatigue life of specimen tested under constant strain, mode. The stiffness was evaluated for the linear portion of the stress-strain curve (normally the relationship is linear up to the 3/4th of the tensile failure stress) as follows:

$$S_{3/4} = \frac{3/4\sigma_{tf}}{\epsilon_{3/4}} \qquad (4.6)$$

where;

$S_{3/4}$ = Stiffness at 3/4th tensile stress

σ_{tf} = tensile stress at failure,

$\epsilon_{3/4}$ = tensile strain at 3/4th failure stress,

Maupin's results showed that stiffer mixes had shorter fatigue lives.

Elliott and Herrin [4.5] used the relationships established by Maupin to develop the following relative life equation based on the split tensile strength:

$$\log\left(\frac{N_a}{N_b}\right) = SF\,[ST_a - ST_b] \qquad (4.7)$$

where;

N_a/N_b = the relative life ratio of two mix variations,

ST_a and ST_b = the split tensile strength of two mix variations,

SF = a strain factor determined to be 0.0163 for typical asphalt pavements.

The stress-strain curve obtained during the split tensile strength test can also be used to calculate toughness of asphalt concrete mixtures. Toughness is the area under the curve up to the point of failure and is defined as the amount of work per unit volume required to cause failure. Toughness is often used as a relative indicator of the resistance of an asphalt concrete mixture to fracture, either fatigue or temperature related [4.6]. High toughness values indicate greater resistance to fracture and vice versa. Materials with high toughness values have high potential to absorb energy without fracture. Little and Richey [4.6] showed that maximum toughness occurs at the same asphalt content regardless of the loading rate or temperature and that the peaks are more well defined at a temperature of 77^0F and with a loading rate of 2"/min.

4.3.2 Split Tensile Strength testing equipment

A modified Marshall Stability test device was used for conducting split tensile strength tests. Split tensile loading caps were used in place of the Marshall Stability breaking head. These caps have loading strips, with curved surface to hold Marshall size (4"× 2.5") specimen between them. These strips allow diametric loading to the specimen sides. The test apparatus is shown in Figure 4.2. Loading was applied to the test specimen at a deformation rate of 2"/minute. The magnitude of loading was monitored and recorded on a strip chart. The strip chart calibration was checked and adjusted at the start of each test period.

Temperature was controlled using the same set up as used for the Resilient modulus testing. In these tests, however, the test was set at 77°F ± 2. Figure 4.3 shows a typical stress-strain curve obtained from the test.

4.3.3 Test Procedure

The split tensile test provides an indirect measure of the tensile strength of a material. As shown in Figure 4.4, the test is conducted with the specimen's cylindrical axis in a horizontal position. The specimen is then subjected to a compressive loading which is applied to opposite sides of the cylindrical surface. Although the loading is compressive, the specimen fails due to tensile stresses generated perpendicular to the vertical plane through the specimen. The magnitude of the tensile stress is:

$$ST = \frac{2P}{(\pi \times L \times d)} \tag{4.8}$$

where;

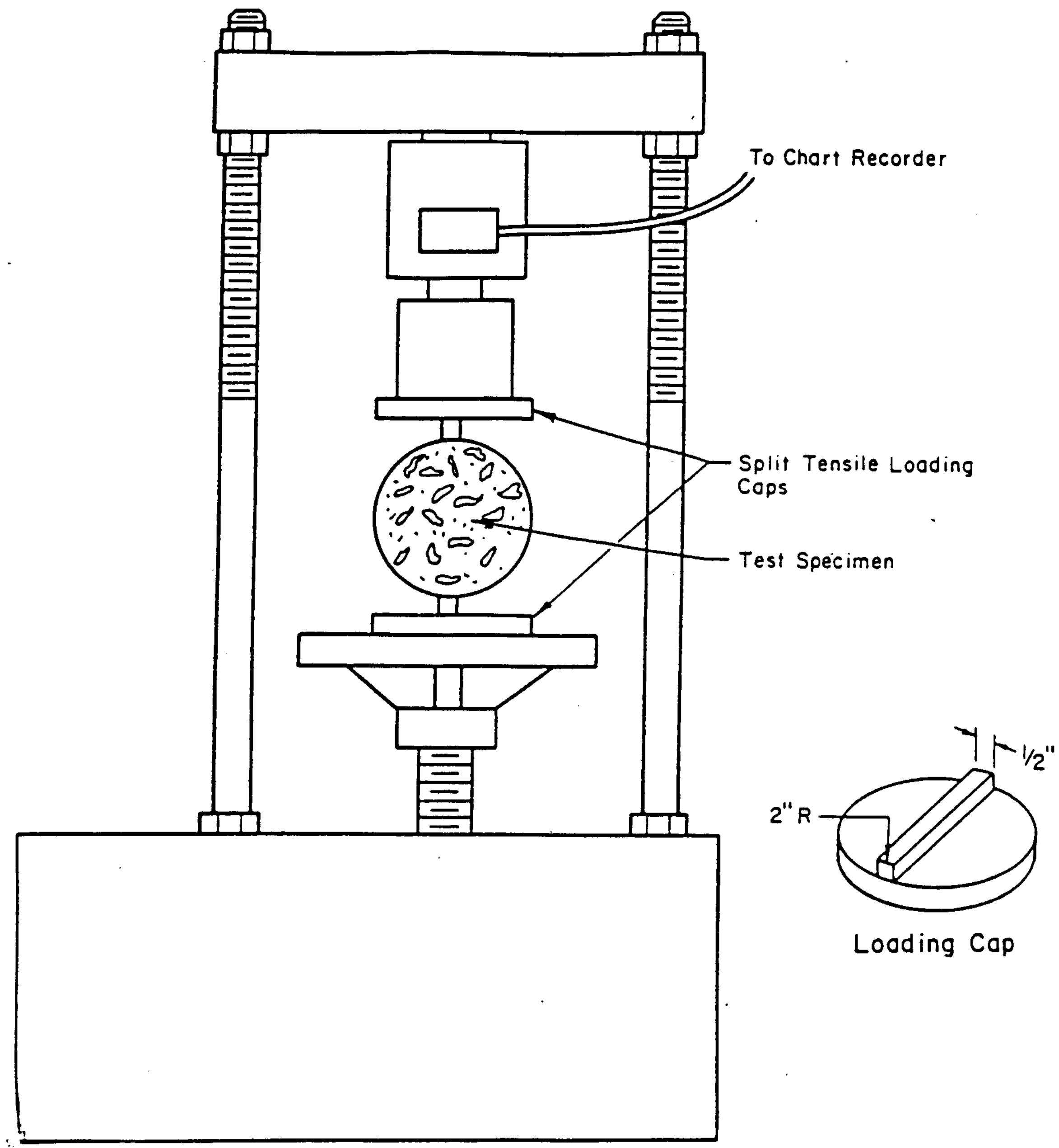


Figure 4.2 Modified Marshall test device with split tensile loading caps.

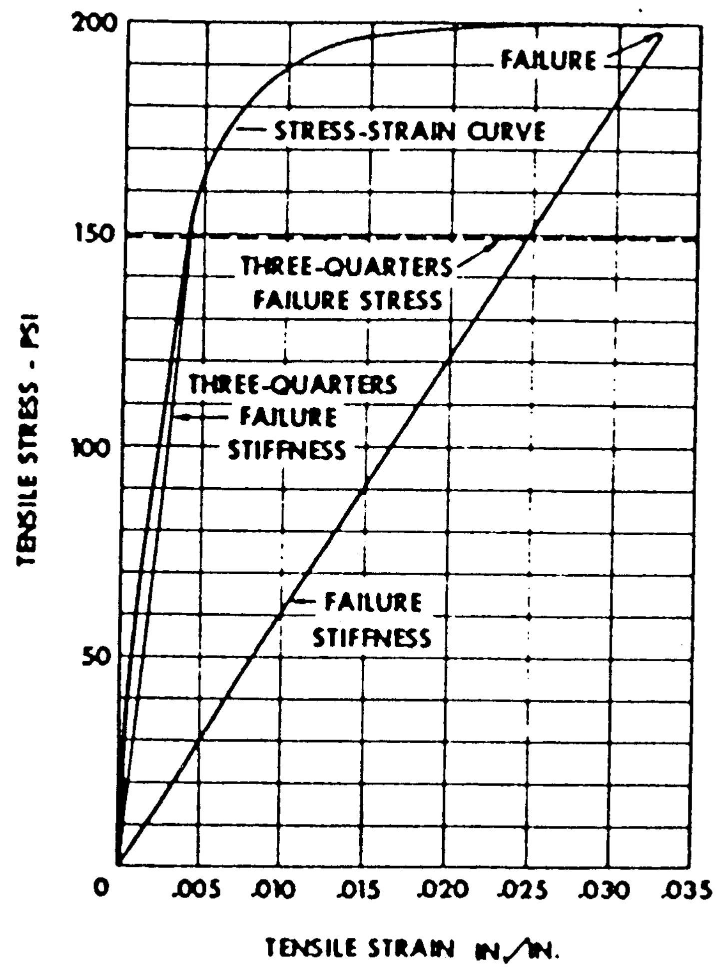


Figure 4.3 Typical Stress-Strain curve for Split tensile strength test.

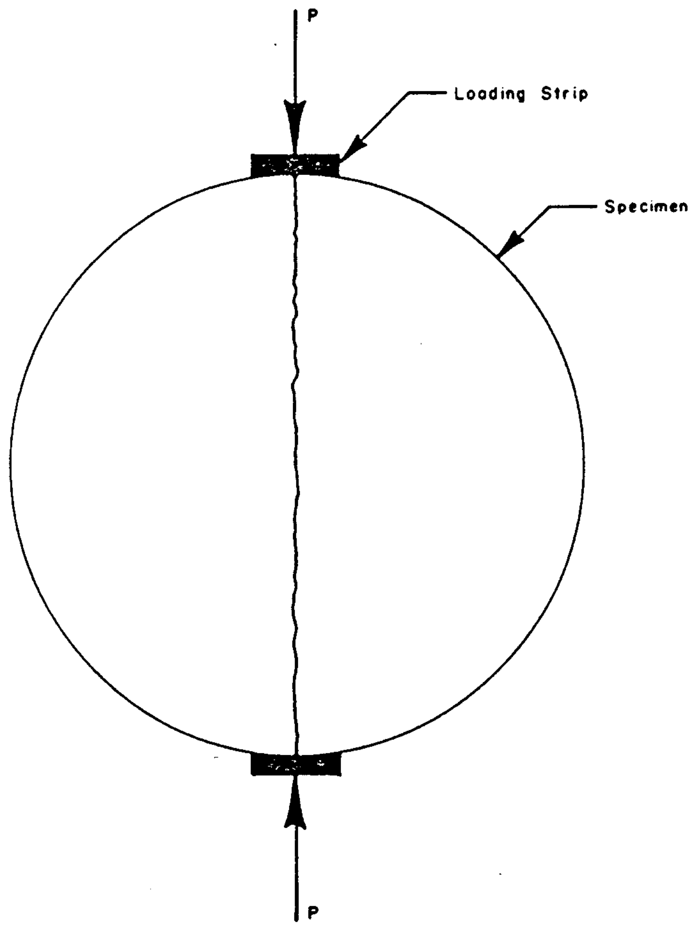


Figure 4.4 The Split Tensile Strength Test.

ST = the tensile stress (psi) perpendicular to the vertical plane of loading,

P = the magnitude of the load (lbs) applied to the specimen,

L = the specimen length, inches,

d = the specimen diameter, inches.

The split tensile strength of the specimen is the maximum tensile stress determined by the above formula using the peak magnitude of loading as recorded on the strip chart. Tables 4 through 6 given in the appendix show the split tensile strengths of the ACHM base course samples from three different sites (Job#10940, Highway 67, Highway 10).

Tensile Stiffness was calculated using Equation 4.6. Tables 7 through 9 given in the appendix show the tensile stiffnesses for the tested mixes.

Toughness was also determined for the three mixes. The area (Work/unit volume) under the load-deformation curve was determined using a planimeter. Results of toughness for the different mixes are presented in Tables 10 through 12 given in the appendix.

Using Equation 4.7, the relative fatigue lives of the three mixes were estimated. Table 4.2 shows a comparison of the estimated fatigue lives. Table 4.3 presents a summary of results obtained by resilient modulus and split tensile strength testing.

Table 4.2 Comparison of relative fatigue lives.

ACHMBC Mix from Highway Project	Tensile Strength (psi)	Predicted life (%)
Highway 10	142.12	100
Highway 67	105.25	25
Project# 10940	80.06	9.7

Table 4.3. Summary of Split Tensile Strength and Resilient Modulus Test Result.

