PB2002-101484

Report No. KS-01-3

FINAL REPORT

MECHANISTIC EVALUATION OF SUPERPAVE LEVEL 1 MIXTURES

Rajesh Kaldate
Mustaque Hossain, Ph.D., P.E.

Kansas State University
Manhattan, Kansas

SEPTEMBER 2001

KANSAS DEPARTMENT OF TRANSPORTATION

Division of Operations
Bureau of Materials and Research

REPRODUCED BY: NTIS
U.S. Department of Commerce
National Technical Information Service
Springfield, Virginia 22161

1. Report No. KS-01-3	**2. Government Accession No.**	**3. Recipient Catalog No.**
4 Title and Subtitle MECHANISTIC EVALUATION OF SUPERPAVE LEVEL 1 MIXTURES		**5 Report Date** September 2001 **6 Performing Organization Code**
7. Author(s) Rajesh Kaldate and Mustaque Hossain, Ph.D., P.E.		**8 Performing Organization Report No.** FHWA-KS-01-3
Performing Organization Name and Address Kansas Department of Transportation Bureau of Materials and Research, Research Unit 2300 Southwest Van Buren Street Topeka, Kansas 66611-1195		**Work Unit No. (TRAIS)** **Contract or Grant No.** C948
Sponsoring Agency Name and Address Federal Highway Administration 3300 South Topeka Boulevard, Suite 1 Topeka, Kansas 66611-2237		**Type of Report and Period Covered** Final Report 06/01/1996 – 08/17/2000 **Sponsoring Agency Code** RE-0119-01

Supplementary Notes
Prepared in cooperation with the U.S. Department of Transportation, Federal Highway Administration

Abstract

This research involved the mechanistic analysis of two recently built pavements using 2 inch (in.) and ¾-in. maximum nominal size Superpave mixtures. Fatigue life equations were developed for these mixtures and a conventional mixture of the KDOT based on the laboratory flexural fatigue tests on asphalt concrete beams. Field deflection tests were done on a 1000-foot (ft.) section on each pavement with a Dynatest 8000 FWD. The deflection results were used to back-calculate layer moduli values. The mechanistic responses namely, horizontal tensile strain at the bottom of the asphalt concrete layer and vertical compressive strain on top of the subgrade layer under an 18 kip Equivalent Single Axle Load (ESAL) were computed for each pavement using an elastic layer analysis software. Fatigue life, in term repetitions of 18 kip ESALs, was predicted for these pavements with the fatigue equations developed in this study. The results were then compared with the life predictions done by the equations developed by the Strategic Highway Research Program (SHRP) and the Asphalt Institute. The results show that the predicted lives obtained by Kansas State University and SHRP equations are close for ¾- in. Superpave mix while those obtained by the KSU and Asphalt Institute equations are close for the conventional 2-in. mix. The 2-inch Superpave mix appeared to have a higher predicted fatigue life than the ¾-in. mix. Overall, the Superpave mixtures appeared to have far superior predicted fatigue lives than the conventional asphalt mix used by KDOT in the past. The predicted life of KDOT conventional mix under similar strains was almost one-third to one-quarter of the Superpave mixes. Based on the analysis of maximum shear stresses to evaluate the rutting potential in these Superpave pavements, it is felt that the required maximum shear stress values in the Superpave Level II mix design for tertiary creep evaluation are not realistic. Further studies are recommended in this area.

Key Words Pavement, Asphalt Concrete Beams, SHRP, Superpave, Rutting, HMA, Air Voids, and QC/QA		**Distribution Statement** No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161	
Security Classification (of this report) Unclassified	**Security Classification (of this page)** Unclassified	**No. of pages** 130	**Price**

Form DOT F 1700.7 (8-72

MECHANISTIC EVALUATION OF SUPERPAVE LEVEL 1 MIXTURES

Final Report

Prepared by

Rajesh Kaldate
Graduate Research Assistant
Kansas State University

and

Mustaque Hossain, Ph.D., P.E.
Associate Professor
Kansas State University

A Report on Research Sponsored By

THE KANSAS DEPARTMENT OF TRANSPORTATION
TOPEKA, KANSAS

KANSAS STATE UNIVERSITY
MANHATTAN, KANSAS

September 2001

NOTICE

The authors and the state of Kansas do not endorse products or manufacturers. Trade and manufacturers names appear herein solely because they are considered essential to the object of this report.

This information is available in alternative accessible formats. To obtain an alternative format, contact the Office of Transportation Information, Kansas Department of Transportation, 915 SW Harrison Street, Room 754, Topeka, Kansas 66612-1568 or phone (785) 296-3585 (Voice) (TDD).

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the views or the policies of the state of Kansas. This report does not constitute a standard, specification or regulation.

ABSTRACT

This research involved the mechanistic analysis of two recently built pavements using 2 inch (in.) and ¾-in. maximum nominal size Superpave mixtures. Fatigue life equations were developed for these mixtures and a conventional mixture of the Kansas Department of Transportation (KDOT) based on the laboratory flexural fatigue tests on asphalt concrete beams. Field deflection tests were done on a 1000-foot (ft.) section on each pavement with a Dynatest 8000 Falling Weight Deflectometer (FWD). The deflection results were used to back-calculate layer moduli values. The mechanistic responses namely, horizontal tensile strain at the bottom of the asphalt concrete layer and vertical compressive strain on top of the subgrade layer under an 18 kip Equivalent Single Axle Load (ESAL) were computed for each pavement using an elastic layer analysis software. Fatigue life, in term repetitions of 18 kip ESALs, was predicted for these pavements with the fatigue equations developed in this study. The results were then compared with the life predictions done by the equations developed by the Strategic Highway Research Program (SHRP) and the Asphalt Institute. The results show that the predicted lives obtained by Kansas State University (KSU) and SHRP equations are close for ¾-in. Superpave mix while those obtained by the KSU and Asphalt Institute equations are close for the conventional 2-in. mix. The 2-inch Superpave mix appeared to have a higher predicted fatigue life than the ¾-in. mix. Overall, the Superpave mixtures appeared to have far superior predicted fatigue lives than the conventional asphalt mix used by KDOT in the past. The predicted life of KDOT conventional mix under similar strains was almost one-third to one-quarter of the Superpave mixes. Based on the analysis of maximum shear stresses to evaluate the rutting potential in these Superpave pavements, it is felt that the required maximum shear stress values in the Superpave Level II mix design for tertiary creep evaluation are not realistic. Further studies are recommended in this area.

KDOT is implementing a new Quality Control/ Quality Assurance (QC/QA) testing program for the pilot Superpave projects and is in the process of developing statistical performance based specifications. The QC/QA program and specifications used for pilot projects were created by drawing upon earlier Hot Mix Asphalt (HMA) project experience as well as guidelines of other agencies. Therefore, detailed documentation and statistical analysis of the QC/QA data acquired for these mixtures on pilot projects are necessary.

The existing KDOT QC/QA program and specifications are described in terms of various requirements of statistical performance based specifications. The test results of volumetric parameters (namely air voids, binder content and compacted pavement density) of 13 mixes on four pilot projects obtained under existing testing program were statistically analyzed. It must be noted that the KDOT specifications, particularly pay factors, were being continuously modified throughout the study period. Therefore, the study results and analysis had to be modified time and again accordingly. KDOT pay factors were calculated and compared with American Association of State Highway and Transportation Officials (AASHTO), National Highway Institute (NHI) and old KDOT pay factors. Based on this analysis, inferences about these Superpave mixes and existing specifications have been drawn. Accordingly, modifications in these have been recommended.

Attempts were made to incorporate air voids as an independent variable in the mechanistic transfer functions developed earlier. This resulted in modification of original fatigue life equations. But the results are not reliable in statistical terms. Yet basic methodology for the development of statistical performance based pay factors has been explained using results obtained in this study. Further studies using a factorial experiment design with at least binder content and air voids as variables are strongly recommended.

ACKNOWLEDGMENTS

The financial support for this study was provided by the Kansas Department of Transportation (KDOT). The authors wish to thank Richard L. McReynolds, P.E., Project Monitor, Richard E. Kreider Jr., P.E., Quality Engineer, and Glenn A. Fager, P.E., Bituminous Research Engineer, all of KDOT, for their cooperation and advice during this study. The specifications part of this study would not have been complete without the valuable input by Rodney G. Maag, P.E., Field Engineer, KDOT. Cooperation of Albert Oyerly and the KDOT Falling Weight Deflectometer (FWD) crew in deflection testing is hereby acknowledged. Participation of Donald Bruns in laboratory testing is also acknowledged.

PROTECTED UNDER INTERNATIONAL COPYRIGHT
ALL RIGHTS RESERVED
NATIONAL TECHNICAL INFORMATION SERVICE
U.S. DEPARTMENT OF COMMERCE

Reproduced from
best available copy.

TABLE OF CONTENTS

ACKNOWLEDGEMENT iv

LIST OF TABLES viii

LIST OF FIGURES x

1 INTRODUCTION 1
- 1.1 Introduction 1
- 1.2 Problem Statement 2
- 1.3 Objectives of the Study 3
- 1.4 Organization of the Report 4

2 BACKGROUND AND SIGNIFICANCE OF WORK 5
- 2.1 Mechanistic-Empirical Method of Pavement Analysis 5
- 2.2 Failure Criteria Based on Flexible Pavement Distresses 6
  - 2.2.1 Low Temperature Cracking 6
  - 2.2.2 Permanent Deformation 6
  - 2.2.3 Fatigue Cracking 8
- 2.3 Selection of Fatigue Cracking as a Distress Mode 9
- 2.4 Fatigue Testing of Asphalt Concrete 9
- 2.5 Improved Fatigue Test Procedure and Equipment in SHRP Research 11
  - 2.5.1 Specimen Size 11
  - 2.5.2 Test Equipment 11
  - 2.5.3 Specimen Testing 12
  - 2.5.4 Analysis of Results 12
- 2.6 Flexural Fatigue Tests on Superpave Level 1 Mixtures 12

3 LABORATORY FATIGUE TESTS: PROCEDURES AND ANALYSIS 13
- 3.1 Fatigue Testing Modes 13
  - 3.1.1 Controlled Stress 13
  - 3.1.2 Controlled Strain 15
- 3.2 Load Variables 17
  - 3.2.1 Load History 17
  - 3.2.2 Load Rate 17
  - 3.2.3 Patterns of Applying Loads 18
- 3.3 Definition of Failure 20
- 3.4 Test Temperature 21
- 3.5 Analysis Methods 21

3.5.1 Linear Fatigue Life Relationships 21

4 RESEARCH ACCOMPLISHED 24
4.1 Selection of Projects and Mixes for the Study 24
4.2 Research Approach for Mechanistic Evaluation 24
4.2.1 SM-2C 25
4.2.2 SM-1B 26
4.2.3 Control Mixture (BM-1B) 27
4.3 Asphalt Beam Sample Preparation 28
4.4 Testing of Samples 30

5 DATA ANALYSIS AND DISCUSSION OF RESULTS 35
5.1 Data Analysis 35
5.2 Results of Correlation Analysis 37
5.3 Developing Fatigue Life Equations 39
5.4 Performance Prediction Using Transfer Functions 43
5.5 Field Data Collection 43
5.6 Pavement Modeling and Back-calculation of Layer Moduli 44
5.7 Prediction of Fatigue Life 46
5.8 Shift Factor 47
5.9 Discussion of Results 49
5.10 Comparison of Fatigue Lives of Superpave Mixtures 50
5.11 Check for Rutting as a Failure Criteria 53

6 PERFORMANCE-BASED SPECIFICATIONS AND QC/QA TESTING 57
6.1 Historical Background 57
6.2 Concept of Quality Assurance System 57
6.3 Objectives of Statistical Performance-Based Specifications 58
6.4 Advantages of Statistical Specifications 59
6.5 Quality Control Plan 60
6.6 Mix Design 60
6.7 Concept of Normal Distribution 60
6.8 Specification Limits as Control Criteria 62
6.9 Statistical Estimates of Population Parameters 64
6.10 Process Control 66
6.11 Acceptance Plan 67
6.12 Lot and Sample Sizes 68
6.13 Selection of Mixture Characteristics 69
6.14 Sampling Procedure 69
6.15 Testing Methods and Frequency 70
6.16 Validation of QC Test Results 70
6.17 Basis of Acceptance 71
6.18 Measures of Quality 74

6.19 Relating Quality to Perfomance - Development of Pay Adjustment Schedules 74
6.20 KDOT Pay Schedules 76
6.21 Evaluating Risks with Operating Characteristic (OC) Curves 78

7 ANALYSIS OF QC/QA VOLUMETRIC TEST DATA 80
7.1 Data Collection 80
7.2 Validation of Contractor's QC Data 81
7.3 Selection of Volumetric Properties for Analysis 82
7.4 Methods of Computing Sample Statistics 82
7.5 Data Analysis 83
7.6 Pay Factors 87
7.7 Operating Characteristic (OC) Curves 89
7.8 Check for Pay Factors 89
7.9 New KDOT Pay Factors 90
7.10 Relating Volumetric Properties with Mechanistic Test Results 91
7.11 Development of Equations Incorporating Air Voids 93

8 STATISTICAL ANALYSIS: RESULTS AND DISCUSSION 95
8.1 Summary of Test Result Statistics 97
8.2 Interpretation of Test Result Statistics 98
8.3 KDOT's Specification Working Ranges 100
8.4 Discussion of Pay Schedules 101
8.5 Combined Pay Factor for Multiple Quality Characteristics 104
8.6 Volumetric Parameters in Fatigue Life Equations 105
8.7 A Fresh Factorial Experiment 106

9 CONCLUSIONS AND RECOMMENDATIONS 108
9.1 Conclusions 108
9.2 Recommendations 110

REFERENCES 112

LIST OF TABLES

TABLE 4.1 Individual and Combined Aggregate Gradations for SM-2C Mix (K-177) ... 26

TABLE 4.2 Individual and Combined Aggregate Gradations for SM-1B Mix (US-50) ... 27

TABLE 4.3 Individual and Combined Aggregate Gradations for Control Mix (BM-1B) ... 28

TABLE 4.4 Test Matrix for Flexural Fatigue Tests ... 34

TABLE 5.1 Superpave Project K-177 SM-2C Mix at 68° F ... 36

TABLE 5.2 Superpave Project US-50 SM-1B Mix at 68° F ... 37

TABLE 5.3 Control (BM-1B) Mix at 68° F ... 38

TABLE 5.4 Results of Correlation Analysis ... 38

TABLE 5.5 Results of Back-calculation Analysis ... 46

TABLE 5.6 Comparison Among Different Fatigue Models and Shift Factors ... 47

TABLE 5.7 Summary of Asphalt Pavement Strains and Fatigue Lives ... 50

TABLE 5.8 Comparison of the Three Test Mixes in a Theoretical Section with Same Modulus ... 52

TABLE 5.9 Shear Stress, τ_{max} at Various Levels to Evaluate Rutting Potential ... 53

TABLE 6.1 KDOT's Specification Working Ranges (Limits) ... 67

TABLE 6.2 KDOT Pay Schedule for Air Voids (Lot Size of Four Tests) ... 77

TABLE 6.3 KDOT Pay Schedules for Compacted Pavement Density ... 77

TABLE 7.1 Sample Statistics for Air Voids Test Results ... 84

TABLE 7.2 Sample Statistics for Air Voids Test Results in terms of Absolute Deviations from Target Value ... 85

TABLE 7.3 Sample Statistics for Binder Content-
Single Point Test Results 85

TABLE 7.4 Sample Statistics for Binder Content-
Moving Average Test Results 86

TABLE 7.5 Sample Statistics for Percent Compacted Pavement Density 86

TABLE 7.6 Air Voids Pay Factor Statistics 87

TABLE 7.7 Binder Content Pay Factor Statistics 88

TABLE 7.8 Compacted Pavement Density Pay Factor Statistics 88

TABLE 7.9 Simple Check for KDOT Pay Schedules 90

TABLE 7.10 New KDOT Pay Schedule for Air Voids
(Lot Size of Four Tests) 91

TABLE 7.11 Comparison of Original and Modified KDOT Pay Schedules 92

TABLE 8.1 Summary of Test Result Statistics 98

TABLE 8.2 Development of Performance-Based Air Voids Pay Schedule 107

LIST OF FIGURES

FIGURE 3.1 Controlled Stress Fatigue Test 15

FIGURE 3.2 Controlled Strain Fatigue Test 16

FIGURE 3.3 Haversine Pattern for Simple Loading 19

FIGURE 3.4 Block Pattern for Compound Loading 20

FIGURE 4.1 Layer Types and Thicknesses of Different Test Sections 25

FIGURE 4.2 Combined Gradation of the Three Test Mixes 29

FIGURE 4.3 Test Set-Up for Flexural Fatigue Testing of Asphalt Concrete Beams 31

FIGURE 4.4 Strain Measurement Gauge Details 32

FIGURE 4.5 Full-Bridge Circuit Details 33

FIGURE 5.1 Relationship Between Number of Repetitions to Failure and Initial Tensile Strain for SM-2C, SM-1B and BM-1B Mixes 42

FIGURE 5.2 Surface Modulus Plot drawn from a Typical FWD Deflection Basin for a K-177 Test Section 45

FIGURE 5.3 Surface Modulus Plot drawn from a Typical FWD Deflection Basin for a US-50 Test Section 45

FIGURE 5.4 Schematic Diagram of the Hypothetical Pavement Sections Used for Prediction of Fatigue Lives Under Equal Moduli Assumptions 51

FIGURE 5.5 Graphical Comparison of Fatigue Lives for SM-2C, SM-1B and BM-1B Mixes 52

FIGURE 5.6 Shear Stress, τ_{max} due to 22.5 kip Load at Various Levels in K-177 (SM-2C) Section 54

FIGURE 5.7 Shear Stress, τ_{max} due to 22.5 kip Load at Various Levels in US-50 (SM-1B) Section 55

FIGURE 5.8 Typical Relationship of Plastic (Permanent) Strain, ε_p, and Load Repetitions, N, for Mixes with Tertiary Creep 56

FIGURE 6.1 Concepts of Accuracy, Precision and Bias 62

FIGURE 6.2 Central Tendency and Variability Control....62

FIGURE 6.3 Normal Distribution with Specification Limits 64

FIGURE 6.4 Probability Distribution of y and Sampling Distribution of y....65

FIGURE 6.5 Typical Quality Control Chart for Binder Content (for SM-1B, US-50) 67

FIGURE 6.6 Distribution of Absolute Deviation in Comparison with Standard Normal Distribution....72

FIGURE 6.7 Typical Distribution of Absolute Deviation for a Sample Size of Four....73

FIGURE 6.8 Conventional OC Curve 79

FIGURE 6.9 Typical OC Curve with an Adjusted Pay Schedule....79

FIGURE 8.1 Original and Modified KDOT Air Voids Pay Schedules with Proposed Pay Equation (Lot Size of Four Tests)....103

CHAPTER 1

INTRODUCTION

1.1 Introduction

The Strategic Highway Research Program (SHRP) was established by Congress in 1987 as a five year, $150 million research program to improve the performance and durability of roads in the United States. The final product is a new system called Superpave, short for **Su**perior **Per**forming Asphalt **Pave**ments. Superpave represents an improved system for specifying asphalt binders and mineral aggregates, developing asphalt mixture design, and analyzing and establishing pavement performance prediction. It incorporates performance-based asphalt materials characterization with the design environmental conditions to improve performance by controlling rutting, low temperature cracking and fatigue cracking (1). The Superpave binder specification and mix design system include various test equipment, test methods and criteria.

The unique feature of the Superpave system is that it is a performance-based specification system. The tests and analysis have direct relationships with field performance. For example, the Superpave asphalt binder tests measure physical properties that can be related directly to field performance by engineering principles.

Superpave mix design and analysis is performed at one of three increasingly rigorous levels with each level providing more information about the mixture's performance capabilities. Superpave Level 1 mix design is an improved material selection and volumetric mix design process. A

Superpave Level 1 design involves selecting asphalt and aggregate materials that meet the Superpave specifications, and then conducting a volumetric analysis of hot mix asphalt (HMA) specimens compacted with the Superpave gyratory compactor. Level 2 mix design procedures use the volumetric mix design as a starting point and include a battery of performance tests to arrive at a series of performance predictions. Level 3 Mixture design includes a more comprehensive array of tests and results to achieve a more reliable level of performance prediction (1).

1.2 Problem Statement

The majority of the state highway system in Kansas serves a low level of traffic volume. As such most of the Superpave mix designs that are being used at present as well as those that will be proposed in the near future will fall under Level 1 category of Superpave mixture design. Current Superpave mixture design requirements for Level 1 mixes consists of:

a) Mixture volumetric requirements,

b) Dust proportion and

c) Moisture susceptibility (2).

Thus, no mechanistic test is available for Level 1 mixtures to predict their field performance directly by engineering principles. This has caused much concern, since the unique feature of the Superpave system (i.e., it is a performance-based specification system) does not seem to cover Level 1 mixtures. For maximizing the benefits of Superpave Level 1 mixtures, some type of mechanistic evaluation was deemed necessary.

KDOT is also pursuing a new quality control/quality assurance (QC/QA) program for the Superpave mixes. As Superpave itself is a performance-based system, the QC/QA program must be developed as a part of statistical performance-based specifications. For the purpose of pilot Superpave projects the QC/QA program is designed by drawing upon historical HMA project experience as well as guidelines of other agencies. It is therefore, desirable to have detailed documentation (for establishing database) and statistical analysis of the QC/QA test data acquired for these mixtures on pilot projects.

1.3 Objectives of the Study

KDOT is evaluating Superpave Level 1 mixtures used on its pilot projects, in the newly built Kansas Testing Laboratory for Civil and Highway Infrastructure of the Kansas State University. The major objective of this research project was to mechanistically evaluate the Superpave Level 1 mixtures used in constructing pilot projects under KDOT's QC/QA program. The mechanistic testing involved four-point flexural fatigue tests under repetitive sinusoidal load on asphalt concrete beam specimens. KDOT fabricated these beam specimens from Superpave Level 1 mixes used on two KDOT QC/QA pilot projects in Manhattan and Hutchinson, Kansas. A mechanistic transfer function in the form of a fatigue life equation relating the fatigue life in repetitions to the tensile strain and possibly other mix properties was expected from this analysis. The predicted performance of these Superpave mixes were compared with that of the conventional mix determined in an earlier KDOT project titled, "A Flexural Fatigue of Asphalt-Rubber Mixes."

The results of QC/QA tests conducted on seven Level 1 mixtures used on these two projects, under KDOT's new QC/QA testing program were to be documented to form a database. Especially, test results of volumetric parameters like air voids and asphalt content were to be covered. Later the scope of the project was extended to cover the QC/QA test results of six more mixes from two other pilot projects executed during the 1997 construction season. These two projects are located in Saline County and Sedgwick County, Kansas. Statistical analysis was planned to be done on this data to draw inferences about the Superpave mixes, existing QC/QA program, specifications and pay factors. The volumetric properties were expected to be possibly correlated with the mechanistic test results. Based on this analysis modifications in the QC/QA program and specifications were planned to be suggested if necessary. The basic framework for the development of statistical performance-based specifications and pay factors for these Superpave mixes would be explained.

1.4 Organization of the Report

This report is divided into nine chapters. Chapter 1 is the introduction to the problem. Chapter 2 describes the background and significance of the work. Literature review on the development of fatigue tests in earlier research is also included. Chapter 3 discusses the fatigue testing and data analysis procedures. Chapter 4 describes the pilot projects, test mixtures, laboratory-testing details, and results obtained. In Chapter 5 analysis of lab and field data and discussion of results is presented. Chapter 6 describes statistical performance based specifications, KDOT's pilot QC/QA program and pay factors. Chapter 7 describes the statistical analysis of the QC/QA data collected on the Superpave pilot projects, followed by Chapter 8, which presents the results and discussions. Finally, Chapter 9 presents the conclusions and recommendations based on this study.

CHAPTER 2

BACKGROUND AND SIGNIFICANCE OF WORK

A majority of state highway agencies are currently designing and analyzing pavements using the current American Association of State Highway and Transportation Officials (AASHTO) Design Guide Methodology that uses structural number based on the equivalent single axle loads (ESALs) and the road-bed soil resilient modulus. The pavement design is then accomplished using layer coefficients and trial layer thickness' to build the required structural number. This method uses regression equations based on American Association of State Highway Officials (AASHO) Road Test results (3). Earlier studies attempted to predict the failure of road sections using structural number and soon found that structural number was not a good measure of time to failure (4). This lack of a strong relationship between structural number and time to failure is a driving force in moving toward mechanistic methods of pavement design.

2.1 Mechanistic-Empirical Method of Pavement Analysis

The mechanistic-empirical methods of pavement design are based on the mechanics of materials that relates an input, such as a wheel load, to an output or pavement response, such as stress or strain (3). The response values are used to predict distresses through transfer functions based on laboratory test and field performance data. Thus, a number of failure criteria, each directed to a specific distress, must be established. This is in contrast to the current AASHTO pavement design method, where general pavement condition is described by the present serviceability index (PSI). It is important to acknowledge that mechanistic-empirical flexible pavement design is a reality. A few state agencies like Kentucky, Illinois, Washington and North Carolina; have developed mechanistic design procedures (4). The Part IV of the 1986 and 1993 AASHTO Guide has described the framework for the mechanistic-empirical design procedures. The latest AASHTO Pavement Design Guide scheduled to be available in year 2003 is expected to be fully mechanistic.

2.2 Failure Criteria Based on Flexible Pavement Distresses

It is generally agreed that fatigue cracking, rutting, and low temperature cracking are the three principal types of distresses to be considered for flexible pavement design (3). These are the distresses analyzed in Superpave research also. Several currently available mechanistic pavement design methods, such as the Shell Pavement Design Guide and DAMA by the Asphalt Institute, use some of these distresses, primarily fatigue cracking and permanent deformation, in defining the failure criteria.

2.2.1 *Low Temperature Cracking*

Low temperature cracking is caused by adverse environmental conditions rather than by applied traffic loads. It is characterized by intermittent transverse cracks that occur at a consistent spacing. Low temperature cracks occur when an asphalt pavement layer shrinks in cold weather. The tensile stresses developed by this shrinkage exceed the tensile strength of the asphalt concrete and it cracks. The asphalt binder plays a key role in low temperature cracking (1). Thus it is more of a material selection problem and has been adequately covered in Superpave asphalt binder specification and material selection system.

2.2.2 *Permanent Deformation*

Permanent deformation is characterized by a surface cross section no longer in its design position (1). Wheel path rutting is the most common form of permanent deformation. Rutting occurs only on flexible pavements. One major finding of the AASHO Road Test was that rutting occurred principally due to decrease in thickness of component layers. About 91 percent of the rutting occurred in the pavement itself with 32 percent in the surface, 14 percent in the base, and 45 percent in the subbase (3). This rutting results from an asphalt mixture without adequate shear strength to resist repeated heavy loads. A weak mixture will accumulate small, but permanent deformations with each truck pass, eventually forming a rut characterized by a downward and lateral movement of the mixture (1). The rutting may occur in the asphalt surface course, or the rutting that shows in the surface may be caused by a weak underlying asphalt course. Using a failure criteria based on correlation with road tests or field performance, various mechanistic models for rutting distress have been developed by Asphalt Institute and Shell, etc.. The approach in these methods is based on the contention that if the *quality of the surface and base courses is well controlled, rutting can be reduced to a tolerable level by limiting vertical compressive strain on the top of subgrade.* However it is felt that, the transfer functions for the rutting of HMA and granular materials are marginal and require further development. Thus it must be noted that rutting in the surface layer relating to permanent deformation in thick HMA layer is best considered by material selection and mix design practices like Superpave (5). A similar conclusion drawn by Wisconsin Department of Transportation (WSDOT) states that rutting in asphalt concrete is primarily a mix design issue (6). It is suggested that the design procedure should include a check for the rutting potential after the thickness design is completed using a fatigue failure model. If unsatisfactory, the selection of different mixture design procedures and practices should be made until the rut depth is reduced to the acceptable limit (3).