Sample from Highway or Job #	Tensile Strength (psi)		Tensile Strain at Faillure (inch/inch)		Toughness (psi.in/in)		Tensile Stiffness at 3/4th of the maximum tensile stress (psi)		Resilient Modulus (ksi)	
	Mean	COV. %	Mean	COV. %	Mean	COV. %	Mean	COV. %	Mean	COV. %
Highway 10	142.12	15.31	0.0587	14.83	5.43	27.23	3481.75	17.98	654.50	14.98
Highway 67	105.25	11.31	0.0532	8.91	3.53	14.94	3020.53	14.94	441.75	15.43
Job# 10940	80.06	22.92	0.0480	13.65	2.46	26.03	2693.00	26.13	473.37	16.87

Chapter 5

ANALYSIS AND DISCUSSION OF TEST RESULTS

This project was started in July, 1991 as a six month study with limited testing of material samples from three construction projects. The original study plan only included resilient modulus testing and development of comparisons between the AHTD base material and the bituminous stabilized base used at the AASHO Road Test. The immediate objective of the study was satisfied in a letter preliminary report submitted November 7, 1991 that recommended a layer coefficient of 0.34.

At no cost to the project, the testing was extended to include all of the testing reported in Chapter 4. The extended work delayed completion of the study but provided additional basis for selecting an appropriate layer coefficient. A draft final report on all the testing and analyses was submitted in June 1994. This report indicated that a coefficient value greater than 0.34 might be justified but did not recommend a higher value since only three projects had been tested. As a result of this indication, AHTD elected to extend the study further by adding testing of samples from additional construction projects. The extended work plan called for the testing of samples from three additional projects but five were actually sampled and tested.

This chapter reports the results of the testing and analyses of samples from both the three original construction projects and the five additional projects. For some of the data, only results from the three original projects are reported. This is because the only tests performed on the samples from the additional projects were those believed to provide a direct indication of an appropriate coefficient value.

5.1 Resilient Modulus Tests

The AASHTO Guide contains recommended relationships between layer coefficients and resilient modulus. The relationship in the Guide for asphalt base material is shown in this report as Figure 2.4. The Guide also incorporates a resilient modulus/layer coefficient relationship in its flexible pavement overlay design procedures. Both of these were used with the resilient modulus test results to estimate layer coefficients for each project.

The mean resilient modulus and estimated layer coefficients from each project are listed in Table 5.1. Note that all of the resilient modulus values exceed the maximum value on Figure 2.4. To estimate coefficients based on this figure it was necessary to develop an extrapolation of the relationship. These extrapolated values must be viewed with great caution. In general those much in excess of about 0.4 are believed to be unrealistically high.

Table 5.1 Estimated Layer Coefficients Using AASHTO Guide Resilient Modulus Relationships

Site Location	Mean Resilient Modulus (ksi)	Layer Coefficient from Equation 5.1	Layer Coefficient Extrapolated from Figure 2.4
Highway 67	441.8	0.34	0.36
Job #10940	473.4	0.35	0.38
Highway 10	654.5	0.39	0.49
Job #20095	501.0	0.36	0.40
Job #7995	638.5	0.39	0.48
Job #060614	697.4	0.40	0.52
Job #060641	558.7	0.37	0.43
Job #60105	554.6	0.37	0.43

The other layer coefficient estimates are based on relationship developed in Appendix NN of the "AASHTO Guide for the Design of Pavement Structures, 1986". This relationship is also used in the 1993 AASHTO Guide for the determination of SNeff for overlay design. This relationship is:

$$a_2=0.0045(E_r)^{1/3} \qquad (5.1)$$

Note that these values are lower than those estimated from Figure 2.4.

5.2 Split Tensile Strength

The split tensile test data are summarized in Table 5.2. This table also contains an estimate of the fatigue lives of the mixes if tested at strain levels typical of those expected from an 18 kip single axle load on a relatively thin (3 to 4 inches of asphalt mix) flexible pavement [4.5]. The estimates are based on the relationship developed by Maupin and Freeman (Equation 4.5). Although these numbers do not directly provide an indication of acceptable layer coefficients, they do suggest that the mixes can be expected to perform well in fatigue which could justify the use of higher coefficients.

5.3 Resilient Modulus and Split Tensile Strength Properties

Resilient modulus and split tensile strength of the specimens were plotted to see if any correlation exists between the two properties (Figure 5.1 through 5.3). If a

Table 5.2 Mean Split Tensile Test Results and Estimated Fatigue Lives.

Mix from Highway Project	Split tensile Strength (psi)	Tensile stiffness at 3/4th of maximum tensile stress (psi)	Tensile strain at failure (in/in)	Estimated load applications to failure by Eq 4.5 at micro-strain of 200
Job #10940	80.1	2693	0.0480	3.00E+06
Highway 67	105.3	3020	0.0532	7.74E+06
Highway 10	142.1	3481	0.0587	3.09E+07
Job #20095	88.1	2448.7	0.0389	4.05E+06
Job #7995	92.0	2510.4	0.0385	4.69E+06
Job #060614	100.4	2753.3	0.0382	6.44E+06
Job #060641	122.5	2730.6	0.0453	1.48E+07
Job #60105	103.9	2619.3	0.0417	7.34E+06

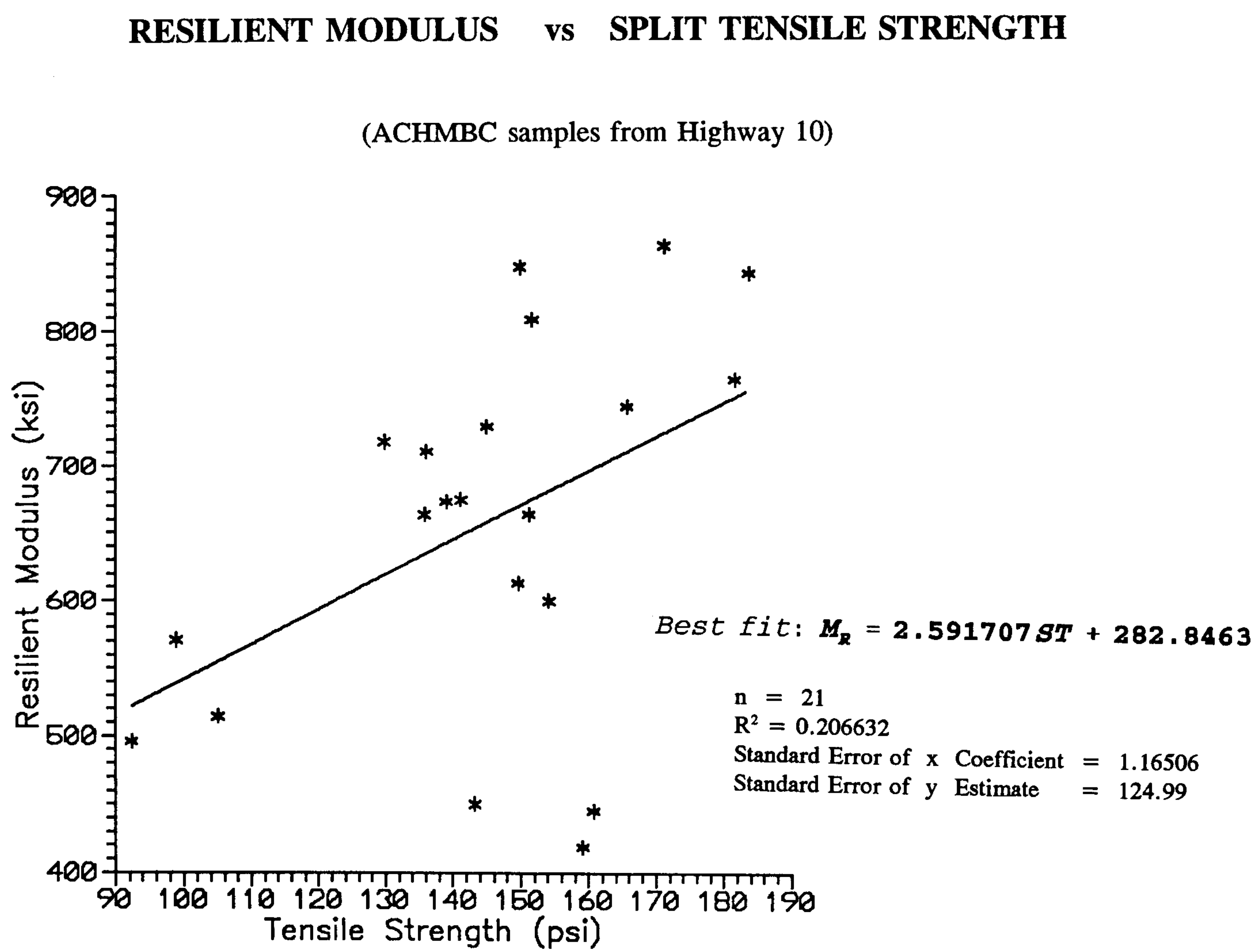
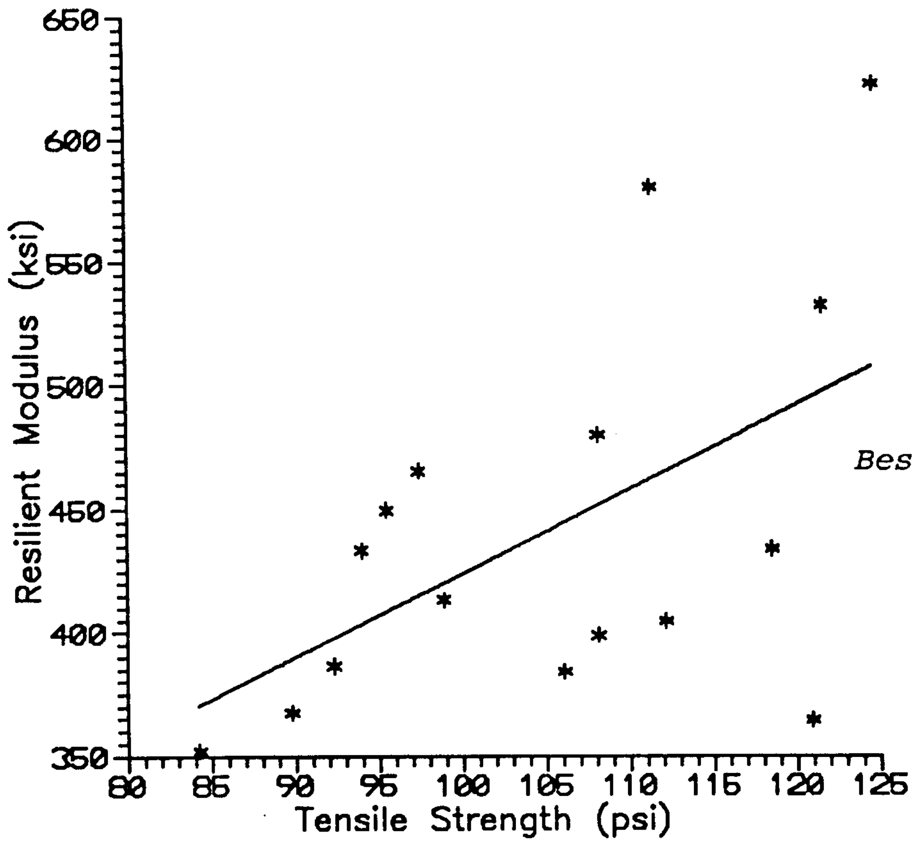


Figure 5.1 Correlation between Split tensile strength and Resilient modulus values for ACHMBC samples from Highway 10.

RESILIENT MODULUS vs SPLIT TENSILE STRENGTH

(ACHMBC samples from Highway 67)



Best fit: $M_R = 3.394546\,ST + 84.46842$

n = 16
R^2 = 0.294367
Standard Error of x Coefficient = 1.40463
Standard Error of y Estimate = 68.07

Figure 5.2 Correlation between Split tensile strength and Resilient modulus values for ACHMBC samples from Highway 67.

RESILIENT MODULUS vs SPLIT TENSILE STRENGTH

(ACHMBC samples from Job # 10940)

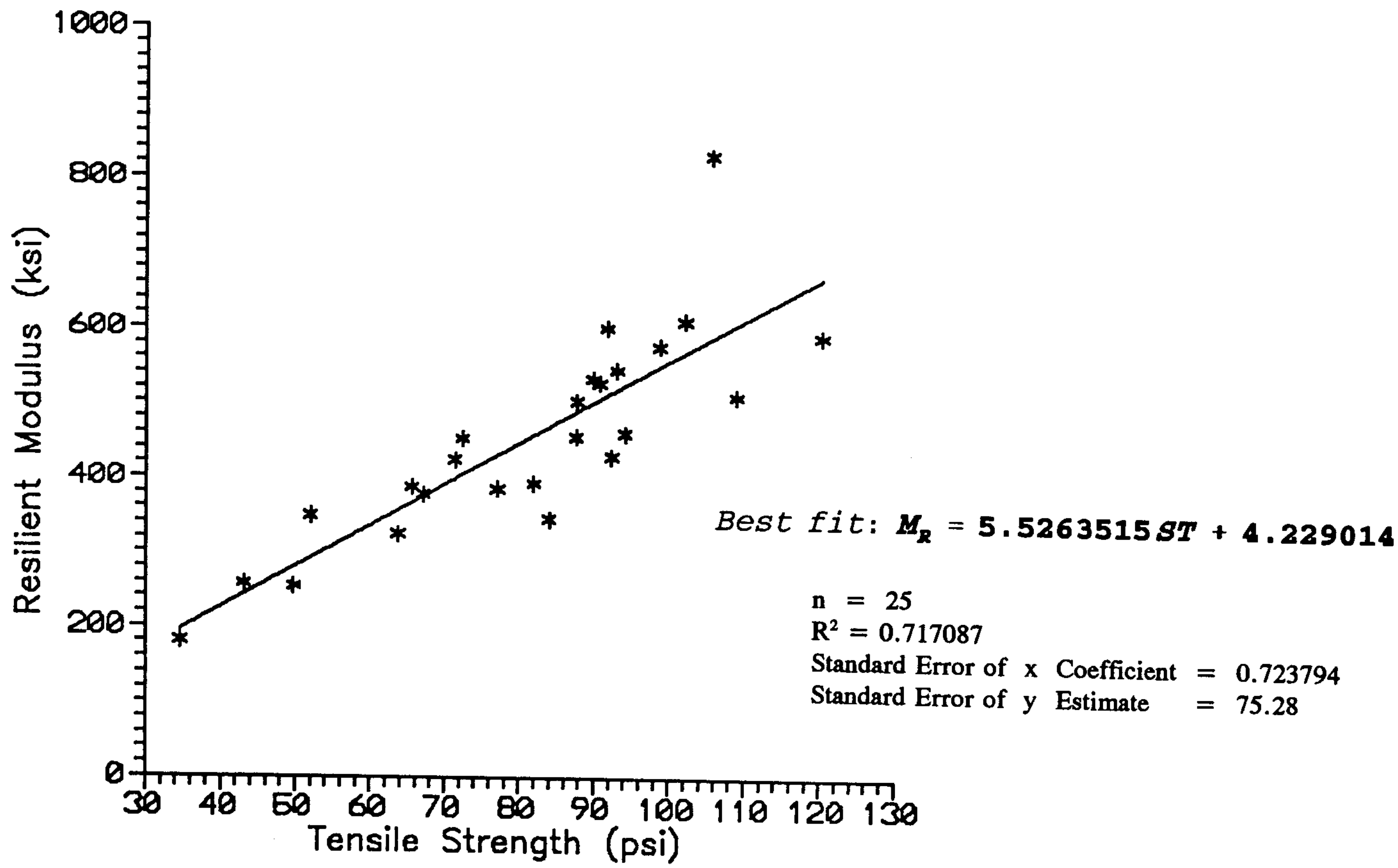


Figure 5.3 Correlation between Split tensile strength and Resilient modulus values for ACHMBC samples from Job # 10940.

strong correlation were found this might provide a means to estimate resilient modulus from a simpler test. Although there appears to be some correlation, the relationship does not appear to be sufficiently strong to be used. The data from all three mixes are plotted on Figure 5.4.

5.4 Comparison of AASHO and AHTD Bituminous Stabilized Bases

The samples provided by AHTD from the highway projects Job# 10940 and Highway 10 were compared with the AASHO bituminous stabilized base course on basis of estimated and measured resilient modulus, Marshall properties, aggregate gradation, and crushed particle content. The comparison is summarized in Table 5.3.

5.4.1 Gradation

Figure 5.5 presents a comparison of the AASHO base gradation with the maximum density line (gradation reference line or Fuller's curve). In general for a mix to achieve maximum density its gradation should follow the maximum density line as closely as possible. The AASHO gradation falls significantly above the maximum density line. Largest deviations are in the amount of material passing the 1/2 inch, No.4 and No.40 sieves. This gradation would be characterized by reduced contact area between the coarser aggregates resulting in limited aggregate interlock and frictional resistance.

RESILIENT MODULUS vs SPLIT TENSILE STRENGTH

(ACHMBC samples from Highway 67, Highway 10 and Job # 10940)

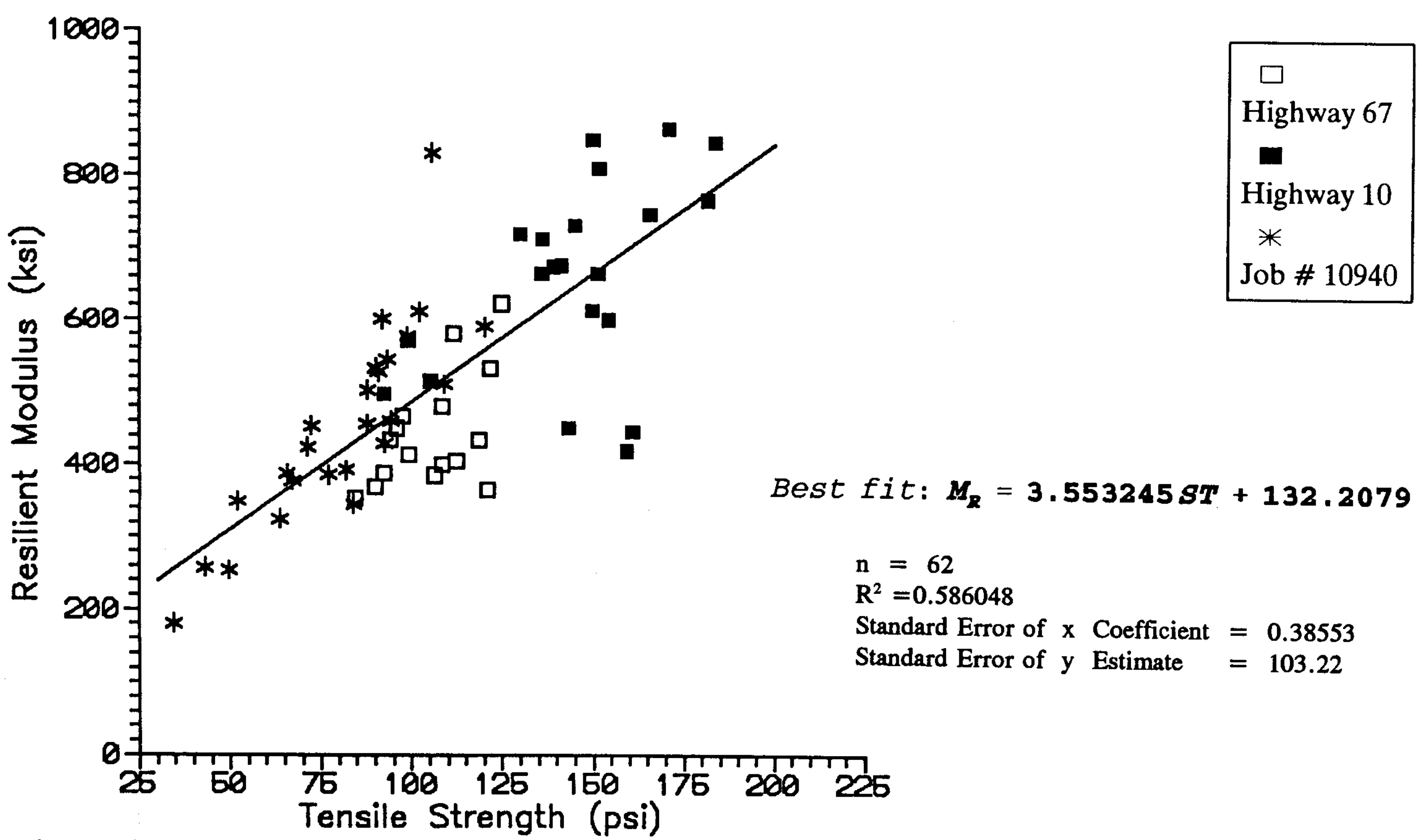


Figure 5.4 Correlation between Split tensile strength and Resilient modulus values for ACHMBC samples from Highway 10, Highway 67 and Job # 10940 plotted together.

Table 5.3. Comparison of ACHM base course material properties with the AASHO Road Test bituminous stabilized base course

Material Property	AASHO Base Course	AHTD Mix Specifications	Mix Design (Highway 10)	Mix Design (Job# 10940)	Mix Design (Highway 67)
Base Course material	Uncrushed, natural sand gravel	Mixture of gravel / crushed stone	same as specified	same as specified	Data not available
Asphalt Content (%)	5.2 (85-100 pen. grade asphalt)	3 to 5 ±0.4	4.6 grade A.C-30	4.1 grade A.C -20	Data not available
Marshall Stability (lbs)	1600	1000 (minimum)	2204	2475	Data not available
Marshall Flow (1/100)	10	---	8.69	8.8	Data not available
% Air Voids	6.2	3 to 8	5.37	4.5	Data not available
Minimum % of Crushed particles	none	At least 15 % retained on #4	same as specified	same as specified	Data not available
Resilient Modulus (ksi)	360-470* estimated	560-650* estimated	655 measured	473 measured	442 measured

*Resilient modulus estimated from Asphalt Institute equation.

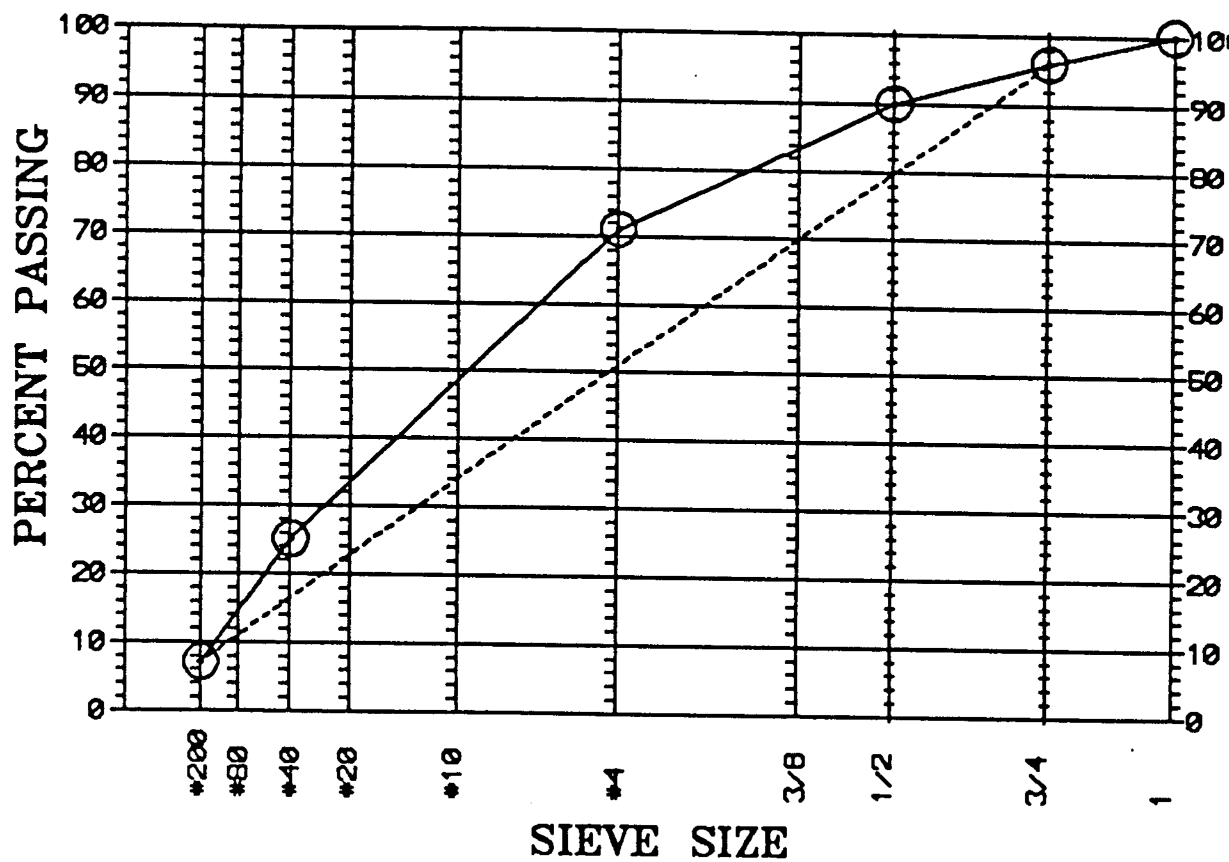


Figure 5.5 Comparison of the AASHO bituminous stabilized base course gradation with the maximum density line.