2.2.3 *Fatigue Cracking*

Fatigue cracking occurs in asphalt pavements when the applied loads are repeated on the asphalt materials, causing cracks to form due to fatigue. An early sign of fatigue cracking is intermittent longitudinal cracks in the traffic wheel path. Fatigue cracking is usually caused by a number of factors occurring simultaneously. Obviously repeated heavy loads must be present. Thin pavements are prone to high deflections under heavy wheel loads. High deflections lead to the increase of horizontal tensile stresses at the bottom of the asphalt layer, leading to fatigue cracking. Often, fatigue cracking is merely a sign that a pavement has sustained the design number of load applications, in which case the pavement's useful life is simply exhausted. Thus assuming that the fatigue cracking occurs at the end of the design period, it would be considered a natural progression of the pavement design strategy. Since pavements must be designed for the repeated loadings caused by traffic if they are to give satisfactory service over a reasonable period of time (1). If the observed cracking occurs much sooner than the design period, it may be a sign of under design. Fatigue is the dominant pavement failure mechanism for flexible pavements as reported by studies for developing mechanistic model at both Washington and North Carolina State Department of Transportations' (4).

The best ways to overcome fatigue cracking are: choose adequate design ESALs, keep the subgrade dry, design for adequate thickness, use pavement materials not excessively weakened by moisture and use HMA having sufficient resilience to withstand normally expected deflections. Only the last two items can be strictly addressed using a material selection and mixture design system like Superpave (1). Thus to design satisfactorily for the fatigue distress, it is absolutely necessary to develop a mechanistic model that relates an input pavement response like tensile strain to the actual field fatigue behavior of the asphalt mixes.

2.3 Selection of Fatigue Cracking as a Distress Mode

In view of the above, it was decided to consider fatigue cracking as the critical distress mode for the flexible pavements. As such the mechanistic modeling efforts in this study were aimed at developing a transfer function based on fatigue cracking. Besides as one of the pilot projects is a composite pavement, rutting was expected to be significant. Thus, it was decided to check the rutting potential of the Superpave mixes.

Failure criteria were available from several sources. This study used the fatigue failure criteria developed under National Cooperative Highway Research Program (NCHRP) Project 1-10 B, which deals with the development of pavement structural subsystems. This fatigue criteria expects that fatigue cracking would be limited to approximately 20 to 25 percent of the total area. This failure criteria for fatigue cracking is based on empirical data taken from the AASHO Road Test observations (7).

2.4 Fatigue Testing of Asphalt Concrete

Because of its complex nature, fatigue failure is difficult to analyze and to design against. The fatigue analysis system developed by SHRP-A003A researchers recognizes that a given mixtures fatigue performance in situ depends on critical interactions between mixture properties and in situ conditions (e.g. pavement structure, traffic loading or environmental conditions) (8). Miners cumulative damage concept has been widely used to predict fatigue cracking. It is widely accepted that the maximum tensile strain at the underside of the asphalt concrete layer governs the initiation of fatigue cracking in situ. Thus, the allowable number of load repetitions that a pavement can service is related to this tensile strain.

A host of test methods have been developed for the fatigue testing of asphalt concrete. F.N. Hveem, California Division of Highways, was one of the first to investigate the effects of repeated loadings on asphalt pavements. When investigating pavement flexibility, he developed a fatigue testing device capable of testing small beams cut from asphaltic pavements (9).

Various other tests were developed in the early stages of fatigue testing by Hennes and Chen, Nijboer, Van der Poel, Monismith and Pell (10). In the beam tests, third-point loading mode was used by Deacon at University of California at Berkeley (UCB) (3). The same loading mode was used in this study. The advantage of the third point loading over the center point loading is the existence of a constant bending moment over the middle third portion of the specimen, so any weak spot due to non-uniform material properties should show up in test results.

As fatigue testing has progressed, engineers have become more aware of the complexities entailed and many approaches for analyzing the fatigue life of pavements have been adopted. Monismith and several other researchers have developed and refined test methods using various types of beam specimens (10). The specimen size used by Deacon at UCB was 1.5-in. width and depth and 15 in. long. To reduce the test variability in its studies, the Asphalt Institute increased the width and depth to 3-in. (3). Maupin and Freeman recommended a simplified fatigue test based on the indirect tensile mode of loading asphalt concrete specimens (11). In the SHRP, an accelerated performance-related test method for asphalt mixture fatigue was studied in detail (12).

2.5 Improved Fatigue Test Procedure and Equipments in SHRP Research

In the SHRP research, two major improvements were made to the flexural beam fatigue test procedure and equipment.

2.5.1 *Specimen Size*

The square cross section of the pilot test beam was increased from 1.5-in. x 1.5-in. to a rectangular cross-section with a 2.5-in. width and 2.0-in. height. Although, the beam length was restricted to 15-in., the effective beam span was increased from the original 12 to 14 in. in order to minimize shear deformation in the beam.

2.5.2 *Test Equipment*

Specific changes in the test equipment included the following:

a) Automating the specimen clamping procedure by the use of torque motors, simplified mounting and reduced the set-up time by 80 percent for each test.

b) Improvements in the linear and torsional bearings minimized risk of any extraneous stress in the beam specimen.

c) Redesign of various components for accommodating the module within the Universal Testing Machine (UTM).

The new fatigue test equipment, with its hydraulic pressure system, has a more precise control of the stress or strain induced in the specimen than the earlier electropneumatic system. Sinusoidal loads applied at up to 25 Hertz (Hz) frequency, with or without rest periods, can easily be achieved at temperatures ranging between 14° and 104° Fahrenheit (F). After specimen loading, the entire test, including temperature, loading, data acquisition, and data reduction, is controlled by various automated testing system software like FATIGUE. These improvements significantly improved the repeatability of the test in relation to the pilot test program. The use of rolling-wheel compaction in this research virtually eliminated fracturing of aggregate in compacted specimens, which was observed with kneading compaction used earlier.

2.5.3 ***Specimen Testing***

Beam specimens ready for testing were stored at the required temperature for at least 2 hours. All specimens in this test program were tested at 68°F, except for the temperature equivalency study. Specimens were tested under the controlled-strain mode of testing at 10 Hz frequency, corresponding to a sinusoidal load of 0.1 seconds, with no rest periods.

2.5.4 ***Analysis of Results***

Test data were analyzed using the FATIGUE computer program to compute the stress, strain, stiffness, phase angle, and cumulative dissipated energy as functions of the number of load cycles. Fatigue life was defined as the number of cycles corresponding to a 50 percent reduction in initial stiffness; initial stiffness was measured at the 50th load cycle.

2.6 Flexural Fatigue Tests on Superpave Level 1 Mixtures

In 1996, KDOT decided to conduct mechanistic evaluation of Superpave Level 1 mixtures to be used in constructing pilot projects of its QC/QA program. The major focus of this research is to develop transfer functions based on fatigue failure, by conducting third-point flexural fatigue tests on asphalt concrete beams fabricated from the subject test mixes.

After a detailed literature review on fatigue testing methods, the testing was conducted using parameters that offered the most advantages, where a choice was possible. The choice of some test parameters like a rest period between loading, was restricted because of the test equipment constraints. In general, the test protocol followed was almost similar to the one described in the final report for the earlier project referred above.

CHAPTER 3

LABORATORY FATIGUE TESTS: PROCEDURES AND ANALYSIS

This chapter primarily discusses the various laboratory fatigue testing modes and test data analysis procedures. The particular type of testing mode and data analysis method adopted for the study was designated with reasons for their choice. Although mechanistic evaluation based on fatigue testing is the objective, in some instances the discussion includes the use of the test results in pavement design.

3.1 Fatigue Testing Modes

Laboratory fatigue testing methods predominately have used two modes of controlled loading for bituminous specimens. These modes, controlled stress and controlled strain, are designed to hold either the stress or strain at a desired value while an unconstrained variable is monitored.

3.1.1 *Controlled Stress*

The controlled stress mode of testing requires that a pre-decided load of constant value be repeatedly applied to the specimen throughout the testing process as illustrated in Figure 3.1. When this testing mode is used, the deflection of the specimen is monitored to determine the strain corresponding to the applied load. Obviously as the HMA becomes weaker under the action of repeated loads, the strain increases with the number of load repetitions. The controlled stress test mode is used to test the bituminous materials, which provide the primary structural support of the roadway, i.e., it is applicable to thicker pavements, wherein the HMA materials are placed in thickness greater than 6 in. (3).

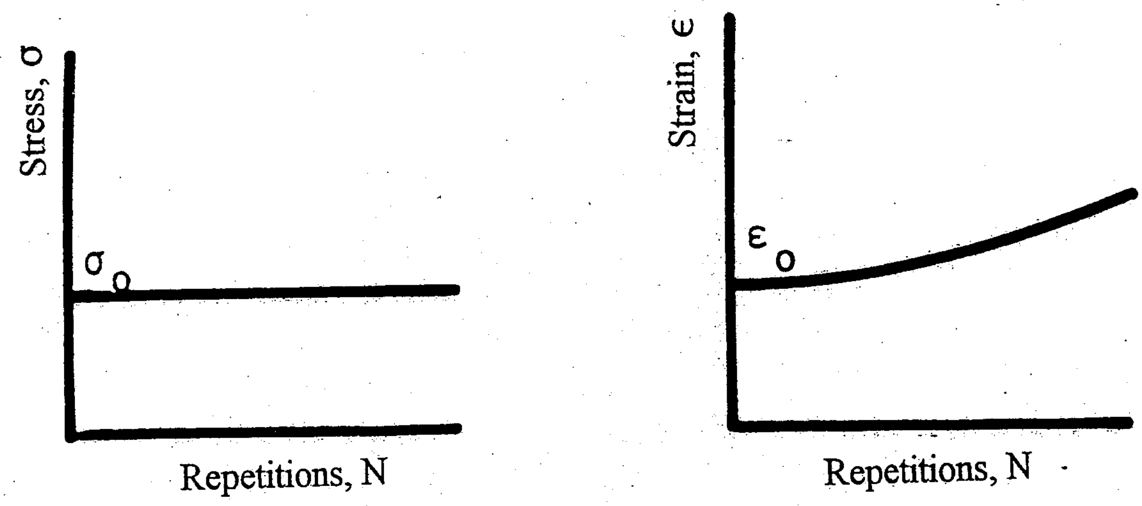


FIGURE 3.1 Controlled Stress Fatigue Test (Source: 3)

3.1.2 *Controlled Strain*

The controlled strain mode of testing is performed by maintaining the strain at a desired level and monitoring the corresponding stress. In this mode, a predetermined value of deflection, or strain, is placed on the specimen, and the load required to produce this deflection is recorded throughout the test. Graphic illustrations of strain vs. cycles to failure and stress vs. cycles to failure are given in Figure 3.2. As the HMA becomes weaker under the action of repeated loads, the stress level needs to be decreased with the number of load repetitions to maintain constant strain. The controlled strain test is used to test bituminous materials used as thin surface layers less than 2 in. thick. The reason being that the surface layer of a bituminous roadway gives little if any structural support (3). Also deflections in this layer are governed by the characteristics of the underlying layers such as subgrade, base material, and bituminous base and are not affected by the decrease in stiffness of HMA.

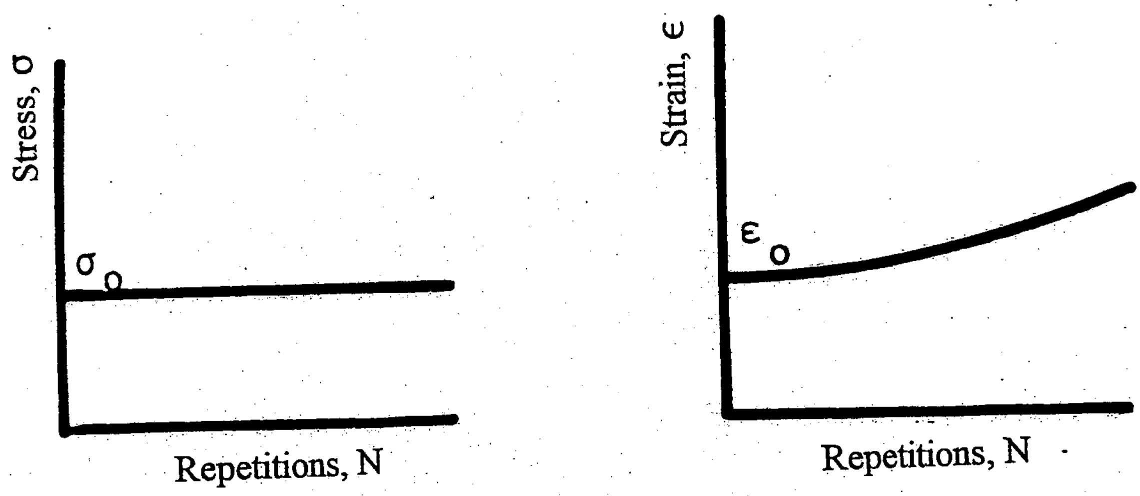


FIGURE 3.2 Controlled Strain Fatigue Test (Source: 3)

It is suggested that the use of controlled stress mode yields more conservative results, since both stress and strain are larger in the controlled stress test (3). The use of controlled stress has the further advantage that failure occurs more quickly and can be more easily defined, while an arbitrary failure criterion, such as stress equal to 50 percent of the initial stress, is frequently used for the controlled strain test (3).

In keeping with the above discussion, it was decided to conduct a controlled stress type of test for this study. The constant stress type of loading is applicable to SM-2C and BM-1B mixes under study, wherein the hot mix asphalt base layer is more than 6 inches thick and is the main load-carrying component. The constant strain type of loading is inapplicable since no thin pavements with HMA less than 2 in. thick was considered. For intermediate thicknesses like 2.5 in. representative of the SM-1B mix under study, a combination of constant stress and constant strain exists. However, the constant stress type of loading was preferred for uniformity. It was felt that due to the existence of 4-in. conventional asphalt layer below SM-1B, the overall behavior would be similar to a thick pavement. Besides the controlled stress mode seems to be more logically representative of the actual field conditions then constant strain test mode.

3.2 Load Variables

Many types of load variables can be used in a laboratory fatigue test. The primary variables are the load history, the rate of load application, and the pattern of applying the load.

3.2.1 *Load History*

A specimen may be subjected to two types of load history - simple and compound. In simple loading, whether controlled stress or controlled strain, the load condition remains unchanged throughout the fatigue test. In compound loading there are changes in the load condition during the test, with a change being defined as a change in the amount of stress or strain applied to the specimen or a change in the environment, such as an increase or decrease in temperature.

Compound loadings can be preprogrammed to simulate the loadings a pavement receives from traffic; however, the process is quite involved, so simple loadings are more widely used and were preferred in this study.

3.2.2 *Load Rate*

The rate of loading is the number of load applications made over a specified period of time. It has been proven that the fatigue life varies with the rate of loading. Tests by Deacon and Monismith (13) indicated that over loading rates ranging from 30 to 100 repetitions per minute, there was a significant decrease in fatigue life as the loading rate increased. In tests performed by Taylor, it was found that loading rates of less than 200 repetitions per minute caused a greater variation in specimen service life than did higher loading rates (14). In this study a loading period of 0.1 seconds with no rest period was employed to minimize the effect of loading rates on repetitions to failure.

3.2.3 *Patterns of Applying Loads*

The type and duration of the loading i.e. the loading pattern used in the repeated load test must match that existing in actual field conditions as closely as possible. As a wheel load approaches a given point on the pavement, the stress at that point starts increasing from a null value and reaches a peak when the load is exactly on top of it. As the load moves away, the stress drops to zero in an exactly reverse fashion. Therefore it has seemed logical to assume the stress pulse to be a haversine or triangular loading. The load patterns commonly used are block, sinusoidal, and haversine.

The haversine pattern for a simple loading is shown in Figure 3.3. Compound loading tests are done predominately with the block pattern; however, haversine and sinusoidal patterns may also be used (11). There are two ways in which a compound loading can be applied - sequentially or randomly. A sequential compound block-loading pattern is illustrated in Figure 3.4.

The haversine pattern is used mostly for simple loading rather than compound loading. It constitutes the compressive half of the sine curve, in which the simply supported beam is loaded and then enough tension is applied to force it back to the neutral axis. This pattern is preferred over the sinusoidal pattern because it more closely resembles the loadings of roadway pavements. The surface layer of a roadway undergoes both tensile and compressive forces during a wheel loading; however, the compressive forces far outweigh the tensile forces (11). Yet in the matter of fatigue cracking, the tensile forces are far more critical than the compressive forces.

Barksdale investigated the vertical stress pulses measured in the AASHO Road Test. The stress pulse time was related to the vehicle speed by the haversine function, "sin ($\pi/2 + \pi t/d$)", where "t is time" and "d is pulse time". This yielded a pulse time of 0.028 for a vehicle speed of 40 miles per hour (mph), which is very low. Boltzmann calculated the haversine loading pulse time based on "$\sin^2$ ($\pi/2 + \pi t/d$)". This gave a pulse time of 0.1 seconds, which checks more closely with the actual stress pulse measurements in pavements. However, in view of the fact that the vehicle speed varies greatly, it is recommended that a haversine load with the duration of 0.1 second and a rest period of 0.4 seconds be used (3). As per recommendations, a haversine loading with 0.1 second pulse time was used for the testing. However, due to the limitations of the available testing equipment no rest period could be applied between repeated loadings. This should not be a cause for concern, because some experts feels that although the effect of rest period is not known, it is probably insignificant (3). Also the SHRP research described earlier used zero rest period in their fatigue testing.

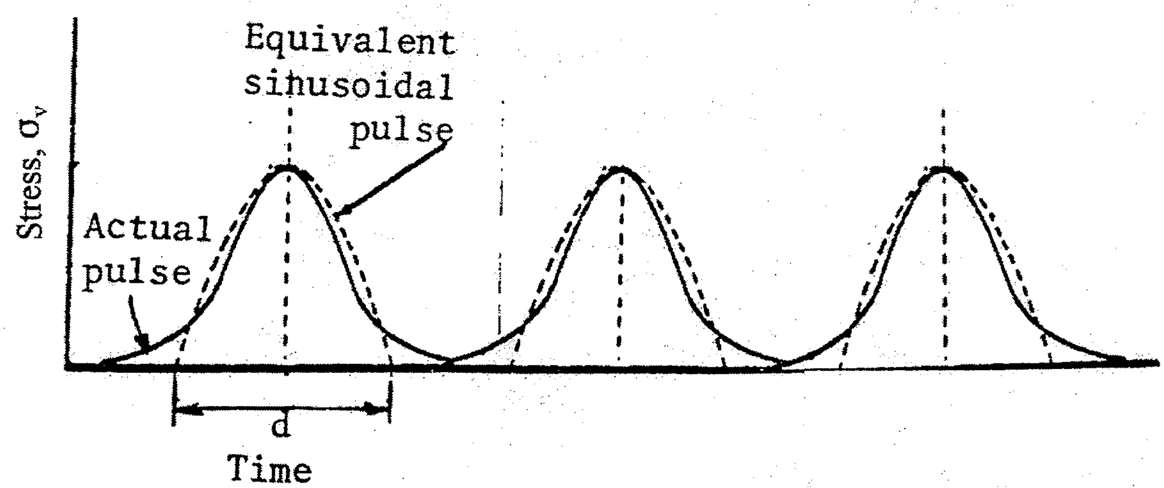


FIGURE 3.3 Haversine Pattern for Simple Loading (Source: 3)

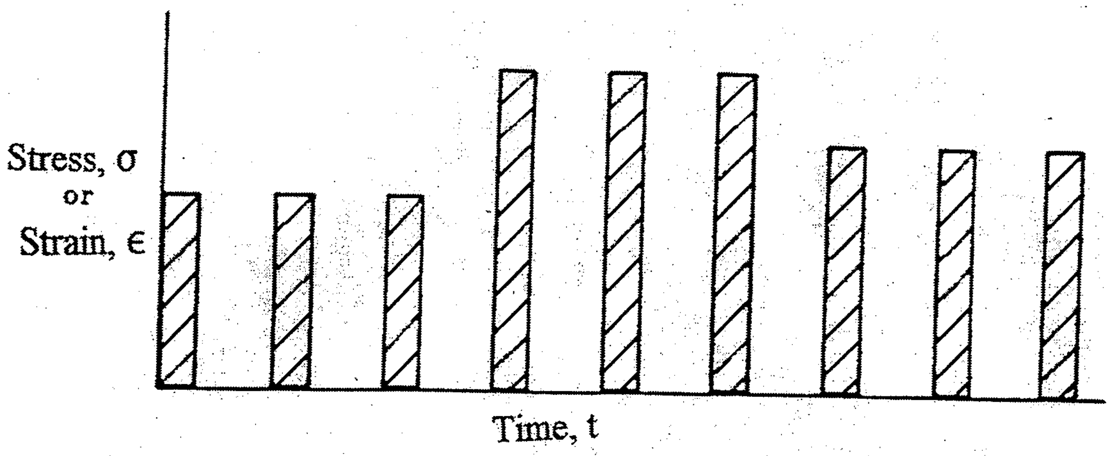


FIGURE 3.4 Block Pattern for Compound Loading (Source: 11)

3.3 Definition of Failure

Failure of a specimen is generally defined as the point at which it no longer has the ability to satisfactorily withstand a desired load (13). For fatigue tests, the failure condition varies depending on the mode of testing used.

In the controlled stress mode of testing, fatigue life is defined as the number of loadings required for the specimen to completely fracture. In the controlled strain mode of testing the dynamic load applied to the specimen is recorded after the first 200 to 300 load applications, and the fatigue life is said to be reached when the dynamic load reduces to a predetermined percentage (50 to 75) of the initial dynamic load. It has been reported by Epps and Monismith that 25 and 50 percent reductions in stiffness correspond to small and extensive crack propagations, respectively (15).

The use of constant stress mode of testing in the study implied a complete fracture of the test beam specimen as the definition of failure. This was also more appropriate as the individual test time was reduced and there was no ambiguity regarding the point of failure occurrence.

3.4 Test Temperature

The new methodology of SHRP-A003A described earlier, proposes limiting fatigue tests at one temperature and expressing the destructive effects of the anticipated traffic in the field as ESALs at that given temperature (68° F was used by SHRP). The approach simplifies testing, which increases productivity and reduces costs (8). As such all specimens in this study were tested at 68° F.

3.5 Analysis Methods

3.5.1 *Linear Fatigue Life Relationships*

In recent years a number of mechanistically based design methodologies for asphalt concrete have been developed. These include Shell International Petroleum Co., Asphalt Institute, SHRP, NCHRP, TRRL, etc. to name a few. In all of these procedures, the major design criteria is the tensile strain at the bottom of the asphalt concrete layer, incorporated to control fatigue cracking in the asphalt layer. Each of these procedures has utilized a specific set of fatigue data (7).

The methods used to analyze the data are the same regardless of the size of the specimen used. The basic equations for extreme fiber stress, stiffness modulus, and extreme fiber strain are listed below (3). The equations apply to a beam of uniform cross section, which is simply supported at the ends and loaded by two symmetrical, concentrated loads applied near the center.

$$\sigma = \frac{3aP}{bd_2} \quad (3.1)$$

$$E_s = \frac{Pa(3\ell^2 - 4a^2)}{4bd^3t} \quad (3.2)$$

$$\varepsilon = \frac{12td}{3\ell^2 - 4a^2} \quad (3.3)$$

where

σ	=	extreme fiber stress (psi)
a	=	2 (reaction span length - distance between load clamps) (in)
P	=	dynamic load applied to deflect beam (lbs)
b	=	specimen width (in)
d	=	specimen depth (in)
E_s	=	flexural stiffness modulus based on deflection (psi)
ℓ	=	reaction span length (in)
I	=	specimen moment of inertia (in^4)
t	=	dynamic deflection of beam center (in)
ε	=	extreme fiber strain of mix in region of equal moment calculated from deflection of beam center (in/in)

For constant strain tests, the stress, σ, and fatigue life, N_f, can be correlated using a least squares regression analysis that results in a linear log-log plot of σ versus N_f (16). This relationship is shown in the form:

$$N_f = K_1(1/\sigma)^n \quad (3.4)$$

where

N_f	=	number of load applications to failure
K_1	=	constant depending on the mix
σ	=	extreme fiber bending stress, psi
n	=	constant (slope of regression line)

With the above equation, the fatigue life for a given bending stress can be estimated.

For constant stress test, a similar relationship can be established for strain, ε, versus fatigue life, N_f (16). This relationship is also obtained from a least squares regression analysis, and is shown as:

$$N_f = K_2(1/\varepsilon)^{n2} \qquad (3.5)$$

where

N_f	=	number of load applications to failure
K_2	=	constant depending on the mix
ϵ	=	initial bending strain based on center point deflection of specimen
n_2	=	constant (slope of regression line)

As the subject study used a constant stress mode of fatigue testing, the data was generally modeled in the form of equation 3.5 to obtain the desired transfer function. Generally the data, in all the earlier studies mentioned above, are expressed in a similar equation.

CHAPTER 4

RESEARCH ACCOMPLISHED

4.1 Selection of Projects and Mixes for the Study

Two recently built Superpave projects of KDOT, K-177 near Manhattan, Kansas, and US-50 near Hutchinson, Kansas, were selected for this study. These projects are designated as pilot projects in KDOT's QC/QA program. The pavement structural section is detailed in Figure 4.1. K-177 has 1 in. SM-1T as surface layer, 8 in. SM-2C as base course, and 8 in. crushed rock subbase. US-50 has a 1 in. SM-1T as a top layer over 1.5 in. of SM-1B base course as a part of the present project. The older structure was a composite section consisting of 4 in. of conventional asphalt layer BM-7 over 9 in. of JRCP. The third, denoted as control section, is a hypothetical design having a 9 in. BM-1B conventional asphalt mix over 8 in. crushed rock subbase and is almost similar to K-177 structure. The various mixes described above using KDOT nomenclature are:

SM-1T: 3/8-in. nominal maximum size Superpave mix (surface layer)

BM-7: 3/4-in. max aggregate size conventional (Marshall) mix (base layer)

JRCP: Jointed Reinforced Concrete Pavement.

Other mix designations namely, SM-2C, SM-1B and BM-1B are described later.

4.2 Research Approach for Mechanistic Evaluation

The research involved fatigue testing of the two Superpave base mixes SM-2C and SM-1B using the process covered by the KDOT special provision 90P-3346-Rev. A conventional asphalt mix, BM-1B was studied as a "control" mix design. The fatigue testing for this mix was done under the KDOT project referred earlier (10) and the results obtained have been incorporated in the present study.

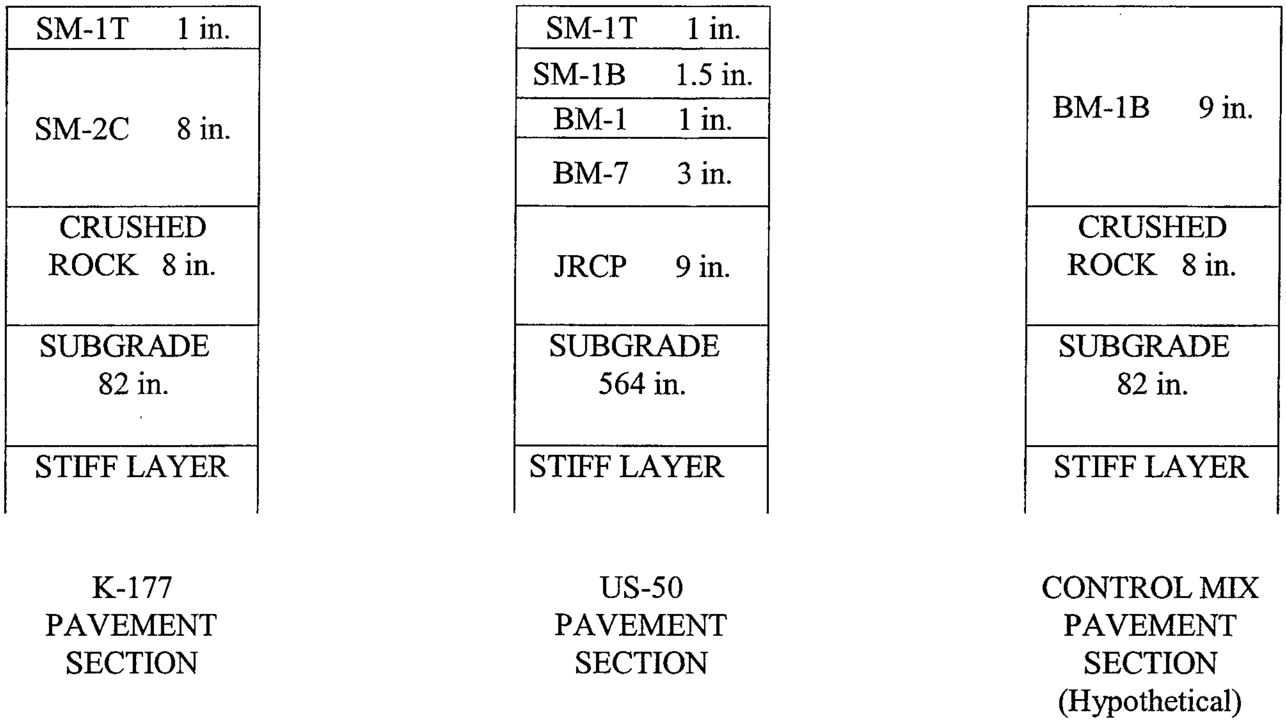


FIGURE 4.1 Layer Types and Thicknesses of Different Test Sections

*Existing conventional asphalt layers over concrete layer.

Notes: SM-1T: 3/8 in. nominal maximum size Superpave mix (surface course);

SM-1B: 2-in. nominal maximum size Superpave mix (base course);

SM-2C: 2-in. nominal maximum size Superpave mix (base course);

BM-1: 3/8-in. maximum aggregate size conventional asphalt (Marshall) mix (surface course);

BM-7: 3/4-in. max aggregate size conventional asphalt (Marshall) mix (base course); and

JRCP: Jointed Reinforced Concrete Pavement

4.2.1 *SM- 2C*

SM-2C is a ¾-inch nominal maximum size Superpave Level 1 mix. The design aggregate structure consisted of 66 percent Martin-Marietta rocks (CS-1) from Riley County, 5 percent manufactured sand from Fogle Quarry, Franklin County, 8 percent coarse sand (SSG-1) and 21 percent fine sand (SSG-2) from Wamego Sand, Wabaunsee County. Table 4.1 shows the individual and combined cumulative aggregate gradation for the test mix (See Figure 4.2). The design binder content was 5.1 percent. The asphalt cement used was an AC-10 produced by Coastal Derby of El Dorado, Kansas.

TABLE 4.1 Individual and Combined Aggregate Gradations for SM-2C Mix (K-177)

Sieve Size	Percent Aggregate Retained				
	CS-1 (66%)	SSG-1 (8%)	SSG-2 (21%)	Manufactured Sand (5%)	Combined Cumulative (100%)
¾ in.	0	0	0	0	0
2 in.	20	0	0	0	13
3/8 in.	50	0	0	0	33
No. 4	92	13	5	0	63
No. 8	95	50	18	18	71
No. 16	95	83	41	57	81
No. 30	96	93	64	76	88
No. 50	96	98	88	89	94
No. 100	96	99	95	94	96
No. 200	96	100	99	96	96.9

4.2.2 *SM-1B*

SM-1B is a 2-in. nominal maximum size Superpave Level 1 mix. The design aggregate structure consisted of 19 percent and 25 percent coarse aggregates (CS-1 and CS-2, respectively) from Lyon County, 28 percent coarse screenings (CS-1A) from Lyon County, and 28 percent coarse sand (SSG-1) from Reno County. Table 4.2 shows the individual and combined cumulative aggregate gradation for this mix (See Figure 4.2). The design binder content was 5.7 percent. The asphalt cement used was an AC-20 produced by Total of Arkansas City, Kansas.

TABLE 4.2 Individual and Combined Aggregate Gradations for SM-1B Mix (US-50)

Sieve Size	Percent Aggregate Retained				
	CS-1 (19%)	CS-1A (28%)	CS-2 (25%)	SSG-1 (28%)	Combined Cumulative (100%)
¾ in.	0	0	0	0	0
2 in.	28	0	0	1	6
3/8 in.	62	10	0	5	16
No. 4	95	74	14	15	46
No. 8	96	96	41	40	67
No. 16	97	97	59	67	79
No. 30	97	98	73	84	88
No. 50	98	98	79	97	93
No. 100	98	98	83	99	95
No. 200	98	98	87	99	95.5

4.2.3 *Control Mixture (BM-1B)*

The control mixture studied was a KDOT designation BM-1B mixture. It is a 2-in. nominal maximum size mixture. The combined gradation consisted of 45 percent 2-in. Bedding from Fogle Quarry; 30 percent Martin-Marietta Screening; and 25 percent Kansas River sand. Table 4.3 shows the individual and combined cumulative aggregate gradation for the test mix. Figure 4.2 shows that the gradation just satisfies the requirements of Superpave 2 inch nominal maximum size. The binder content according to the Marshall mix design was 5 percent, and the binder used was an AC-10. At this binder content, the mix had an air void content of 2.8 percent, VMA of 13 percent, and VFA of 78.5 percent. The estimated VMA at 4 percent air voids will be 13.1 percent, approximately 1 percent below minimum VMA of 14 percent required by Superpave for 2 in. nominal maximum size.

4.3 Asphalt Beam Sample Preparation

The asphalt mix used for preparing the Superpave sample beams was sampled from the hot mix plants by diverting the delivery chutes from the dump trucks at the respective Superpave projects. The mixtures were reheated to the compaction temperature before beam fabrication. The control sample, a conventional KDOT BM-1B type mix, was prepared in the laboratory and not aged.

For beam fabrication, the asphalt concrete mix was placed in a rectangular mold of 3 in. x 4 in. x 16 in. size and compacted by a California kneading compactor (manufactured by Cox & Sons) at the Materials Research Center of KDOT in Topeka, Kansas. The compaction was done in two layers. The foot-pressure of the kneading compactor was successively increased to 100 psi, 200 psi, and 300 psi for each of the approximately 40 tamps per layer. After the mold was removed from the kneading machine it was placed under a static load for five minutes. The mold was then placed in a refrigerator for 20 minutes to cool. After cooling, the beam was extracted from the mold with a hydraulic jack.

TABLE 4.3 Individual and Combined Aggregate Gradations for Control Mix (BM-1B)

Sieve Size	Percent Aggregate Retained				
	2 in Bedding Fogle Quarry (45%)	Shilling Screening (30%)	Shilling Sand (25%)	Combined Cumulative (100%)	KDOT Spec.
2 in.	0	0	0.3	0	0 - 10
3/8 in.	37.8	3.8	0.7	18.3	12 - 26
No. 4	99.7	36.8	5.2	57.2	39 - 56
No. 8	99.8	56.2	19.3	66.6	60 - 76
No. 16	99.8	67.6	41.8	75.6	72 - 87
No. 30	99.8	73.0	65.8	83.3	79 - 92
No. 50	99.8	81.4	91.1	92.1	84 - 95
No. 100	99.8	84.3	98.2	94.8	88 - 98
No. 200	99.8	85.3	98.3	95.0	92 - 98

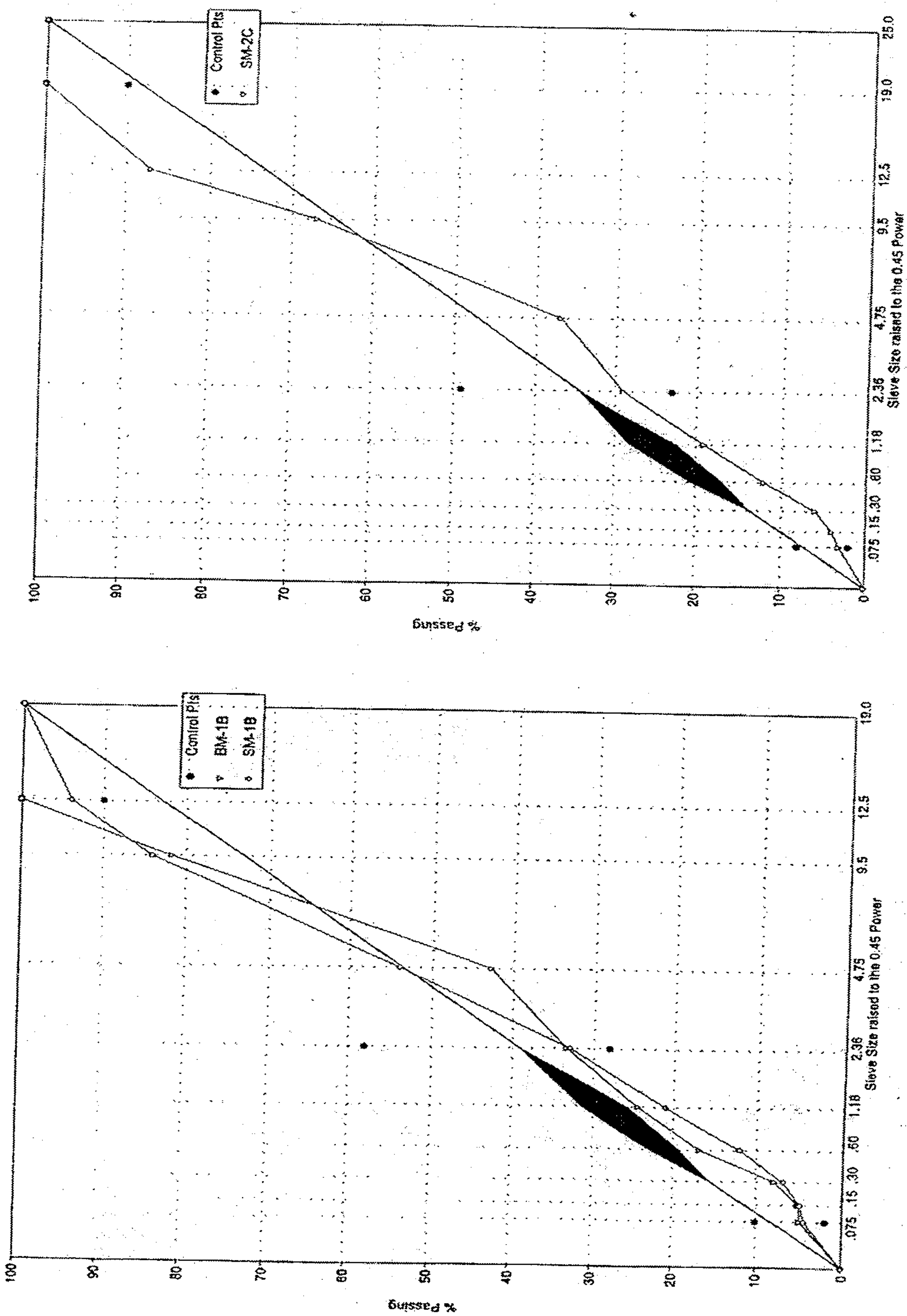


FIGURE 4.2 Combined Gradation of the Three Test Mixes

4.4 Testing of Samples

Twenty beams each of SM-2C and SM-1B mixes were received from KDOT. The mass and dimensions of these beam specimens were measured. One specimen of each mix was statically tested in a three-point loading mode using a static universal testing machine at 68°F. The results were used to estimate the ultimate load carried by each mix in flexure. The fatigue loads applied in the later dynamic flexural fatigue tests were fractions of this ultimate load. Initially the fractions were chosen at random. And the corresponding load repetitions to failure were monitored. Based on the initial results, the level of stresses for the remaining tests were selected so that the specimens would fail within a range of 1,000 to 100,000 repetitions.

Flexural fatigue tests on asphalt beams were performed in this experiment under controlled stress type loading with simple loading history. The beams were tested in a third-point mode of loading as shown in Figure 4.3 using the test equipment described earlier. The repeated fatigue loads were applied in a haversine loading pattern of 0.1 second duration with no rest period. The test temperatures were 68°F for all three mixes. For each specimen the deflection at the center of the beam after 200 cycles of load repetition was measured with a strain gauge at the bottom fiber of the beam. The hacksaw blade strain gauge used is shown in Figure 4.4. A full-bridge set up was used to connect the strain gauge in an electrical circuit and a digital strain gauge indicator was used for read out. Figure 4.5 illustrates the full-bridge set up used. The deflection reading was found from the calibration chart of the electrical strain gauge with a micrometer. After the strain reading was taken at 200^{th} cycle, the strain measurement set-up was dismantled and the samples were loaded repeatedly to failure (i.e. full-depth cracking). The number of cycles needed to cause failure was noted.

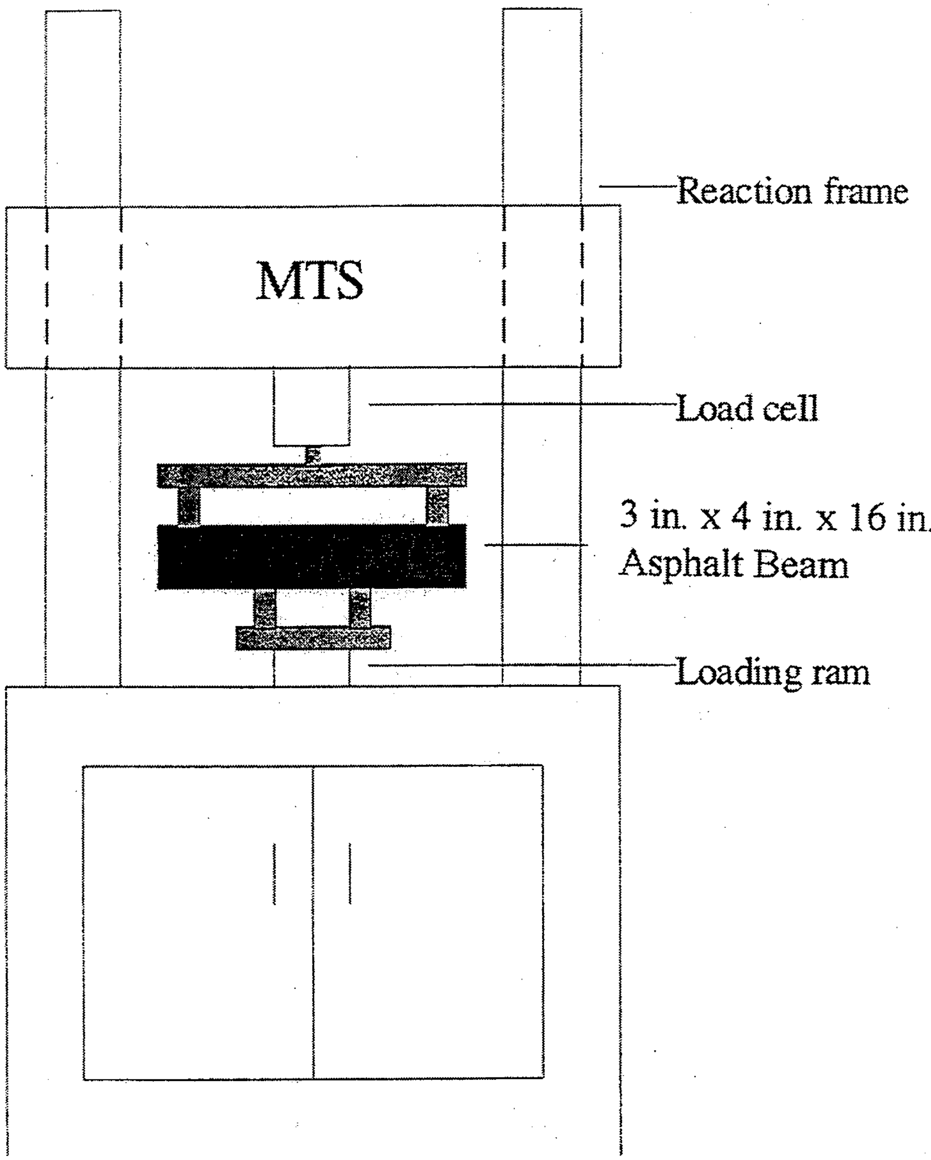


FIGURE 4.3 Test Set-Up for Flexural Fatigue Testing of Asphalt Concrete Beams

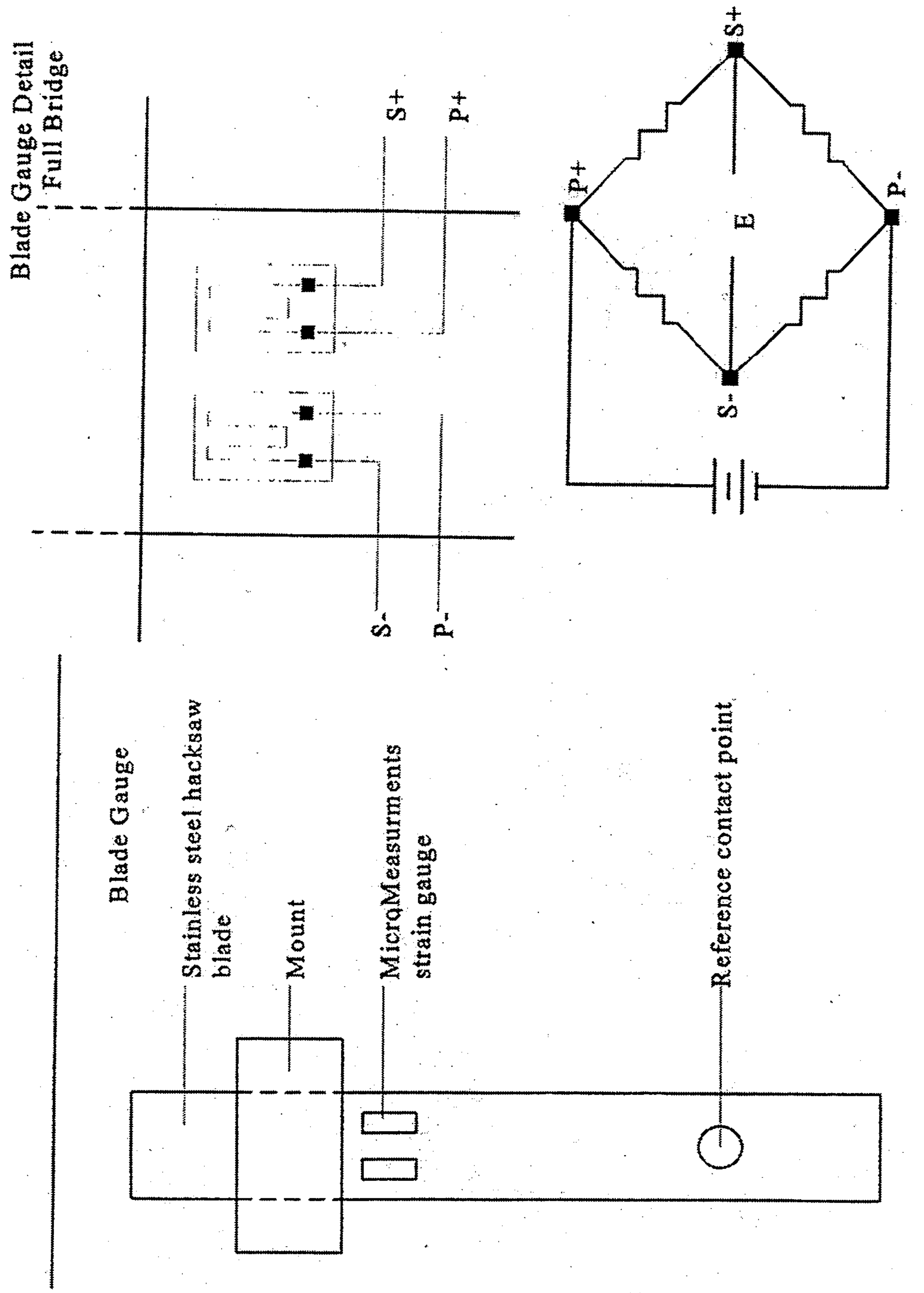


FIGURE 4.4 Strain Measurement Gauge Details

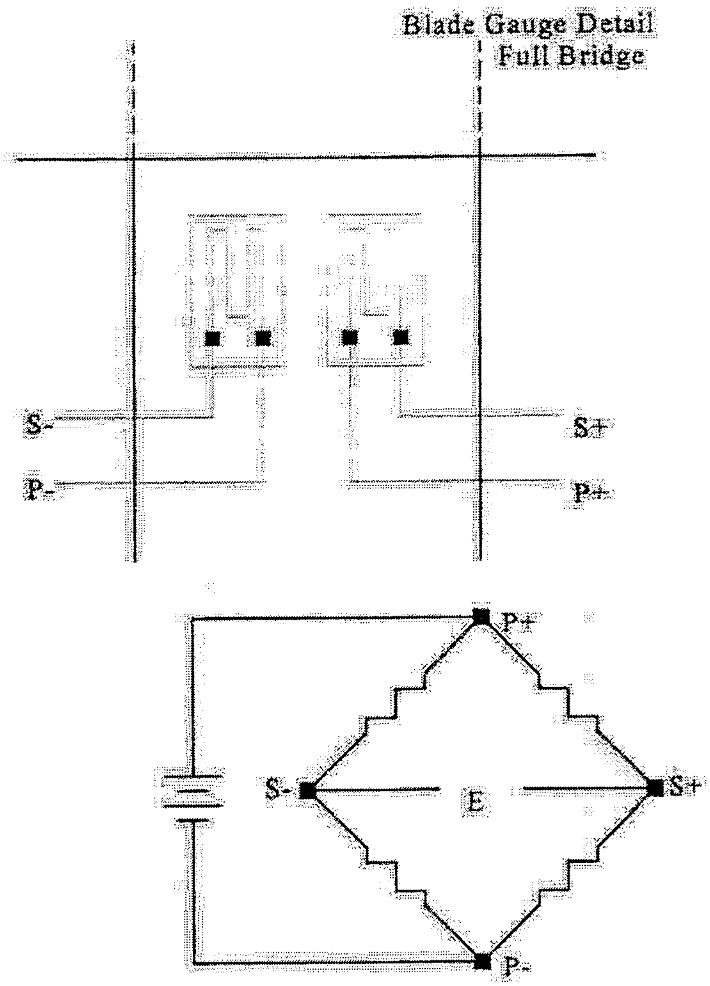


FIGURE 4.5 Full-Bridge Circuit Details

The total number of beams tested were 19 + 1 (companion for static ultimate load testing) = 20 specimens for the SM-1B mix, 20 + 1 = 21 specimens for SM-2C mix, and 9 + 1 = 10 for BM-1B mix. The stress levels ranged from 0.1 to 0.8 times of the ultimate flexural strength of the mixes. The measured repetitions to failure ranged from 426 to 102,557. The test matrix is shown in Table 4.4. A few specimen test results were discarded from analysis due to their premature failure caused by localized defects in the beam structure.

After the fatigue testing, the least damaged half of the test specimen was sawed on the fractured side to obtain a 6-in. long sample. These were used to determine the bulk specific gravity of each specimen using KT-15 test procedure. The specimen was then grounded by a hammer and maximum specific gravity of the loose specimen was determined by using KT-39 (Rice) test. The results were used to estimate the air voids percentage of each specimen.

TABLE 4.4 Test Matrix for Flexural Fatigue Tests

Mixture Type	Range of Stress Levels Applied (as percentage of ultimate load)	Range of Load Repetitions of Failure	Number of Samples for Flexural Fatique	
			Tested	Analyzed
SM-2C (K-177)	0.1 – 0.5	4399 – 86401	20	18
SM-1B (US-50)	0.25 – 0.8	10900 – 102557	19	18
Control (BM-1B)	0.3 – 0.7	426 – 3596	9	7

CHAPTER 5

DATA ANALYSIS AND DISCUSSION OF RESULTS

5.1 Data Analysis

This chapter describes the analysis of flexural fatigue test data and discusses the results obtained. The dynamic stiffness modulus and the initial strain of each test were determined at the 200th repetition by using Equations 5.1 and 5.2, respectively (3). These are same as Equations 3.2 and 3.3 provided earlier.

$$E_s = \frac{Pa(3\ell^2 - 4a^2)}{4bd^3t} \qquad (5.1)$$

$$\varepsilon_t = \frac{12td}{(3\ell^2 - 4a^2)} \qquad (5.2)$$

Dynamic stiffness modulus must be determined at the initial stage, since it is an input to compute initial tensile strain. The 200^{th} repetition is chosen, because experts opine that after 100 to 200 repetitions the plastic strains are sufficiently dampened in a repeated load test (3).

Tables 5.1, 5.2, and 5.3 show the fatigue test results for SM-2C, SM-1B and BM-1B at 68°F, respectively. The tables also show the air voids percentage of each beam specimen. The stiffness of the SM-2C mixture varied from 64 ksi to 158 ksi with a mean value of 102.6 ksi. The coefficient of variation was 29 percent. The air voids percentage of these beams were also higher. The average air void percentage was 11.5 percent with a coefficient of variation 7.1 percent. The stiffness values of the SM-1B mixtures were less variable with an average value of 150 ksi and a coefficient of variation of 17 percent. The average air void of the mixtures was also low at 6.4 percent with a coefficient of variation 16 percent. The control, BM-1B mixtures had an average stiffness of 73.6 ksi with a coefficient of variation of 24 percent. The average air void for these dense-graded samples was 4.2 percent with a coefficient of variation 31.1 percent.