Figure 5.6 shows the comparison of the job mix for ACHM base course from Job# 10940 with the maximum density line. The mix gradation follows the maximum density line very closely and in the coarser fraction of gradation almost falls on the maximum density line. The largest deviation of the mix gradation from the maximum density line is about 3% on the sieve No.40. Recommended limits of tolerances by AHTD are also shown in the figure. This particular gradation is believed to follow the maximum density line more closely than desired but still should be a better gradation than that used for the AASHO base.

Figure 5.7 shows that the job mix for ACHM base course from Highway 10 follows the maximum density line more closely in the coarser range fraction than in the finer range. The largest deviation is of 8% on sieve No. 10. Recommended limits of tolerances by AHTD are also shown in the figure. This gradation also appears to be superior to that used for the AASHO base.

5.4.2 Marshall Properties

The Marshall properties were available for only two of the three ACHM base courses tested. Both of these were found to be superior to the Marshall properties of the AASHO base. In fact, the Marshall properties of the Job # 10940 and Highway 10 ACHM bases are superior to the Marshall properties of the AASHO binder and surface mixes.

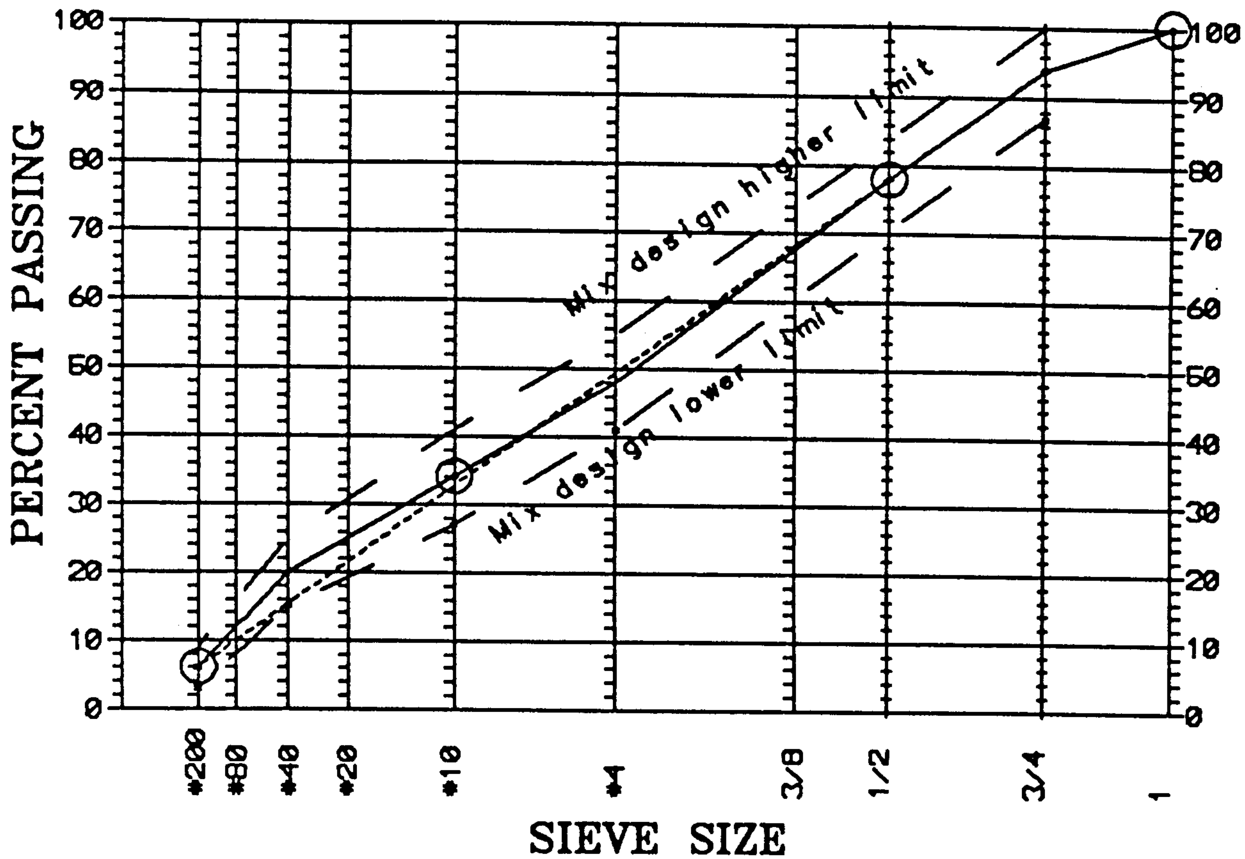


Figure 5.6 Comparison of AHTD's ACHM base course mix gradation from Job#10940 with the maximum density line.

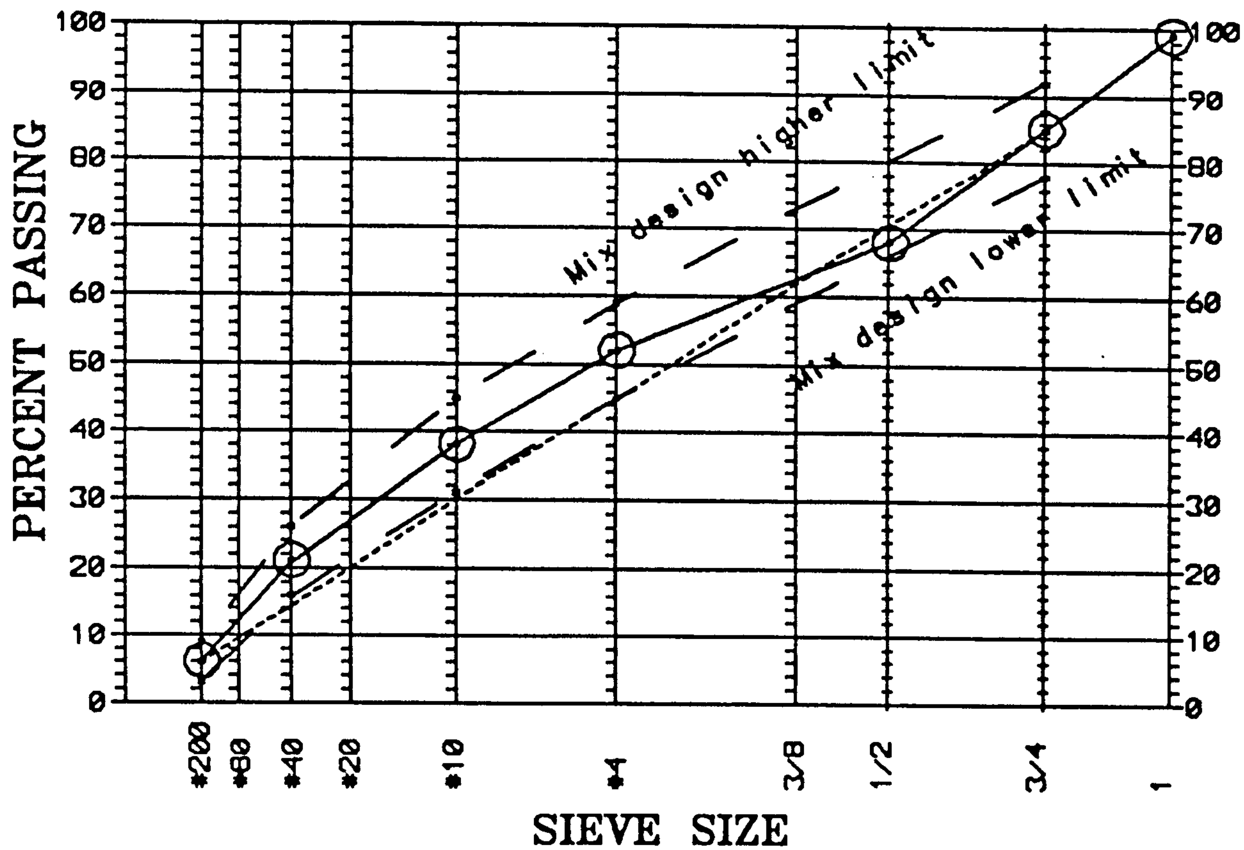


Figure 5.7 Comparison of AHTD's ACHM base course mix gradation from Highway 10 with the maximum density line.

5.4.3 Crushed Particle Content

The AHTD ACHM base course is also superior to the AASHO base in terms of crushed particle content. The AASHO base did not have any crushed material, whereas the ACHM base courses have at least 15% of crushed particles. Crushed particles increase stability through the interlocking of the angular coarser size particles.

5.4.4 Resilient Modulus

Resilient modulus tests were not conducted at the time of the Road Test; thus, no actual resilient modulus measurements exist for the AASHO base material. However, the modulus can be estimated using a relationship developed by The Asphalt Institute [3.17]. Using this relationship, the AASHO base would be expected to have a resilient modulus of 360 to 470 ksi. A similar estimate of the modulus for AHTD's ACHM base produces an expected range of 560 to 650 ksi. By these estimates, the AHTD base is clearly superior.

The resilient moduli of AHTD ACHM base samples were determined by conducting diametral resilient modulus test on 62 Marshall size samples. The average modulus from the Highway 10 mix (Table 5.3) is somewhat higher than the estimated range while the averages from the other two mixes are lower than the estimated range. Nevertheless, even the lowest average (442) is in the upper range of values estimated for the AASHO base.

5.5 AAMAS Relative Fatigue Classification

NCHRP Project 1-10B developed an asphalt-aggregate analysis system (AAMAS) for evaluating and designing asphalt mixes. One part of AAMAS is a relationship developed between resilient modulus and tensile strain at failure for the AASHO Road Test binder and surface mixes. NCHRP Report 338 [2.12] uses this relationship to classify mixes according to their fatigue characteristics in comparison with the AASHO Road Test mixes. Mixes falling above this relationship are regarded as having fatigue characteristics better than those of the AASHO binder and surface mixes. Those falling below are considered to be poorer and more susceptible to fatigue cracking. The AAMAS relationship is:

$$\log \epsilon_t = 4.503 - 0.2595 \log E_R \tag{5.2}$$

where:

ϵ_t = tensile strain at failure,

E_R = total resilient modulus of the asphaltic concrete mix.

This relationship is based on total resilient modulus which was not measured in this study. The resilient modulus measured is referred to as the instantaneous resilient modulus. Nevertheless the total resilient modulus can be estimated using the instantaneous values. Figure 5.8 is a plot of data from NCHRP 338 [2.12]. From this data the total