TABLE 5.1 Superpave Project K-177 SM-2C Mix at 68°F

Sample ID	N_f (repetitions)	ϵ_t (micro-strain)	E_s (ksi)	% Air Voids
SM-2C-2	4569	880	113	11.77
SM-2C-5	35310	486	158	11.82
SM-2C-6	21631	869	98	11.35
SM-2C-8	30630	481	137	10.4
SM-2C-9	27549	715	83	11.46
SM-2C-10	86285	237	158	11.07
SM-2C-11	8100	751	125	10.62
SM-2C-12	9997	692	113	11.17
SM-2C-14	8647	568	120	10.28
SM-2C-15	4399	925	74	11.05
SM-2C-16	15123	672	70	10.41
SM-2C-17	31228	280	126	11.72
SM-2C-18	31695	420	86	12.36
SM-2C-19	50123	351	77	12.75
SM-2C-20	47565	322	83	11.48
SM-2C-21	37048	421	64	12.01
SM-2C-22	80021	293	86	13.27
SM-2C-23	86401	232	76	11.3

TABLE 5.2 Superpave Project US-50 SM-1B Mix at 68°F

Sample ID	N_f (repetitions)	ϵ_t (micro-strain)	E_s (ksi)	% Air Voids
SM-1B-2	102557	339	164	7.56
SM-1B-3	58064	449	190	5.19
SM-1B-4	34815	628	164	7.61
SM-1B-5	49467	616	163	7.07
SM-1B-6	26847	809	127	6.3
SM-1B-7	40908	649	168	5.88
SM-1B-8	32860	612	164	7.65
SM-1B-9	30970	540	192	7.67
SM-1B-10	20031	896	115	7.05
SM-1B-11	40600	738	143	5.7
SM-1B-12	24863	890	117	6.43
SM-1B-13	35013	578	174	6.68
SM-1B-14	10900	1082	115	6.32
SM-1B-15	18502	836	153	6.76
SM-1B-16	30545	889	138	4.04
SM-1B-17	23077	1086	112	4.86
SM-1B-18	82665	376	159	5.96
SM-1B-19	26102	963	141	6.34

5.2 Results of Correlation Analysis

A simple one-to-one correlation analysis was done among the number of cycles to fatigue failures, initial tensile strain, flexural stiffness and percent air voids. Every possible permutation of pairs was analyzed, except initial tensile strain and flexural stiffness. The results of the analysis are tabulated in Table 5.4.

TABLE 5.3 Control (BM-1B) Mix at 68°F

Sample ID	N_f (repetitions)	ϵ_t (micro-strain)	E_s (ksi)	% Air Voids
CON – 2	979	1067.00	60	2.8
CON – 3	578	1467.00	64	6.5
CON – 4	3596	645.00	50	3.6
CON – 5	426	1395.00	78	4.9
CON – 6	485	1325.00	83	4.0
CON – 7	1155	721.00	77	4.5
CON – 8	2037	523.00	103	2.8

TABLE 5.4 Results of Correlation Analysis

MIX	VARIABLES	Repetitions (N_f) (R^2/p)	Strain (ϵ_t) (R^2/p)	Stiffness (E_s) (R^2/p)	Air Voids (V_a) (R^2/p)
SM-2C	(N_f)	-	0.68/0.00	0.00/0.99	0.19/0.07
	(ϵ_t)	0.68/0.00	-	-	0.14/0.12
	(E_s)	0.00/0.99	-	-	0.09/0.24
	(V_a)	0.19/0.07	0.14/0.12	0.09/0.24	-
SM-1B	(N_f)	-	0.68/0.00	0.23/0.04	0.01/0.68
	(ϵ_t)	0.68/0.00	-	-	0.13-0.15
	(E_s)	0.23/0.04	-	-	0.09/0.22
	(V_a)	0.01/0.68	0.13/0.15	0.09/0.22	-
BM-1B	(N_f)	-	0.61/0.04	0.08/0.55	0.21/0.30
	(ϵ_t)	0.61/0.04	-	-	0.43/0.11
	(E_s)	0.08/0.55	-	-	0.04/0.66
	(V_a)	0.21/0.30	0.43/0.11	0.04-0.66	-

As expected the repetitions to failure was significantly affected by the initial tensile strain with the lowest level of significance, p-value = 0.04 for BM-1B mix. However, the coefficient of determination, R^2 values for all three mixes hovered around 0.65, which was not sufficient to establish a good linear correlation. The stiffness had significant effect on the number of cycles to failure for SM-1B mix at p-value of 5 percent, but displayed insignificant R^2= 0.23. Stiffness did not show significant effect on repetitions to failure for SM-2C and BM-1B mixes. This is well expected in a constant stress-type testing. The number of repetitions to failure was significantly affected by percent air voids (p-value = 0.07) for the SM-2C mix, but showed poor correlation in terms of R^2= 0.19. No significant effects were observed for the SM-1B and BM-1B mixes in this respect. Previous research has shown that the number of repetitions to failure is significantly reduced if the air voids are higher than 8 percent (12).

Stiffness was found to be unaffected by percent air voids for all three mixes. Similarly percent air voids did exhibit some effect on the initial tensile strain at p-value = 0.15 for all three mixes. But then no good correlation was evident in terms of R^2 values.

The fatigue test data analysis described in Chapter 3 is based on a power relationship between repetitions to failure and initial tensile strain. Hence, the correlation analysis was repeated considering natural log of each variable. It was observed that generally, the R^2 and p values showed little change. But in the case of repetitions to failure and initial tensile strain the R^2 values rose to around 0.85, which was considered good enough to develop the equations of this study.

5.3 Developing Fatigue Life Equations

The number of cycles to fatigue failure showed an excellent correlation to the initial tensile strain in the log-log scales. Therefore, it was decided to develop simple transfer functions based on the form shown below. The same generic equation has been used by all agencies in their research work described earlier.

$$N_f = f_1(\varepsilon_t)^{-f2} \qquad (5.3)$$

Where "N_f" is the number of repetitions to failure, "f_1" is a fatigue constant that is the value of "N_f" when "ε_t" = 1, and "f_2" is the inverse slope of the straight line.

Accordingly, the initial strains measured were plotted against the number of load repetitions to failure on log scales for all three mixes. The resulting relationships obtained are straight lines of the form shown in Equation 5.3. The equations were originally developed using the TABLECURVE software program. This software program is capable of developing graphs for a variety of best fit equations, to choose from. The following relationships were obtained for different mixtures at 68°F:

SM-2C:

$$N_f = 0.0336 \cdot \varepsilon_t^{-1.766} \qquad (R^2 = 0.81) \qquad (5.4)$$

SM-1B:

$$N_f = 1.264 \cdot \varepsilon_t^{-1.397} \qquad (R^2 = 0.88) \qquad (5.5)$$

BM-1B:

$$N_f = 0.000446 \cdot \varepsilon_t^{-2.1012} \qquad (R^2 = 0.85) \qquad (5.6)$$

Figure 5.1 shows the fatigue relationships developed for SM-2C, SM-1B and BM-1B. The equations showed good coefficient of determination values ($R^2 > 0.80$) and the p-values for the independent variable, tensile strain, were less than 0.006 for all three mixes. The above statistical analysis was checked using the SAS statistical analysis software also.

Some agencies, like the Asphalt Institute and Shell Petroleum have incorporated the dynamic modulus as an independent variable in their transfer functions in the form shown below:

$$N_f = f_1 (\varepsilon_t)^{-f2} (E^*)^{-f3} \qquad (5.7)$$

where: E* is the complex dynamic modulus, which is unique for a given mix. The above studies used fatigue test data from a number of mixes (>17) and could thus incorporate the dynamic modulus in the transfer functions. Since only two dynamic moduli for the two Superpave mixes would be available in this study, it could not be included in the transfer functions.

Laboratory tests have shown that under the same initial strain, the number of repetitions to failure decreases with the increase in stiffness modulus. Therefore it is suggested (3) that the generic equation can be expanded as:

$$N_f = f_1(\varepsilon_t)^{-f2}(\mathrm{E}_s)^{-f3} \qquad (5.8)$$

In this study stiffness modulus was available for each sample. Hence, attempts were made to use the following combinations: 1) tensile strain and flexural stiffness, and 2) tensile strain, flexural stiffness and air voids as independent variables. The statistical analysis showed that, addition of more independent variables increased the R^2 values of the basic equation by 1 to 5 percent for the Superpave mixes and up to 17 percent for BM-1B mix. But, the p-values for the variables, stiffness and air voids, were generally greater than 0.10 for most equations. However, as the stiffness modulus is stress sensitive, it cannot be used as a dynamic modulus in the linear elastic system (3). Since the pavement models for performance prediction (described later) were analyzed as linear elastic systems, the equations involving stiffness modulus were not studied any further.

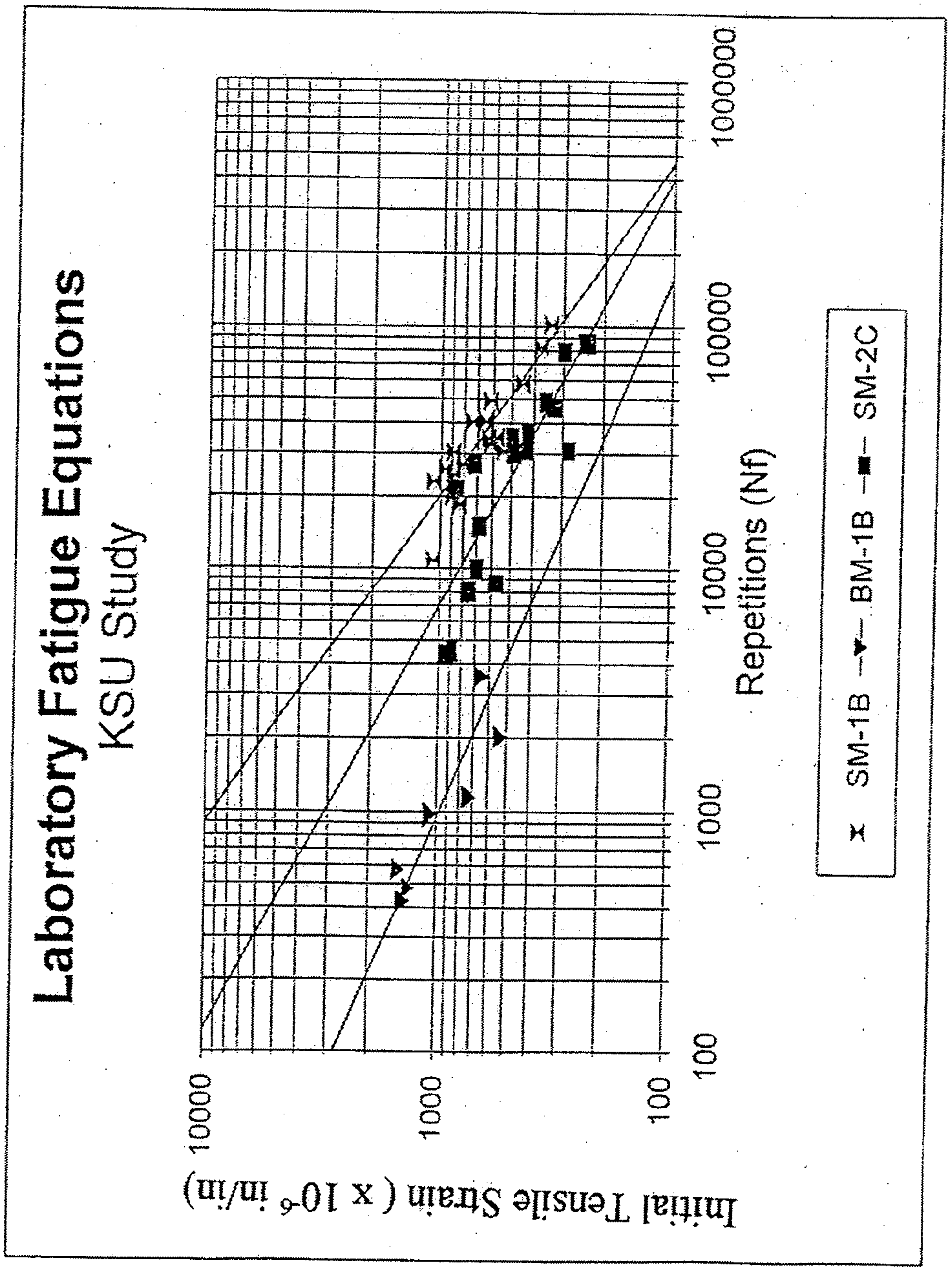


FIGURE 5.1 Relationship Between Number of Repetitions to Failure and Initial Tensile Strain for SM-2C, SM-1B and BM-1B Mixes.

5.4 Performance Prediction Using Transfer Functions

Mechanistic analysis is based on mechanics of materials that relates to an input such as a wheel load, to an output or pavement response such as tensile strain. This response is then used as input in the transfer functions developed from laboratory tests to predict performance or distress. The fatigue lives of the newly built Superpave projects were to be predicted using the transfer functions developed in this study. This required the mechanistic pavement response of tensile strain at the bottom of the asphalt concrete layer under an equivalent single axle load (ESAL) as an input to predict the fatigue life in ESALs. The mechanistic response was estimated by a two step procedure. Falling Weight Deflectometer (FWD) tests were conducted on the Superpave project sections and the deflection results were used to back calculate layer moduli. The pavements were modeled as multi-layer elastic systems based on as-built sections and back-calculated moduli. The mechanistic response (tensile strain) was estimated for each pavement model under an 18-kip ESAL using an elastic layer analysis software.

5.5 Field Data Collection

Two 1000 foot sections on the newly built K-177 and US-50 projects were selected for testing with the Dynatest 8000 FWD. Deflection data were collected at 100-ft intervals, i.e. at eleven locations on each of the test sections in May 1997, approximately seven to ten months after construction. The sections were in excellent condition and the Superpave mixes in these pavements were adequately compacted. The pavement surface temperatures during the tests were 63° F and 63 to 64° F for K-177 and US-50, respectively. The first sensor was located at the center of the loading plate with six others at a uniform radial distance of 12 inches apart. Three drops of the FWD load were used for target loadings of 7, 9, and 15 kips. Tests were done on the outer wheel path of the travel lane.

5.6 Pavement Modeling and Back-calculation of Layer Moduli

The pavements were modeled in this study as multi-layered elastic systems. An automated back calculation program, EVERCALC, developed by the Washington State Department of Transportation was used in the back-calculation process. The back-calculation results were also cross-checked with the Texas Transportation Institute's MODULUS back-calculation program. The deflection basins corresponding to the target loading of 9 kip were used in this comparison.

In the back-calculation process, very good convergence was obtained by assuming a saturated layer with modulus of 50 ksi (behaving as a stiff layer due to water table) below the subgrade for both test sections. The surface modulus plots shown in Figures 5.2 and 5.3 indicated the presence of a bedrock on the K-177 test section and an infinite subgrade at the US-50 test section, respectively. Although the back-calculated moduli estimated under these assumptions showed good convergence, the results were highly variable and unsatisfactory. The depths of the stiff layer for K-177 and US-50 were estimated to be 6.8 ft and 47 ft, respectively by EVERCALC, and matched the depths estimated by MODULUS, presumably due to similar algorithms used by both programs.

In the back-calculation analysis, the surface and base layers, i.e. the 1-in. SM-1T and 8-in. SM-2C layers on K-177, and 1-in. SM-1T and 1.5-in. SM-1B layers on US-50 were grouped together as a single layer. The appropriate temperature correction was applied to the asphalt layer moduli back-calculated by the EVERCALC program following the 1986 AASHTO Pavement Design Guide algorithm to convert into 68°F. The results of back-calculation analysis are shown in Table 5.5.

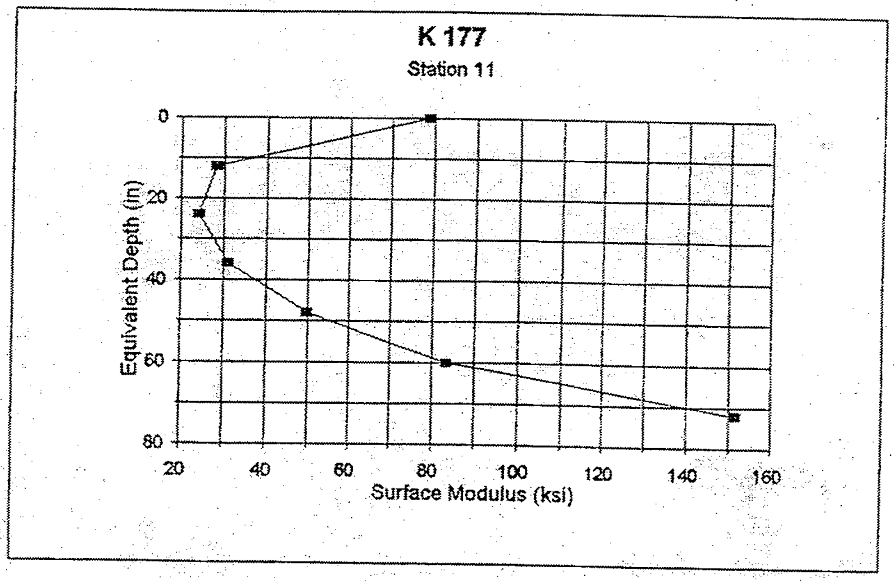


FIGURE 5.2 Surface Modulus Plot drawn from a Typical FWD Deflection Basin for a K-177 Test Section

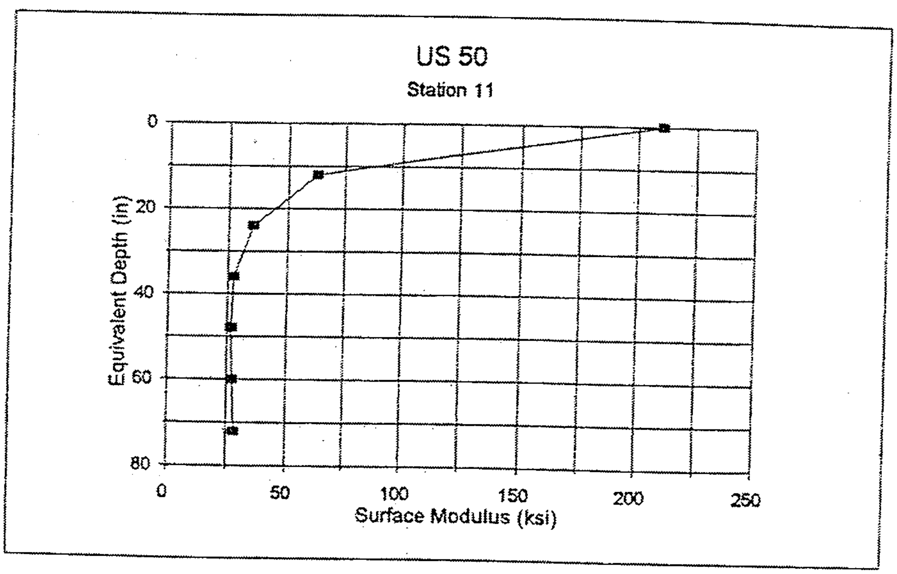


FIGURE 5.3 Surface Modulus Plot drawn from a Typical FWD Deflection Basin for a US-50 Test Section

TABLE 5.5 Results of Back-calculation Analysis

Layer No. (from top)	K-177			US-50		
	Layer Composition	Thickness (in.)	Back. Moduli (ksi)	Layer Composition	Thickness (in.)	Back. Moduli (ksi)
1	SM-1T SM-2C	9	223.5	SM-1T SM-1B	2.5	433.4
2	Aggregate Subbase	8	42.7	BM-1 BM-7	4	711.7
3	Subgrade	81.6	12	JRCP	9	2024
4	Saturated Stiff Layer	Infinite	50	Subgrade	564	25.1
5				Saturated Stiff Layer	Infinite	50

5.7 Prediction of Fatigue Life

For prediction of fatigue lives of SM-2C, SM-1B and BM-1B mixtures, the as-built K-177 and US-50 Superpave pavement sections and a hypothetical section modeled similar to K-177 section (in which SM-2C is replaced by BM-1B) were used, respectively (See Figure 4.1). On each section, the tensile strain at the bottom of the asphalt base layer was computed corresponding to a single axle dual wheel load of 18 kip with 100 psi tire pressure. The EVERSTRS elastic layer analysis software of the Washington State Department of Transportation (WSDOT) was used in strain computation. These strain values were substituted as input into the three equations developed in this study for predicting fatigue lives of SM-2C, SM-1B and BM-1B mixes.

These results were compared with those obtained from equations developed by other agencies. The strain values and other volumetric parameters were used to predict fatigue lives using the (I) SHRP A-003A Laboratory Testing Method (12) and (ii) The Asphalt Institute Fatigue Equations (7). These equations were chosen for comparison because the Asphalt Institute equation is widely used for pavement design and the SHRP equations were developed on Superpave mixtures. Table 5.6 shows the fatigue equations and associated shift factors used with each equation.

5.8 Shift Factor

Laboratory predicted and field observed flexible pavement (distress) performance does not compare favorably. This difference is attributed to the following factors (17):

TABLE 5.6 Comparison Among Different Fatigue Models and Shift Factors

DESIGN METHOD	FATIGUE EQUATIONS	SHIFT FACTOR
SHRP A-003A LABORATORY TEST METHOD	Top Lift (air voids = 6.8%): $N_f = 4.06 \times 10^{-8} \times \epsilon_t^{-3.348}$ Bottom Lift (air voids = 3.7%): $N_f = 8.36 \times 10^{-8} \times \epsilon_t^{-3.420}$	13
ASPHALT INSTITUE	$N_f = 00.432\ C\ \epsilon_t^{-3.291} \lvert E^* \rvert^{-0.854}$ $C = 10^M$ $M = 4.84\left(\frac{V_b}{V_a + V_b} - 0.69\right)$	18.4
KSU STUDY	SM-2C: $N_f = 0.0336.\ \epsilon_t^{-1.766}$ (at 68°F) ($R^2 = 0.81$) SM-1B: $N_f = 1.264.\ \epsilon_t^{-1.397}$ (at 68°F) ($R^2 = 0.88$) BM-1B: $N_f = 0.000446.\ \epsilon_t^{-2.1012}$ (at 68°F) ($R^2 = 0.85$)	100*

* As per suggestion by Brown (18).

Notes:
N_f = Fatigue life (reps),
ε_t = Initial strain (in/in),
E^* = Dynamic ($\simeq$ Elastic) modulus (psi),
C = Fatigue life multiplying factor,
V_b = Volume of binder (%),
V_a = Volume of air voids (%).

i) Wheel loads on actual pavements are not applied at the same location and usually have longer and variable rest periods;

ii) Difference in loading conditions, multi-level loading, loading sequence;

iii) Frictional forces between the asphalt layer and the base layer;

iv) Residual stresses caused by the plasticity of the pavement layers;

v) Dilatancy stresses from the expansion of paving materials under load, which builds up large confining pressures under passing wheel loads; and

vi) Complicated environmental conditions in the field.

Therefore, laboratory developed transfer functions must incorporate a shift factor to adjust predicted performance to more realistically reflect field-observed pavement performance. In other words, the factor f_1 in Equation 5.1 needs to be modified for lab to field calibration. It is quite difficult to develop a shift factor based on sound mechanical theory that accounts for the effects of all the above stated factors. The 1986 AASHTO guide emphasized the importance of field calibration activities. The most widely used shift factor of 13 presented by Finn came from the evaluation of AASHO Road Test data. (17). Pell indicated that the shift factor might range from 5 to 700 (3). Brown (18) has suggested a shift factor of 100 for no rest period testing condition as was used in the present study. Results from 16 test sections, in the North Carolina Department of Transportation's (NCDOT) recently completed US-421 project near Siler City, show that the shift factor ranged from 0.28 to 3.52 (4). WSDOT studies have shown that shift factors for six in-service pavements exhibiting fatigue cracking are more like 3 to 6. In separate works, Van Dijk and Mahoney have presented the shift factor as 8 and 5, respectively (6).

Due to differences in materials, test methods, and structural models, a variety of shift factors can be expected. The shift factor may very well be the most unpredictable variable in the mechanistic analysis of flexible pavements (19). For the present study it was decided to use the shift factor of 100 following the suggestions by Brown (18).

5.9 Discussion of Results

Table 5.7 shows the computed strains and associated fatigue lives predicted by each equation. For K-177, all equations barring the KSU equation show lower fatigue life for the SM-2C layer than the BM-1B layer on control section, although the BM-1B layer has same thickness. In the SHRP equation, the higher life of BM-1B mix (nearly 3 times) is primarily due to lower strain, while in the Asphalt Institute's equation it is nearly 22 times higher, primarily due to lower C value for SM-2C. The lower C values were obtained due to the higher percentage of air voids in that mix. For SM-2C the KSU equations have predicted nearly twice the life of BM-1B due to a higher value of f_1 in the SM-2C equation.

Overall, the results obtained by the KSU and the SHRP equations are close for SM-2C mix, compared to the prediction by the Asphalt Institute. The 10-year cumulative traffic on K-177 used in the AASHTO design guide method was 900,000 ESALs. The predicted fatigue lives for BM-1B mix by KSU and AI equations are closed when compared with the SHRP predictions. Extremely high fatigue lives were predicted by the SHRP equation for US-50 and BM-1B, which are unlikely to be observed in the field. Except for K-177, the KSU equations have predicted lower lives than the rest.

The predicted life for US-50 is exceedingly high by any method. This is essentially due to very low tensile strain at the bottom of the asphalt base layer due to the composite structure of the pavement. It is actually difficult to decide the failure criteria for these types of pavement. Rutting due to vertical compressive strain on the top of the subgrade layer also is not a good indicator since the plastic strains have essentially stabilized over the years.

TABLE 5.7 Summary of Asphalt Pavement Strains and Fatigue Lives.

Section	Load (kip)	Tensile Strain at the bottom of the asphalt base layer ($x10^{-6}$ in/in)	Fatigue Life (Repetitions, Millions)		
			SHRP A-003A	Asphalt Institute	KSU Study
K-177 (SM-2C)	18	166.7	9.1	0.6	15.8
US-50 (SM-1B)	18	14.2	41339	8675.6	751.6
Control (BM-1B)	18	117.1	30.3	12.9	8.2

5.10 Comparison of Fatigue Lives of Superpave Mixtures

A meaningful comparison between fatigue lives of the SM-2C and SM-1B Superpave mixes is not possible due to the difference in the K-177 (flexible) and US-50 (composite) pavement structures. In order to compare these Superpave mixtures, another theoretical structure (similar to K-177) was assumed with a 9-in. thick Superpave layer and an 8-in. thick aggregate base. The moduli for the aggregate base, subgrade and stiff layer were those obtained in the back calculation analysis for K-177. However, the modulus of the Superpave mixes was fixed at a standard value of 435 ksi. Since SM-2C is a younger mix in comparison to SM-1B and also SM-2C uses a softer asphalt than SM-1B, it was felt that using the low back calculated moduli of SM-2C would not yield a good comparison. The three pavement cross-sections used in this analysis are shown in Figure 5.4.