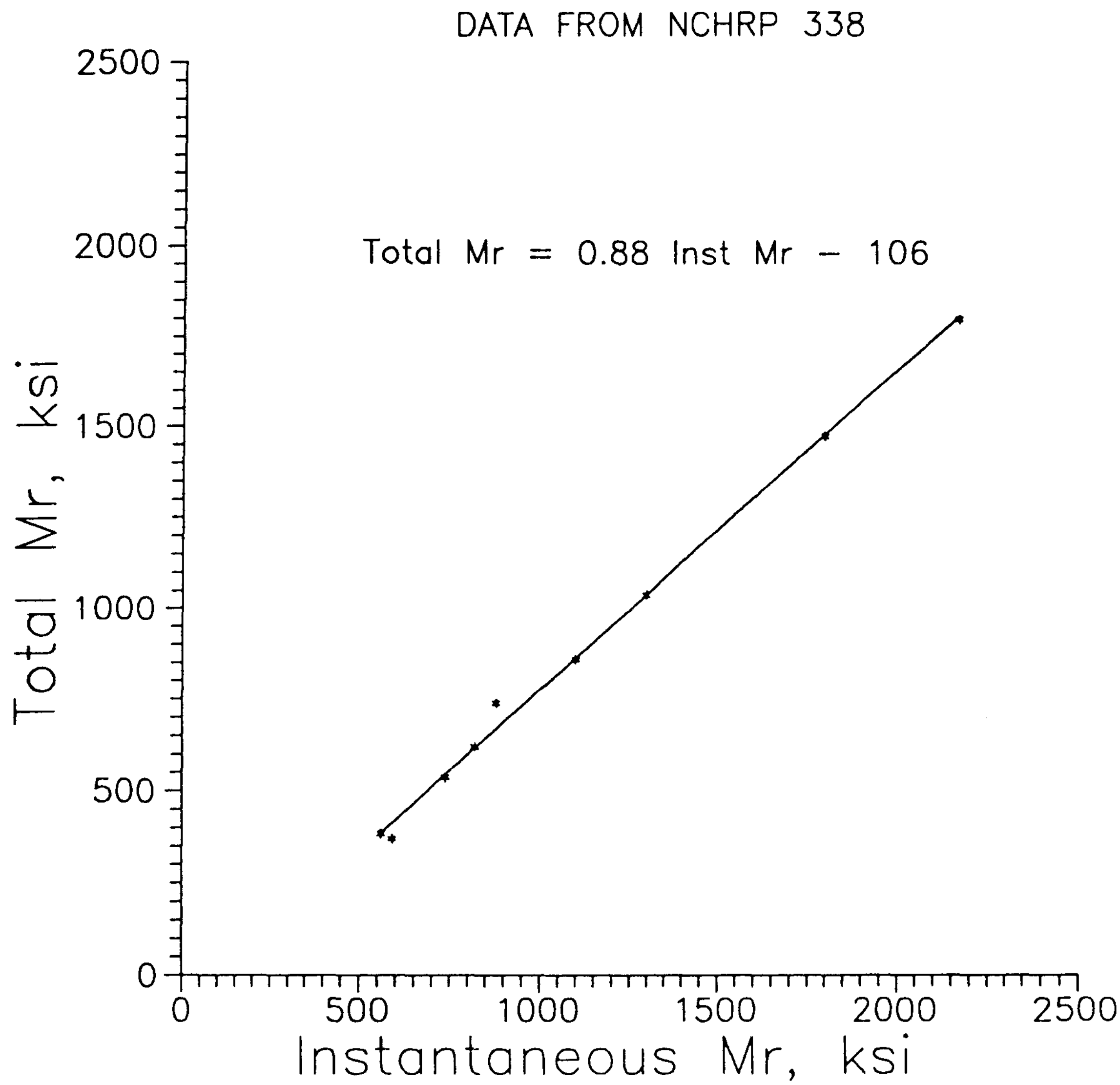


Figure 5.8 Relationship between total resilient modulus and instantaneous resilient modulus from NCHRP 338 [2.12].

resilient modulus can be estimated using the following equation.

$$E_{RT} = 0.88 E_{RI} - 106 \tag{5.3}$$

where:

E_{RT} = total resilient modulus,

E_{RI} = instantaneous resilient modulus,

The estimated average values of the total resilient moduli from equation 5.3 and their corresponding tensile strain values at failure are shown in Table 5.4 for the AHTD base mixes tested. Figure 5.9 presents a plot of these values relative to the NCHRP 338 [2.12] relationship. All eight mixes fall above the relationship. This suggests that ACHM base mixes tested in this study are superior to the AASHO surface and binder in terms of fatigue behavior. If the mixes are superior to the AASHO surface and binder, they can certainly be judged to be superior to the AASHO base.

5.6 Summary

By the comparisons developed in this chapter, the ACHM base course specified by AHTD is shown to be superior to the asphalt stabilized base course used at the AASHO Road Test. The gradations of the AHTD's mixes sampled and tested are superior. The crushed particle content requirement of the AHTD

Table 5.4 Mean Resilient Moduli and Tensile Strain values for AHTD's ACHM

Samples from Highway Project	Tenslie Strain at Failure (inch/inch)	Instantaneous Resilient Modulus (ksi)	Total Resilient Modulus (ksi)
Highway 10	0.0587	654	469
Highway 67	0.0532	441	283
Job #10940	0.0480	473	310
Job #20095	0.0389	501.0	334.9
Job #7995	0.0385	638.5	455.9
Job #060614	0.0382	697.4	507.7
Job #060641	0.0453	558.7	385.7
Job #60105	0.0417	554.6	382.0

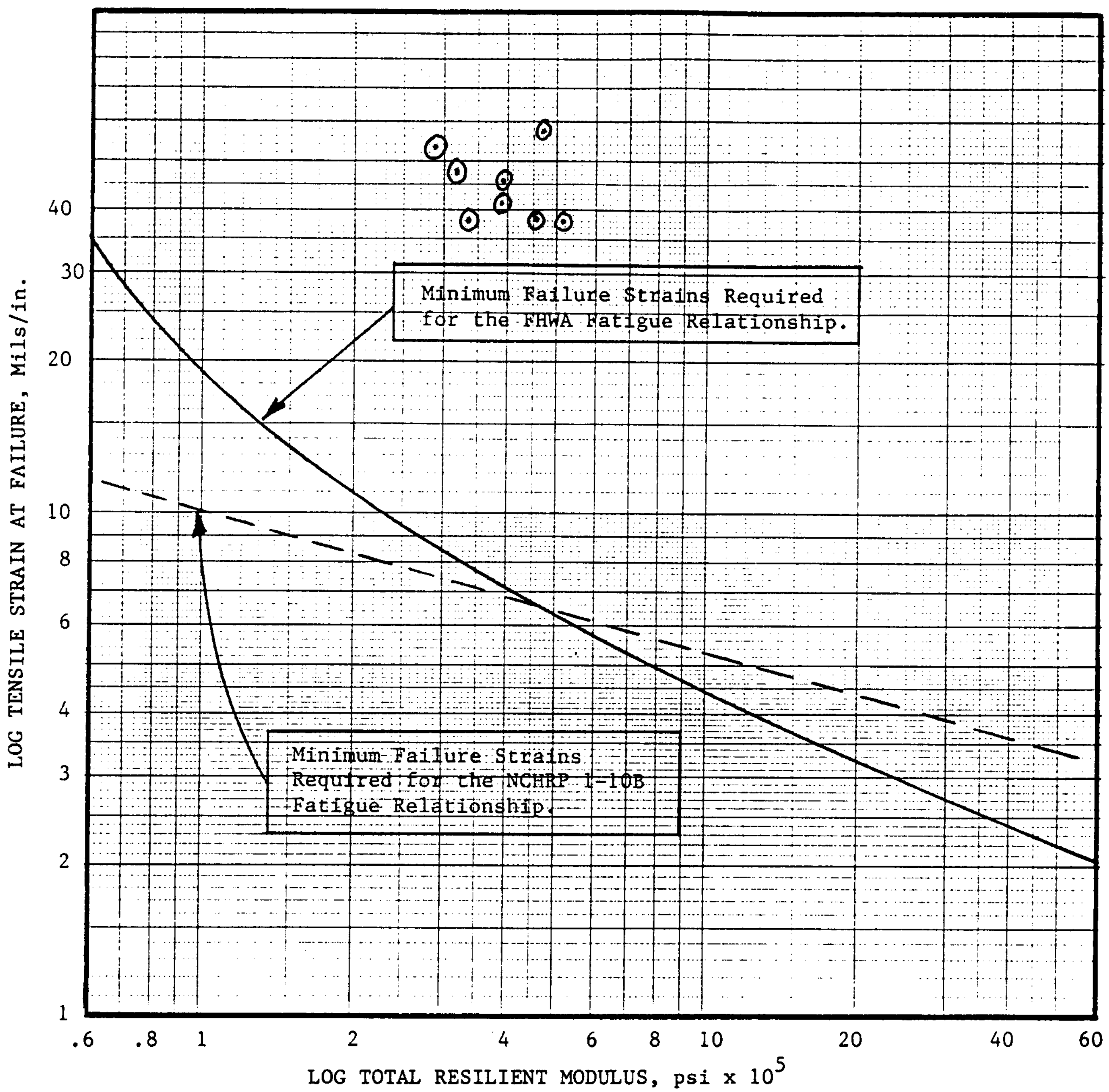


Figure 5.9 Plot of Test Data on NCHRP 338 Fatigue Relationship for AASHO Surfacing Mixture.

specification adds to the superiority and its effect is demonstrated by the higher Marshall stability values. The AHTD mixes also appear to be superior in terms of resilient modulus. In fact, using the relationship developed for AAMAS [2.12], the ACHM base mixes tested appear to even be superior to the AASHO surface and binder mixes with respect to fatigue.

In light of all of this, it can be concluded that AHTD's ACHM base course has a pavement structural value greater than that of the AASHO Road Test asphalt stabilized base.

Chapter 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 General Comments

This project has been solely devoted to the determination of the proper layer coefficient to use in designing flexible pavements containing AHTD's ACHM base course. The pavement designer should recognize that structural numbers, layer coefficients, and thickness are not the only factors to consider in arriving at a satisfactory pavement design.

Numerous investigators have demonstrated that the structural number approach to pavement design has serious limitations. The structural number approach does not directly consider any particular pavement failure mode. For example, the use of an appropriate coefficient and an "adequate" structural number does not assure that excessive, early pavement rutting will not occur. Other engineering and material properties must also be considered. In general, these limitations can be avoided by the designer exercising prudent engineering judgement based on past experience with similar designs in the area.

It also should be pointed out that, despite the clear superiority of the ACHM base, the project investigators did develop some reservations relative to the ACHM base mixture specifications. The primary concern is with the minimum

Marshall stability of 1000 with no limitation on flow or minimum air voids. It is suggested that consideration be given to increasing the minimum stability to 1500 and to adding flow and air void limitations. For high volume highways, consideration might also be given to requiring Class 6 or higher aggregates.

6.2 Recommendation Development

All tests and analyses conducted during this study demonstrate that AHTD's ACHM base course is superior to the AASHO Road Test asphalt stabilized base. Since the structural layer coefficient of 0.34 was estimated for the AASHO base, these study results clearly justify the use of a coefficient greater than 0.34.

The method recommended by the AASHTO Guide for selecting a layer coefficient is based on the material's resilient modulus (Figure 2.4). Resilient modulus tests were conducted on 116 Marshall size specimens obtained from eight highway projects located in Arkansas. From the AASHTO Guide selection method, the layer coefficients for the ACHM base courses for the eight projects ranged from 0.36 to 0.52. The values much greater than 0.34 are recognized as being somewhat questionable since they represent an extrapolation beyond the limits of the AASHTO figure. Nevertheless they do demonstrate help demonstrate that a value in excess of 0.34 is warranted abd suggest that the minimum reasonable value may be 0.36.

The flexible pavement overlay design procedure in the AASHTO Guide contains another resilient modulus relationship (Equation 5.1) that was used to estimate layer coefficients. This procedure is used with back calculated pavement stiffness (i.e. resilient modulus) to estimate the effective structural number of the existing pavement. The layer coefficients estimated using this method ranged from 0.34 to 0.40 for the eight projects.

In general, the layer coefficients typically used to design flexible pavements having asphalt stabilized base are believed to be conservative. A study of the long term behavior of the AASHO pavements after they were incorporated into Interstate 80 [3.26] showed that flexible pavements with asphalt stabilized bases perform much better than would be predicted by the AASHTO Guide design procedures. In particular, the 0.34 structural layer coefficient value assigned to the AASHO base was found to be quite conservative when the total asphalt thickness exceeds about 12.5 inches. A value of 0.44 was suggested by that study for the base coefficient when the total asphalt thickness is 12.5 inches or more.

In light of all of this, it is concluded that the layer coefficient for AHTD's ACHM base should be increased significantly above the 0.25 value initially assumed and being used at the time the study was initiated. Based on the superior characteristics of the ACHM base, a value as high as

0.40 might be justified. However, a value this high would need to used with caution particularly for pavements having a total asphalt thickness of less than 10 inches. It is therefore recommended that a layer coefficient of 0.36 be adopted.

REFERENCES

Chapter 2

2.1 "AASHTO Guide for the Design of Pavement Structures", American Association of State Highway and Transportation Officals, Washington, D,C., 1986.

2.2 The AASHO Road Test Report: History and Description of Project.HRB, Special Report. 61A, 1962.

2.3 The AASHO Road Test Report: Report 5 Pavement Research,HRB, Special Report. 61E, 1962

2.4 AASHO Interim Guide for Design of Pavement Structures. American Association of State Highway Officals, Washington, D.C. 1972.

2.5 Dunn, H.D. Jr., "A Study of Four Stabilized Base Courses", Ph.D. Dissertation, The Pennsylvania State University, Department of Civil Engineering, November, 1974.

2.6 B.J. Coree and T.D. White., "AASHTO Flexible Pavement Design Method: Fact or Fiction?" Transportation Research Record 1286. 1990, pp 206-216.

2.7 Irick, P.E.," An Introduction to Guidelines for Satelite Studies of Pavement Performance." NCHRP Report 2A, 1964.

2.8 Hudson, W.R., "Guidelines for Satellite Studies of Pavement Performance," NCHRP Report 2A (1964)

2.9 Van Til, C.J., B.F.McCullough, B.A. Vallerga and R. G. Hicks, " Evaluation of AASHO Interim Guides for Design of Pavement Structuress," NCHRP Report 128, 1972.