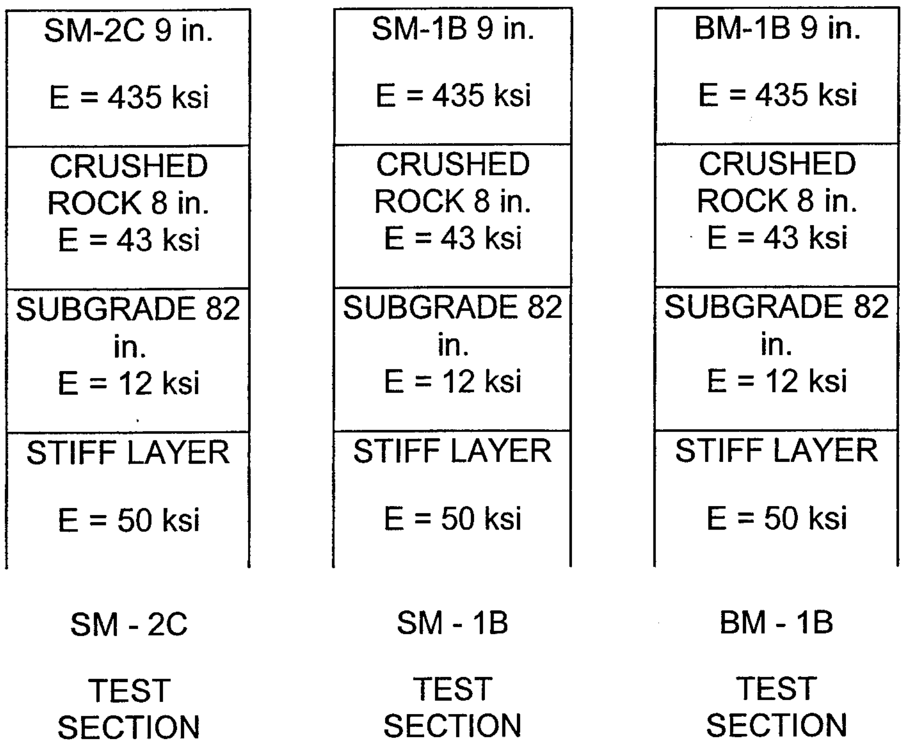


FIGURE 5.4 Schematic Diagram of the Hypothetical Pavement Sections Used for Prediction of Fatigue Lives Under Equal Moduli Assumptions

The critical tensile strains at the bottom of the asphalt layer were estimated for three different load levels of 9, 18, and 22.5 kips. The critical tensile strains and the corresponding predicted fatigue lives obtained by using the KSU equations are tabulated in Table 5.8. The graphical comparison of fatigue lives is shown in Figure 5.5. The results show that the SM-1B mix has a better predicted fatigue life than the SM-2C mix. The increase in fatigue life is remarkable at higher load levels. This is primarily due to the fact that the f_1 value in the SM-1B equation is nearly 38 times higher than that of the SM-2C equation. This was unaffected by the lower f_2 value in the SM-2C equation. One reason for this could be that the air voids percentage of the SM-2C beam samples was almost twice that of the SM-1B samples. It seems from the statistical analysis and previous research that air voids percentage, at higher levels, will significantly affect (lower) the number of repetitions. Besides the asphalt content of SM-1B mix is 11% higher than the SM-2C mix. The Superpave mixtures appeared to have far superior predicted fatigue lives than the conventional asphalt mixes used by KDOT in the past. The predicted lives of BM-1B mix under similar strains is almost one-third to one-quarter of the Superpave mixes.

TABLE 5.8 Comparison of the Three Test Mixes in a Theoretical Section with Same Modulus

LOAD LEVEL (kips)	STRAIN ($x10^{-6}$ in/in)	PREDICTED LIFE (in million repetitions)		
		SM-2C	SM-1B	BM-1B
9	57.5	103.3	106	36.4
18	111	32.1	42.4	9.2
22.5	137	22.3	31.5	6

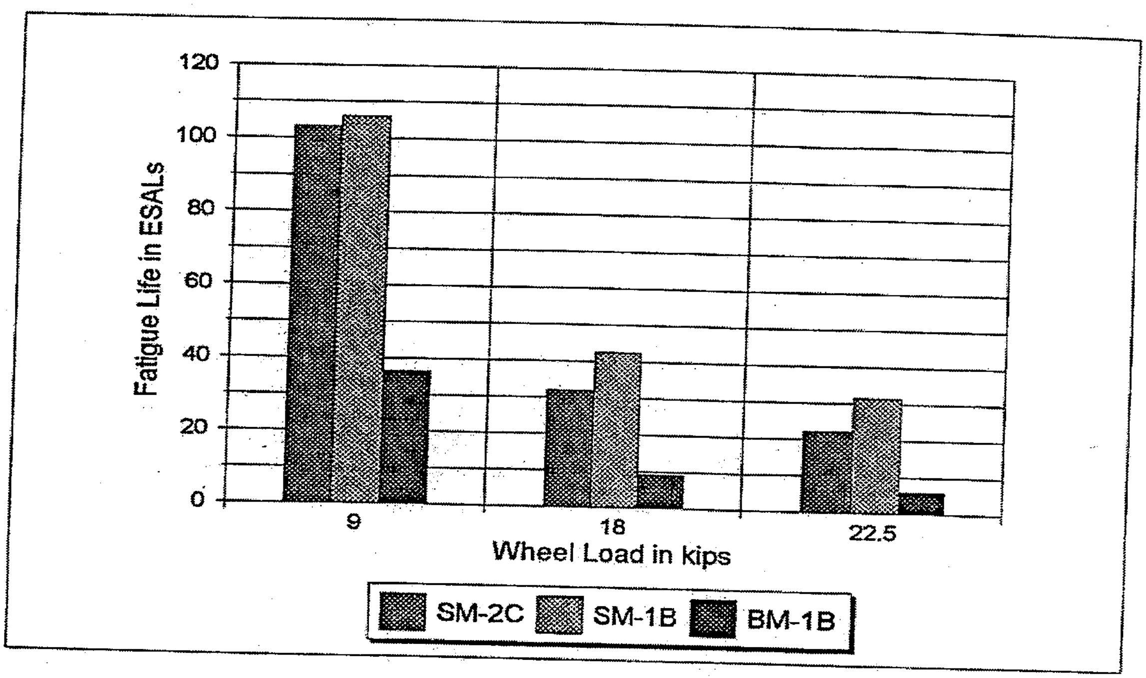


FIGURE 5.5 Graphical Comparison of Fatigue Lives for SM-2C, SM-1B and BM-1B Mixes

5.11 Check for Rutting as a Failure Criteria

Since both Superpave mixtures showed very good predicted fatigue lives. The rutting potential of these mixtures was evaluated by studying the maximum shear stresses at various levels in the pavement cross sections on K-177 and US-50. The maximum shear stresses were evaluated near the surface, at a few points within the asphalt layer and at the interfaces of the Superpave and underlying layers for a 22.5 kip axle load using ELSYM5 layered elastic system software program. The respective back-calculated moduli were used in this analysis. The results are shown in Table 5.9.

TABLE 5.9 Shear Stress, τ_{max} at Various Levels to Evaluate Rutting Potential.

US-50 (SM-1B mix)				K-177 (SM-2C)			
Depth from Surface (in)	**Shear Stress, τ_{max} at A (psi)**	**Shear Stress, τ_{max} at B (psi)**	**Shear Stress, τ_{max} at C (psi)**	**Depth from Surface (in)**	**Shear Stress, τ_{max} at A (psi)**	**Shear Stress, τ_{max} at B (psi)**	**Shear Stress, τ_{max} at C (psi)**
0.1	13.2	27.2	6.47	0.1	6.85	27.1	21.4
0.5	17.7	29.0	6.87	0.5	5.0	31.2	19.5
1.0	22.6	26.0	9.69	1.0	13.5	29.2	16.7
2.5	24.5	21.8	8.93	2.0	23.5	24.1	15.9
6.5	9.25	7.67	1.63	9.0	26.5	25.4	24.1
15.5	24.6	25.9	26.6	17.0	5.4	5.74	5.94

Note: A: at the center of the wheel load;
B: at the edge of the tire;
C: in-between the center of two-wheel loads.

The maximum shear stress is almost always encountered at points below the edge of the wheel load. For K-177 (SM-2C), the maximum shear stresses were encountered within the top 1 in., i.e., in SM-1T. The shear stress was also high throughout the 8 in. thick SM-2C up to its interface with the aggregate base (See Figure 5.6). On US-50 (SM-1B), maximum shear stresses were also encountered within the top 1 in. layer (SM-1T). The shear stress decreased only slightly in the 1.5

in. thick SM-1B layer up to its interface with the old BM-7 layer. High shear stress was also encountered again at the bottom of the JRCP layer (See Fig. 5.7).

For Superpave Level 2 mix design, SHRP has developed a screening process called tertiary creep evaluation to identify a mix that exhibits tertiary plastic flow leading to gross mix instability. If the mix fails this screening test, it will be necessary to either make adjustments to the mix proportioning or to redesign the mix completely. If the mix passes the test requirements it will be subjected to the battery of performance tests. Figure 5.8 illustrates conceptually the relationship between tertiary plastic flow and the performance prediction model (20).

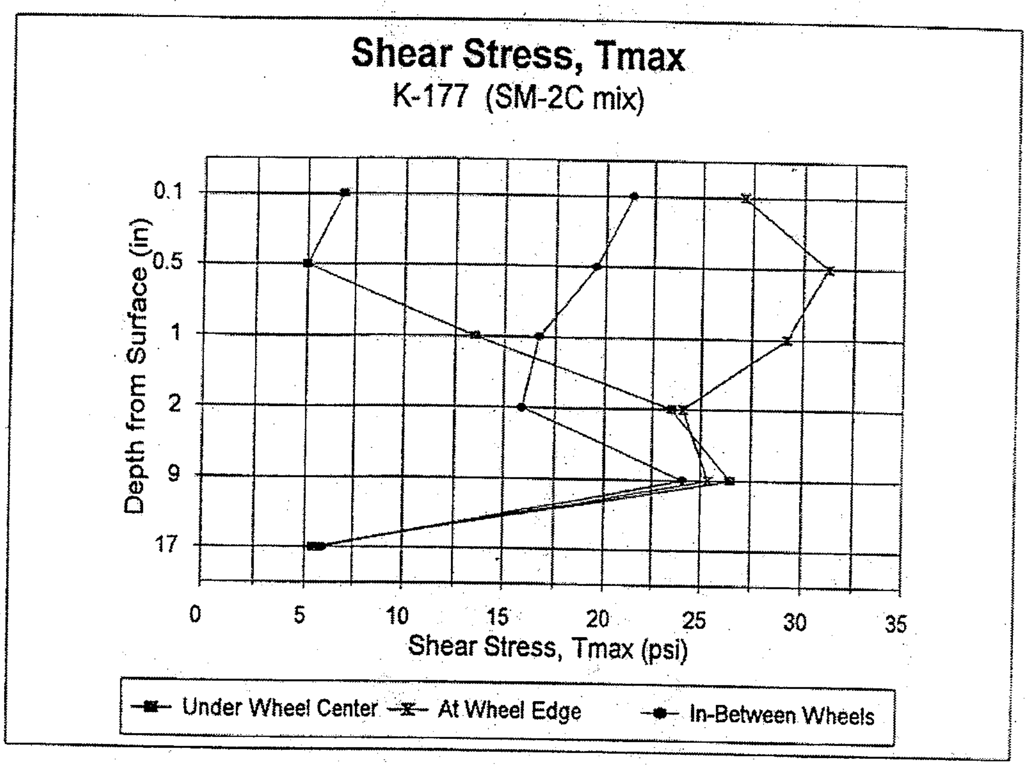


FIGURE 5.6 Shear Stress, τ_{max} due to 22.5 kip Load at Various Levels in K-177 (SM-2C) Section

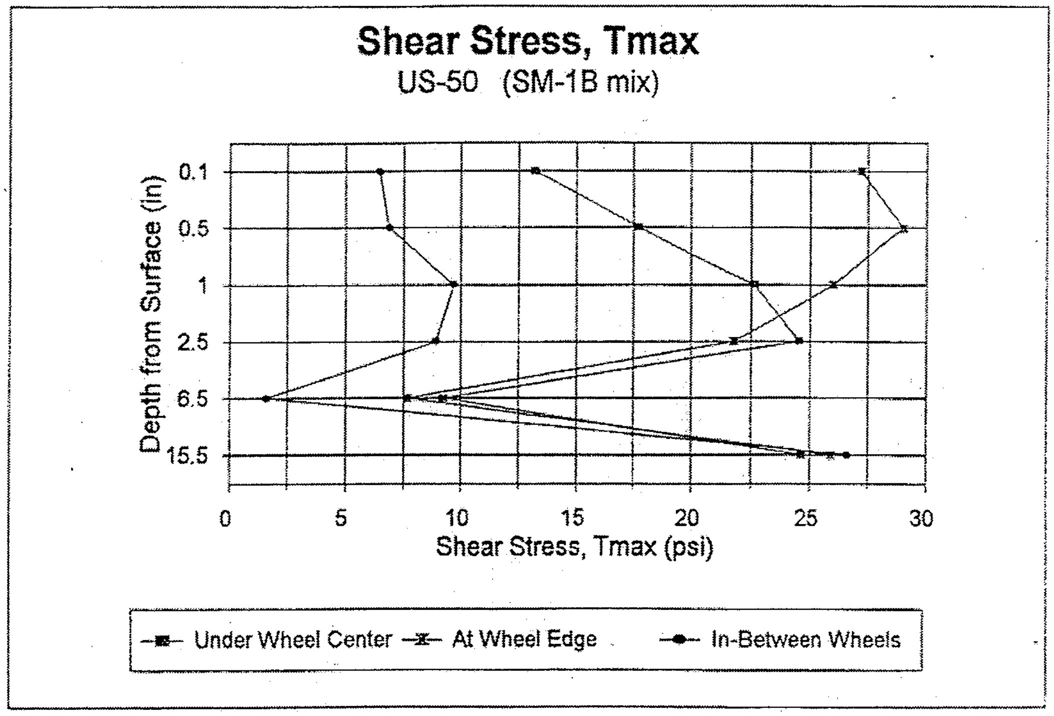


FIGURE 5.7 Shear Stress, τ_{max} due to 22.5 kip Load at Various Levels in US-50 (SM-1B) Section

However, the maximum shear stresses of 29 psi and 31.2 psi obtained for K-177 and US-50, respectively, are much higher than the SHRP recommended value of 14.2 psi for evaluating the tertiary creep of the Superpave mixes in Level 2 design. For granular base, like K 177, the value was even lower (20). Based on the analysis of the maximum shear stresses on these Superpave pavements it is felt that the recommended shear stress values in Superpave Level 2 mix design for tertiary creep evaluation are not realistic. Further studies are needed in this area.

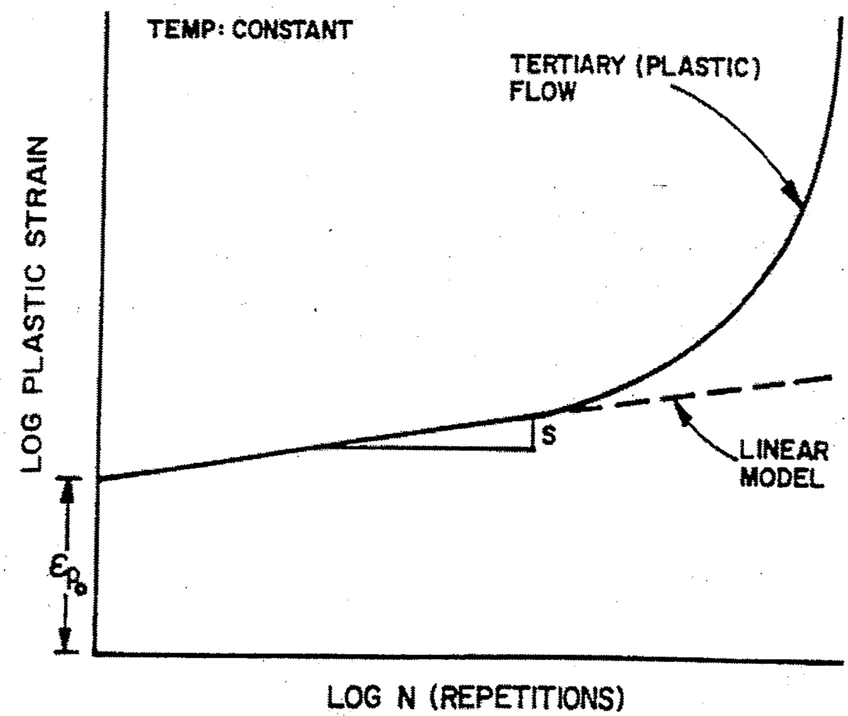


FIGURE 5.8 Typical Relationship of Plastic (Permanent) Strain, ε_p, and Load Repetitions, N, for Mixes with Tertiary Creep (Source: 20)

CHAPTER 6

PERFORMANCE-BASED SPECIFICATIONS AND QC/QA TESTING

6.1 Historical Background

The AASHO Road Test (1960) and the Blatnik House Committee Report on highway construction quality (1962), coupled with the initiative of FHWA, caused widespread development of performance-related statistical specifications by various highway agencies (21). With the development of experience, new knowledge and emergence of the computer, statistical specification writing developed into a state-of-the-art scientific activity. The recently completed NCHRP 10-26A project led to the development and demonstration of a conceptual framework for performance related specifications for hot-mix asphalt concrete mixtures and to an identification of research needs to develop fully functional and reliable performance-related specifications (22).

The present KDOT Standard Specifications are in the traditional method type form. The development of statistical performance-based specifications is in progress.

6.2 Concept of Quality Assurance System

The concept of quality assurance system gradually evolved from the development of statistical specifications' (21). This study used the following definitions of QC/QA as given in AASHTO R-10-921:

> *Quality Control*: The activities that have to do with making the quality of a product what it should be.

> *Quality Assurance*: The activities that have to do with making sure that the quality of a product is what it should be.

The AASHTO Quality Assurance Guide Specification defines "QA" as 'all those planned and systematic actions necessary to provide adequate confidence that a product or service will satisfy given requirements for quality.' This definition considers QA to be an all-encompassing concept, which includes QC, acceptance, and independent assurance (IA) (23).

At present KDOT is pursuing a new QC/QA testing program for the Superpave mixes on the pilot projects. As Superpave itself is a performance-based system, the QC/QA program is intended to be developed as a part of statistical performance-based specifications. As a part of this effort, KDOT has drafted additional requirements, which are partially in the form of performance-based specifications. These, along with the existing KDOT Standard Specifications, were adopted as the specifications for the Superpave projects.

6.3 Objectives of Statistical Performance-Based Specifications

The objectives of statistical performance-based specifications are:

i) To communicate to the contractor in a clear and unambiguous manner exactly what is required.

ii) To make the contractor responsible for the construction process control, while the agency assumes responsibility for judging the acceptability of the finished work.

iii) To develop adjusted pay schedules for providing sufficient incentive to the contractor to produce better quality work, while assessing the pay reductions for poor quality work in the agency's interest. Ideally the specification should pay 100 percent for acceptable quality work on average. It should be fair and equitable for work that differs from desired quality level.

iv) To provide specifications that are realistic in defining acceptable quality levels (AQLs) and rejectable quality levels (RQLs).

v) To make clear to the contractor what target quality level must be aimed for to receive 100 percent payment (24).

6.4 Advantages of Statistical Specifications

The advantages of statistical specifications are:

i) Realization of the existing inherent and testing variabilities for various construction parameters makes the specifications more realistic.

ii) Clearly defined responsibilities reduce likelihood of contractual disputes. Under these specifications, the contractor is responsible for construction process control and agency is responsible for final acceptance of the work.

iii) Based on predefined acceptance criteria and random sampling procedures, risks to both contractor and agency can be minimized and balanced.

iv) The adjusted pay factor schedules provides an unambiguous procedure for accepting work that falls between acceptable and rejectable limits.

v) Random sampling procedures avoid biases and lead to a more reliable estimate of the as-built construction quality (21).

vi) Statistical specifications are easier to write, to interpret, to enforce and to apply.

vii) They produce accurate data obtained with valid random sampling procedures. This data may be analyzed later to improvise the specifications further (24).

6.5 Quality Control Plan

The importance of the quality control function cannot be overemphasized. Deming states that 'Quality must be built into the work. It cannot be inspected in"(25). John Ruskin said, "Quality is never an accident. It is always the result of intelligent effort." This effort starts with an approved quality control plan (25).

If the contractor is to assume the responsibility of process control, it is imperative that the contractor should posses a functional and responsive QC plan. The specifications must provide guidelines for the QC plan, to ensure at least minimum level of QC and uniform bidding (23). KDOT requires all contractors to submit a quality control plan using an example provided in the Department's SD/SF manual as a guideline (26).

6.6 Mix Design

The mix design is a necessary and critical part of the effort to control the quality of the construction item. Tests should be performed during the mix design development to ensure that material manufactured in accordance with the mix design specifications will perform as required by the contract. KDOT requires the contractors to submit trial mix design (with supporting test results) which after an approval process will be accepted as the design job mix formula (JMF). The details of the mix design requirements (based on Superpave guidelines) for 7 different mix types are given in the specifications' (26).

6.7 Concept of Normal Distribution

It is fully impossible to achieve production matching the exact characteristics of design JMF. The basic edifice of statistical specifications is that the measurements of all mix properties will vary widely around their target mean values (μ), usually in the form of a bell-shaped normal distribution. The foundation of statistical specifications lies in recognizing and dealing with the sources of variability. These may be material, process or sampling and testing variability. The primary function of statistical data analysis is to estimate the mean and variability of the measured characteristics in a sound statistical manner under the concept of normal distribution.

Quality is judged by accuracy and precision of selected properties of the finished product. Accuracy will be determined in terms of proximity of the estimated mean μ to target value set by the approved JMF, which is usually a constant for a particular project. Precision will be measured in terms of variability of measured values (See Figures 6.1 and 6.2) (24). Figure 6.2 (left) shows two distributions of the measured values of a mix property. Both distributions have mean equal to target value, but with different standard deviations (σ). The second distribution, with a smaller variability represents a higher level of control. Figure 6.2 (right) shows two distributions, each with different means and standard deviations. The first distribution with mean equal to the target value has better central tendency control (accuracy), while the second distribution with smaller standard deviation has better variability control (precision) (27).

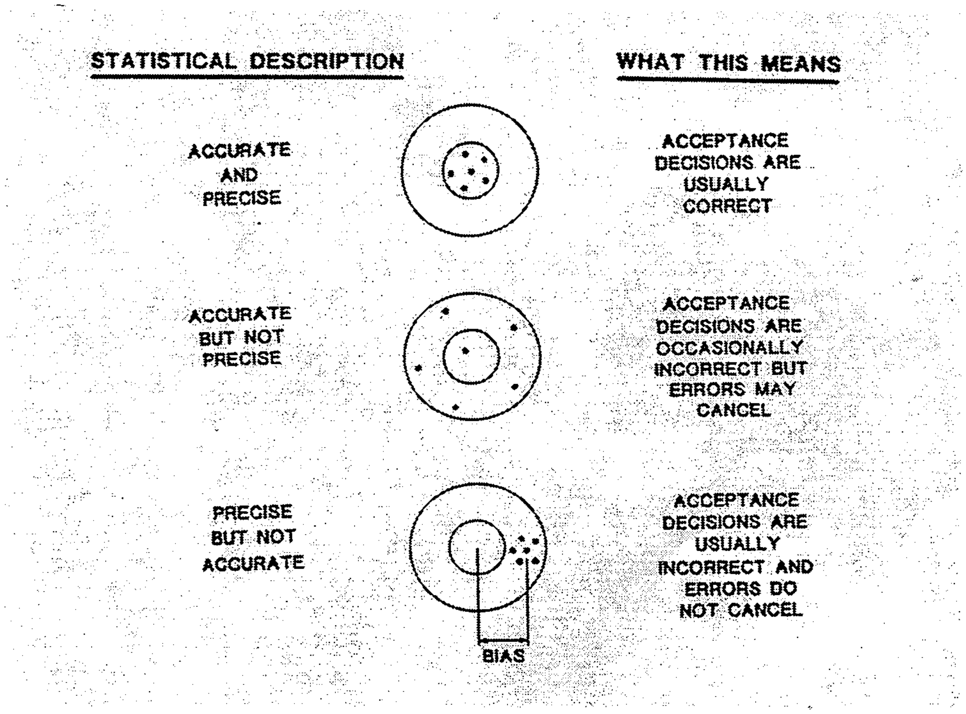


FIGURE 6.1 Concepts of Accuracy, Precision and Bias (Source: 24)

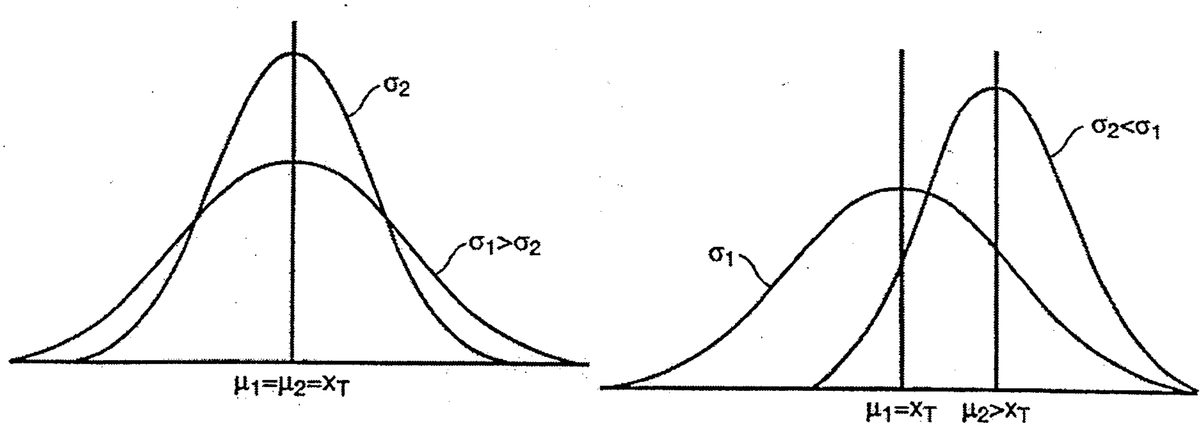


FIGURE 6.2 Central Tendency and Variability Control (Source: 27)

6.8 Specification Limits as Control Criteria

Specifications aim to control both the mean as well as variability by setting limits about target values. The estimate of variability, which is usually the standard deviation (σ), is used to set up realistic specification limits about the target JMF value assuming normal distribution of the measure of that property.

The empirical rule states that for a normal distribution, the intervals $\mu \pm \sigma$, $\mu \pm 2\sigma$, and $\mu \pm 3\sigma$ will contain approximately 68, 95, and 100 percent of the test results, respectively (28). An Upper and Lower Specification Limits, which mark the allowable deviation from the target value, can be set in terms of "Cσ", where "C" is a constant and "σ" is the standard deviation of the measured property. The value of "C" depends upon the level of variability that is decided to be acceptable (for example, 2 times σ, 2.5 times σ, etc.) and is a rather subjective management decision. The specifying agency should study contractor capabilities, state of the art recommendations, and available historical database and then set acceptable ranges based on these factors coupled with the criticality of the item being specified.

A normal distribution with upper and lower specification limits is shown in Figure 6.3. Thus the tails of these distributions may extend into the hatched areas representing unacceptable performance. But as long as the measured value of a characteristic falls within the range bounded by the limits, it is considered to be part of an acceptable population with the target value as the mean. And on the whole, the occurrence of a relatively small percentage of test results falling outside specification limits will be considered normal.

The hatched area also represent the seller's risk (α), which is the probability that a satisfactory product is rejected. Buyer's risk (β) is the probability that an unsatisfactory product is accepted. To reduce the buyer's risk and to break the production into manageable size portions, the specifications are developed for the mean of multiple samples (Lot mean values) instead of single samples (sublot test values). Note that this does not affect the seller's risk in any way.