2.10 "AASHTO Interim Guide for Design of Pavement Structures," American Association of State, Highway and Transportation Officials, Washington D.C., 1981

2.11 Kallas, B.F. and Riley, J.C., "Mechanical Properties of Asphlat Pavement Materials." Proc., Second International Conference on the Structural Design of Asphalt Pavements,University of Michigan (1967)

2.12 "Asphalt Aggregate Mixture Analysis System" March ,1991. Federal Highway Adminsration Authority, Washington, D.C,.

2.13 Mamlouk et al., "Implementation of the 1986 AASHTO Guide at the City and County Levels," Transportation Reaserch Record 1215, Transportation Research Board, 1989.

2.14 Chastain, Sr., W.E. and Schwartz, D.R., "AASHO Road Test Equations Applies to the Design of Bituminous Pavements in Illinois", Highway Research Record 90, Highway Research Board, 1965.

2.15 Elliott, R.P, "Proposal for Structural layer coefficeint for Asphalt Concrete Hot Mix Base Courses for Arkansas", 1991.

Chapter 3

3.1 J.F. Shook and F.N. Finn. "Thickness Design Relationships for Structural Design of Asphalt Pavements," Proceedings of the 1st International Conference on the Structural Design of Asphalt Pavements, Ann Arbor, Michigan, 1962.

3.2 Monismith, C.L,. R. L., and Chan, C. K., "Load Transmission Charactoristics of Asphalt Treated Base Courses," Proceedings. Second International Conference of the Structural Design of Asphalt Pavements, University of Michigan, 1967.

3.3 Vaswani, N.K., "Design of Pavements Using Deflection Equations from AASHO Road Test Results", Highway Research Record 239, Transportation Research Board, 1968,

3.4 Vaswani, N. K., "Design of Flexible Pavements in Virginia Using AASHO Road Test Results", Highway Research Record 291, Transportation Rearch Board, 1969.

3.5 Vaswani, N. K., "Optimum Structural Strenght of Materials in Flexible Pavements", Highway Research Record 329, Transportation Research Board 1970.

3.6 Vaswani, N. K., "Method for Separately Evaluating Structural Performance of Subgrades and Overlying Flexible Pavements", Highway Research Record 362, Transportation Research Board 1971.

3.7 Chu, T. Y., Humphries, W. K., Fletcher, O. S., "Application of the AASHO Road Test Findings to the Design of Flexible Pavement Structures on South Carolina," Highway Research Record 131, Transportation Research Board, 1966

3.8 Lukanen, E.O., "Application of AASHO Road Test Results to Design of Flexible Pavements in Mennesota-Final Report".

3.9 Lukanen, E.O., "Evaluation of Full-Depth Asphalt Pavements", Investigation No. 195, Office of Materials, Research and Standards, Minnesota Department of Transportation.

3.10 Thompson, M.R., "A Preliminary Study of Lime Soil Coeffficients", Unpublished Report prepared for Illinois Department of Transporations, March 1968.

3.11 Thompson, M. R., "Proposal for a study of a Mechanistic Evaluation of Illinois Flexible Pavement Design Procedures", University of Illinois at Urbana-Champaign,1978.

3.12 Rada, G., and Witczak, M. W., "Material layer coefficients of unbound granular materials from resilient modulus," Highway Research Record No. 852, Transportation Research Board, 1982.

3.13 Van Til, C.J., B.F.McCullough, B.A. Vallerga and R. G. Hicks, " Evaluation of AASHO Interim Guides for Design of Pavement Structuress," NCHRP Report 128,1972.

3.14 Kalankamary P. George,."Structural Layer Coefficient for Flexible Pavement," Journal of Transportation Engineering, American Society of Civil Engineering, Vol.110, No.2, March 1984.

3.15 Eks, Martin, "On the Right Track", Asphalt Institute Quarterly, The Asphalt Institute, College Park, Maryland, October 1961.

3.16 Skok, E.L. and Finn, F.N.., "Theoretical Concepts Applied to Asphalt Concrete Pavements", Proceedings, 1st International Conference on Structural Design of Asphalt Pavements, Ann Arbor, Michigan, 1962.

3.17 "Thickness Design-Full Depth Asphalt Pavement Structures for Highways and Streets", MS-1 (Revised 8th Edition), Asphalt Institute, 1970.

3.18 Thickness Design-Full Depth Asphalt Pavement Structures for Highways and Streets", MS-1, The Asphalt Institute, 1981.

3.19 Kamel, N.I.,Phang,W.,Morris, J.,and Haas, R.C.G.,"Layer Analysis of the Brampton Test Road and Application to Pavement Design", Highway Research Record 466, Transportation Research Board, 1973."

3.20 Phang, W.A. "Four Years Experience at the Brampton TestRoad", Report RR-153, Ontario Departement of Highways, October 1969.

3.21 Jung, F.W., Phang,W.A., " Elastic layer anlaysis related to performance in flexible design", Report RR 191, Research and Development Division, Ministry of Transportation and Communications, Ontario, Canada, March, 1975.

3.22 Takeshita, H., "Consideratins on the Structural Number", Proceedings, 2nd International Conference on the Structural Design of Asphalt Pavements, Ann Arbor, Michigan,1967.

3.23 Dunn,H. D. Jr., "A Study of Four Stabilized Base Courses", Ph.D. Dissertation, The Pennsylvania State University, Departement of Civil Engineering, November, 1974.

3.24 Wang, M.C. and Larson, T.D., "Performance Evaluation for Bituminous Concrete Pavements at the Pennsylvania State Test Track", Transportation Research Record 632, Transportation Research Board, 1977.

3.25 M.C Wang and T.D. Larson, "Evaluation of Structural Layer Coefficients of stabilized base courses materials. Transportation Research Record 725, 1979.

3.26 Elliott, R.P., "Rehabilitated AASH(T)O Road Test-Analyis of Performance Data Reported in Illinois Phyisical Report 76," National Asphalt Pavement Associaiton, April 1981.

3.27 M.I. Darter and A.J. Devos. Structural Analysis of Asphaltic Cold Mixture Used in Pavement Bases. Civil Engineering Studies, Transportation Engineering Series 19, Illinois Cooperative Highway Research Program Series 171, University of Illinois, Urbana, Aug, 1977.

3.28 M.Gomez, M.R. Thompson, "Structural Coefficients and Thickness Equivalency Ratios", University of Illinois, Urbana, Illinois, June,1983.

3.29 B.J. Coree and T.D. White., "AASHTO Flexible Pavement Design Method: Fact or Fiction?" Transportation Research Record 1286. 1990, pp 206-216.

3.30 Anastasios M. Ionnides, " Theoritical Implications of the AASHTO 1986 Nondestructive Testing Method 2 for Pavement Evaluation", Transportation Research Record 1307.

3.31 R.L. Terrel and C.L. Monismith. "Evaluation of Asphalt-Treated Base Course Marterials. Proc., Association of Asphlat Paving Technologisits Technical Sessions, Atlanta, Georgia, 1968.

3.32 B.S Coffman,G.Ilves, and W.Edwards, "Theoretical Asphaltic Concrete Equivalences, Highway Research Record 239, Highway Reasearch Board, 1968.

3.33 F.P. Nichols. Dsicussion of "Design of Fexible Pavemennts in Virginia Using AASHO Road Test Result," by N.K. Vaswani, Highway Research Record 291, Highway Reaserch Board, 1969.

3.34 A.M. Ioannides, " Correlation between Layer Moduli, Layer Coefficients and In-Situ Pavement Structural Numbers", NCHRP Project 20-7 Task 39, 1989.

3.35 Liddle, W.J., "Application of AASHO Road Test Results to the Design of Flexible Pavement Structures", Proceedings, 1st International Conference on the Structural Design of Asphlat Pavements, Ann Arbor, Michigan, 1962.

Chapter 4

4.1 Kenndey, T.W., and Anagnos, J.N., "Procedures for the Static and Repeated load Indirect Tensile Tests," Center for Transportation Research, Report No.183-14, 1983.

4.2 Materials, C.P., "Tenative Mix Design Criteria for Gap-Graded Bituminous Surfacings," National Institute for Road Research, Pretoria, South Africa, January 1974.

4.3 G.W. Maupin, Jr., and J.R, Freeman, Jr. Simple Procedure for Fatigue Characteristics of Bituminios Concrete. Report FHWA-RD-76-102. FHWA, U.S. Department of Transportation, 1976.

4.4 Maupin, G.W., "Results of Indirect Tensile Tests Related to Asphalt Fatigue," Virginia Highway and Transportation Research Council, Nov. 1971.

4.5 Elliott, R.P., and Herrin, M., "Relative Life Effects of Mix Composition and Density Variation," Proceedings of the Association of Asphalt Paving Technologists, 1985.

4.6 Little, Dallas, N. and Richey, Barry, L.," A Mixture Design Procedure Based on The Failure Envelope Concept, " Proceedings of the Association of Asphalt Paving Technologists, 1983.

Chapter 5

5.1 How, L.S., "Effect of Crushed Particle Content on Asphalt Mix properties", Master's Theis, Departement of Civil Engineering, University of Arkansas, August 1989.

APPENDIX A

Test Results from Three Original Projects

Table 1. Resilient Modulus of Asphalt concrete base course samples from - Job #10940.

Sample #	Layer Position	Av. M_R on the first axis (ksi)	Av. M_R on the Second axis (ksi)	Average MR for the Layer (ksi)	Average M_R for the sample (ksi)	Standard Deviation	Coefficient of Variation (%)
A	Top	348.80	405.50	377.10	470.33	123.54	26.26
	Middle	423.47	423.40	423.43			
	Bottom	577.16	643.77	610.46			
B	Top	622.65	558.15	590.40	581.87	253.29	43.53
	Middle	353.67	295.10	324.40			
	Bottom	880.40	781.20	830.80			
C	Top	374.80	408.94	391.87	412.01	41.36	10.03
	Middle	409.40	359.80	384.64			
	Bottom	528.08	391.20	459.53			
D	Top	223.42	138.05	180.74	217.09	51.40	23.67
	Bottom	256.18	250.70	253.44			
E	Top	367.46	322.03	344.74	329.50	66.05	20.05
	Middle	282.80	231.54	257.16			
	Bottom	403.80	369.40	386.60			
F	Top	544.83	508.60	526.70	527.47	72.82	13.80
	Middle	437.85	472.20	455.00			
	Bottom	579.50	621.91	600.70			
G	Top	400.00	456.70	428.60	502.48	104.43	20.78
	Bottom	569.58	583.11	576.35			
H	Top	354.02	343.11	348.56	434.32	78.30	18.02
	Middle	511.33	492.70	502.00			
	Bottom	438.31	466.47	452.40			
I	Top	497.63	522.60	510.12	529.00	17.37	3.28
	Middle	506.32	559.36	532.55			
	Bottom	501.00	587.70	544.33			

Mean M_R for all samples = 473.37 ksi
Standard Deviation for all samples = 79.87
Coefficient of Variation for all samples = 16.87 %

Table 2. Resilient Modulus of Asphalt concrete base course samples from Highway 10

Sample #	Layer Position	Av. M_R on the first axis (ksi)	Av. M_R on the Second axis (ksi)	Average MR for the Layer (ksi)	Average M_R for the sample (ksi)	Standard Deviation	Coefficient of Variation (%)
A	Top	570.54	629.50	600.02	680.54	83.21	12.22
	Middle	655.70	695.10	675.40			
	Bottom	749.30	783.30	766.20			
B	Top	612.30	615.30	613.30	625.70	213.44	34.11
	Middle	422.80	414.96	418.83			
	Bottom	817.66	872.38	845.02			
C	Top	546.00	483.00	514.50	589.58	106.17	18.00
	Bottom	702.30	627.00	664.65			
D	Top	425.60	464.97	445.30	535.88	166.25	31.02
	Middle	455.00	446.79	450.90			
	Bottom	763.72	659.20	711.46			
E	Top	488.00	503.83	495.92	533.36	52.95	9.92
	Bottom	609.70	532.00	570.80			
F	Top	749.00	870.80	809.90	782.85	98.40	12.57
	Middle	584.80	762.70	673.75			
	Bottom	806.46	923.30	864.90			
G	Top	654.30	674.19	664.25	756.46	130.39	17.23
	Bottom	868.90	828.42	848.66			
H	Top	743.12	717.74	730.44	731.64	13.47	1.84
	Middle	747.91	689.80	718.86			
	Bottom	710.00	781.23	745.61			

Mean M_R for all samples = 654.50 ksi
Standard Deviation for all samples = 98.04
Coefficient of Variation for all samples = 14.98 %