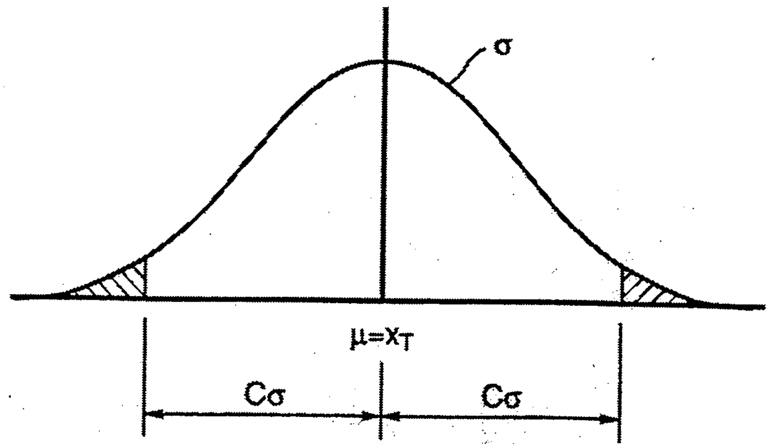


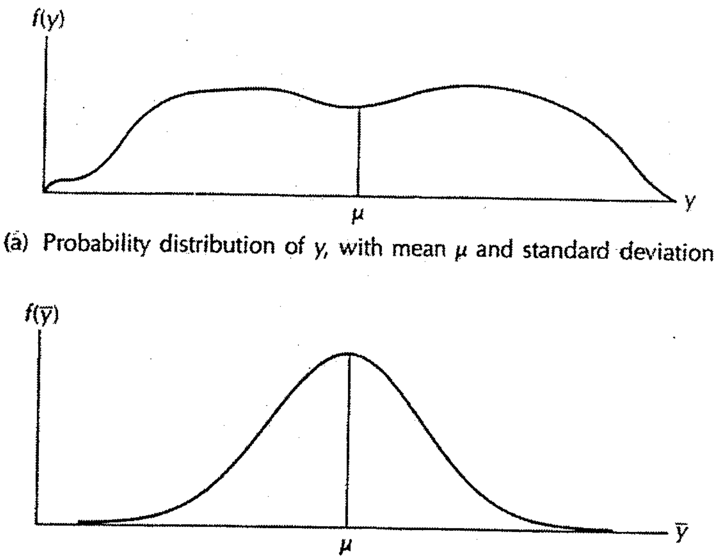
FIGURE 6.3 Normal Distribution with Specification Limits (Source: 27)

6.9 Statistical Estimates of Population Parameters

It must be noted that the mean (μ) and standard deviation (σ) described earlier are measures of the entire population i.e. the entire mix production. They are referred to as population parameters (28). However, in practice we do not test the entire quantity of the mix produced, but test a predetermined number of samples randomly selected from this population. Thus in practice, μ and σ are unknown and the important assumption of normal distribution of the population is not always true.

The Central Limit Theorem provides a solution to these problems. It states that, 'If random samples of n measurements each, are repeatedly drawn from a population with a finite mean μ and a standard deviation σ, then when n is large, the relative frequency histogram for the sample means y , obtained from repeated sampling will be approximately normal with mean μ and a standard deviation σ/ _n. The quantity σ/_n is referred to as the standard error of y. The sample mean and sample standard deviation are called sample statistics. Note that the theorem does not assume or require a normally distributed population (See Figure 6.4) (28).

The Central Limit Theorem will hold for $n > 30$, because for $n < 30$ the sampling distribution will follow a t distribution, which is slightly skewed then the normal distribution. However, if the underlying population is symmetric, the theorem will hold for $n < 30$ also (28).



(a) Probability distribution of y, with mean μ and standard deviation σ

(b) Sampling distribution of $\bar{y}$, with mean μ and standard error $\sigma/\sqrt{n}$

FIGURE 6.4 Probability Distribution of y and Sampling Distribution of y (Source 28)

Many times in practice, we are provided with only one sample for the entire population. As a rough rule, the statistics for this single sample can be used directly as point estimates of the population parameters, when the sample size n is larger than 30 (28).

It has also been shown that, the sample standard deviation s, is an unbiased estimator of the population standard deviation. If we were to draw a very large number of samples, each of size n, from the population and compute the s for each sample, then the average sample standard deviation should be equal to the population standard deviation (28).

6.10 Process Control

Quality control is aimed at ensuring that the mix produced in the plant is the same as the JMF approved in the laboratory. An important feature of QC operation is to keep the production process under control, quickly determine when it goes out of control and take remedial measures to bring it within control. So long as the measured value of a characteristic falls within the range formed by the specification limits, the process is considered to be in control. Graphical techniques provide much simplicity in statistical analysis. A statistical quality control chart serves as a visual monitor of the health of the production process by detecting 'out of control' and 'shift in quality' situations.

KDOT specifies the required production in terms of specification working ranges (limits) for 11 mix parameters as shown in Table 6.1. KDOT uses dual criteria of single point test value and four-point moving average for specification limits. If two consecutive single point test results or any one four-point moving average value fails to fall within the respective established limits, KDOT requires suspension of mix production until appropriate corrective measures are adopted (26).

KDOT requires establishment of QC charts to monitor previously mentioned 11 mix characteristics and the compacted pavement density as part of quality control requirements (26). A typical QC chart for asphalt content test data for SM-1B mix on US-50 project is shown in Figure 6.5.

TABLE 6.1 KDOT's Specification Working Ranges (Limits)

Mix Characteristic	Tolerance from JMF Single Test Value	Tolerance from JMF 4 Point Moving Average Value
Binder Content	+/- 0.6%	+/- 0.3%
Gradation:- All applicable sieves for the mix	NA	zero tolerance
Air Voids at N_{des} gyrations	+/- 2.0%	NA
Voids in Mineral Agg.	1.0% below min.	zero tolerance
Voids Filled with Asphalt	NA	zero tolerance
Course Agg. Angularity	zero tolerance	NA
Sand Equivalent	zero tolerance	NA
Fine Agg. Uncompacted Voids	zero tolerance	NA
Tensile Strength Ratio	zero tolerance	NA
Density at Nini and Nmax	NA	zero tolerance

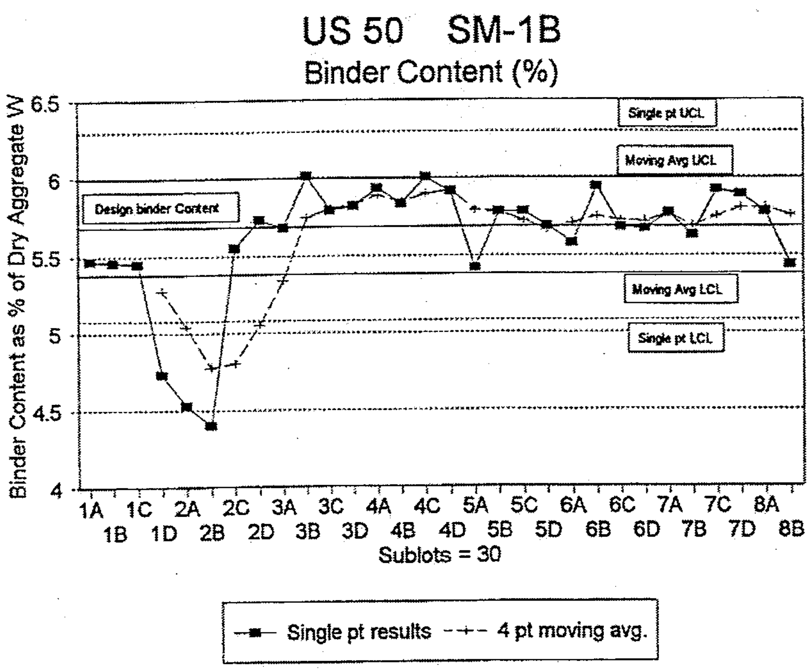


FIGURE 6.5 Typical Quality Control Chart for Binder Content (for SM-1B, US-50)

6.11 Acceptance Plan

The statistical acceptance plan defines a set of rational procedures to be used by the agency to decide the degree of compliance with contract requirements and value of product delivered by the contractor based on statistical probabilities (23). An acceptance plan defines the lot size, the characteristics on which acceptance will be based, the sample size, the random sampling procedure, the test method and the procedure for analysis of test results to arrive at an acceptance decision. In an attribute-type plan, the sample is either accepted or rejected. A variable plan makes efficient use of all information (mean, standard deviation, etc..) to arrive at an acceptance decision with greater discriminating power. Variable plans produce a continuous result, which is more suitable for developing adjusted pay schedules (21).

The KDOT mix acceptance plan is a combination of variable types (for air voids and compaction) and attribute-type for all other properties. Acceptance will be decided on a lot basis after satisfactory test results for all applicable tests have been obtained (26).

6.12 Lot and Sample Sizes

The definitions of lot and sublot sizes are of prime importance in developing a statistical specification. A single lot should be a homogeneous isolated quantity produced from a single source under similar conditions so that it can be considered as a population. Lot sizes can be based either on quantity or time basis. The sample size is basically governed from a standpoint of adequacy for conducting statistical analysis and is limited to a minimum of three. The sample size divides the lot into equal size sublots.

KDOT has defined a standard mix production lot size on a quantitative basis as 3,000 tons. The sample size specified is four, resulting in four sublots of 750 tons in each lot. However, for the roadway density determination lot size is defined on a time basis as one day's production. This is subject to a minimum of 1,000 tons daily production (quantitative basis). Each lot is divided into five sublots with two tests per each sublot (26).

6.13 Selection of Mixture Characteristics

A critical aspect of specification development is the selection of control properties that are more critical in determining product performance to a degree that is logical, equitable and legally defensible. Because of property interrelationships, continual control of all mix properties is not necessary. A practical and manageable set can be selected for monitoring under quality control, a further critical subset can be isolated from these for basing payment decisions.

KDOT specifications include QC tests to ensure conformance of 18 important mix parameters. Out of these, 11 properties shown in Table 6.1 along with compacted pavement density are monitored under quality control operations using control charts. Among these payment is based on air void percentage and density only (26).

6.14 Sampling Procedure

The assumption of random sampling is the most critical theoretical condition on which statistical analysis is based. Only when all vestiges of personal bias are removed can the laws of statistical probability be relied upon to function properly (21). Random sampling ensures that every possible sample quantity in a lot possesses an equal opportunity to be selected as sample. Pure random, stratified random and discrete stratified random are the commonly used sampling procedures. KDOT does not specify a particular random sampling procedure but requires that the procedure used must be approved by the Engineer. The Contractor or KDOT can generate the random locations before sampling (26).

6.15 Testing Methods and Frequencies

During construction operations, QC testing should be performed at a frequency set forth in the contractor's QC plan (approved by the agency). The agency must determine the acceptance testing frequency depending upon practical constraints such as personnel, time and whether it has decided to use QC test results for drawing acceptance decision. It must be noted that the frequency of testing (either contractor's QC or agency's acceptance) used for Lot acceptance must be at least equal to the sample size required for statistical analysis. The agency should also establish the testing frequency for the independent assurance program. In general, the IA testing frequency is 10 percent of the acceptance testing frequency (23). An alternative method bases the IA frequency on source, where the personnel and equipment will be verified at a predetermined frequency irrespective of location.

KDOT has developed a sampling and testing frequency chart for 18 mix properties, covering the test methods to be adopted and the testing frequency under QC, Acceptance and IA for each property. KDOT has chosen to base IA testing frequency on source, i.e. the certified technicians will be verified once every year (26).

6.16 Validation of QC Test Results

Acceptance based solely on the agency's testing requires an uneconomically high-test frequency. To increase the amount of available data for basing acceptance decisions, the contractor's test results, which are a legitimate source, can be incorporated. But a statistically correct validation system must be used to ensure the reliability and accuracy of the contractor's test results against those obtained by the agency. In this case, the contractor's QC testing frequency must be equal to the statistically required sample size. The agency's testing frequency will be much lower but should be statistically sufficient to form a basis for the validation process.

KDOT has chosen to use the QC test results for acceptance. KDOT follows the validation procedure suggested in AASHTO's Implementation Manual for Quality Assurance (26). According to present testing frequency requirements, ten QC and five verification tests are available per Lot for compaction density. While four QC and one verification test are available per Lot for air voids and binder content.

6.17 Basis of Acceptance

The objective of the acceptance program is to determine the degree of compliance with the contract requirements and the value of the finished product. AASHTO recommends the use of Percent Within Limit (PWL) analysis, also popularly known as Quality Level Analysis, for this purpose (23). This variability unknown procedure controls both central tendency and variability. Assuming normal distribution and random sampling it estimates the portion of each lot of construction item that may be expected to be within specified tolerance limits and is strongly preferred by the highway engineers (21). Figure 6.3 displays the concept of PWL analysis. The unshaded portion between the specification limits represents the PWL.

However, KDOT has preferred the use of a methodology based on variability known procedure using mean of absolute deviations from target values for air voids analysis (26). This method proposed by Frazier, et.al. at the Highway Research Center, Auburn University was published in the Transportation Research Board's (TRB) Transportation Research Record 1389 and its use as an alternative method has also been suggested by AASHTO (23). This method based on the variability known concept, also controls both central tendency and variability (27). Figure 6.6

compares the distribution of absolute deviations from target values with the normal distribution. Figure 6.7 shows a typical distribution of absolute deviations from target values for a sample size of four. For the standard distribution of absolute deviation, (corresponding to a standard normal distribution with mean = 0 and standard deviation = 1) the mean is given by (27)

$$\mu' = \sqrt{\left(\frac{2}{\pi}\right)} \qquad (6.1)$$

$$\sigma' = \sqrt{(1 - (\frac{2}{\pi}))} \qquad (6.2)$$

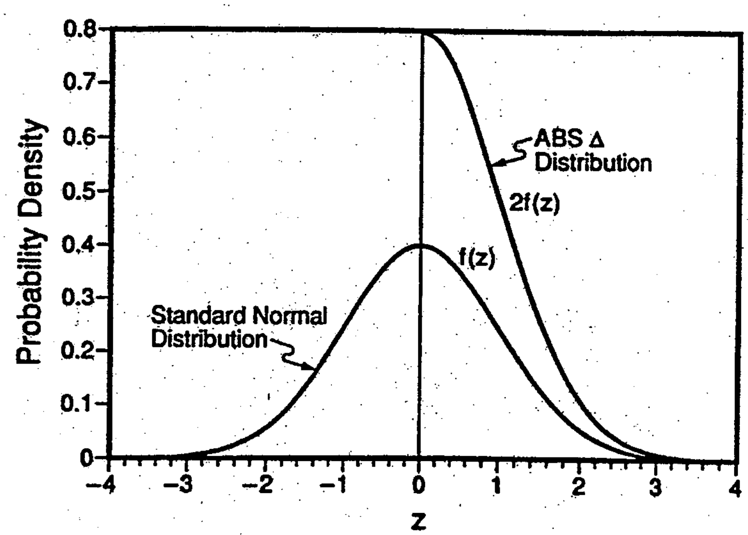


FIGURE 6.6 Distribution of Absolute Deviation in Comparison with Standard Normal Distribution (Source: 27)

For absolute deviation distribution of a parameter with historically known or assumed standard deviation (SD), mean and standard deviation are given as (27)

$$\mu = \mu' * SD \quad (6.3)$$

$$\sigma = \sigma' * SD \quad (6.4)$$

The shape of this distribution is a function of sample size, n. As n increases the shape approaches that of normal distribution according to the Central Limit Theorem. Thus, the mean will remain the same as n=1, for any other sample size, the standard deviation will change as (27)

$$\sigma_n = \frac{o'}{\sqrt{(n)}} \quad (6.5)$$

The quality level analysis or the alternative method can be applied to govern any parameter. But for compacted pavement density analysis, KDOT uses a simple average for lot and lowest test value for a sublot method (26).

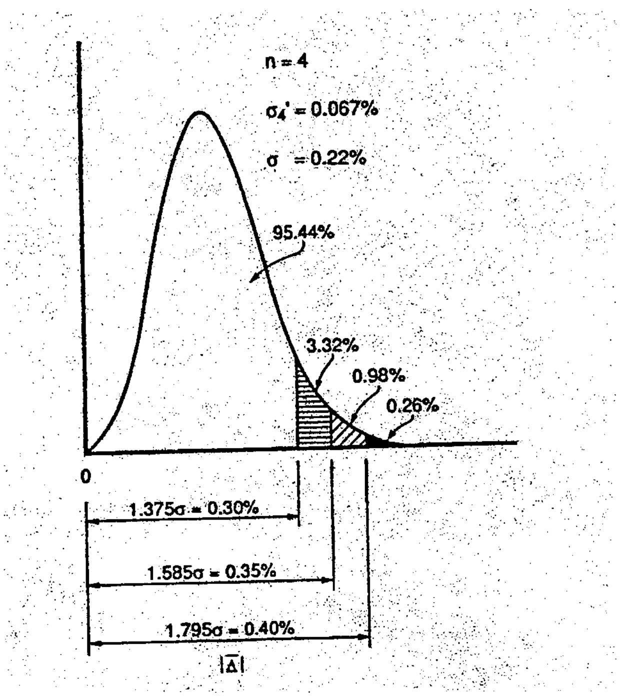


FIGURE 6.7 Typical Distribution of Absolute Deviation for a Sample Size of Four (Source: 27)

6.18 Measures of Quality

The measure of quality implies the mode of measurement for the quality of a product in terms of its properties like asphalt content, air voids, etc.. If the measure of quality is strongly related to the final performance of the product, then it can be used as a scale for deciding payment. The PWL is a popular statistical measure of quality, which estimates the percentage of the work falling within specification limits and is strongly related to actual performance. The acceptable quality level AQL, is that level of measure of quality, which the specifying agency is willing to accept at 100 percent payment. The rejectable quality level, RQL is that level of measure of quality, which is so deficient that it warrants immediate repair and/or replacement and should be chosen with restraint. In between AQL and RQL, acceptance at a reduced payment is made using the adjusted pay schedules (21).

In KDOT specifications, the Lot mean of absolute deviations from the target value is the measure of quality in air voids analysis. While the average of 10 test results covering each Lot and the lowest sublot test value are the measures of quality for density analysis (26).

6.19 Relating Quality to Performance - Development of Pay Adjustment Schedules

A common feature of most performance-based statistical specifications is the adjusted pay schedule. Based on the procedure used to arrive at the original design parameters of the pavement, the as-built parameters can be used to back-calculate the fraction of design loadings the pavement will actually be able to sustain (21). If the actual life exceeds the design life then the agency has profited in terms of cost per unit life and vice versa. The appropriate pay adjustment is the present worth of the sum of these credits or debits and, thus may be positive or negative, respectively. Based on (mechanistic) design principles and the concept of liquidated damages, pay adjustment schedules provide the most practical approach of deciding the actual value of a product falling between AQL and RQL.

The development of pay schedules/equations is essentially formulating the relationship between product quality and performance. Selection of consistent limits for applications of various levels of pay adjustments is similar to the selection of acceptance criteria (27). The pay schedule/equation should be based on material properties, which are significant predictors of pavement performance and can be controlled during production. The quality is quantified in terms of measures of quality for these properties. Historical databases or sound engineering principles provide the life cycle costs (performance) associated with different quality levels. This data is used along with the principle of liquidated damages to establish rational performance-based pay schedule/equation.

To the extent possible pay schedules must be based on design models that relate mix properties to the pavement performance. Unfortunately, in case of flexible pavements, it is not feasible to actually monitor the fundamental mix properties incorporated in design models (like tensile strain, dynamic modulus, creep, etc..), under a QC/QA program for deciding pay adjustments. On the other hand, for construction characteristics commonly measured under QC/QA (like air voids and binder content), their qualitative relationship with performance is not well established and known only in a vague manner. **This is the basic reason underlying the need to develop equations relating important mix characteristics routinely monitored under QC/QA (like air voids and binder content) to product performance. It is imperative to at least incorporate these properties in the present design models.**

Presently theoretical considerations and engineering judgement provide insights to aid in selection of AQLs and RQLs. The decision process is somewhat subjective and selection of various levels of the constant C is often based on tolerable probabilities for pay reduction. The usual approach is to set limits, which are intuitively correct and consistent with concepts of causes of failure in final products. Intuitively, if the measured variability is greater than historical variability, the probabilities of pay reduction should be more and vice versa. Similarly, the increase in deviation from the target value will lead to decrease in performance and should be subjected to corresponding increase in pay reduction (27).

6.20 KDOT Pay Schedules

KDOT uses pay schedules based on air voids and density to evaluate final value of the finished item. These were adopted from the Minnesota DOT specifications (29). A separate price adjustment clause covers the deficiencies arising out of all other properties and authorizes the agency's engineer to accept the lot at the lowest adjusted price (26). The pay schedule for air voids (Lot size of 4 sublot test results) is given in Table 6.2. The mean of absolute deviations from the target value for four test results (D4) covering a lot is used to estimate the pay factor. For conditions providing less or more than four (3, 5 and 6) sublot test results alternative schedules are given in the specifications. The air voids pay adjustment factor is multiplied with the bid price per ton to get the adjusted price for that lot (26).

TABLE 6.2 KDOT Pay Schedule for Air Voids (Lot Size of Four Tests)

Air Voids Deviation Value	Pay Factor
0.00 ≥D4≥0.35	1.03
0.36 ≥D4≥0.55	1.02
0.56 ≥D4≥1.05	1.00
1.06 ≥D4≥1.24	0.95
1.25 ≥D4≥1.40	0.90
1.41 ≥D4	0.80

The pay schedule for compacted pavement density is given in Table 6.3. The average of 10 test values covering a Lot and the lowest test value of a sublot are used to estimate two pay factors. These are combined to get the density pay adjustment factor, which is multiplied with the bid price per ton to get the adjusted price for that lot. The tables provide separate values for SM-1T mix, which is used in surface layers (26).

TABLE 6.3 KDOT Pay Schedules for Compacted Pavement Density

% of Maximum Specific Gravity Average of Lot (10 Density Tests)*	Pay Factor A (% of Contract Price)	
	All Mixes Except SM-1T	SM-1T
93.0% or Greater	1.02	1.02
92.0% to 92.9%	1.01	1.01
91.0% to 91.9%	1.00	1.00
90.0% to 90.9%	0.99	1.00
89.0% to 89.9%	0.965	0.99
88.0% to 88.9%	0.94	0.965
87.0% to 87.9%	0.90	0.94
86.0% to 86.9%	0.70	0.90
Less than 86.0%	0.70	0.70

(continued on page 78)

TABLE 6.3 KDOT Pay Schedules for Compacted Pavement Density (cont)

% of Maximum Specific Gravity Lowest Average of any sublot (2 Density Tests)	Pay Factor B (% of Contract Price)	
	All Mixes Except SM-1T	SM-1T
91.0% or Greater	1.02	1.02
90.0% to 90.9%	1.01	1.01
89.0% to 89.9%	1.00	1.00
88.0% to 88.9%	0.99	1.00
87.0% to 87.9%	0.97	0.99
86.0% to 86.9%	0.94	0.97
85.0% to 85.9%	0.90	0.94
84.0% to 84.9%	0.80	0.90
Less than 84.0%	·0.80	0.80

6.21 Evaluating Risks with Operating Characteristic Curves

An important step in the development of statistical specification is the construction of operating characteristic (OC) curve. This determines, in advance, if the acceptance procedure and pay schedules will function as intended. The OC curve estimate the risks to both the agency (β) and the contractor (α), which can then be controlled at suitable levels to develop equitable and effective specifications. The contractor can also use them to develop appropriate bid price and production strategies (24).

A conventional OC curve for the attribute-type specifications is shown in Figure 6.8. Probability of acceptance is the ordinate and quality levels are indicated on the abscissa. The contractor's risk, α of acceptable (AQL) material being rejected and the agency's risk, β of rejectable (RQL) material being accepted are illustrated in Figure 6.8. Figure 6.9 displays an OC curve for the variable acceptance plan based on adjusted pay schedule (23). A common mistake is to assume that OC curve and adjusted pay schedule are the same. The pay schedule provides the pay factor to be awarded for a particular level of quality, while the OC curve determines whether the use of pay schedule will achieve desired results in the long run or not (21).

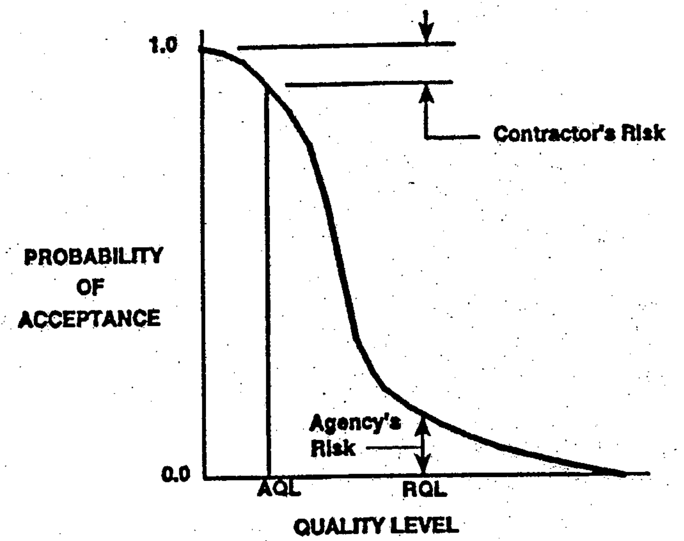


FIGURE 6.8 Conventional OC Curve (Source: 23)

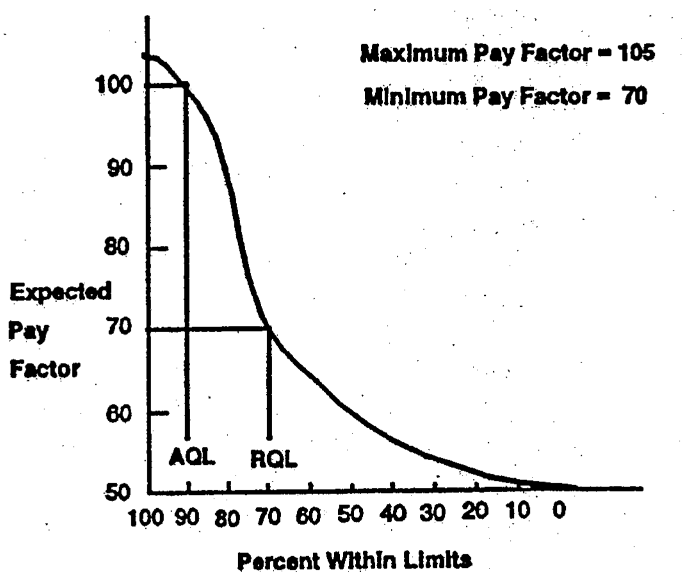


FIGURE 6.9 Typical OC Curve with an Adjusted Pay Schedule (Source: 23)

CHAPTER 7

ANALYSIS OF QC/QA VOLUMETRIC TEST DATA

7.1 Data Collection

The complete QC/QA test data for US-50, which was a smaller project, covered results for 19 sublots of SM-1T (surface) and 30 sublots of SM-1B (base) mixes. The data for K-177, which is a larger project was received in stages. Data for 14 sublots of SM-1T (surface) mix and 85 sublots of SM-2C (base) mix for the northbound lanes were received first. Later the data for 24 sublots of SM-1T (surface) mix, 26 sublots of SM-2C (base) mix and 46 sublots of SR-2C (base) mix used on the south bound lanes was received and analyzed. The test results for SM-2C and SM-1T mixes from the two phases of K-177 project were analyzed separately. Because of the time gap between production, they were assumed to be representing different populations. Besides for the SM-1T mixes, the binder content for the two different phases is not the same.