Table 3. Resilient Modulus of Asphalt concrete base course samples from Highway 67

Sample #	Layer Position	Av. M_R on the first axis (ksi)	Av. M_R on the Second axis (ksi)	Average M_R for the Layer (ksi)	Average M_R for the sample (ksi)	Standard Deviation	Coefficient of Variation (%)
A	Top Bottom	364.11 457.92	370.67 408.79	367.45 433.40	400.42	46.63	11.64
B	Top Bottom	359.32 458.45	345.00 475.40	352.16 465.43	408.79	19.59	4.79
C	Top Bottom	357.55 418.25	371.59 391.00	364.50 404.60	384.55	28.35	7.37
D	Top Bottom	363.90 443.37	405.02 424.36	384.50 433.87	409.18	34.92	8.53
E	Top Bottom	375.04 401.09	397.60 425.60	386.31 413.40	399.85	19.16	4.79
F	Top Bottom	371.04 463.93	426.35 495.89	398.75 480.00	439.40	57.45	13.07
G	Top Bottom	424.56 526.50	474.48 633.68	449.52 580.07	514.79	92.31	17.93
H	Top Bottom	519.80 591.70	544.13 652.60	531.96 622.16	577.03	63.78	11.05

Mean M_R for all samples = 441.75 ksi
Standard Deviation for all samples = 68.16
Coefficient of Variation for all samples = 15.43 %

Table 4. Split Tensile Strength test results for ACHMBC sample from Job # 10940

Sample #	Layer Position	Tensile Strength (psi)	Average Tensile Strength (psi)	Standard Devation	Coefficient of Variation (%)	Tensile Strain at failure (in/in)	Average Strain at failure (in/in)
A	Top Middle Bottom	67.07 71.30 101.98	80.12	19.05	23.78	0.046828 0.039023 0.046828	0.044226
B	Top Middle Bottom	120.14 63.66 105.48	96.43	29.31	30.39	0.052031 0.033820 0.052031	0.045961
C	Top Middle Bottom	81.89 77.03 94.09	84.33	8.78	10.42	0.067640 0.049429 0.067640	0.061569
D	Top Bottom	34.59 49.73	42.16	10.70	25.39	0.059835 0.044226	0.052031
E	Top Middle Bottom	83.97 43.10 65.53	64.2	20.46	31.88	0.048280 0.048280 0.036421	0.044327
F	Top Middle Bottom	90.71 87.56 91.70	89.99	2.16	2.40	0.041624 0.041624 0.039023	0.040757
G	Top Bottom	92.31 98.67	95.49	4.49	4.71	0.046828 0.036421	0.041625
H	Top Middle Bottom	51.96 87.69 72.34	70.66	17.93	25.36	0.062437 0.046828 0.039023	0.049429
I	Top Middle Bottom	108.86 89.87 92.98	97.23	10.18	10.47	0.062437 0.049429 0.044226	0.052031

Mean Tensile Strength = 80.06 psi Standard Deviation = 18.35 Coefficient of Variation = 22.92 %
Mean Tensile Strain at failure = 0.047995 in/in Standard Deviation = 0.006553 Coefficient of Variation = 13.65 %

Table 5. Split Tensile Strength test results for ACHMBC sample from highway 10

Sample No.	Layer Position	Tensile Strength (psi)	Average Tensile Strength (psi)	Standard Devation	Coefficient of Variation (%)	Tensile Strain at failure (in/in)	Average Strain at failure (in/in)
A	Top Middle Bottom	154.16 141.18 181.43	159.13	20.51	12.91	0.06764 0.04943 0.05203	0.05637
B	Top Middle Bottom	149.60 159.15 183.49	164.08	17.47	10.65	0.08064 0.07805 0.05203	0.07024
C	Top Bottom	105.04 151.19	128.11	23.56	18.39	0.04682 0.04942	0.04812
D	Top Middle Bottom	160.74 143.24 136.07	146.68	12.69	8.65	0.05723 0.07284 0.08325	0.07111
E	Top Bottom	92.31 98.89	95.60	4.65	4.87	0.04682 0.04942	0.04812
F	Top Middle Bottom	151.57 139.04 170.98	153.86	16.09	10.46	0.05723 0.05723 0.05854	0.05767
G	Top Bottom	135.75 149.79	142.77	9.92	6.95	0.05203 0.05984	0.05593
H	Top Middle Bottom	144.97 129.79 165.52	146.76	17.93	12.22	0.05723 0.06244 0.04943	0.05637

Avg. Tensile Strength = 142.12 psi Standard Deviation = 21.75 Coeff. of Variation = 15.31 %

Avg. Tensile Strain at failure = 0.05868 in/in Standard Deviation = 0.008705 Coeff. of Variation = 14.83 %

Table 6. Split Tensile Strength test results for ACHMBC sample from highway 67

Sample No.	Layer Position	Tensile Strength (psi)	Average Tensile Strength (psi)	Standard Devation	Coefficient of Variation (%)	Tensile Strain at Failure (in/in)	Average Strain at failure (in/in)
A	Top Bottom	89.76 93.97	91.86	2.97	3.24	0.057234 0.049429	0.053331
B	Top Bottom	84.26 97.44	90.85	9.32	10.26	0.046828 0.052031	0.049429
C	Top Bottom	120.96 112.19	116.57	6.20	5.23	0.044226 0.052031	0.048128
D	Top Bottom	106.10 118.58	112.34	8.82	7.85	0.062437 0.057230	0.059833
E	Top Bottom	92.31 98.89	95.60	4.65	4.86	0.046820 0.049420	0.048120
F	Top Bottom	108.16 108.16	108.16	0	0	0.052031 0.052031	0.052031
G	Top Bottom	95.49 111.41	103.45	11.25	10.88	0.067640 0.052031	0.059835
H	Top Bottom	121.60 124.77	123.18	2.24	1.82	0.057234 0.052031	0.054632

Avg. Tensile Strength = 105.25 psi Standard Deviation = 11.90 Coeff. of Variation = 11.31 %
Avg. Tensile Strain at failure = 0.053167in/in Standard Deviation = 0.004738 Coeff. of Variation = 8.91 %

Table 7. Stiffness of ACHMBC samples from Job # 10940

Sample #	Layer Position	Tensile Stiffness at 3/4th of tensile Stress (psi)	Mean (psi)	Standard Devaiton	Coefficient of Variation (%)
A	Top Middle Bottom	2274.65 2936.70 3919.70	3043.68	827.72	27.19
B	Top Middle Bottom	3148.46 2622.05 2764.43	2844.98	272.27	9.57
C	Top Middle Bottom	1888.71 2220.59 1750.06	1953.30	241.78	12.38
D	Top Bottom	1107.88 2205.78	1656.83	776.33	46.85
E	Top Middle Bottom	2689.84 1825.82 2952.00	2489.22	589.28	23.67
F	Top Middle Bottom	3631.90 3505.91 4005.17	3714.33	259.64	6.99
G	Top Bottom	2956.77 4063.50	3510.13	782.56	22.29
H	Top Middle Botom	1440.33 2527.70 2978.98	2315.67	790.92	34.19
I	Top Middle Botom	2851.43 2467.38 2821.38	2713.40	213.58	7.87

Mean Tensile Stiffness at 3/4th of the maximum tensile stress = 2693.97 psi
Standard Deviation = 703.83
Coefficient of Variation = 26.13 %

Table 8. Tensile Stiffness of ACHMBC samples from Highway 67

Sample #	Layer Position	Tensile Stiffness at 3/4th of maximum tensile Stress (psi)	Mean (psi)	Standard Devaiton	Coefficient of Variation (%)
A	Top Bottom	2353.43 2851.73	2602.08	352.35	13.54
B	Top Bottom	2699.20 2687.25	2687.25	286.80	10.67
C	Top Bottom	4102.52 3234.20	3668.36	613.99	16.73
D	Top Bottom	2550.00 3418.72	2984.36	614.28	20.58
E	Top Bottom	2956.49 3167.33	3061.91	149.09	4.87
F	Top Bottom	2970.00 3282.20	3126.10	220.76	7.06
G	Top Bottom	1966.40 2919.93	2443.17	674.25	27.56
H	Top Botom	3186.90 3996.75	3591.80	572.62	15.94

Mean Tensile Stiffness at 3/4th of the maximum tensile stress = 3020.63 psi
Standard Deviation = 444.00
Coefficient of Variation = 14.70 %

Table 9. Tensile Stiffness of ACHMBC samples from Highway 10.

Sample #	Layer Position	Tensile Stiffness at 3/4th of maximum tensile Stress (psi)	Mean (psi)	Standard Devaiton	Coefficient of Variation (%)
A	Top Middle Bottom	3423.98 3716.83 4548.12	3896.31	583.16	14.96
B	Top Middle Bottom	2695.49 3058.68 4921.87	3558.68	1194.44	33.56
C	Top Bottom	3364.65 4358.93	3861.79	703.06	18.20
D	Top Middle Bottom	2989.50 2949.60 3268.87	3069.33	173.96	5.66
E	Top Bottom	2128.87 2192.17	2160.52	44.75	2.04
F	Top Middle Bottom	3361.00 3083.95 3793.58	3412.84	426.61	12.50
G	Top Bottom	3913.42 3598.48	3755.95	222.69	5.93
H	Top Middle Botom	3798.42 2993.28 5613.83	4135.17	1342.34	32.46

Mean Tensile Stiffness at 3/4th of the maximum tensile stress = 3481.32 psi
Standard Deviation = 626
Coefficient of Variation = 17.98 %

APPENDIX B

Test Results from Five Additional Projects

Table 1. Resilient Modulus of Asphalt concrete base course samples from - Job #20095

Sample #	Layer Position	Av. Mr on the First axis (ksi)	Av. Mr on the Second axis (ksi)	Average Mr for the Layer (ksi)	Average Mr for the sample (ksi)	Standard Deviation	Coefficient of Variation (%)
1	Top	357.4	431.9	394.7	444.3	43.0	9.7
	Middle	417.5	517.6	467.6			
	Bottom	457.7	483.6	470.7			
2	Top				490.6	2.4	0.5
	Middle	493.7	484.0	488.9			
	Bottom	484.7	499.9	492.3			
3	Top	492.2	445.6	468.9	525.3	79.7	15.2
	Middle						
	Bottom	611.8	551.4	581.6			
4	Top	487.5	611.3	549.4	509.6	64.7	12.7
	Middle	552.1	536.7	544.4			
	Bottom	456.9	413.0	435.0			
5	Top				535.2	-----	-----
	Middle	551.9	518.4	535.2			
	Bottom						

Mean Mr for all samples = 501.0 ksi
Standard Deviation for all samples = 47.5
Coefficient of Variation for all samples = 9.5 %

Table 2. Resilient Modulus of Asphalt concrete base course samples from - Job #7995

Sample #	Layer Position	Av. Mr on the First axis (ksi)	Av. Mr on the Second axis (ksi)	Average Mr for the Layer (ksi)	Average Mr for the sample (ksi)	Standard Deviation	Coefficient of Variation (%)
6	Top	364.4	348.9	356.7	437.5	114.2	26.1
	Middle						
	Bottom	518.2	518.2	518.2			
7	Top	677.3	678.2	682.3	682.3	-----	-----
	Middle						
	Bottom						
8	Top	591.2	617.1	604.2	575.8	40.3	7.0
	Middle						
	Bottom	530.1	564.5	547.3			
9	Top	813.7	754.9	784.3	716.9	95.4	13.3
	Middle						
	Bottom	600.1	698.7	649.4			
10	Top	705.2	798.3	751.8	743.9	11.2	1.5
	Middle						
	Bottom	717.7	754.1	735.9			

Mean Mr for all samples = 638.5 ksi
Standard Deviation for all samples = 65.3
Coefficient of Variation for all samples = 12.0 %

Table 3. Resilient Modulus of Asphalt concrete base course samples from - Job #060614

Sample #	Layer Position	Av. Mr on the First axis (ksi)	Av. Mr on the Second axis (ksi)	Average Mr for the Layer (ksi)	Average Mr for the sample (ksi)	Standard Deviation	Coefficient of Variation (%)
11	Top	652.8	682.2	667.5	664.7	4.0	0.6
	Middle						
	Bottom	675.0	648.6	661.8			
12	Top	737.6	735.7	736.7	683.3	110.5	16.2
	Middle	781.1	732.6	756.9			
	Bottom	538.0	574.6	556.3			
13	Top	734.3	724.2	729.3	733.6	6.0	0.8
	Middle						
	Bottom	717.3	758.3	737.8			
14	Top	737.4	801.1	769.2	683.6	121.1	17.7
	Middle	598.1	597.7	597.9			
	Bottom						
15	Top	610.3	634.7	622.5	721.7	140.3	19.4
	Middle						
	Bottom	679.2	961.9	820.9			