During the fall of 1997, KDOT expanded the scope of the present study. Statistical data analysis of test results covering six more mixes from two new pilot projects constructed during the 1997 construction season was to be included. Accordingly, data for the two projects namely, the I-70 project in Saline County, Kansas, and K-96 project in Sedgwick County, Kansas, was received and analyzed. The data for I-70 project covered 50 sublots of SM-2C mix, 68 sublots of SR-2C (base) mix and 60 sublots of SR-2C (shoulder) mix. While, the data for K-96 project covered 13 sublots of SM-1T (surface) mix, 60 sublots of SR-2C (base) mix and 28 sublots of SR-2C (shoulder) mix. New data has since been analyzed and the findings incorporated into the study.

7.2 Validation of Contractor's QC Data

KDOT's verification test results were available for US-50; I-70; and K-96 projects but not for K-177 project. Appropriateness of statistical analysis done for K-177 data is subject to validation of contractor's QC test results against agency's test results. For the US-50, I-70 and K-96 projects, Contractor's QC test results were validated before conducting statistical analysis using the procedure recommended in AASHTO's Implementation Manual for Quality Assurance (23). However, the binder content test results for SR-2C (shoulder) and SM-1T mixes on K-96 could not be validated due to insufficient number of verification tests results, 1 and 2, respectively.

As explained earlier, present testing frequency requirements provide 10 QC and five verification test results per Lot for compaction density. Therefore, the AASHTO procedure can be used to validate Contractor's test results on a Lot wise basis for compacted density from the first Lot itself. This can also be achieved with a minimum of three verification tests per Lot.

At present four QC and one verification tests are available per Lot for air voids and binder content. Hence the AASHTO procedure cannot be employed for a Lot wise validation of Contractor's test results. At least three Lots must be produced, before the AASHTO procedure can be used with a minimum of three verification test results. At that stage, an inference of invalidity for the Contractor's test results can be of limited use, since the 3 Lots in question are already in place.

Thus the agency's verification testing frequency is inadequate for Lot wise validation of air voids and binder content QC test results. While, it is more than adequate for compacted pavement density results. Also during data analysis it was observed that the agency's actual verification testing frequency is much lower than that stated in the specifications.

7.3 Selection of Volumetric Properties for Analysis

Under the existing QC/QA system, test data for 18 mix properties are available. But due to low testing frequency for many properties, like aggregate angularity, gradation, etc., the number of test results (data points) generated were insufficient for statistical analysis.

An important task of this research was to correlate the volumetric properties with the mechanistic test results. AI and SHRP have incorporated volumetric properties like air voids into their transfer functions. The volume of air voids and binder content were chosen for this analysis because it is felt that these strongly affect mixture performance and mechanistic response. Due to interrelationship among different volumetric properties, analysis of other dependent properties like VMA, VFA, etc. was not felt necessary. The compacted pavement density was also analyzed as it is used for deciding pay adjustment. In fact, some researchers feel that compacted density is the most important factor in judging mixture performance (30).

7.4 Methods of Computing Sample Statistics

The sample statistics (y and s), can be used to estimate corresponding population parameters (μ and σ). In present situation, assuming the entire production for a mix type to be the population, we can compute the sample statistics for it under three different interpretations.

i) Central Limit Theorem: The four test results covering each lot will represent a stratified random sample, with each test result on a sublot representing an individual sample measurement. Thus the number of lots produced will correspond to the number of repeated sampling and the sample size, n will be equal to 4. We can calculate the sample mean, y for each lot (sample) as the average of four test results covering that lot. Then the mean of all y values will estimate the population mean μ and the standard deviation of y values multiplied by square root of sample size n, will estimate the population standard deviation σ.

ii) Rough Rule to be Used in Case of a Single Sample: Each test result covering each sublot will represent a sample measurement. The set of all test results for a particular mix will constitute a single sample of size n. Thus the total number of sublots produced will correspond to the sample size n, which needs to be greater than 30. We can calculate the sample mean y and the sample standard deviation s of all test results covering the entire mix production. These will then serve as point estimates of the population mean μ and population standard deviation σ, respectively. Note that the population mean estimated by approaches (i) and (ii) will be the same.

iii) Method 3: Similar to Central Limit Theorem, each lot is considered as a stratified sample with sublot test results as individual sample measurements. The sample standard deviation s of four test results covering each lot (sample) is computed. Then the mean of all s values will estimate the population standard deviation σ. This method does not estimate the mean μ.

7.5 Data Analysis

The data received for the 13 mixes used on four pilot Superpave projects was catalogued in QuattroPro (Version 6.1) spreadsheets. This also aided the creation of statistical quality control charts. The mean and standard deviation for the test results of three mix properties namely air voids, binder content and percent compacted pavement density, were calculated using three different approaches described earlier. The test statistics are presented in Tables 7.1, 7.2, 7.3, 7.4 and 7.5.

TABLE 7.1 Sample Statistics for Air Voids Test Results

Project	Mix Designation	APPROACH FOR COMPUTING SAMPLE STATISTICS					
		i) Central Limit Theorem		ii) Rough Rule For One Sample		Method #3 For Standard Deviation	
		MEAN	STD. DEV	MEAN	STD. DEV	MEAN	STD. DEV
US-50	SM-1T	4.9	2.96	4.76	1.53	-	0.96
	SM-1B	3.35	1.52	3.32	1.04	-	0.78
K-177	SM-1T	3.09	0.4	3.08	0.63	-	0.65
	SM-2C	3.92	1.62	3.93	0.97	-	0.59
	SM-1T	3.59	1.28	3.59	0.89	-	0.68
	SM-2C	4.18	1.64	4.11	1.23	-	0.92
	SR-2C	3.79	1.92	3.77	1.09	-	0.58
I-70	SM-2C	4.12	1.00	4.13	0.62	-	0.39
	SR-2CB	3.90	0.96	3.90	0.64	-	0.45
	SR-2CS	3.63	1.16	3.63	0.75	-	0.49
K-96	SM-1T	4.47	2.06	4.38	1.30	-	1.03
	SR-2CB	4.02	1.02	4.02	0.77	-	0.63
	SR-2CS	3.88	0.80	3.88	0.74	-	0.67

TABLE 7.2 Sample Statistics for Air Voids Test Results in terms of Absolute Deviations from Target Value

Project	Mix Desig-nation	APPROACH FOR COMPUTING SAMPLE STATISTICS					
		i) Central Limit Theorem		ii) Rough Rule For One Sample		Method #3 For Standard Deviation	
		MEAN	STD. DEV	MEAN	STD. DEV	MEAN	STD. DEV
US-50	SM-1T	1.33	2.38	1.22	1.18	-	0.65
	SM-1B	1.03	0.96	1.04	0.68	-	0.52
K-177	SM-1T	1.00	0.12	1.00	0.49	-	0.49
	SM-2C	0.79	0.78	0.78	0.56	-	0.48
	SM-1T	0.81	0.68	0.81	0.53	-	0.47
	SM-2C	0.88	0.92	0.90	0.83	-	0.74
	SR-2C	0.91	0.80	0.91	0.63	-	0.51
I-70	SM-2C	0.48	0.42	0.49	0.40	-	0.32
	SR-2CB	0.49	0.54	0.49	0.42	-	0.35
	SR-2CS	0.59	0.86	0.59	0.59	-	0.4
K-96	SM-1T	1.08	0.62	1.05	0.8	-	0.83
	SR-2CB	0.59	0.50	0.59	0.48	-	0.45
	SR-2CS	0.61	0.38	0.61	0.41	-	0.40

TABLE 7.3 Sample Statistics for Binder Content - Single Point Test Results

Project	Mix Desig-nation	APPROACH FOR COMPUTING SAMPLE STATISTICS					
		i) Central Limit Theorem		ii) Rough Rule For One Sample		Method #3 For Standard Deviation	
		MEAN	STD. DEV	MEAN	STD. DEV	MEAN	STD. DEV
US-50	SM-1T	5.55	0.4	5.56	0.2	-	0.11
	SM-1B	5.61	0.6	5.61	0.4	-	0.24
K-177	SM-1T	5.51	0.16	5.52	0.11	-	0.09
	SM-2C	5.14	0.3	5.14	0.24	-	0.2
	SM-1T	6.53	0.5	6.53	0.37	-	0.3
	SM-2C	5.46	0.54	5.44	0.35	-	0.27
	SR-2C	5.48	0.7	5.46	0.42	-	0.22
I-70	SM-2C	4.64	0.44	4.63	0.30	-	0.23
	SR-2CB	3.93	0.30	3.93	0.22	-	0.18
	SR-2CS	4.25	0.42	4.25	0.26	-	0.17
K-96	SM-1T	6.82	0.04	6.82	0.62	-	0.33
	SR-2CB	5.20	0.24	5.20	0.20	-	0.18
	SR-2CS	5.57	0.34	5.57	0.23	-	0.18

TABLE 7.4 Sample Statistics for Binder Content - Moving Average Test Results

Project	Mix Designation	APPROACH FOR COMPUTING SAMPLE STATISTICS					
		i) Central Limit Theorem		ii) Rough Rule For One Sample		Method #3 For Standard Deviation	
		MEAN	STD. DEV	MEAN	STD. DEV	MEAN	STD. DEV
US-50	SM-1T	5.56	0.22	5.58	0.12	-	0.08
	SM-1B	5.64	0.64	5.62	0.33	-	0.08
K-177	SM-1T	5.53	0.10	5.53	0.05	-	0.02
	SM-2C	5.13	0.30	5.14	0.16	-	0.07
	SM-1T	6.55	0.34	6.56	0.20	-	0.11
	SM-2C	5.50	0.52	5.48	.27	-	0.12
	SR-2C	5.49	0.62	5.48	0.33	-	0.12
I-70	SM-2C	4.55	0.60	4.55	0.39	-	0.19
	SR-2CB	3.91	0.24	3.92	0.13	-	0.06
	SR-2CS	4.23	0.32	4.23	0.19	-	0.08
K-96	SM-1T	6.91	0.22	6.91	0.13	-	0.09
	SR-2CB	5.20	0.20	5.20	0.11	-	0.06
	SR-2CS	5.56	0.30	5.57	0.15	-	0.05

TABLE 7.5 Sample Statistics for Percent Compacted Pavement Density

Project	Mix Designation	APPROACH FOR COMPUTING SAMPLE STATISTICS					Average of Lowest Sublot Density
		i) Central Limit Theorem		ii) Rough Rule For One Sample		Method #3	
US-50	SM-1T	89.7	1.50	89.6	1.67	1.61	87.8
	SM-1B	90.6	1.32	90.6	1.26	1.14	89.33
K-177	SM-1T	92.9	2.46	92.7	2.39	2.26	89.8
	SM-2C	91.7	2.84	91.7	1.57	1.06	90.5
	SM-1T	89.0	2.1	89.0	1.84	1.4	87.4
	SM-2C	93.1	1.9	93.1	1.4	1.17	91.9
	SR-2C	92.3	3.18	92.3	1.79	1.18	90.8
I-70 Combined 235 Sublots	SM-2C	92.65	1.65	92.66	0.99	0.63	91.92
	SR-2CB						
	SR-2CS						
K-96	SM-1T	92.11	2.12	91.99	1.17	0.87	90.99
	SR-2CB						
	SR-2CS						

7.6 Pay Factors

For the two pilot projects, air voids and compacted pavement density pay factors were calculated for each lot of each mix using KDOT pay schedules described earlier and a few other methods like AASHTO, NHI (21), and Nebraska Department of Roads (NDOR) (29) for comparison. Pay factors were also calculated for binder content using the AASHTO and NHI methods. The summary statistics of the pay factors calculated for air voids, binder content and compacted pavement density are shown in Tables 7.6, 7.7 and 7.8, respectively.

TABLE 7.6 Air Voids Pay Factor Statistics

Project	Mix Desig-nation	AIR VOIDS PAY FACTORS					
		1. KDOT Pay Factors		2. AASHTO Pay Factors		3. NHI Pay Factors	
		MEAN	STD. DEV	MEAN	STD. DEV	MEAN	STD. DEV
US-50	SM-1T	0.96	0.04	97.06	17.75	94.0	24.6
	SM-1B	0.96	0.05	101.98	4.67	101.98	4.67
K-177	SM-1T	0.98	0.03	104.74	0.46	104.74	0.46
	SM-2C	0.99	0.04	104.06	2.56	104.06	2.56
	SM-1T	0.98	0.05	103.39	3.95	103.39	3.95
	SM-2C	0.97	0.37	99.1	5.53	99.1	5.53
	SR-2C	0.96	0.05	101.69	4.65	101.69	4.65
I-70	SM-2C	0.99	0.03	104.94	0.23	104.94	0.23
	SR-2CB	1.02	0.02	104.68	1.33	104.68	1.33
	SR-2CS	1.00	0.06	103.51	4.5	103.01	6.41
K-96	SM-1T	0.95	0.05	97.14	5.96	97.14	5.96
	SR-2CB	1.01	0.01	104.72	0.74	104.72	0.74
	SR-2CS	1.01	0.01	104.88	0.31	104.88	0.31

TABLE 7.7 Binder Content Pay Factor Statistics

Project	Mix Designation	BINDER CONTENT PAY FACTORS 1. KDOT Pay Factors MEAN	1. KDOT Pay Factors STD. DEV	2. AASHTO Pay Factors MEAN	2. AASHTO Pay Factors STD. DEV	3. NHI Pay Factors MEAN	3. NHI Pay Factors STD. DEV
US-50	SM-1T	-	-	103.45	3.46	103.45	3.46
	SM-1B	-	-	99.63	10.26	95.33	19.92
K-177	SM-1T	-	-	105	0	105	0
	SM-2C	-	-	104.06	3.14	104.06	3.14
	SM-1T	-	-	96.39	12.97	92.78	21.47
	SM-2C	-	-	92.05	11.9	79.22	41.39
	SR-2C	-	-	95.89	15.63	92.79	21.54
I-70	SM-2C	-	-	95.11	10.55	90.52	19.44
	SR-2CB	-	-	101.61	5.47	101.43	5.93
	SR-2CS	-	-	103.23	3.23	103.23	3.23
K-96	SM-1T	-	-	93.88	2.13	93.88	2.13
	SR-2CB	-	-	104.08	2.00	104.08	2.00
	SR-2CS	-	-	101.71	5.62	101.71	5.62

TABLE 7.8 Compacted Pavement Density Pay Factor Statistics

Project	Mix Designation	COMPACT DENSITY PAY FACTORS 1. KDOT Pay Factors MEAN	1. KDOT Pay Factors STD. DEV	2. AASHTO Pay Factors MEAN	2. AASHTO Pay Factors STD. DEV	3. Nebraska DOR Pay Factors MEAN	3. Nebraska DOR Pay Factors STD. DEV
US-50	SM-1T	0.98	0.02	65.19	7.26	0.60	0.22
	SM-1B	0.99	0.02	72.85	10.39	0.76	0.17
K-177	SM-1T	1.02	0.02	90.85	9.60	0.99	0.02
	SM-2C	1.01	0.03	86.10	16.56	0.80	0.19
	SM-1T	0.96	0.04	68.29	18.81	0.49	0.15
	SM-2C	1.03	0.01	100.60	6.50	0.99	0.02
	SR-2C	1.02	0.02	91.58	14.60	0.92	0.11
I-70	SM-2C	1.03	0.01	0.97	0.05	102.64	6.65
	SR-2CB						
	SR-2CS						
K-96	SM-1T	1.02	0.01	0.93	0.07	96.00	11.62
	SR-2CB						
	SR-2CS						

7.7 Operating Characteristic (OC) Curves

The OC curve determines whether a given pay schedule will perform as desired in the long run or not and estimates the risks to both the agency and the contractor. The process to develop an OC curve for a given pay equation needs a computer generated set of random test results.

However, if a large number of actual field test results covering an entire mix production are known, as in the present study, a simple check for the pay schedules can be done. The mean test value of the mix property, on which the payment is based (say air voids), is determined for all lots produced and the corresponding pay factor is calculated. Next the average of the individual pay factors for all lots in that mix production is estimated. If this average pay factor matches the pay factor for the mean test value, the pay schedule is performing satisfactorily.

The development of OC curves follows the same logic, but uses numerous sets of randomly generated test values instead of actual test values. The sets are generated so that the random test values represent a normal distribution with a standard deviation equal to that assumed in development of the pay schedule and their mean values coincide with each level covered in the pay schedule. Then the average pay factor for each set is calculated by computer simulation. These average pay factors when plotted against the corresponding mean test values will give the OC curve. If the pay factor given by the OC curve does not match the pay factor from the pay schedule for a given test value, the pay schedule is not performing as intended.

7.8 Check for Pay Factors

The simple check for the satisfactory operation of KDOT pay schedules was conducted. The test values for air voids and percent compacted pavement density for the 13 mixes on four projects were assumed to be normally distributed. The average pay factors were drawn from the Tables 7.6 and 7.8, while the average test values for the air voids and density were determined from Tables 7.2 and 7.5, respectively. The pay factor for the mean test values were estimated from the KDOT pay schedules (26). The results are shown in Table 7.9.

TABLE 7.9 Simple Check for KDOT Pay Schedules

Project	**Mix Designation**	**PAY FACTORS**					
		AIR VOIDS			**DENSITY**		
		Avg. Pay Factor	**Mean Test Value Abs.Dev**	**Pay Factor for Mean**	**Avg. Pay Factor**	**Mean Test Value Abs.Dev**	**Pay Factor for Mean**
US-50	**SM-1T**	0.96	1.22	0.95	0.98	89.6/87.8	0.94
	SM-1B	0.96	1.04	1.00	0.99	90.6/89.3	0.99
K-177	**SM-1T**	0.98	1.00	1.00	1.02	92.7/89.8	1.01
	SM-2C	0.99	0.78	1.00	1.01	91.7/90.5	1.01
	SM-1T	0.98	0.81	1.00	0.96	89.0/87.4	0.98
	SM-2C	0.97	0.90	1.00	1.03	93.1/91.9	1.03
	SR-2C	0.96	0.91	1.00	1.02	92.3/90.8	1.02
I-70	**SM-2C**	0.99	0.49	1.02	1.03	92.7/91.9	1.03
	SR-2CB	1.02	0.49	1.02			
	SR-2CS	1.00	0.59	1.00			
K-96	**SM-1T**	0.95	1.05	1.00	1.02	92/91	1.03
	SR-2CB	1.01	0.59	1.00			
	SR-2CS	1.01	0.61	1.00			

7.9 New KDOT Pay Factors

During the course of this study, KDOT slightly modified the air voids and compacted density pay schedules. This modification essentially consisted of incorporating pay factor equation for some steps within the stepped pay schedule. For example, the original air voids pay schedule (for Lot size of 4 sublots) shown in Table 6.2 is modified as shown in Table 7.10. The pay factors were also calculated under the new scheme and the average pay factors calculated by the original schedule and the modified schedule are shown in Table 7.11.

7.10 Relating Volumetric Properties with Mechanistic Test Results

The present study, like others described earlier, has developed the transfer function for fatigue failure based on the following generic form

$$N_F = F_1(\varepsilon_T)^{-f2} \qquad (7.1)$$

Implementing specifications based on fundamental mix properties (like strain, stiffness and creep) is difficult because most agencies lack the capabilities for actual measurement of these properties. Therefore relationships that relate these fundamental properties to construction variables that are monitored by traditional specifications (e.g. air voids and binder content) may be used (22). In other words, traditional mix parameters can be incorporated into the generic equation instead of or along with fundamental properties.

TABLE 7.10 New KDOT Pay Schedule for Air Voids (Lot Size of Four Tests)

Air Voids Deviation Value	Pay Factor
0.00<D4<0.35	1.030
0.36<D4<0.55	1.000 + 0.15 (0.55 - D)
0.56<D4<1.05	1.000
1.06<D4<1.40	1.000 – 0.44 (D - 1.05)
1.41<D4	0.800

TABLE 7.11 Comparison of Original and Modified KDOT Pay Schedules

Project	Mix Designation	AVERAGE PAY FACTORS			
		AIR VOIDS		DENSITY	
		Original Pay Schedule	Modified Pay Schedule	Original Pay Schedule	Modified Pay Schedule
US-50	SM-1T	0.96	0.960	0.98	0.978
	SM-1B	0.96	0.959	0.99	0.988
K-177	SM-1T	0.98	0.997	1.02	1.016
	SM-2C	0.99	0.986	1.01	1.008
	SM-1T	0.98	0.979	0.96	0.957
	SM-2C	0.97	0.978	1.03	1.032
	SR-2C	0.96	0.955	1.02	1.016
I-70	SM-2C	0.99	0.987	1.03	1.028
	SR-2CB	1.02	1.018		
	SR-2CS	1.00	0.999		
K-96	SM-1T	0.95	0.939	1.02	1.019
	SR-2CB	1.01	1.012		
	SR-2CS	1.01	1.006		

The Asphalt Institute fatigue equation incorporates the air voids and binder content in the form of a correction factor (7). More recently, the SHRP researchers have also incorporated air voids in their equations (8). Therefore, it was decided to incorporate air voids as a variable in the transfer functions developed earlier.

Since the effective binder contents were constant values for the three mixtures studied, it could not be used as a variable. The effect of field compaction lies presumably in increasing density and reducing air content of the compacted asphalt mixes. However, the mechanisms of this vital process in affecting pavement performance are not fully understood (30). In the present study, although the compacted density (as a percent of Gmm) for each test beam was measured, it was not used as a variable. Because the lab density and field density are very different, and also the air voids, calculated on the same basis, were already being used as variables.

7.11 Development of Equations Incorporating Air Voids

As binder content was not used as a variable, the new equations were not modeled incorporating a correction factor like the Asphalt Institute equation. Instead the air void was included as an independent variable in addition to the tensile strain in the original equations. The following generic form was used for modeling the equations,

$$N_f = f1\,(\epsilon_t)^{f2}(Va)^{f3} \qquad (7.2)$$

where

N_f = number of repetitions to fatigue failure

fx = fatigue coefficients

ϵ_t = tensile strain at the bottom of asphalt concrete layer

Va = air voids (%)

The following equations were obtained using SAS for the three mixes studied:

SM-2C:

$$N_f = 0.000593.\varepsilon_t^{-1.651}.Va^{2.017} \quad (R^2 = 0.77) \qquad (7.3)$$

SM-1B:

$$N_f = 2.192.\varepsilon_t^{-1.517}.Va^{-0.775} \quad (R^2 = 0.85) \qquad (7.4)$$

BM-1B:

$$N_f = 0.0083.\varepsilon_t^{-1.686}.Va^{-0.024} \quad (R^2 = 0.81) \qquad (7.5)$$

The addition of air void as an independent variable increased the coefficient of determination, R^2 values obtained by the SAS software by 2, 5 and 0 percents for the SM-2C, SM-1B and BM-1B equations, respectively. The levels of significance, p-values for the variable air void, were 0.29, 0.03 and 0.98 for the SM-2C, SM-1B and BM-1B equations, respectively.

A newly developed SHRP-A003A model, which incorporates air void content, is shown below (8):

$$N_f = 2.5263 * 10^5 * \exp(-0.2007Va) * (\varepsilon_t)^{-3.4134}(E^*)^{-2.1239} \qquad (7.6)$$

This model was also used for incorporating air voids into the transfer function. However, the dynamic stiffness modulus, which is used as a variable in the SHRP model, was left out for reasons explained in Chapter 5. The following equations were obtained using SAS software:

SM-2C:

$$N_f = 0.011.\exp(0.173Va) * (\varepsilon_t)^{-1.65} \quad (R^2 = 0.77) \qquad (7.7)$$

SM-1B:

$$N_f = 1.177.\exp(-0.132Va) * (\varepsilon_t)^{-1.523} \quad (R^2 = 0.85) \qquad (7.8)$$

BM-1B:

$$N_f = 0.007.\exp(0.005Va) * (\varepsilon_t)^{-1.706} \quad (R^2 = 0.81) \qquad (7.9)$$

The incorporation of air voids increased the R2 values obtained using SAS software by 2, 5 and 0 percents for the SM-2C, SM-1B and BM-1B equations, respectively. The p-values were 0.29, 0.03 and 0.98 for the SM-2C, SM-1B and BM-1B equations, respectively. In fact, these statistics match those of the earlier model.

CHAPTER 8

STATISTICAL ANALYSIS: RESULTS AND DISCUSSION

The specifications used for the pilot projects are created by incorporating some performance-based specifications into the earlier method type KDOT Standard Specifications. Some experts feel that in developing performance based specifications current state manuals should not be referenced at all. They argue that it is not possible to change results while keeping the recipe the same (25).

At present KDOT must lay emphasis on reducing the method type specifications to an absolute minimum. For flexible pavements the exact mechanisms by which widely monitored volumetric properties affect the final performance are not well established. For such properties, method type specifications and engineering judgement may be necessary for process control. As improved models describing the performance relationships of these mix properties become available, the minimal method-type specifications and engineering judgement issues can be phased out to develop truly performance- based specifications.

Some experts (24) have suggested that specification limits be based on historical database. While others question the appropriateness of using historical variability obtained from construction test data controlled with traditional specifications since it may be biased (31). Western Association of State Highway Transportation Officials (WASHTO) and National Cooperative Highway Research Program (NCHRP) has recommended development of unbiased databases under model QC/QA specifications for setting up final specification limits. Using the latter approach, the Alabama Department of Transportation gradually implemented their statistical specifications over three construction seasons (31). Similarly, if KDOT continues to develop a statistically sufficient database on its pilot projects, final specification limits can be obtained by the statistical analysis.

There seems to be some doubt regarding the number of verification tests required per Lot. The basic purpose of verification test results is to test validity only and not acceptance. Therefore, it may be argued that the verification test results should be kept at a minimum to save costs. However, it has to be kept in mind that if the QC test results are to form the sole basis of acceptance, there correct validation in accordance with statistical laws is necessary.

To test the validity of QC test results on a lot wise basis, five compacted density and one air void/binder content tests per Lot are more than adequate and inadequate, respectively. At this point it is suggested that, the agency should maintain the verification test frequency at one per Lot strictly for the air voids and binder content test results. Validity on a Lot wise basis can then be achieved by a simple t-test. Then after the first three Lots are validated individually, the 12 QC test results and the three verification test results can be used to test overall validity by the AASHTO procedure. In spite of successful validation at this stage the verification test frequency must be maintained for another three Lots. Each new Lot will thus be validated individually by t-test and overall by the AASHTO procedure. After six Lots when a statistically valid sample size of 30 tests (24 QC and six verification) is obtained, the agency may choose to lower the verification frequency depending on results obtained.

Although more verification tests lower the risks involved, the agency may choose to lower the verification testing frequency from 5 to 3 or 4 and still continue to validate QC test results on a Lot wise basis using the AASHTO procedure. If the agency feels comfortable based on past performance, the verification testing frequency for density may be decreased to one test per Lot after 10 Lots (30 verification results). This will allow Lot wise validation using simple t-test and also AASHTO procedure can be used as new test results get added in the overall test database.

8.1 Summary of Test Result Statistics

Large differences were observed among the standard deviation values computed by the three methods. Theoretically, the results provided by methods 1 and 3 are valid only if the sample size (number of sublots within a Lot) exceeds 30. In contrast, the sample size used in method 2 (total number of sublots) will always be larger than the other two methods.

It is felt that the variability obtained for some mixes should not be used in specification development because of statistically inadequate data points (US-50 - SM-1T, K-177 - SM-1T Phase 1, K-96 - SM-1T). Also the validity of variability obtained for SM-2C on the second phase of K-177 is severely compromised because standard lot sizes, as per definition, were rarely produced. As such, the statistics were recomputed after combining irregular lots into best possible sizes. The best estimate of variability for the three mix properties of 13 mixes studied is shown in Table 8.1. Average value of the standard deviations obtained by the three methods is used; but in some cases extreme outlier values have been discarded.