Mean Mr for all samples = 697.4 ksi
Standard Deviation for all samples = 76.4
Coefficient of Variation for all samples = 10.9 %

Table 4. Resilient Modulus of Asphalt concrete base course samples from - Job #060641

Sample #	Layer Position	Av. Mr on the First axis (ksi)	Av. Mr on the Second axis (ksi)	Average Mr for the Layer (ksi)	Average Mr for the sample (ksi)	Standard Deviation	Coefficient of Variation (%)
16	Top	447.6	478.1	462.9	601.7	100.8	16.7
	Top (II)	655.0	725.9	690.5			
	Middle	570.2	619.7	595.0			
	Bottom	626.1	691.0	658.6			
17	Top	427.2	549.8	488.5	569.2	73.1	12.8
	Middle	524.3	651.6	588.0			
	Bottom	569.3	692.8	631.1			
18	Top	444.1	510.5	477.3	555.0	72.1	13.0
	Top (II)	635.3	549.8	592.6			
	Middle	592.5	678.7	635.6			
	Bottom	504.2	524.8	514.5			
19	Top	381.2	418.3	399.8	477.3	71.3	14.9
	Middle	463.7	520.0	491.9			
	Bottom	542.4	538.0	540.2			
20	Top				590.5	25.2	4.3
	Middle	550.9	594.4	572.7			
	Bottom	503.2	713.3	608.3			

Mean Mr for all samples = 558.74 ksi
Standard Deviation for all samples = 68.5
Coefficient of Variation for all samples = 12.3 %

Table 5. Resilient Modulus of Asphalt concrete base course samples from - Job #60105

Sample #	Layer Position	Av. Mr on the First axis (ksi)	Av. Mr on the Second axis (ksi)	Average Mr for the Layer (ksi)	Average Mr for the sample (ksi)	Standard Deviation	Coefficient of Variation (%)
1	Middle	520.5	474.4	497.5	497.5	42.9	7.7
2	Middle	583.0	563.4	573.2	573.2		
3	Middle	541.1	495.3	518.2	518.2		
4	Middle	672.3	578.9	625.6	625.6		
5	Middle	585.1	539.9	562.5	562.5		
6	Middle	636.9	513.4	575.2	575.2		
7	Middle	537.9	523.1	530.5	530.5		

Mean Mr for all samples = 554.6 ksi
Standard Deviation for all samples = 42.9
Coefficient of Variation for all samples = 7.7 %

Table 6. Split Tensile Strength for ACHMBC samples from - Job #20095

Sample #	Layer Position	Tensile Strength (psi)	Average Tensile Strength (psi)	Standard Deviation	Coefficient of Variation (%)	Tensile Strain at failure (in/in)	Average Strain at failure (in/in)
1	Top	76.0	84.5	8.8	10.4	0.0313	0.0354
	Middle	93.5				0.0418	
	Bottom	84.1				0.0330	
2	Top		97.5	4.9	5.0		0.0382
	Middle	100.9				0.0425	
	Bottom	94.0				0.0338	
3	Top	83.4	93.1	13.6	14.7	0.0400	0.0394
	Middle						
	Bottom	102.7				0.0388	
4	Top	98.0	88.5	10.1	11.4	0.0440	0.0444
	Middle	89.5				0.0468	
	Bottom	77.9				0.0425	
5	Top		77.1	-----	-----		0.0365
	Middle	77.1				0.0365	
	Bottom						

Mean Tensile Strength = 88.1 psi — Standard Deviation = 7.9 — Coefficient of Variation = 8.9 %

Mean Tensile Strain at failure = 0.0389 in/in — Standard Deviation = 0.0035 — Coefficient of Variation = 9.0 %

Table 7. Split Tensile Strength for ACHMBC samples from - Job #7995

Sample #	Layer Position	Tensile Strength (psi)	Average Tensile Strength (psi)	Standard Deviation	Coefficient of Variation (%)	Tensile Strain at failure (in/in)	Average Strain at failure (in/in)
6	Top	90.1				0.0463	
	Middle		109.5	27.4	25.1		0.0438
	Bottom	128.9				0.0413	
7	Top	94.2				0.0398	
	Middle		94.2	-----	-----		0.0398
	Bottom						
8	Top	88.0				0.0468	
	Middle		95.5	10.5	11.0		0.0422
	Bottom	102.9				0.0375	
9	Top	79.2				0.0320	
	Middle		85.3	8.6	10.1		0.0342
	Bottom	91.4				0.0363	
10	Top	78.1				0.0318	
	Middle		75.6	3.6	4.8		0.0326
	Bottom	73.0				0.0333	

Mean Tensile Strength = 92.0 psi   Standard Deviation = 12.6   Coefficient of Variation = 13.7 %

Mean Tensile Strain at failure = 0.0385 in/in   Standard Deviation = 0.0049   Coefficient of Variation = 12.8 %

Table 8. Split Tensile Strength for ACHMBC samples from - Job #060614

Sample #	Layer Position	Tensile Strength (psi)	Average Tensile Strength (psi)	Standard Deviation	Coefficient of Variation (%)	Tensile Strain at failure (in/in)	Average Strain at failure (in/in)
11	Top	104.2				0.0363	
	Middle		107.4	4.5	4.1		0.0388
	Bottom	110.5				0.0413	
12	Top	79.5				0.0393	
	Middle	112.4	98.4	17.0	17.3	0.0350	0.0364
	Bottom	103.4				0.0350	
13	Top	95.7				0.0430	
	Middle		92.5	4.6	5.0		0.0424
	Bottom	89.2				0.0418	
14	Top	127.1				0.0333	
	Middle	85.0	106.1	29.8	28.1	0.0330	0.0332
	Bottom						
15	Top	96.4				0.0425	
	Middle		97.7	1.8	1.8		0.0400
	Bottom	98.9				0.0375	

Mean Tensile Strength = 100.4 psi   Standard Deviation = 6.2   Coefficient of Variation = 6.2 %

Mean Tensile Strain at failure = 0.0382 in/in   Standard Deviation = 0.0035   Coefficient of Variation = 9.2 %

Table 9. Split Tensile Strength for ACHMBC samples from - Job #060641

Sample #	Layer Position	Tensile Strength (psi)	Average Tensile Strength (psi)	Standard Deviation	Coefficient of Variation (%)	Tensile Strain at failure (in/in)	Average Strain at failure (in/in)
16	Top	115.8	112.1	7.1	6.3	0.0425	0.0446
	Top (II)	118.9				0.0520	
	Middle	111.1				0.0438	
	Bottom	102.6				0.0400	
17	Top	119.4	120.4	10.2	8.5	0.0463	0.0446
	Middle	131.1				0.0425	
	Bottom	110.7				0.0450	
18	Top	135.7	129.2	20.6	15.9	0.0438	0.0432
	Top (II)	109.6				0.0443	
	Middle	116.2				0.0460	
	Bottom	155.2				0.0388	
19	Top	136.9	124.0	12.2	9.9	0.0443	0.0474
	Middle	122.4				0.0515	
	Bottom	112.6				0.0463	
20	Top		126.9	4.2	3.3		0.0465
	Middle	123.9				0.0425	
	Bottom	129.9				0.0505	

Mean Tensile Strength = 122.5 psi Standard Deviation = 6.7 Coefficient of Variation = 5.5 %
Mean Tensile Strain at failure = 0.0453 in/in Standard Deviation = 0.0017 Coefficient of Variation = 3.7 %

Table 10. Split Tensile Strength for ACHMBC samples from - Job #60105

Sample #	Layer Position	Tensile Strength (psi)	Average Tensile Strength (psi)	Standard Deviation	Coefficient of Variation (%)	Tensile Strain at failure (in/in)	Average Strain at failure (in/in)
1	Middle	88.6	103.9	13.6	13.1	0.0400	0.0417
2	Middle	118.7				0.0465	
3	Middle	113.5				0.0365	
4	Middle	95.1				0.0433	
5	Middle	110.2				0.0390	
6	Middle	86.0				0.0430	
7	Middle	115.0				0.0438	

Mean Tensile Strength = 103.9 psi Standard Deviation = 13.6 Coefficient of Variation = 13.1 %
Mean Tensile Strain at failure = 0.0417 in/in Standard Deviation = 0.0034 Coefficient of Variation = 8.1 %

Table 11. Stiffness ACHMBC samples from - Job #20095

Sample #	Layer Position	Tensile Stiffness at 3/4 of Tensile Stress (psi)	Mean (psi)	Standard Deviation	Coefficient of Variation (%)
	Top	2851.4			
1	Middle	2281.3	2689.2	355.7	13.2
	Bottom	2934.9			
	Top				
2	Middle	2461.0	2640.4	253.6	9.6
	Bottom	2819.7			
	Top	2176.5			
3	Middle		2415.7	338.2	14.0
	Bottom	2654.8			
	Top	2146.6			
4	Middle	1944.7	2112.3	153.4	7.3
	Bottom	2245.7			
	Top				
5	Middle	2386.0	2386.0	-----	-----
	Bottom				

Mean Tensile Stiffness at 3/4 of the maximum tensile stress = 2448.7 psi
Standard Deviation = 230.6
Coefficient of Variation = 9.4 %

Table 12. Stiffness ACHMBC samples from - Job #7995

Sample #	Layer Position	Tensile Stiffness at 3/4 of Tensile Stress (psi)	Mean (psi)	Standard Deviation	Coefficient of Variation (%)
6	Top	2017.3	2508.0	693.9	27.7
	Middle				
	Bottom	2998.6			
7	Top	2522.7	2522.7	-----	-----
	Middle				
	Bottom				
8	Top	1940.8	2336.6	559.7	24.0
	Middle				
	Bottom	2732.3			
9	Top	2732.2	2710.2	31.1	1.1
	Middle				
	Bottom	2688.2			
10	Top	2758.0	2474.4	401.1	16.2
	Middle				
	Bottom	2190.8			

Mean Tensile Stiffness at 3/4 of the maximum tensile stress = 2510.4 psi
Standard Deviation = 133.8
Coefficient of Variation = 5.3 %

Table 13. Stiffness ACHMBC samples from - Job #060614

Sample #	Layer Position	Tensile Stiffness at 3/4 of Tensile Stress (psi)	Mean (psi)	Standard Deviation	Coefficient of Variation (%)
	Top	2920.6			
11	Middle		2765.7	219.1	7.9
	Bottom	2610.8			
	Top	2112.7			
12	Middle	3093.8	2693.0	514.6	19.1
	Bottom	2872.4			
	Top	2393.7			
13	Middle		2330.5	89.4	3.8
	Bottom	2267.2			
	Top	4100.2			
14	Middle	2932.7	3516.5	825.5	23.5
	Bottom				
	Top	2295.5			
15	Middle		2460.6	233.5	9.5
	Bottom	2625.7			

Mean Tensile Stiffness at 3/4 of the maximum tensile stress = 2753.3 psi
Standard Deviation = 461.2
Coefficient of Variation = 16.7 %

Table 14. Stiffness ACHMBC samples from - Job #060641

Sample #	Layer Position	Tensile Stiffness at 3/4 of Tensile Stress (psi)	Mean (psi)	Standard Deviation	Coefficient of Variation (%)
16	Top	2573.0	2447.6	118.5	4.8
	Top (II)	2287.0			
	Middle	2468.5			
	Bottom	2461.7			
17	Top	2403.7	2633.9	322.2	12.2
	Middle	3002.2			
	Bottom	2495.9			
18	Top	3061.1	3078.6	838.7	27.2
	Top (II)	2491.9			
	Middle	2490.8			
	Bottom	4270.4			
19	Top	3042.9	2653.7	339.8	12.8
	Middle	2416.3			
	Bottom	2501.8			
20	Top		2839.0	364.1	12.8
	Middle	3096.4			
	Bottom	2581.5			

Mean Tensile Stiffness at 3/4 of the maximum tensile stress = 2730.6 psi
Standard Deviation = 238.9
Coefficient of Variation = 8.7 %

Table 15. Stiffness ACHMBC samples from - Job #60105

Sample #	Layer Position	Tensile Stiffness at 3/4 of Tensile Stress (psi)	Mean (psi)	Standard Deviation	Coefficient of Variation (%)
1	Middle	2372.9	2619.3	354.2	13.5
2	Middle	2676.6			
3	Middle	3124.1			
4	Middle	2320.3			
5	Middle	2952.4			
6	Middle	2150.7			
7	Middle	2737.8			

Mean Tensile Stiffness at 3/4 of the maximum tensile stress = 2619.3 psi
Standard Deviation = 354.2
Coefficient of Variation = 13.5 %