TABLE 8.1 Summary of Test Result Statistics

Project	Mix Design	AIR VOIDS			BINDER CONTENT		DENSITY	
		Mean	Std. Dev.	Std. Dev Abs. Dev.	Std. Dev. Single Point	Std. Dev. 4-Point Moving	Mean	Std. Dev.
US-50	SM-1T	4.83	1.25	0.92	0.24	0.14	89.7	1.59
	SM-1B	3.34	1.11	0.72	0.41	0.35	90.6	1.24
K-177	SM-1T	3.09	0.56	0.37	0.12	0.06	92.8	2.37
	SM-2C	3.93	0.78	0.61	0.25	0.18	91.7	1.82
	SM-1T	3.59	0.95	0.56	0.39	0.22	89.0	1.78
	SM-2C	4.15	1.26	0.83	0.39	0.30	93.1	1.49
	SR-2C	3.78	1.20	0.65	0.45	0.36	92.3	2.05
I-70	SM-2C	4.12	0.67	0.38	0.32	0.39	92.66	1.09
	SR-2CB	3.9	0.68	0.44	0.23	0.14		
	SR-2CS	3.63	0.80	0.62	0.28	0.20		
K-96	SM-1T	4.43	1.46	0.75	0.48	0.15	92.05	1.39
	SR-2CB	4.02	0.81	0.48	0.21	0.12		
	SR-2CS	3.88	0.74	0.40	0.25	0.17		

8.2 Interpretation of Test Result Statistics

The mean target value is 4 percent for all mixes. The specification working ranges use a standard deviation of 0.66 percent for air voids test values. The air voids pay schedule (for lot size of 4 sublots) is based on a standard deviation of 0.20 for mean absolute deviation values. Table 8.1 shows that for the first two projects of 1996, mean air voids vary from 3.09 to 4.83, the standard deviation ranges from 0.56 to 1.26 for air voids test values, and from 0.37 to 0.92 for absolute deviation values. Thus good accuracy was achieved but the same is not true for precision. While for the two 1997 projects mean air voids varied from 3.63 to 4.43, the standard deviation ranges from 0.67 to 1.43 for air voids test values, and from 0.38 to 0.75 for absolute deviation values. Thus the accuracy improved but precision could not be bettered.

The specification working ranges for binder content assume a standard deviation of 0.2 percent for single point test results and 0.1 percent for 4-point moving average. Table 8.1 shows that the standard deviation varies from 0.12 to 0.45 for single point and from 0.06 to 0.36 for 4-point moving average test results for the two 1996 projects (US-50 and K-177). For binder content property, good accuracy is achieved but the same is not true for precision. In 1997, for I-70 and K-96 standard deviation varies from 0.21 to 0.48 for single point and from 0.12 to 0.39 for 4-point moving average test results. Thus though the accuracy was maintained, the precision may have dropped slightly.

For compacted pavement density no specification limits are set, while pay schedule provides full payment for lot mean density ranging upwards from 91 percent of Gmm. KDOT historical database (of 1488 main-line paving) test results show a weighted mean density of 95.72 percent of Marshall density with a standard deviation of 1.8 percent (29). Table 8.1 shows a lot mean densities ranging from 89 to 93.1 (percent of Gmm), while standard deviations range from 1.24 to 2.37 (percent of Gmm) for 1996 projects. Fair accuracy and precision were achieved. While in 1997, I-70 project had a mean density of 92.6 percent with a standard deviation of 1.09. The K-96 project had a mean density of 92.05 percent with a standard deviation of 1.39. Thus, both accuracy and precision improved. However, this may have been due to combined calculation for three different types of mixes on these projects.

It can be stated from the observations of Table 8.1 that the variabilities (standard deviations) of the three volumetric properties are significantly different for different types of mixes. It has been shown that for asphalt mixes, a substantial portion of variability comes from material variation or construction process itself (31). Since material variability, process variability and/or sampling and testing variability will be different for different type of mixes, it is felt that it is incorrect to specify a common specification limit and/or pay schedule for all types of mixes. This has also been found in a NCHRP study (31). It is suggested that KDOT should examine the possibility of specifying different specification limits and pay schedules for different types of mixes, at least on a broad bases. The variabilities found in this study and through future data collection should be used for this purpose. .

8.3 KDOT's Specification Working Ranges

Table 6.1 shows that a zero tolerance is specified for the single point test results of certain properties to seek exact uniformity with the approved JMF. It must be noted that even the approved JMF itself is a random sample selected from a large possible population of acceptable mix formuli. The concept of zero tolerance goes against the basic foundation of statistical specifications, which lies in accepting certain level of variability as normal. It is strongly felt that the 'zero tolerance' set as the specification limit for some mix properties must be reconsidered. In the absence of well-established performance relationships, engineering judgement or AASHTO values may be substituted.

Specification limits also exist for the 4-point moving average test results, particularly for binder content. Some experts believe that the usefulness of moving average concept as a basis for mixture acceptance is doubtful. Since each single point test result appears in four different moving average results used as acceptance criteria, there is a lack of statistical independence. Also, computer simulation has shown that the use of moving average provides no gain in the discriminating power obtained from the use of single point test results. Yet, it tends to reduce statistically variability in the long run (24). Hence it is felt that the use of 4-point moving average criteria should be continued as a process control tool (in QC charts), but its continuation as a basis for acceptance should be reconsidered.

8.4 Discussion of Pay Schedules

KDOT uses stepped pay schedules and has recently replaced some 'stepped pay factors' by pay equations within the same framework. Continuous pay equations are more desirable than stepped pay schedules because they reduce the disputes relating to rounding-off of test values especially at the boundary of a 'step'. Besides they have essentially the same long-term performance as stepped pay schedules (24). This is also observed in Table 7.11, where on an average, the pay factors calculated by the original and modified pay schedules are found to be the same. The replacement of the modified stepped pay schedules by a single continuous pay equation will be beneficial.

Table 7.6 shows that the mean KDOT air voids pay factors are considerably lower (by about 0.05) than those of AASHTO and NHI. This difference decreased to 0.035 in 1997 probably due to better accuracy. The check of air voids pay schedule (Table 7.9) revealed that the average of actual pay factors applied are lower than the pay factor for mean absolute deviation by about 0.03 in 1996. This difference also decreased to 0.01 in 1997. Although this difference is significant, a new pay schedule is not warranted. This is partially caused by the comparatively large step in the Mean Absolute Deviation, D4 (ranging from 0.56 to 1.05) corresponding to the pay factor of 1.000. A continuous pay equation as shown in Figure 8.1 is suggested for replacing the air voids pay schedules for Lot size of 4 sublots. This curve is established using the pivotal points of the original stepped pay schedule, and curves for other Lot sizes can be obtained similarly.

Binder content significantly affects pavement performance and has been included in the Asphalt Institute's fatigue life equation. It is felt that this important mix property should form a basis for pay adjustment in addition to its existing use in process control. Table 7.7 shows the widely ranging binder content pay factors calculated by AASHTO and NHI methods using the KDOT specification limits. The use of a pay schedule/equation based on binder content is highly recommended.

Table 7.8 shows that the mean KDOT density pay factors are much higher (by about 0.25) than those of AASHTO and Nebraska DOT in 1996. In 1997 the difference was reduced to 0.08 probably due to better accuracy and precision. But again this may have been due to combined calculations for the three different mixes. Table 7.9 shows that the pay schedule is performing very well. The average of actual pay factors closely matches the pay factor for mean lot density values for most mixes. However, two problems must be noted. First, a system based only on averages and using the lowest sublot density will be biased on the lower side. Second, this pay schedule places only a lower bound on the variability of compacted pavement density.

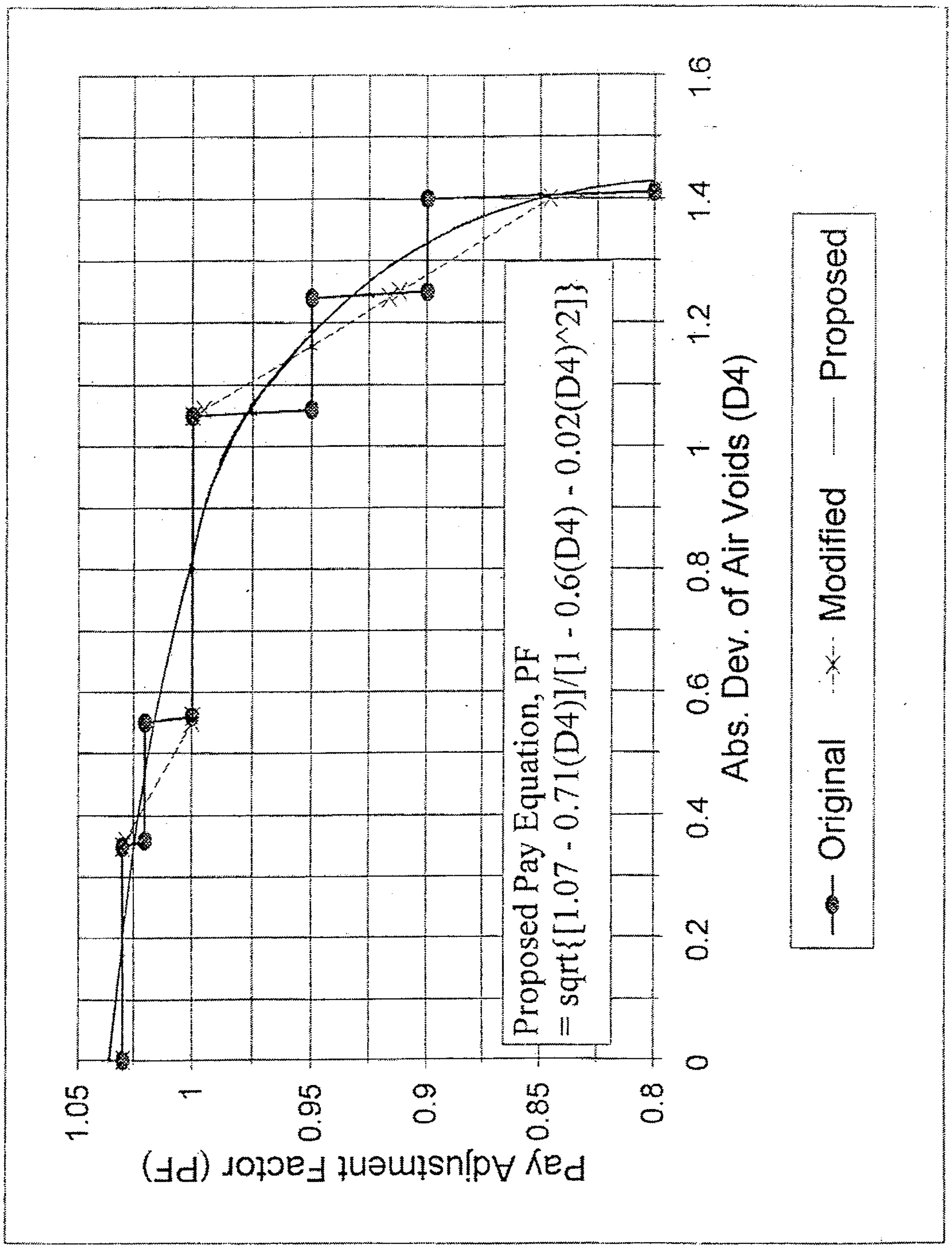


FIGURE 8.1 Original and Modified KDOT Air Voids Pay Schedules with Proposed Pay Equation (Lot Size of Four Tests)

AASHTO density pay factors are based on the PWL concept with a target mean of 93 percent of Gmm, bound by slightly unsymmetric limits (lower limit of 91 percent and upper limit of 96 percent). Upper limit is applied because higher density beyond a certain optimum level (93 percent according to AASHTO) is also detrimental to pavement performance. It is felt that the present pay adjustment system must include an upper limit on density test values. This will also help to control the variability. A pay equation/schedule based on the absolute deviation method (similar to air voids) with a target mean of 93 percent of Gmm and using limits based on realistic variability obtained for the mixtures in this study should be developed.

8.5 Combined Pay Factor for Multiple Quality Characteristics

When statistical specifications provide pay factors for multiple quality characteristics, they must be combined to obtain an overall pay factor for the construction item. For example, assume that a single transfer function incorporating various mix properties is developed. Then the life of each Lot produced can be predicted by using respective test results as inputs. Applying the principle of liquidated damages to the difference between the predicted and the design life, the actual pay factor can be computed. Thus a continuous overall pay factor equation with the mix properties as variables can be developed.

In absence of such relationships, multiplication of individual pay factors is considered a good method to arrive at an overall pay factor (21). It is felt that the Lot size for both air voids and density must be made the same (preferably a day's production). This will allow the multiplication of air voids and density pay factors to arrive at a combined pay factor. The different number of sublots within a Lot (4 for air voids and 5 for density) need not be changed. This multiplication method will also allow for later inclusion of new pay factors (say, based on binder content).

8.6 Volumetric Parameters in Fatigue Life Equations

The need for the mechanistic transfer functions incorporating routinely monitored mix properties (like air voids and binder content) in developing performance-based specifications has been explained earlier. In the absence of exact relationship between these properties and performance, the existing pay schedules for these properties can be developed based on theoretical considerations, past experience and engineering judgement.

The modified transfer functions (Equations 7.3 to 7.9) obtained in this study by incorporating air voids as a variable were not found to be statistically appropriate. The contribution of the air voids variable in the R^2 values was less than 5 percent, while except for SM-1B mix, the p value was more than 0.1. This was expected since the air void values were clustered around 11 and 5 percents for SM-2C and SM-1B respectively, with little variability. The primary reason for this was that air void was not planned to be used as a control variable during the experimental design.

However, the equation for SM-1B (Eqn.7.4), which has a good p-value (0.03) can be used to demonstrate the concept of developing performance based pay adjustment schedules. Consider six Lots having varying levels of air voids as shown in Column 2 of Table 8.2. Air voids levels were chosen to match 'steps' of original KDOT pay schedule. For a constant strain of 111 micro-in/in, the laboratory performance lives were predicted by Equation 7.4. Thus, for every 1 percent increase in air voids, a decrease in fatigue life of 121,963 ESALs can be expected. If the predicted life at 4 percent air voids (767,986 ESALs) is considered to be the design ESALs, this results in a decrease in design life by 15.9 percent.

However, this does not mean that for every 1 percent increase in air voids, the pay adjustment factor should be (100 - 15.9)/100, i.e. 0.841. The pay adjustment is designed to withhold sufficient payment at the time of construction to cover the future repair costs made necessary due to premature failure caused by deficiency in construction quality. The reverse explanation applies for bonus pay factors. Using the principle of liquidated damages (Eqn. 8.1) it is possible to develop an appropriate pay schedule for various levels of expected lives corresponding to various levels of air voids as shown in Table 8.2.

$$PF = [1 + C_o (R^{Ld} - R^{Lp}) / C_p (1 - R^{Lo})] \qquad (8.1)$$

where

PF = pay factor (percent)
C_p = present unit cost of pavement ($/sq. yd)
C_o = present unit cost of overlay ($/sq. yd)
L_d = design life of pavement (years)
L_p = predicted life of pavement (years)
L_o = expected life of overlay (years)
R = (1 + annual inflation rate/100)/(1 + annual interest rate/100)

To illustrate the development of pay schedule the following values are assumed:

C_p = $15 /sq. yd
C_o = $10 /sq. yd
L_d = (ESALs) = 10yrs @ 76,800 ESALs/ year
L_p = (predicted life in ESALs/ 76,800 ESALs)yrs
L_o = 5 yrs
$R = (1 + 4/100)/(1 + 8/100) = 0.963$

8.7 A Fresh Factorial Experiment

From the previous discussion, it is quite clear that true performance-based pay equations can be developed only when exact relationships between the mix volumetric properties and pavement performance have been established. The modeling of a valid transfer function incorporating volumetric properties requires new test data to be obtained under a statistically-designed controlled experiment. The following factorial experiment is recommended to include the air void and binder content as controlled variables in the fatigue life equation.

1) Test Type: Flexural fatigue testing of asphalt concrete beams

2) Test Conditions: Similar to the current study

3) Controlled Factorial Variables

   a) Air Voids: @ 2, 3, 4, 5 and 6%

   b) Binder content: @ -0.6, -0.3, 0, 0.3 and 0.6% of the Superpave design binder content for the test mix

4) Sample size: Total of 50 beam specimens per each test mix (@ 2 beams per factorial combination)

TABLE 8.2 Development of Performance-Based Air Voids Pay Schedule

Constant C (times σ)	**Air Voids (%)**		**Predicted Life (ESALs)**	**Predicted Life (years)**	**Perform. Based PF***	**Original KDOT PF**
	Test Result	**Abs. Dev.**				
0	4.0	0	767986	10	1.030	1.03
0.25	4.35	0.35	719649	9.37	0.966	1.03/1.02
0.75	4.55	0.55	695010	9.05	0.933	1.02/1.00
2.5	5.05	1.05	641060	8.35	0.859	1.00/0.95
3.0	5.24	1.24	622971	8.11	0.833	0.95/0.90
3.5	5.4	1.4	608618	7.92	0.813	0.90/0.80

Note: *To provide for bonus, 0.03 is added o the PF calculated by Equation 8.1.

CHAPTER 9

CONCLUSIONS AND RECOMMENDATIONS

9.1 Conclusions

Based on the results of this study, the following conclusions can be drawn:

1. The predicted fatigue lives obtained by the KSU and SHRP equations are close for SM-2C mix when compared with the prediction by the AI equation. However, the predicted fatigue lives for KDOT conventional BM-1B mix by the KSU and AI equations are close when compared with the prediction by the SHRP equation.

2. Except for SM-2C, the KSU equations have predicted lower lives for other mixes than the SHRP and AI equations.

3. The SM-1B mix appeared to have a better predicted fatigue life than the SM-2C mix. The air voids percentage of the SM-2C beam samples was almost twice that of the SM-1B samples. It is felt that air voids percentage, at higher levels, will significantly affect (lower) the number of repetitions. Besides, the asphalt content of SM-1B mix is 11% higher than the SM-2C mix.

4. Overall, the Superpave mixtures appeared to have far superior predicted fatigue lives than the asphalt mixes used by KDOT in the past. The predicted lives of BM-1B mix under similar strains are one-third to one-quarter of the Superpave mixes.

5. The Asphalt Institute fatigue equation may not be applicable to the Superpave mixtures due to different fatigue behavior of these mixes. On the other hand, extremely high fatigue lives were predicted by the SHRP equation for SM-1B and BM-1B, which are unlikely to be observed in the field.

6. Based on the analysis of the maximum shear stresses on these Superpave pavements, it is felt that the recommended maximum shear stress values in Superpave Level 2 mix design for tertiary creep evaluation are not realistic.

7. Firm conclusions for all mixes, cannot be drawn from the statistical analysis of the volumetric test results due to factors like statistically insufficient data points and irregular lot sizes. However, the variability (standard deviation) of volumetric properties is significantly different for different types of mixes.

8. For air voids accuracy increased from 1996 to 1997 but precision could not be improved. For binder content accuracy was maintained but precision may have dropped slightly. The changes in accuracy and precision of compacted pavement density from 1996 to 1997 indicate an improvement in both accuracy and precision. But due to combined calculation for three different types of mixes on both I-70 and K-96 projects this cannot be concluded reliably.

9. The KDOT pay schedules for density and air voids are performing as desired. However, when compared to AASHTO, the mean KDOT density pay factors are much higher (by about 0.25), while air voids pay factors are lower (by about 0.05). The later introduction of equations to replace some stepped pay factors produced the same average pay factor over the entire mix production as expected.

10. Air void and binder content must be included in the mechanistic transfer function to develop performance based pay equations. The present attempt to incorporate air void in the fatigue life equation did not yield statistically good results. This was because air void was not planned to be used as a variable during experimental design.

9.2 Recommendations

Based on the results of this study, the following recommendations are made:

1. The recommended maximum shear stresses for tertiary creep evaluation in Superpave Level 2 design need to be studied further.

2. The shift to performance-based specifications from the present combination of method type and performance-based specifications must be completed in a phased manner. At present emphasis must be laid on specifying only the final result as clearly as possible, while reducing the method-type specifications to an absolute minimum.

3. The zero tolerance set as the specification limit for some mix properties must be reconsidered. Also, the use of 4-point moving average as a dual criteria for mix acceptance must be reconsidered.

4. The need to set different specification limits and pay equations/schedules for different type of mixes should be studied. The variabilities obtained in this study can be used for this purpose.

5. A fresh factorial experiment using air void and binder content as controlled variables is needed to develop truly performance-based pay equations. Irrespective of this, the use of binder content as a basis for pay adjustment must be considered.

6. It is felt that the present method for computing density pay factor should be replaced by a pay equation based on the absolute deviation method (similar to air voids) using realistic variability obtained for the mixtures in this study. This will also set double sided limits and help control variability.

7. At present, the replacement of stepped pay schedules by a continuous pay equation is recommended.

8. A uniform Lot size for both air voids and density should be specified solely on time basis, preferably a day's production. This will allow the use of a combined pay factor, which shall be a product of air voids and density pay factors. The different number of sublots within a Lot (4 for air voids and 5 for density) need not be changed. However, an adequate number of verification test results must be collected as suggested earlier.

REFERENCES

1. Asphalt Institute. Superpave Level 1 Mix Design. Superpave Series No. 2 (SP-2). Lexington, Kentucky. August 1995.

2. McGennis, R. B. et al. Background of SUPERPAVE Asphalt Mixture Design and Analysis. Final Report No. FHWA-SA-95-003, Federal Highway Administration, Washington, D.C., November 1994.

3. Huang, Y.H. Pavement Analysis and Design. 1st Edition, Prentice Hall, NJ, 1992.

4. Corley-Lay, J.B. AEfforts by North Carolina Department of Transportation to Develop Mechanistic Pavement Design System. Transportation Research Record 1539, Transportation Research Board, National Research Council, Washington, D.C., 1996, pp. 18-24.

5. Thompson, M.R. A Mechanistic-Empirical Flexible Pavement Design: An Overview. Transportation Research Record 1539, Transportation Research Board, National Research Council, Washington, D.C., 1996, pp. 1-5.

6. Mahoney, J.P. Examination of Washington State Department of Transportation Transfer Functions for Mechanistic-Empirical Asphalt Concrete Overlay Design. Transportation Research Record 1539, Transportation Research Board, National Research Council, Washington, D.C., 1996, pp. 25-32.

7. The Asphalt Institute. Research and Development of the Asphalt Institute's Thickness Design Manual (MS-1) Ninth Edition. Research Report No. 82-2. The Asphalt Institute, Lexington, KY, 1982.

8. Sousa, J.B. et al. "Application of SHRP Mix Performance-Based Specifications." Transportation Research Record 1454, Transportation Research Board, National Research Council, Washington, D.C., 1996, pp. 154-162.

9. Hveem, F.N. "Pavement Deflections and Fatigue Failures," Bulletin, Highway Research Board, Washington, 1955, pp. 43-87.

10. Hossain, M. et al. "A Study of Flexural Fatigue Behavior of Asphalt-Rubber Mixes." Final Report. Kansas State University, Manhattan, KS, 1997.

11. Maupin, G.W. and J.R. Freeman. "Simple Procedure for Characterization of Bituminous Concrete." Report No. FHWA-RD-76-102, Virginia Highway & Transportation Research Council, Charlottesville, Virginia, June 1976.

12. SHRP. Fatigue Response of Asphalt-Aggregate Mixes, Final Report, SHRP-A-404, Strategic Highway Research Program, National Research Council, Washington, D.C., 1994.

13. Deacon, J.A., and C.L. Monismith. "Laboratory Flexural Fatigue testing of Asphaltic Concrete with Emphasis on Compound Loading," Paper presented at the Annual Meeting of the Highway Research Board, Washington, D.C., January 1966.

14. Taylor, I.F. Asphaltic Road Materials in Fatigue, Thesis, University of Nottingham, October 1968.

15. Epps, J.A., and C.L. Monismith. "Fatigue of Asphalt Concrete Mixtures - Summary of Existing Information," Fatigue of Compacted Bituminous Aggregate Mixtures, ASTM STP 508, American Society for Testing and Materials, 1972, pp. 19-45.

16. Kallas, B.F., and V.P. Puzinauskas. "Flexure Fatigue tests on Asphaltic Paving Mixtures," ASTM STP 508, Fatigue of Compacted Bituminous Aggregate Mixtures, American Society for Testing and Materials, 1972, pp. 47-65.

17. Kim, Y.R. et al. "Healing in Asphalt Concrete Pavements: Is it Real?" Transportation Research Record 1454, Transportation Research Board, National Research Council, Washington, D.C., 1996, pp. 89-96.

18. Brown, S.F. AA Simplified Fundamental Design for Bituminous Pavements. Journal of Highway Engineers, Vol. 21 No. 8-9, London, U.K., 1974, pp.14-23.

19. Discussion with Dr. M. Hossain, Kansas State University, Manhattan, KS, September 1997.

20. SHRP. The Superpave Mix Design Manual for New Construction and Overlays. Final Report, SHRP-A-407. The Strategic Highway Research Program, National Research Council, Washington, D.C., 1994.

21. Weed, R.M. Statistical Specification Development. 2nd Edition. Report No. FHWA/NJ-88-017. New Jersey Department of Transportation, NJ, 1989.

22. Fernando, E.G. "Demonstration of Potential Benefits of Performance-Oriented Specifications." Transportation Research Record 1353, Transportation Research Board, National Research Council, Washington, D.C., 1992, pp. 73-81.

23. AASHTO. Implementation Manual for Quality Assurance. A Report of the AASHTO Highway Subcommittee on Construction. AASHTO, Washington, D.C.,1996.

24. Afferton, K. et al. "Managing Quality: Time for a National Policy." Transportation Research Record 1340, Transportation Research Board, National Research Council, Washington, D.C., 1992, pp. 3-39.

25. Shilstone,J.M.Sr. "Quality Management for Concrete Pavement Under Performance Standards." Transportation Research Record 1340, Transportation Research Board, National Research Council, Washington, D.C., 1992, pp. 48-55.

26. Hossain, M et al. Handbook of Superpave Quality Control/Quality Assurance (QC/QA) Analysis. Kansas State University, Manhattan, KS, 1997.

27. Frazier, P.Jr. et al. "Development of New Criteria for Control of Hot-Mix Asphalt Construction." Transportation Research Record 1389, Transportation Research Board, National Research Council, Washington, D.C., 1993, pp. 1-8.

28. Ott, R.L. An Introduction to Statistical Methods and Data Analysis. 4th Edition, Duxbury Press, Belmont, CA, 1993.

29. Facsimilie communications with Rodney Maag, P.E. Field Engineer, Kansas Department of Transportation, November 1997.

30. Abd El Halim, A.O. et al. AEffect of Field Compaction Method on Fatigue Life of Asphalt Pavement.@ Transportation Research Record 1469, Transportation Research Board, National Research Council, Washington, D.C., 1994, pp. 43-49.

31. Frazier, P.Jr. et al. AHot-Mix Asphalt Mix Properties Measured for Construction Quality Control and Assurance.@ Transportation Research Record 1469, Transportation Research Board, National Research Council, Washington, D.C., 1994, pp. 9-17.